MILITARY INSTITUTE OF SCIENCE AND TECHNOLOGY (MIST)

DEPARTMENT OF CIVIL ENGINEERING

GEOTECHNICAL ENGINEERING LABORATORY

TERZAGHI LAB

GEOTECHNICAL ENGINEERING SESSIONAL



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SECTION 1

GENERAL INSTRUCTION FOR GEOTECHNICAL ENGINEERING SESSIONAL

General Discussions

At undergraduate level, CE 342 (or EWCE 342) is the only course that will require the students to come to Geotechnical Engineering Laboratory except the students take major on Geotechnical Engineering in Level 4. This course will give the students the opportunity to gain "hands-on" experience with some of the laboratory tests and techniques used in soil testing as observed in any typical soil test reports of today. In this course, the students will gain experience in both the performance of soils tests and the subsequent analysis of that data for some of the fundamental soil mechanics laboratory procedures.

Course Contents

There are total ten tests designated for this course. These tests are described in the following sections:

3. Field Identification Test
4. Grain Size Analysis
5. Specific Gravity Test
6. Atterberg Limit Test
7. Relative Density Test
8. Unconfined Compression Test
9. Compaction Test
10. Permeability Test
11. Direct Shear Test
12. Consolidation Test

Reference Books

- Engineering Properties of Soil and Their Measurement – Joseph E Bowles.
- Soil Testing for Engineers — T William Lambe.
- Any other books referred by the instructors of CE 341.

Modus Operendi

- Students will be divided into few groups.
- At the end of each class, the course instructor will assign the students with the test for the next class. If there is any change in the program — students will be notified through proper channel.
- The students are required to read about the test and come prepared to the laboratory.
- There could be tests at the beginning of each class to assess the preparation for the subsequent laboratory test.
- Students will draw required apparatus from the laboratory assistant and do the test.
- Students will note the test result and do the calculations and get in checked by the course instructor.
- At the beginning of the following week, the students should submit the complete report.

Grading

In this course, students will be graded based on the marks obtained out of 150. Student's activity will be constantly monitored for the subsequent evaluation. The distribution of marks is a follows:

- Laboratory attendance - 20.
- Report writing - 50.
- Laboratory performance - 10.
- Quiz (Total 2) - 50
- Viva - 20

Items to be carried in Each Class

The following items a student must carry while entering

- Geotechnical Engineering Laboratory.
- Reference books, hand-outs.
- Calculator.
- Flexi-curve.
- Scale (preferably steel).
- Pen, pencil, eraser etc.
- Graph papers (normal, semi-log, log).
- Plain papers.

Few Tips to Get Good Grading

- Laboratory attendance is mandatory! It will pay the students in many ways.
- CE 342 (or EWCE 342) will follow CE 341 (or EWCE 341). So, the students should prepare themselves accordingly and read the laboratory handouts for that week's work before coming to laboratory.
- Dedicate the three hours for the laboratory.
- Students should write their own reports individually by themselves following the instructions.
- Prior to any test, the course instructor will give you some briefing on that test — students should listen those carefully for quiz and viva.
- Students should do each and every test procedure by themselves.
- Maintain discipline.

SECTION 2

GENERAL GUIDELINES FOR WRITING LAB REPORTS

General Discussions

The answer to a problem in soil engineering is normally obtained by first determining the properties of the soil in question and then employing these properties to work out the solution. Since the soil at every site is different, the soil involved in each different problem must be evaluated. Often this evaluation can be approximated from knowledge of geology of the site or from experience with similar soils. Usually, the soil properties must be determined by laboratory or field tests. Test report is a very important part for that purpose. In order to give the students some help in writing report on any test covering all minor details and to maintain uniformity this guideline has been prepared.

Report Writing

- Each student shall submit a separate report on each test.
- All reports shall be hand written on a design pad and be bound in a file.
- The report should be brief but self-explanatory.
- Advantage should be taken of tabular and graphical methods of presenting data.
- In addition to the subject matter, clarity, conciseness, method of presentation, legibility and neatness of the report will receive consideration in the grading of a report.
- Grading is also largely dependent on the quality of writing the discussion for each individual test result.
- **Copying of any report will result in adverse effect on the course as a whole**.

File

On the cover of the file the following items must be present:

Subject. Geotechnical Engineering Sessional I.

Name and Roll No of the Student. xxxxx.

Level and Term. xxxxx.

Session. xxxxx.

Index Sheet

An index sheet containing test no, test name, page no and grade is to be placed just after the cover page of the file. See Table 2-1.

Table 2-1. General Format of Index Sheet

Experiment No.	Name of the Experiment	Page No. From To	Grading

Cover Page of Individual Report

It should be as shown in Table 2-2, covering the full page and should be written by hand for each test report.

Table 2-2 General Format of Cover Page

Course No:	Name of the Course	Group No.
Test No. Test Name.		
Date of Performance Date of Submission	Signature of the Course Instructor	Name. Student No. Level & Term Session.

Structure of the Report

a. **Objective of the Test**: What is to achieve/ attain through this test?
b. **Significance of the Test**: Purpose of the test utility of the test, condition required for this test etc.
c. **Equipment/ Apparatus Used**: List down the name of the equipment/ apparatus used during the test.

d. **Test Data Sheet:** Data sheet should be signed by the instructor prior to reporting
e. **Sample Calculations:** All equations or formulae used should be clearly stated with reference. All calculations should be submitted in concise form
f. **Test Results:** The results of the tests should be submitted in tabular or preferably in graphical form, whenever possible.
g. **Discussions:** A brief discussion of all the salient features of the tests, as in the tables and diagrams, should be included. The test results may be compared with pertinent data given in any books or publications and definite conclusions should be drawn.

Submission of the Report

Reports on the tests in a particular class shall be submitted in the next available class or the beginning of next week, as per the advice of the concerned course instructor. 50% will be deducted from the total marks for the late (with a maximum of 2 weeks) submission of the report. **Any** student who, without satisfactory reasons or excuse, fails to nit a report within 2 weeks of the date it is due, is subjected to dismissal of particular report.

Tips on Graph Plotting

Graphs are the largest single cause of re-submission for the sessional report. Few tips for drawing graphs are given here:

a. Labelling

- Provide a title for the graph.
- Label the axes in words and symbols. Do not forget to provide `units'.
- Label curves to distinguish one from another or points of interest.

b. Plotting

- Theoretical curves should be plotted as continuous curves, but in such a way as to distinguish them from the experimental curves. Use `French Curves' or `Flexi-Curves' to draw these; not in freehand.
- Experimental points should be clearly marked and ringed.
- Experimental curves should be a 'best fit' to the experimental points. They need not pass through all points and must not be drawn freehand. Again, use French or flexi-curves to draw these.

- If a plot is defined as linear, fit the best straight line to the experimental points. Do not force this line to pass through the origin; mark the zero-error.
- If the slope of the line is required, do not simply divide the two Mute values of one set of coordinates. Take a genuine tangent slope.

c. Miscellaneous

- Use the space available on a sheet of graph paper. If slopes hay to be measured, the greatest accuracy is achieved by having the line as near as possible to 45°. This can be done by suitably choosing the axes of the graph.
- Plot graphs so that they can be distinctly seen i.e. use a HB pencil.

Tips on Writing Discussion on the Test Results

In the discussion section of the report, the students are expected to show their grasp over the test performed. So it is advisable to think for a while before writing the discussion. Here are few tips to guide. Try to address the following questions:

- What does the test result signify?
- Can the numerical results be applied directly to design and similar uses, or are they of value only for comparison with other results?
- Considering the methods of testing and the kind of material, what are the limitations of the test results, or how reliable are the test data?
- Are the assumed conditions satisfied?
- What is most likely to go wrong or produce incorrect results?

Conclusion

The students are requested to keep their all 'antennas on' during any test to observe all minor details to write a very good and informative report.

SECTION 3

FIELD IDENTIFICATION TEST

(VISUAL MANUAL PROCEDURE)

References

- ASTM D 2488 - 17 – Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)
- Bowles, Joseph E., Engineering Properties of Soils and Their Measurement (4^{th} Edition)

General Discussion

This test process covers procedures for the description of soils for engineering purposes. This practice also describes a procedure for identifying soils, at the option of the user, based on the classification system described in Test Method D 2487. The identification is based on visual examination and manual tests. ***It must be clearly stated in reporting identification that it is based on visual-manual procedures***. These methods are necessarily less precise and more subjective than the methods based on laboratory tests. The accuracy of their results depends on the experience of the operation.

The principal term used by civil engineers to describe soils are **gravel**, **sand**, **silt** and **clay**. Most natural soils consist of a mixture of two or more of these constituents, and many contain an admixture of organic material in a partly or fully decomposed state. The mixture is given the name of the constituent that appears to have the most influence on its behavior, and the other constituents are indicated by adjectives. Thus a silty clay has predominantly the properties of a clay but contains a significant amount of silt and an organic silt is composed primarily of silt-sized mineral matter but contains a significant amount of organic material.

A broad classification of soil is

1. **Coarse-grained** - Gravels and sands,
2. **Fine-grained** - silts and clays.

The distinction between the above two is based on whether the individual particles can be differentiated by the naked eye. The method of describing coarse-grained soils differs from those

appropriate for fine-grained soils. Therefore, the procedures are discussed under separate headings.

Equipment

1. Spatula
2. Wooden hammer
3. Beaker
4. Glass rod / Stirrer
5. 1/8 inch dia. steel rod

Test procedure

Preliminary classification

The soil is first classified as **"coarse grained"** on **"fine grained"** following the procedure mentioned below

1. A representative sample is spread on a flat surface.
2. All particles larger than 75 mm are removed and their proportions of the whole are estimated and recorded. Only the fraction of the sample smaller than 75 mm is classified. A visual estimate is made of the percentage (by volume) of **fine** and **coarse** fraction.[1]
3. If the soil contains more than 50 percent of coarse fraction it is termed as "coarse grained soil"
4. If the soil contains more than 50 percent fine of fraction it is termed as "fine grained soil".

Coarse and fine grained soil are further classified following separate procedures, as follows:

Coarse Grained Soils

Gravel, Sand and Fines

1. The percentages of the following particle fractions are estimated

(i) Gravel fraction (75 mm – 4.75 mm or approximately 5 mm),

(ii) Sand fraction (4.75 or 5 mm – 75 micron), and

(iii) Soil fines, i.e., silt and clay fraction (smaller than 75 micron).

[1]The soil particles larger than 75 micron constitute the "coarse" fraction and those smaller than 75 micron constitute the "fine" fraction. Coarse particles feel harsh and gritty when rubbed between the fingers. Fine particles feel characteristically smooth and soapy and only very slightly sticky. 75-micron particles are about the smallest that can be seen individually by the naked eye.

2. If the gravel fraction is greater than sand fraction, identify the soil as "gravel" (G). The gravel fraction may be further divided into:

(i) Coarse gravel (75 mm-20 mm) and

(ii) Fine gravel (20 mm-4.75 mm).

3. If the gravel fraction is equal to or less than sand fraction, then the soil is identified as "sand" (S). The sand fraction may be further divided into:

(i) Coarse sand (4.75 mm – 2 mm),

(ii) Medium sand (2 mm – 425 micron), and

(iii) Fine sand (425 micron – 75 micron)

4. The soil further is identified as “clean gravel” or "clean sand" if the percentage of fines is estimated to be less than 5 percent. It is identified as "gravel with fines" or "sand with fines" if the percentage of fines is estimated to be more than 15%.

5. The "clean gravels" or "clean sand" are classified as follows:

The soil is identified as ***well graded gravel*** (GW), or as a ***well graded sand*** (SW) if there is good representation of all particle sizes.

The soil is identified as ***poorly graded gravel*** (GP), or as a ***poorly graded sand*** (SP) if it contains predominantly of one size (uniformly graded) or it has a wide range of sizes with some intermediate size(s) obviously missing (gap graded).

6. "Gravel with fines" or "Sand with fines" are classified as follows:

(a) If the other course grained constituent is less than 15% then:

Silty gravel (GM) or silty sand (SM) if the fines have little or no plasticity, or Clayey gravel (GC) or clayey sand (SC), if the fines are of low to medium or high plasticity.

(b) If the other course grain constituent is greater than 15%

Silty gravel Clayey gravel	With sand
Silty sand Clayey sand	With gravel

7. If the percentage of fine is in between 5~15% then the group name shall be a combination from the two columns below:

Well graded gravel	With clay
Poorly graded gravel	With silt
Well graded sand	
Poorly graded sand	

8. The shape of the Coarse-grained particles should be identified as follows:

Table 3-1: Criteria for Describing Angularity of Coarse – grained particles

Description	**Criteria**
Angular	Particles have sharp edges and relatively plane sides with Unpolished surfaces
Sub angular	Particles are similar to angular description but have rounded edges.
Sub rounded	Particles have nearly plane sides but have well-rounded corners and edges.
Rounded	Particles have smoothly curved sides and no edges.

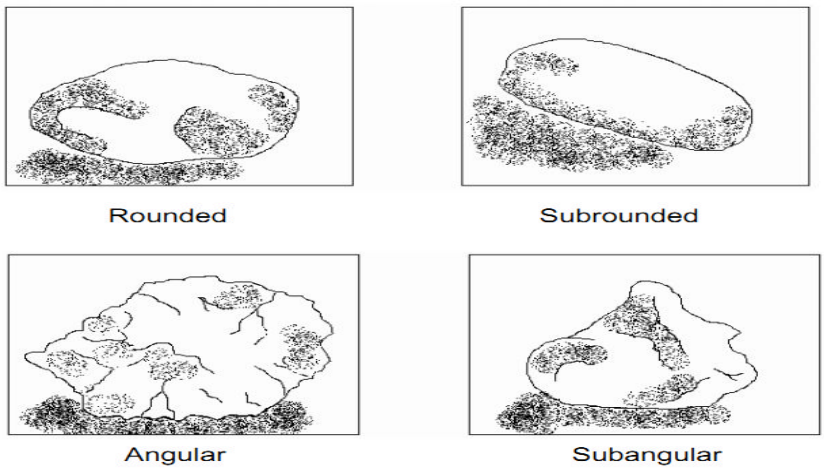


Figure 3 -1: Shapes of coarse-grained soil particles.

Boundary classifications

First, the coarser soil should be assumed, when there is a choice, the classification is to be completed and assigned the appropriate symbol.

Then beginning where the choice was made, the finer soil is assumed, the classification is to be completed and assigned the second group symbol according to Unified Soil Classification System (USCS). The examples are as follows: GW-GP, GM-GC, GW-GM, GW-GC SW-SP, SM-SC, SW-SM, SW-SC, GW-SW, GP-SP, GM-SM, GC-SC, SM-ML, SC-CL, etc.

Identification of Peat.

A sample composed primarily of vegetable tissue in various stages of decomposition that has a fibrous to amorphous texture, usually a dark brown to black color, and an organic odor, shall be designated as a highly organic soil and shall be identified as peat (PT), and not subjected to the identification procedures described.

Fine-grained Soils

Inorganic silt, which constitutes the coarser portion of the microscopic soil fraction, possesses little or no plasticity or cohesion primarily of very fine rounded quartz grains are called rock flour The most plastic varieties containing an appreciable quantity of flake- shaped particles are called plastic silt.

Clay is predominantly an aggregate of microscopic and submicroscopic flake-shaped crystalline minerals. It is characterized by the typical colloidal properties of plasticity, cohesion and ability to absorb ions. These properties are exhibited over a wide range of water content.

The distinction between silt and clay cannot be based on particle size because the significant physical properties of the two materials are related only indirectly to the size of the particles. Furthermore, since both are microscopic, physical properties other than particle size must be used as criteria for field identification. Usually the following four field tests serve the purpose of field identification of a fine-grained soil

(1) Dry strength

(2) Dilatancy or shaking test

(3) Plasticity

(4) Dispersion

Dry Strength Test

a. From the specimen, enough material has to be selected to mold into a ball about 1 inch (25 mm) in diameter. The material is to be molded until it has the consistency of putty, adding water if necessary.

b. From the molded material, at least three test specimens is to be made. ***A test specimen shall be a ball of material about 0.5 inch (12.5 mm) in diameter***. The test specimen should be allowed to dry in air, or sun, or by artificial means, as long as the temperature does not exceed 60^0C.

c. If the test specimen contains natural dry lumps, those are about 0.5 inch (12.5 mm) in diameter may be used in place of the molded balls. ***It should be kept in mind that the process of molding and drying usually produces higher strengths than are found in natural dry lumps of soil***.

d. The strength of the dry balls or lumps is to be tested by crushing between the fingers. The strength should be noted as none, low, medium, high, or very high in accordance with the criteria in Table 3-2. If natural dry lumps are used, the results of any of the lumps that are found to contain particles of coarse sand should not be used.



Figure 3-2: Procedure for different identification tests.

e. The presence of high strength water soluble cementing materials, such as calcium carbonate, may cause exceptionally high dry strengths. The presence of calcium carbonate can be detected from the intensity of the reaction with dilute hydrochloric acid.

Table 3-2: Criteria for Describing Dry Strength.

Description	Criteria
None	The dry specimen crumbles into powder with mere pressure of handling.
Low	The dry specimen crumbles into powder with some finger pressure.
Medium	The dry specimen breaks into pieces or crumbles with considerable finger pressure.
High	The dry specimen cannot be broken with finger pressure. Specimen will break into pieces between thumb and a hard surface.
Very High	The dry specimen cannot be broken between the thumb and a hard surface.

Dilatancy Test

a. From the specimen, enough material should be selected to mold into a ball about 0.5 inch (12.5 mm) in diameter. The material should be molded, adding water if necessary, until it has a soft, but not sticky consistency.

b. The soil ball should be smoothed in the palm of one hand with the blade of a knife or small spatula. It has to be shacked horizontally, striking the side of the hand vigorously against the other hand several times. The reaction of water is to be noted appearing on the surface of the soil. Then the sample has to be squeezed by closing the hand or pinching the soil between the fingers, and noted the reaction as none, slow, or rapid in accordance with the criteria in Table 3-3.

c. The reaction is the speed with which water appears while shaking, and disappear while squeezing.

Table 3-3: Criteria for Describing Dilatancy.

Description	Criteria
None	No visible change in the specimen.
Slow	Water appears slowly on the surface of the specimen during the shaking and does not disappear or disappears slowly upon squeezing.
Rapid	Water appears quickly on the surface of the specimen during shaking and disappears quickly upon squeezing.

Toughness or Plasticity Test

a. Following the completion of dilatancy test, the test specimen is shaped into an elongated pat and rolled by hand on a smooth surface or between the palms into a thread about 1/8 inch (3 mm) in diameter. If the sample is too wet to roll easily, it should be spread into a thin layer and allowed to lose some water by evaporation. The sample should be folded threads and re-rolled repeatedly until the thread crumbles at a diameter about 1/8 inches. The thread will crumble at a diameter of 1/8 inches when the soil is near the plastic limit. The pressure has to be noted required to roll the thread near the plastic limit. Also, the strength of the thread has to be. After the thread crumbles, the pieces should be lumped together and kneaded until the lump crumbles. The toughness of the material has to be noted during kneading.



Figure 3 - 3: Toughness and plasticity Test

b. The toughness of the thread and lump should be described as low, medium, or high in accordance with the criteria in Table 3-4.

c. On the basis of observations made during the toughness test, the plasticity of the material should be described in accordance with the criteria given in Table 3-5.

Table 3-4: Criteria for Describing Toughness of Plastic Thread

Description	Criteria
Low	Only slight pressure is required to roll the thread near the plastic limit. The thread and the lump are weak and soft
Medium	Medium pressure is required to roll the thread to near plastic limit. The thread and lump has medium stiffness.
High	Considerable pressure is required to roll the thread to near the plastic limit. The thread and lump have very high stiffness.

Table 3-5: Criteria for Describing Plasticity.

Description	Criteria
Non-plastic	A 1/8 inches (3 mm) thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be re-rolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time for rolling and kneading to reach the plastic limit. The thread can be re-rolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than plastic limit.

Dispersion Test

a. A rough estimate of the relative amounts of sand, silt and clay in a soil can be made by dispersion test. A small quantity of the soil is dispersed with water in a glass cylinder or beaker and then allowed to settle.

b. The coarser particles fall out first and the finest particles remain in suspension the longest.

c. Usually, sands settle in 30 - 60 sec, silt settles in 15 - 60 min and clay remain in suspension for several hours. Remember, due to flocculation of clay particles, the result may be erroneous.



Figure 3-4: Dispersion Test

Deciding on Inorganic or Organic Fine Grained Soil

One should decide whether the soil is an inorganic or organic fine grained soil following the respective steps.

a. **Identification of Inorganic Fine Grained Soil**

(1) The soil should be identified as ***lean clay***, **CL**, if the soil has medium to high dry strength, no or slow dilatancy, and medium toughness and plasticity. See Table 3-6.

(2) The soil should be identified as ***flat clay***, **CH**, if the soil has high to very high dry strength, no dilatancy, and high toughness and plasticity. See Table 3-6.

(3) The soil should be identified as ***silt***, **ML**, if the soil has no to low dry strength, slow to rapid dilatancy, and low toughness and plasticity, or is non-plastic. See Table 3-6.

(4) The soil should be identified as ***elastic silt***, **MH**, if the soil has low to medium dry strength, no to slow dilatancy, and low to medium toughness and plasticity. See Table 3-6.

(5) These properties are similar to those for lean clay. However, the silt will dry quickly on hand and have a smooth, silky feel when dry. Some soils that would classify as MH in accordance with the criteria in Test Method D 2487 are visually difficult to distinguish from lean clay, CL. It may be necessary to perform laboratory tests for proper identification.

Table 3-6: Identification of Inorganic Fine Grained Soils.

Soil Symbol	Dry Strength	Dilatancy	Toughness
ML	None to low	Slow to rapid	Low or thread cannot be formed
CL	Medium to high	None to slow	Medium
MH	Low to medium	None to slow	Low to medium
CH	High to very high	None	High

b. **Identification of Organic Fine Grained Soil**

(1) The soil should be identified as ***organic soil*, OL/OH**, if the soil contains enough organic particles to influence soil properties. Organic soils usually have a dark brown to black color and may have an organic odor. Often, organic soils will change color, for example, black to brown, when exposed to air. Some organic soils will lighten in color significantly when air dried. Organic soils normally will not have a high toughness or plasticity. The thread for the toughness test will be spongy.

(2) In some cases, through practice and experience, it may be possible to further identify the organic soils as organic silts or organic clays, OL or OH. Correlations between dilatancy, dry strength, toughness tests, and laboratory tests can be made to identify organic soils in certain deposits of similar materials of known geologic origin.

Few Important Notes

a. If the soil is estimated to have 15 - 20% sand or gravel, or both, the words 'with sand' or 'with gravel' (whichever is more predominant) shall be added to the group name. For example, 'lean clay with sand, CL' or 'silt with gravel, ML'. If the percentage of sand is equal to the percentage of gravel, use 'with sand'.

b. If the soil is estimated to have 30% or more sand or gravel or both, the words 'sandy' or 'gravelly' shall be added to the group name. Add the word 'sandy', if there appears to be more sand than gravel. Add the word 'gravelly', if there appears to be more gravel than sand. For example, 'sandy lean clay, CL', 'gravelly fat clay, CH' or 'sandy silt, ML'. If the percentage of sand is equal to the percentage of gravel, use 'sandy' to the group name. For example, 'silty gravel with cobbles, GM'.

c. Finally compare the result with Table 3-7.

Table 3-7: Field Identification of Fine Grained Soils

Typical Name	Dry Strength	Dilatancy Reaction	Toughness of Plastic Thread	Time to Settle in Dispersion Test
Sandy silt	None to very low	Rapid	Low	30 sec to 15 min
Silt	Very low to low	Rapid	Low	15 to 60 min
Clayey silt	Low to medium	Rapid to slow	Medium	15 min to several hours
Sandy clay	Low to high	Slow to none	Medium	30 sec to several hours
Silty clay	Medium to high	Slow to none	Medium	15 min to several hours
Clay	High to very high	None	High	Several hours to days
Organic silt	Low to medium	Slow	Low	15 min to several hours
Organic clay	Medium to very high	None	High	Several hours to days

WORKED OUT EXAMPLE

Test Serial No. **1.**

Name of the Test. **Field Identification Test**

Date Tested: **January 07, 2009**

Tested By:

Project Name: **Airport Road – Rokeya Sharoni Link Road Project**

Sample Number: **Sample-1, Soil Surface Top**

Sample-2, 1m below the Surface

1. **Identification of Peat**.

Sample No	Color	Odor	Remarks
1.	Gray	Nil	Not Organic soil
2..	Reddish Brown	Nil	Not Organic organic

2. **Dry Strength Test**.

Sample No	Test Observation	Description
1.	The dry specimen breaks into pieces or with considerable finger pressure.	Medium
2..	The dry specimen cannot be broken between the thumb and a hard surface.	Very High

3. **Dilatancy Test**

Sample No	Test Observation	Description
1.	Water appears slowly on the surface of the specimen during the shaking and disappears slowly upon squeezing.	Slow
2..	No visible change in the specimen.	None

4. **Toughness Test**

Sample No	Test Observation	Description
1.	Medium pressure is required to roll the thread to near plastic limit. The thread and lump has medium stiffness.	Medium
2..	Considerable pressure is required to roll the thread to near the plastic limit. The thread and lump have very high stiffness.	High

5. **Plasticity Test**

Sample No	Test Observation	Description
1.	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be re-rolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.	Medium
2..	It takes considerable time for rolling and kneading to reach the plastic limit. The thread can be re-rolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.	High

6. **Dispersion Test**

Sample No	Dispersion Time	Remarks
1.	1 hr 50 min	Water gives a clear look.
2..	6 hrs 35 min	Even after this time water remained unclear. It is obvious that it will take more time to settle.

7. **Field Identification Test Result**

Serial No	Observation / Test	Sample 1	Sample 2
1.	Organic/Inorganic	Inorganic	Inorganic
2.	Dry Strength Test	Medium	Very High
3.	Dilatancy Reaction	Slow	None
4.	Toughness Test	Medium	High
5.	Plasticity Test	Medium	High
6.	Dispersion Time	1 hr 50 min	6 hrs 35 min
7.	Typical Name	Clayey silt	Clay
8.	Symbol	MH	CH

FIELD IDENTIFICATION TEST

Group No:

Name of the Student. Student No.

Test Ser. No. Date of test performed:

Project._____

BH No._____ Sample no. _____ Depth _____

Color _____ Odor _____

Name of the test	Observation	Probable soil Type (considering individual test)	Conclusion on soil type considering all the four field tests
Dry strength			
Dilatancy			
Toughness of plastic thread			
Dispersion			

Typical Name:

Soil Symbol:

_____

Sign. of the Course Instructor

SECTION 4

GRAIN SIZE ANALYSIS

References

- ASTM D 7928 - 17 - Standard Test Method for Particle-Size Distribution (Gradation) of Fine-Grained Soil Using the Sedimentation (Hydrometer) Analysis.
- ASTM D 6913 - 17 - Standard Test Method for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis.
- Bowles, Joseph E., Engineering Properties of Soils and Their Measurement (4th Edition).
- Lambe, T.W., Soil Testing for Engineers.
- ASTM D 6026 - 06 – using Significant Digits in Geotechnical Data

General Discussion

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

1. Sieve Analysis

Sieve analysis is performed to determine the gradation (or grain size distribution) of different particle sizes of coarse grained soil (sand) or coarser portion of a soil containing both coarse (sand & silt) and fine (silt & clay) particles. A sieve analysis consists of shaking the soil through a stack of wire screens with openings of known sizes; the definition of particle diameter for a sieve test is therefore the side dimension of a square hole. It should be noted that sieve analysis does not give any idea about the shape of the particles.

Sieves are made of woven wires with square openings. Table - 4 gives a list of U.S. standard sieve numbers with the corresponding size of openings. For all practical purposes #200 sieve is the sieve with the smallest opening that should be used for the test. The sieves that are commonly used in the test have diameter of 8 inch (203mm).

Actually for the purpose of gradation of soils there may be one of the following three tests

1. Sieve analysis – Applicable when the soil contains almost no fines
2. Hydrometer analysis – Applicable when the soil is fine grained
3. Combined analysis i.e. both sieve and hydrometer analysis – Applicable when the soil contains sizable portion of both fine and coarse fraction.

Table 4-1: U.S. Sieves

Sieve no.	Opening (mm)	Sieve no.	Opening (mm)
4	4.75	35	0.500
5	4.00	40	0.425
6	3.35	45	0.355
7	2.80	50	0.300
8	2.36	60	0.250
10	2.00	70	0.212
12	1.70	80	0.180
14	1.40	100	0.150
16	1.18	120	0.125
18	1.00	140	0.106
20	0.85	200	0.075
25	0.71	270	0.053
30	0.60	400	0.038

The particle-size analysis is presented as a semilog plot of percent finer versus particle size. From this plot, information about grain size distribution is obtained. Besides the gradation curve, other parameters namely uniformity coefficient Cu, coefficient of curvature, Cc are determined and all these information are used for classification.

It is pointed here that FM (Fineness Modulus) is determined from sieve analysis done on fine and coarse aggregates. However, FM has almost no meaning and use in geotechnical interpretation and understanding of soil behavior as a foundation material

Equipment

Figure 4-1 shows the equipment used for grain size analysis by sieve. The list of the equipment is as following:

- #4, #8, #16,#30, #50, #100, #200 sieve
- Pan
- Lead
- Brush
- Container
- Spoon
- Bowel
- Balance
- Sieve shaker

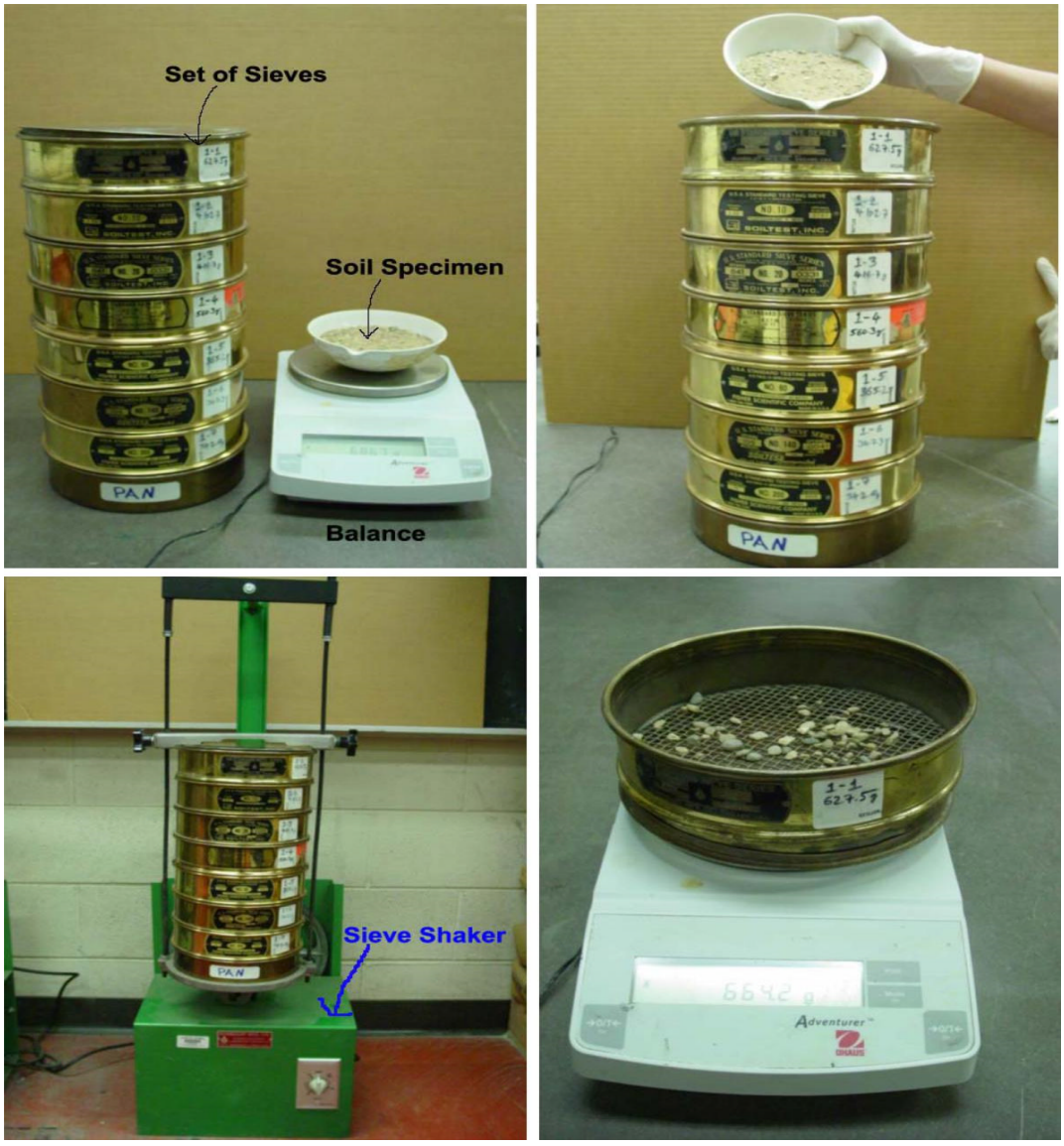


Figure 4-1: Equipment used for sieve analysis.

Test Procedure

1. Sieve Analysis

a. The weight of each sieve as well as the bottom pan to be used in the analysis should be determined and recorded.

b. The weight of the given dry soil sample should be recorded.

c. It should be made sure that all the sieves are clean, and assembled in the ascending order of sieve numbers (#4 sieve at top and #200 sieve at bottom). The pan should be placed below #200 sieve. The soil sample should be poured into the top sieve and the cap should be placed over it.

d. The sieve stack should be placed in the mechanical shaker and shaked for 10 minutes.

e. The stack should be removed from the shaker and carefully weighed and the weight of each sieve with its retained soil should be recorded. In addition, it should be remembered to weigh and record the weight of the bottom pan with its retained fine soil.

2. Wash Sieving

a. For Clay

i. First the oven-dry sample should be crushed with the help of a wooden hammer.

ii. Then 100 or 500 gm (preferably 500 gm; for disturbed sample collected during SPT, it is difficult to get 500 gm sample) sample should be measured and mixed with water in a bowl. The sample in that condition should be kept for an hour.

iii. This sample should be washed through #200 sieve with the help of a controlled water flow.

iv. Special care should be taken to avoid any overflow or other losses.

v. Once the flowing water is clean, sample should be taken to an oven-proof bowl with the help of a wash bottle.

vi. Then it should be put in the oven for drying.

vii. Procedure described in Para 1 should be followed.

b. For Sand.

(1) The 500 gm oven-dry sample should be taken and mixed with water in a bowl

(2) This sample should be washed through #200 sieve with the help of a controlled water flow.

(3) Special care should be taken to avoid any overflow or other losses.

(4) Once the flowing water is clean, sample should be taken to an oven-proof bowl with the help of a wash bottle.

(5) Then it should be put in the oven for drying.

(6) Procedure described in Para 1 should be followed.

Data Analysis

Sieve Analysis.

a. The mass of soil retained on each sieve should be obtained by subtracting the weight of the empty sieve from the mass of the sieve + retained soil, and this mass should be recorded as the weight retained on the data sheet. The sum of these retained masses should be approximately equals the initial mass of the soil sample. A loss of more than two percent is unsatisfactory.

b. The percent retained on each sieve should be calculated by dividing the weight retained on each sieve by the original sample mass.

c. The percent passing (or percent finer) by starting with 100 percent and subtracting the percent retained on each sieve should be calculated as a cumulative procedure.

For example: Total mass = 500 g
Mass retained on No. 4 sieve = 9.7 g
Mass retained on No. 10 sieve = 39.5 g

For the No.4 Sieve:
Quantity passing = Total mass - Mass retained
= 500 - 9.7 = 490.3 g

The percent retained is calculated as;

% retained = Mass retained/Total mass

= (9.7/500) X 100 = 1.9 %

From this, the % passing = 100 - 1.9 = 98.1 %

For the No. 10 Sieve:

Quantity passing = Mass arriving - Mass retained = 490.3 - 39.5 = 450.8 g

% Retained = (39.5/500) X 100 = 7.9 %

% Passing = 100 - 1.9 - 7.9 = 90.2 %

d. A semi-log plot of grain size vs. percent finer has to be made.

e. Using the gradation curve, the percentage of **'clay'**, **'silt'** and **'sand'** sized particles should be determined according to ASTM and MIT textural classification (see Fig,1.13. of "Foundation Engineering- Peck, Hensen and Thornburn")

f. D_{10}, D_{30} and D_{60} should be determined from the graph and calculate the following :

$$C_u = \frac{D_{60}}{D_{10}}$$

$$C_c = \frac{D_{30}^2}{D_{60} X D_{10}}$$

2. Hydrometer Analysis

Grain size distribution of soil which contain significant amount of finer particles (silt and clay) cannot be done by sieve analysis. Hydrometer analysis is required to determine the grain size distribution of the finer portion. For many natural soils we require both sieve analysis and hydrometer analysis to obtain the complete gradation of the coarse and fine fraction. In the sieve analysis test #200 sieve (opening 0.074 mm) is used as the finest sieve. Now hydrometer analysis will be performed on a fine grained soil that passes #200 sieve.[1]

Determination of particle diameter

Determination of particle diameter using hydrometer is based on Stoke's Law which states that the terminal velocity (v) of a freely falling sphere through a medium is proportional to the square of the

[1] According to ASTM D422, when combined analysis is required, the sample is to be divided into two parts. Sieve analysis is to be done on the coarser portion and hydrometer analysis is to be done into finer portion. Division of the sample into two portion is to be done by either of #4 (4.75mm), #10 (2.00 mm), #40 (0.425mm) or #200 (0.074 mm) sieve depending on the sp.gr.of particles. However, for our natural soils separation on #200 sieve will be sufficient.

diameter (D) of the particle i.e. $v \alpha D^2$. From this we obtain

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

Where, v = velocity (cm/s)

γ_s = Unit wt. of soil particles (gm/cc)

γ_w = Unit wt. of water (gm/cc)

η= Viscosity of water ($\frac{gm \,.\, s}{cm^2}$)

D = Diameter of particle (cm)

Therefore, if the velocity of fall v is known, then diameter of the particle can be calculated as

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

To understand the details of hydrometer analysis it is necessary to have an idea about the apparatus "hydrometer". There are two types of ASTM soil hydrometers. One is designated ASTM 151H and the other is designated ASTM 152H.

The readings obtained from these two hydrometers have different meanings. Both can be used for hydrometer analysis. But, calculation procedures for these two are different. Here we have described the procedure for ASTM 152H hydrometer that you will use.

The hydrometer, made of glass, has a stem and a bulb. When it is inserted into a liquid, it floats or submerges in an upright position with its bulb on the downside. The stem of the hydrometer has graduation**.** ASTM 152H hydrometers are calibrated such that when inserted in a soil water suspension the reading on the hydrometer stem indicates grams of soil that are in 1000cc suspension of the density at the center of the bulb. Thus, when the hydrometer reading is zero, it indicates pure water and when the reading is 30 it means that there are 30 gm of soil in 1000cc soil suspension. Usually there are readings from 0 to 60 on a hydrometer stem. It can be easily conceived that as the density of suspension increases, more of the stem will come out of the suspension, and so the hydrometer reading will be increased. If we prepare a soil suspension in a jar and take hydrometer readings at different time intervals from the instant when the suspension is not disturbed by stirring or any other means, then successive hydrometer readings will decrease because particles of soil will start to fall and finally settle at the bottom of the jar resulting in reduced density of the suspension**.** Now, suppose at any time interval t, the distance from the c.g. (center of gravity) of the bulb to the surface of suspension is L. Because the location of c.g. of the hydrometer bulb is related to the sp.gr. of the suspension around it, L may be

considered to be the distance of fall of a particle in time t. Therefore, velocity of the particle will be v=L/t.

$$\text{Therefore, } D = \sqrt{\frac{18\eta}{\gamma_s - \gamma_w}} \cdot \sqrt{\frac{L}{t}}$$

If we want to put t in minute, L in cm, η in poise (1 gm-sec per square cm =g poise =980.7 poise) and want to obtain D in mm then the above equation should be modified as

$$D\,(mm) = \sqrt{\frac{18\eta}{980.7 * (\gamma_s - \gamma_w)}} \cdot \sqrt{\frac{L}{t * 60}} * 10$$

which upon simplification becomes

$$D\,(mm) = \sqrt{\frac{30\eta(poise)}{980.7 * [\gamma_s(\frac{gm}{cm^3}) - \gamma_w(\frac{gm}{cm^3})]}} \cdot \sqrt{\frac{L(cm)}{t(min)}}$$

$$\text{or } D\,(mm) = \sqrt{\frac{30\eta(poise)}{980.7 * (G_s - 1)\gamma_w(\frac{gm}{cm^3})}} \cdot \sqrt{\frac{L(cm)}{t(min)}}$$

The above equation is sometimes written as

$$D = K.\sqrt{\frac{L}{t}}$$

$$\text{Where, } K = \int(T, G_s, \eta)$$

For example for T=20^{0}C, γ_w = 0.9971, G_s =2.65,

However, when we insert the hydrometer in a soil-water suspension we do not measure L directly. Instead, if we measure the distance L_1 and L_2 from the c.g. of the bulb corresponding to two hydrometer readings R_1 and R_2 then we can calculate L easily for any hydrometer reading R from the equation (see fig.4-2)

$$L = \frac{L_b}{2} + L_1 - \frac{L_1 - L_2}{R_2 - R_1}.(R - R_1) \quad \text{.....} \qquad 1$$

Here we need to notice that as we insert the hydrometer in the jar containing the soil-water suspension, the water level in the jar rises by $\frac{V_b}{A_j}$ resulting in rise of the c.g. of the hydrometer bulb by half of this i.e. $\frac{V_b}{2.A_j}$ from the level where it would be if this rise have not occurred (see fig.); V_b=Volume of the hydrometer bulb and A_j = Cross-sectional Area of the hydrometer jar. Thus equation 1 need to be modified as

$$L = \frac{Lb}{2} + L1 - \frac{L1 - L2}{R2 - R1}.(RL - R1) - \frac{Vb}{2Aj} \qquad 2$$

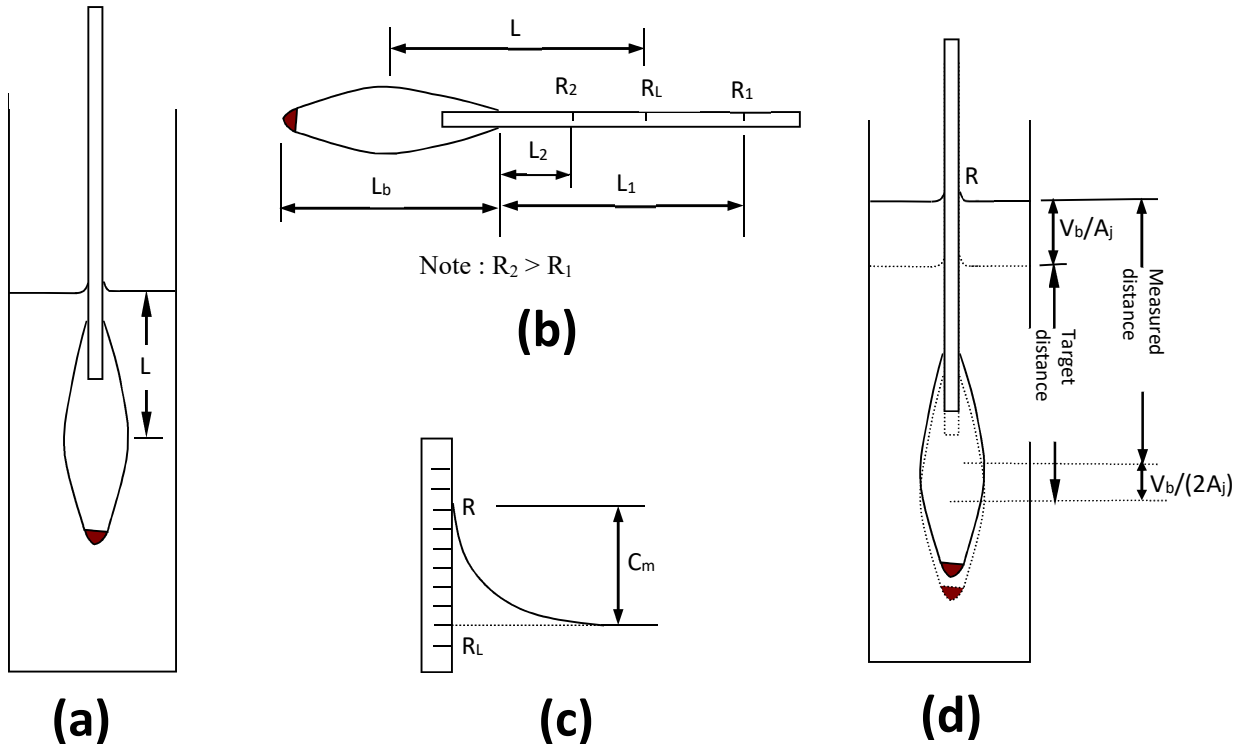


Figure 4-2: Schematic diagrams showing hydrometer readings and corrections.

Furthermore, in the soil-water suspension we can read the upper meniscus, whereas we need the reading at the lower meniscus. So, if R_o be the observed reading, then to put into equation 2, R_L should be

calculated as (see fig.4-2)

$$R_L = R_o - C_m \qquad 3$$

For ASTM 152H hydrometers for R_1=0 the distance L_1=10.5 cm and for R_2=50, the distance L_2 =2.3 cm, L_b=14 cm,V_b=67 cm^3 . Coss-sectional area of hydrometer jar is 27.8 cm^2. For these values, Equation 2 becomes

$$L = 16.29 - 0.164 * R_L \text{ , (L in cm)} \qquad 2a$$

Equation 2a is sometimes presented in tabular form as in Table – 4-2 wherefrom values of L can be obtained for any hydrometer reading.

Table 4-2: Values of Efective depth R_L for hydrometer for which (L_1=10.5 cm, L_2=2.3 cm, R_1=-, R_2=50, L_b=14.0 cm, V_b= 67 cm^3, A_j=27.8 cm^3)

Hydrometer reading, R_L	L	Hydrometer reading, R_L	L	Hydrometer reading, R_L	L	Hydrometer reading, R_L	L
0	16.3	15	13.8	30	11.4	45	8.9
1	16.1	16	13.7	31	11.2	46	8.8
2	16.0	17	13.5	32	11.0	47	8.6
3	15.8	18	13.3	33	10.9	48	8.4
4	15.6	19	13.2	34	10.7	49	8.3
5	15.5	20	13.0	35	10.6	50	8.1
6	15.3	21	12.9	36	10.4	51	7.9
7	15.1	22	12.7	37	10.2	52	7.8
8	15.0	23	12.5	38	10.1	53	7.6
9	14.8	24	12.4	39	9.9	54	7.4
10	14.7	25	12.2	40	9.7	55	7.3
11	14.5	26	12.0	41	9.6	56	7.1
12	14.3	27	11.9	42	9.4	57	6.9
13	14.2	28	11.7	43	9.2	58	6.8
14	14.0	29	11.5	44	9.1	59	6.6

Determination of percent finer

It has been mentioned earlier that ASTM 152H hydrometers are calibrated such that for soil of sp.gr of 2.65, the hydrometer reading after an elapsed time t, indicates the grams of soil in suspension at that instant. This means that if R_c be the hydrometer reading, then (corresponding to the diameter determined using this reading)

$$\text{Percent finer, } N = \frac{R_c}{W_s} * 100 \qquad 4$$

Where, W_s=Weight (in gms) of soil used to prepare the suspension and R is obtained from R_0 after applying some corrections.

Because the sp.gr.of a soil may vary from 2.65 a correction factor is introduced in equation 4. Thus,

$$\text{Percent finer, } N = a * \frac{R_c}{W_s} * 100 \qquad 5$$

Where 'a' is the correction factor introduced to accommodate the variation of sp.gr.from 2.65. The correction factor a can be computed as

$$a = \frac{G_s * 1.65}{(G_s - 1) * 2.65} \qquad 6$$

Values of 'a' for different sp.gr. of soil particles may also be obtained from Table 4-5.

The reading R_c to be used in equation 5 should be obtained from the observed hydrometer reading R_o after applying the following corrections

(a) correction for meniscus (C_m). This correction is always additive.

(b) zero correction (C_z). This correction is introduced because a deffloculating agent is used in the preparation of the soil water suspension, which increases the hydrometer reading. This correction is always subtractive.

(c) Temperature correction (C_T). ASTM 152H hydrometers are calibrated at 20^0C. So if the test temperature is different from soil water suspension a correction is needed. The values for temperature correction are given in Table 4-6.

$$R_c = R_o + C_m - C_z \pm C_T \qquad 7$$

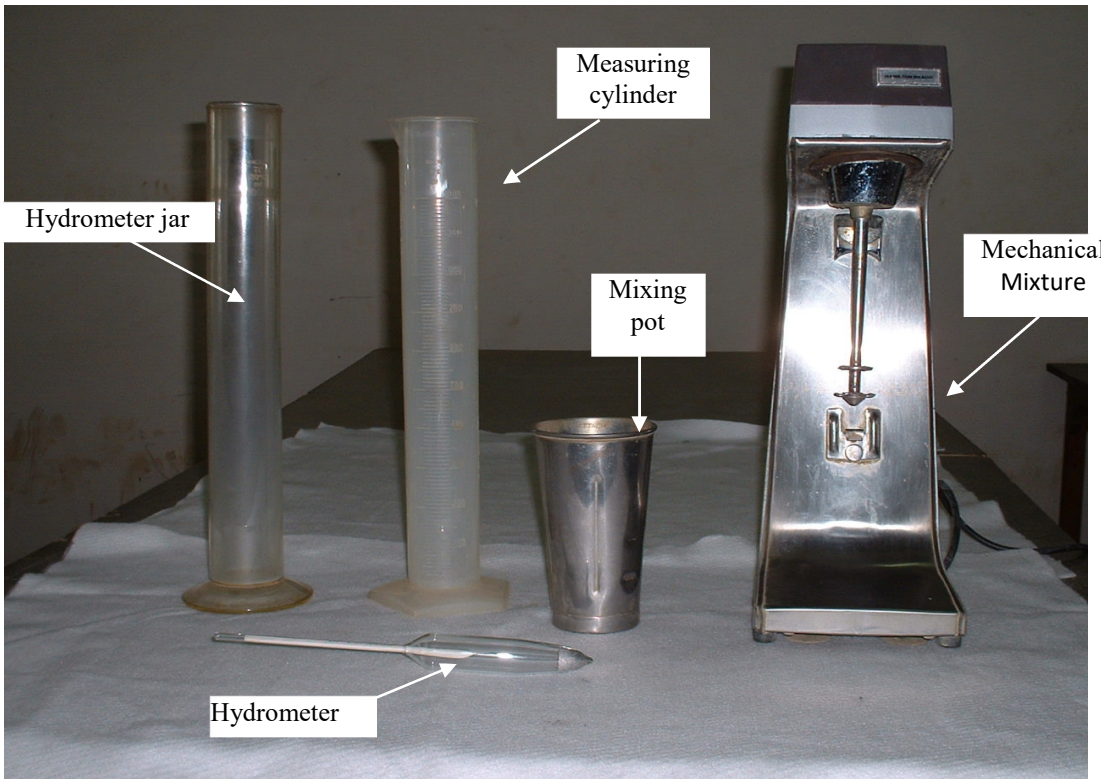
Now consider that hydrometer analysis is performed on a soil for which percent of material finer for #200 sieve is P. Then with respect to the original soil

Percent finer, $N' = N * \frac{P}{100}$

Equipment

Figure 4-3 shows the equipment used for grain size analysis by hydrometer. The list of the equipment is as following

- Hydrometer
- Hydrometer jar
- Measuring cylinder / graduated jar
- Can
- Wash bottle
- Stop watch
- Mechanical mixture & mixing pot
- Balance
- Thermometer
- Water bath



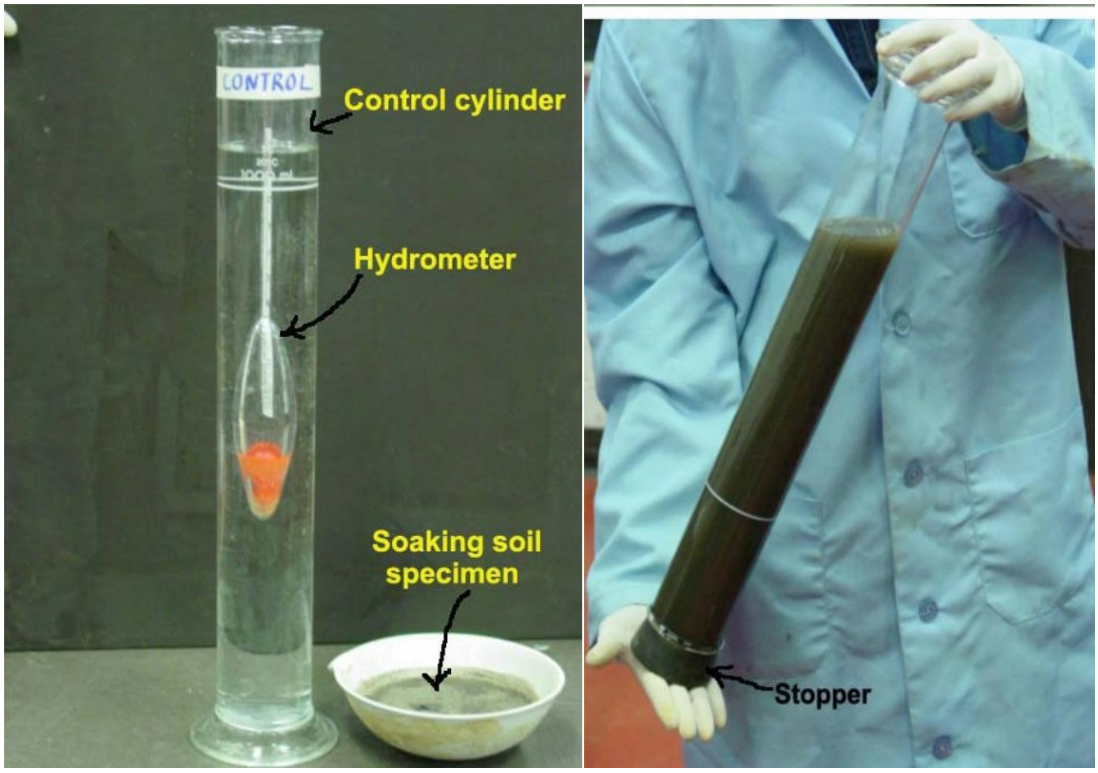


Figure 4-3: Equipment used for hydrometer analysis.

Test Procedure:

a. 50 gm of fine soil from the bottom pan of the sieve set should be taken and should be placed it into a beaker, and added 125 mL of the dispersing agent (sodium hexametaphosphate, 40 gm/liter)) solution. The mixture should be stirred until the soil is thoroughly wet. The soil should be kept soak for at least ten minutes.

b. While the soil is soaking, 125 mL of dispersing agent should be added into the control cylinder and it should be filled with distilled water to the mark. The reading should be taken at top of the meniscus formed by the hydrometer stem and the control solution. A reading less than zero is recorded as a negative (-) correction and a reading between zero and sixty is recorded as a positive (+) correction. This reading is called the zero correction. The meniscus correction is the difference between the top of the meniscus and the level of the solution in the control jar (Usually about +1). The control cylinder has to be shaken in such a way that the contents are mixed thoroughly. The hydrometer and thermometer should be inserted into the control cylinder and the zero correction and temperature should be noted respectively.

b. The soil slurry should be transferred into a mixer by adding more distilled water, if necessary, until mixing cup is at least half full. Then the solution has to be mixed for a period of two minutes.

c. Immediately the soil slurry should be transferred into the empty sedimentation cylinder. Distilled water has to added up to the mark.

d. The open end of the cylinder should be covered up with a stopper and it should be secured with the palm of hand*. Then the cylinder should be turned upside down and back upright for a period of one minute. (The cylinder should be inverted approximately 30 times during the minute).

f. Immediately after placing the hydrometer jar on the table, the hydrometer has to be inserted in the soil water suspension, the timer should be started and readings should be taken at elapsed times of 0.25, 0.5, 1, and 2 minute.

Note - When placing the hydrometer in the suspension, unnecessary agitation should be avoided by proceeding slowly, so that about 10 s is required for the operation. Several trial operations is helpful in placing the hydrometer.

* A rubber stopper, if available, may be used instead of the palm.

g. After 2 minutes, the hydrometer should be removed from the soil water suspension and placed in the water jar.

h. The suspension in the hydrometer jar should be remixed as in step e and should be repeated steps f. If the later set of four readings (at elapsed time of 0.25, 0.5, 1 and 2 minutes) are almost same as the previous set then these readings should be recorded in continue with step i otherwise steps e to h should be repeated.[2]

i. The reading is taken by observing the top of the meniscus formed by the suspension and the hydrometer stem. The hydrometer is removed slowly and placed back into the control cylinder. Very gently it has to be spinned in control cylinder to remove any particles that may have adhered.

j. Hydrometer readings should be taken after elapsed time of ¼, ½, ¾, 1, 2 , 4, , 8, 15, 30, 60 minutes and 2, 4, 8, 24, 48 and 96 hours.

k. With the hydrometer placed in the water jar, upper and lower meniscus readings should be taken to find meniscus correction C_m.[3]

Table 4-3: Specific Gravity of water at different temperature (from T.W.Lambe, Soil testing for Engineers)

0^0C	0	1	2	3	4	5	6	7	8	9
	0.9999	0.9999	1.0000	1.0000	1.0000	1.0000	1.0000	0.9999	0.9999	0.9998
10	0.9997	0.9996	0.9995	0.9994	0.9993	0.9991	0.9990	0.9988	0.9986	0.9984
20	0.9982	0.9980	0.9978	0.9976	0.9973	0.9971	0.9968	0.9965	0.9963	0.9960
30	0.9957	0.9954	0.9951	0.9947	0.9944	0.9941	0.9937	0.9934	0.9930	0.9926
40	0.9922	0.9919	0.9915	0.9911	0.9907	0.9902	0.9898	0.9894	0.9890	0.9885
50	0.9881	0.9876	0.9872	0.9867	0.9862	0.9857	0.9852	0.9848	0.9842	0.9838
60	0.9832	0.9827	0.9822	0.9817	0.9811	0.9806	0.9800	0.9795	0.9789	0.9784
70	0.9778	0.9772	0.9767	0.9761	0.9755	0.9749	0.9743	0.9737	0.9731	0.9724
80	0.9718	0.9712	0.9706	0.9699	0.9693	0.9686	0.9680	0.9673	0.9667	0.9660
90	0.9653	0.9647	0.9640	0.9633	0.9626	0.9619	0.9612	0.9605	0.9598	0.9591

[2] *It is usual to leave the hydrometer in the soil-water suspension for the first two readings, and then remove and reinsert for each of the other two readings. However, little error is introduced if it is left in the suspension for all four readings*

[3]*Usually, Cm=0.5. Take zero correction Cz to be 3.5. To obtain zero correction, the same amount of deflocculating solution as used in the test (125 ml of 4% NaPO3) is mixed with water and placed in a 1000 ml cylindrical jar and hydrometer reading taken. The difference of this reading and hydrometer reading in pure water gives the zero correction Cz*

Table 4-4: Viscosity of water in millipoise at different temperature (from T.W.Lambe, Soil testing for Engineers)

Temp	0	1	2	3	4	5	6	7	8	9
0	17.94	17.32	16.74	16.19	15.68	15.19	14.73	14.29	13.87	13.48
10	13.10	12.74	12.39	12.06	11.75	11.45	1.16	10.88	10.6	10.34
20	10.09	9.84	9.61	9.38	9.16	8.95	8.75	8.55	8.36	8.18
30	8.00	7.83	7.67	7.51	7.36	7.31	7.06	6.92	6.79	6.66
40	6.54	6.42	6.30	6.18	6.08	5.97	5.87	5.77	5.68	5.58
50	5.29	6.40	5.32	5.24	5.15	5.07	4.99	4.92	4.84	4.77
60	4.70	4.63	4.56	4.50	4.43	4.37	4.31	4.24	4.19	4.13
70	4.07	4.02	3.96	3.91	3.86	3.81	3.76	3.71	3.66	3.62
80	3.57	3.53	3.48	3.44	3.40	3.36	3.32	3.28	3.24	3.20
90	3.17	3.13	3.10	3.06	3.03	2.99	2.96	2.93	2.90	2.87
100	2.84	2.82	2.79	2.76	2.73	2.70	2.67	2.64	2.62	2.59

Table 4-5: Correction factor for Different Specific Gravity of Soil Particles

Specific gravity	Correction factor
2.95	0.94
2.90	0.95
2.85	0.96
2.80	0.97
2.75	0.98
2.70	0.99
2.65	1.00
2.60	1.01
2.55	1.02
2.50	1.03
2.45	1.04

Table 4-6: Temperature correction C_T (From Bowels)

Temperature (deg. Celsius)	C_T
15	-1.10
16	-0.90
17	-0.70
18	-0.50
19	-0.30
20	0.00
21	+0.20
22	+0.40
23	+0.70
24	+1.00
25	+1.30
26	+1.65
27	+2.00
28	+2.50

Sample Calculation

Hydrometer Analysis

The following calculations are for the first row of the sample data sheet provided. Columns 1 through 5 are filled out during test. Calculation of other columns are as below:

Column 6 : $R_L = R_0 - C_m = 35 + 0.5 = 35.5$

Column 7 : It becomes convenient to calculate values in column 7 if we reduce Equation 2 to a simplified form

$$L = \frac{Lb}{2} + L1 - \frac{L1 - L2}{R2 - R1}.(R_L - R1) - \frac{Vb}{2A_j}$$

$$L = \frac{14}{2} + 10.6 - \frac{10.6 - 0}{60 - 0}.(R_L - 0) - \frac{58}{2 * 27.8}$$

$= 16.56 - 0.18*R_L$ (where L will be in cm)

Now, for $R_L = 34.5$, $L = 16.56-0.18*34.5 = 10.2$ cm

Column 8 : From Table – 4-3, for $T=28.25^0$ (column 5) $\gamma_w=0.9962$ gm/cc

Column 9 : From Table- 4-4, for $T=28.25^0$ (column 5) η==8.32 millipoise =0.00832 poise

Column 10 : Grain diameter $D\,(mm) = \sqrt{\frac{30\eta(poise)}{980.7 * (G_s - 1)\gamma_w(\frac{gm}{cm^3})}}.\sqrt{\frac{L(cm)}{t(min)}}$

$$= \sqrt{\frac{30 * 0.00832}{980.7 * (2.74 - 1) * 0.9962}}.\sqrt{\frac{10.2}{0.25}}$$

$$= \sqrt{\frac{30 * 0.00832}{980.7 * (2.74 - 1) * 0.9962}}.\sqrt{\frac{10.2}{0.25}}$$

=0.077 mm

Column 11 : From Table – 4-6, for $T=28.25^0$, $C_T = +2.6$

Column 12 : $R_c = R_o + C_m - C_z \pm C_T = 35 + 0.5 - 3.5 + 2.6 = 34.6$

Column 13 : Before calculating values in column 12 we need to find the correction factor for specific gravity, $a = \frac{G_s(1.65)}{(G_s - 1)2.65} == \frac{1.74 * 1.65}{(2.74 - 1) * 2.65} = 0.98$

Percent finer, $N = a * \frac{R_c}{W_s} * 100 = 0.98 * \frac{34.6}{50} * 100 = 67.85$

Column 14 : Percent finer (Combined) $= N' = N * \frac{P}{100} = 67.85 * \frac{21.5}{100} = 14.59$

Significant Figures

1. The Effective Particle Diameter (in mm) should be recorded in 2 or 3 significant digits.
2. The Percent Passing (Gradation) should be recorded in nearest 0.1 or whole number.

Table 4-7: Values of Effective Depth Based on Hydrometer and Sedimentation Cylinder of Specific Sizes.

Hydrometer 151H		Hydrometer 152H			
Actual Hydrometer Reading	Effective Depth, L (cm)	Actual Hydrometer Reading	Effective Depth, L (cm)	Actual Hydrometer Reading	Effective Depth, L (cm)
1.000	16.3	0	16.3	31	11.2
1.001	16.0	1	16.1	32	11.1
1.002	15.8	2	16.0	33	10.9
1.003	15.5	3	15.8	34	10.7
1.004	15.2	4	15.6	35	10.6
1.005	15.0	5	15.5	36	10.4
1.006	14.7	6	15.3	37	10.2
1.007	14.4	7	15.2	38	10.1
1.008	14.2	8	15.0	39	9.9
1.009	13.9	9	14.8	40	9.7
1.010	13.7	10	14.7	41	9.6
1.011	13.4	11	14.5	42	9.4
1.012	13.1	12	14.3	43	9.2
1.013	12.9	13	14.2	44	9.1
1.014	12.6	14	14.0	45	8.9
1.015	12.3	15	13.8	46	8.8
1.016	12.1	16	13.7	47	8.6
1.017	11.8	17	13.5	48	8.4
1.018	11.5	18	13.3	49	8.3
1.019	11.3	19	13.2	50	8.1
1.020	11.0	20	13.0	51	7.9
1.021	10.7	21	12.9	52	7.8
1.022	10.5	22	12.7	53	7.6
1.023	10.2	23	12.5	54	7.4
1.024	10.0	24	12.4	55	7.3
1.025	9.7	25	12.2	56	7.1
1.026	9.4	26	12.0	57	7.0
1.027	9.2	27	11.9	58	6.8
1.028	8.9	28	11.7	59	6.6
1.029	8.6	29	11.5	60	6.5
1.030	8.4	30	11.4		
1.031	8.1				
1.032	7.8				
1.033	7.6				
1.034	7.3				
1.035	7.0				
1.036	6.8				

Table 4-8: Values of **k** for Use in Equation for Computing Diameter of Particle in Hydrometer Analysis

Temperature o*C*	Specific Gravity of Soil Particles								
	2.45	2.50	2.55	2.60	2.65	2.70	2.75	2.80	2.85
16	0.01510	0.01505	0.01481	0.01457	0.01435	0.01414	0.0394	0.01374	0.01356
17	0.01511	0.01486	0.01462	0.01439	0.01417	0.01396	0.01376	0.01356	0.01338
18	0.01492	0.01467	0.01443	0.01421	0.01399	0.01378	0.01359	0.01339	0.01321
19	0.01474	0.01449	0.01425	0.01403	0.01382	0.01361	0.01342	0.01323	0.01305
20	0.01456	0.01431	0.01408	0.01386	0.01365	0.01344	0.01325	0.01307	0.01289
21	0.01438	0.01414	0.01391	0.01369	0.01348	0.01328	0.01309	0.01291	0.01273
22	0.01421	0.01397	0.01374	0.01353	0.01332	0.01312	0.01294	0.01276	0.01258
23	0.01404	0.01381	0.01358	0.01337	0.01317	0.01297	0.01279	0.01261	0.01243
24	0.01388	0.01365	0.01342	0.01321	0.01301	0.01282	0.01264	0.01246	0.01229
25	0.01372	0.01349	0.01327	0.01306	0.01286	0.01267	0.01249	0.01232	0.01215
26	0.01357	0.01334	0.01312	0.01291	0.01272	0.01253	0.01235	0.01218	0.01201
27	0.01342	0.01319	0.01297	0.01277	0.01258	0.01239	0.01221	0.01204	0.01188
28	0.01327	0.01304	0.01283	0.01264	0.01244	0.01255	0.01208	0.01191	0.01175
29	0.01312	0.01290	0.01269	0.01269	0.01230	0.01212	0.01195	0.01178	0.01162
30	0.01298	0.01276	0.01256	0.01236	0.01217	0.01199	0.01182	0.01165	0.01149

Table 4-9: Temperature Correction Factors C_T

Temperature ^oC	Factor C_T
15	1.10
16	-0.90
17	-0.70
18	-0.50
19	-0.30
20	0.00
21	+0.20
22	+0.40
23	+0.70
24	+1.00
25	+1.30
26	+1.65
27	+2.00
28	+2.50
29	+3.05
30	+3.80

Table 4-10: Correction Factors ***a*** for Unit Weight of Solids

Unit Weight of Soil Solids, g/cm^3	Correction factor a
2.85	0.96
2.80	0.97
2.75	0.98
2.70	0.99
2.65	1.00
2.60	1.01
2.55	1.02
2.50	1.04

WORKED OUT EXAMPLE

Grain Size Analysis (Wash Sieving)

Test Serial No. : **04**
Date Tested: **January 25, 2009**
Tested By:

Project Name: **Hatirjheel Project**

Sample Number: **S-1, Embankment Top**.

Weight of Container: **58.52 gm**
Wt. Container+Dry Soil: **558.52 gm**
Wt. of Dry Sample: **500 gm**

Sieve No	Sieve Opening in mm	Wt of the Sieve (gm)	Wt of Sieve + Soil (gm)	Wt of Soil Retained (gm)	% of Soil Retained	Cumulative % Retained	% Finer
# 4	4.76	413.7	413.7	0	0	0	100
# 8	2.38	355.2	355.2	0	0	0	100
# 16	1.19	384.6	384.6	0	0	0	100
# 20	0.84	368	368.3	0.3	0.06	0.06	99.94
# 30	0.59	362.4	365.4	3	0.6	0.66	99.34
# 50	0.297	365	575.5	210.5	42.1	42.76	57.24
# 100	0.149	366.7	591.1	224.4	44.88	87.64	12.36
# 200	0.074	344.6	367.3	22.7	4.54	92.18	7.82
Pan		426.8	465.9	39.1	7.82	100	
				500			

Sample Calculations

For Sieve #50,

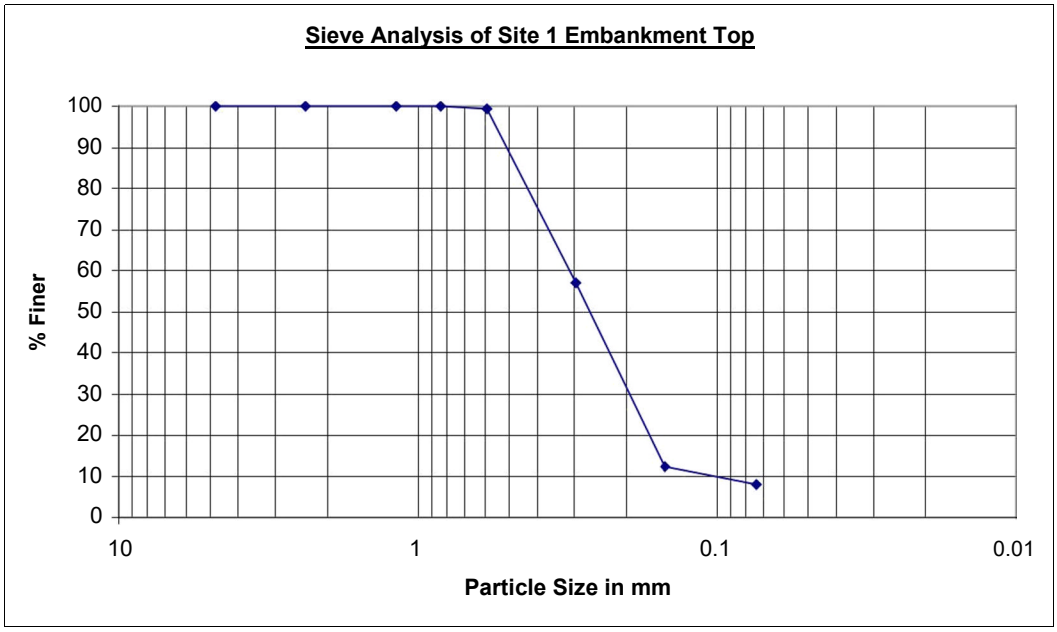
Wt of Soil Retained = {Wt of Soil + Sieve} – Wt of Sieve
= 575.5 – 365 = 210.5 gm

% of Soil Retained = (210.5/500) * 100 = 42. 1

Visual Classification of Soil: **Fine Sand to silty sand**.

Cumulative % Retained = (42. 1 + 0.66) = 42.76

% Finer = (100 – 42. 76) = 57. 24



Quiz

- Find out coefficient of uniformity (C_u) and coefficient of gradation (C_c) from the above graph.

- What should be the fineness modulus (FM) of this soil?

WORKED OUT EXAMPLE

Grain Size Analysis (Hydrometer Method)

Date Tested: **February 02, 2008**
Tested By:
Project Name: **Airport Road**
Sample Number: **S-1, Depth 2m**
Visual Classification of Soil: **Gray Clay**.

Hydrometer Number (if known): **152H**
Specific Gravity of Solids: **2.7**
Dispersing Agent: **4% Solution of Na HexametaPhosphate**
Weight of Soil Sample, W_S: **50 gm**
Zero Correction: **+6** Meniscus Correction: **+1 cm**

Date	Time	Elapsed time t (min)	Room Temp ^{0} C	Actual Hydrometer Reading, R_a	Reading after Meniscus Correction $R=R_a-C_m$	Effective Depth L cm (from Table 4-1)	Value of K (from Table 4-2)	D in mm $= K\sqrt{\frac{L}{t}}$	C_T (from Table 4-3)	a (from Table 4-4)	Corrected Hydrometer Reading R_c	Percent Finer = R_c*a*100/Ws
19/02/08	**9:11**	**0.25**	**22**	**52.5**	**53.5**	**7.5**	**0.01312**	**0.071861**	**+0.40**	**0.99**	**46.9**	**92.862**
19/02/08	**9:11**	**0.5**	**22**	**51.5**	**52.5**	**7.7**	**0.01312**	**0.051487**	**+0.40**	**0.99**	**45.9**	**90.882**
19/02/08	**9:11**	**0.75**	**22**	**51**	**52**	**7.8**	**0.01312**	**0.042311**	**+0.40**	**0.99**	**45.4**	**89.892**
19/02/08	**9:12**	**1**	**22**	**50**	**51**	**7.9**	**0.01312**	**0.036876**	**+0.40**	**0.99**	**44.4**	**87.912**
19/02/08	**9:13**	**2**	**22**	**47.5**	**48.5**	**8.35**	**0.01312**	**0.026808**	**+0.40**	**0.99**	**41.9**	**82.962**
19/02/08	**9:15**	**4**	**22**	**44.5**	**45.5**	**8.85**	**0.01312**	**0.019515**	**+0.40**	**0.99**	**38.9**	**77.022**
19/02/08	**9:19**	**8**	**22**	**40**	**41**	**9.6**	**0.01312**	**0.014372**	**+0.40**	**0.99**	**34.4**	**68.112**
19/02/08	**9:26**	**15**	**22**	**37**	**38**	**10.1**	**0.01312**	**0.010766**	**+0.40**	**0.99**	**31.4**	**62.172**
19/02/08	**9:41**	**30**	**22**	**33.5**	**34.5**	**10.65**	**0.01312**	**0.007817**	**+0.40**	**0.99**	**27.9**	**55.242**
19/02/08	**10:11**	**60**	**22**	**31**	**32**	**11.1**	**0.01312**	**0.005643**	**+0.40**	**0.99**	**25.4**	**50.292**
19/02/08	**11:11**	**120**	**22**	**28.5**	**29.5**	**11.45**	**0.01312**	**0.004053**	**+0.40**	**0.99**	**22.9**	**45.342**
19/02/08	**13:11**	**240**	**22**	**26.5**	**27.5**	**11.8**	**0.01312**	**0.002909**	**+0.40**	**0.99**	**20.9**	**41.382**
19/02/08	**17:11**	**480**	**22**	**25**	**26**	**12**	**0.01312**	**0.002074**	**+0.40**	**0.99**	**19.4**	**38.412**
20/02/08	**9:11**	**1440**	**22**	**21**	**22**	**12.7**	**0.01312**	**0.001232**	**+0.40**	**0.99**	**15.4**	**30.492**
21/02/08	**9:36**	**2910**	**21.5**	**18**	**19**	**13.2**	**0.0132**	**0.000889**	**+0.30**	**0.99**	**12.3**	**24.354**
22/02/08	**9:10**	**4319**	**21**	**17**	**18**	**13.3**	**0.01328**	**0.000737**	**+0.20**	**0.99**	**11.2**	**22.176**
24/02/08	**15:32**	**7581**	**22**	**16**	**17**	**13.5**	**0.01312**	**0.000554**	**+0.40**	**0.99**	**10.4**	**20.592**

Sample Calculations (For 1st Reading)

Actual hydrometer reading, R_a = 52.5

Reading after Meniscus Correction R = R_a - C_m = 52.5 - (-1) = 53.5

Effective Depth, L (from Table 4-1) = 7.5 cm (using interpolation)

Value of K (from Table 4-2) = 0.01332 (for room temperature 22°C and soil specific gravity 2.65)

D in mm $= K\sqrt{\frac{L}{t}} = 0.01334\sqrt{\frac{7.5}{0.25}} =$ **0.071861**

Temperature correction, C_T (from Table 4-3) = +0.40

Correction factor for unit weight of soil, a (from Table 4-4) = 0.99

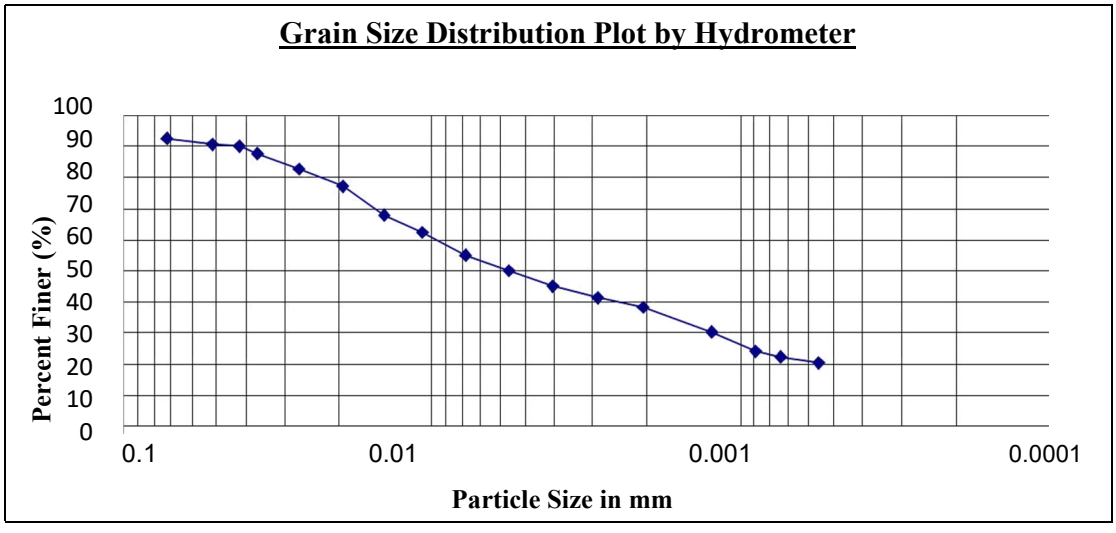
Corrected Hydrometer Reading, R_c = R_a - zero correction + C_T

= {52.5 – (+6) + 0.4}

= 46.9

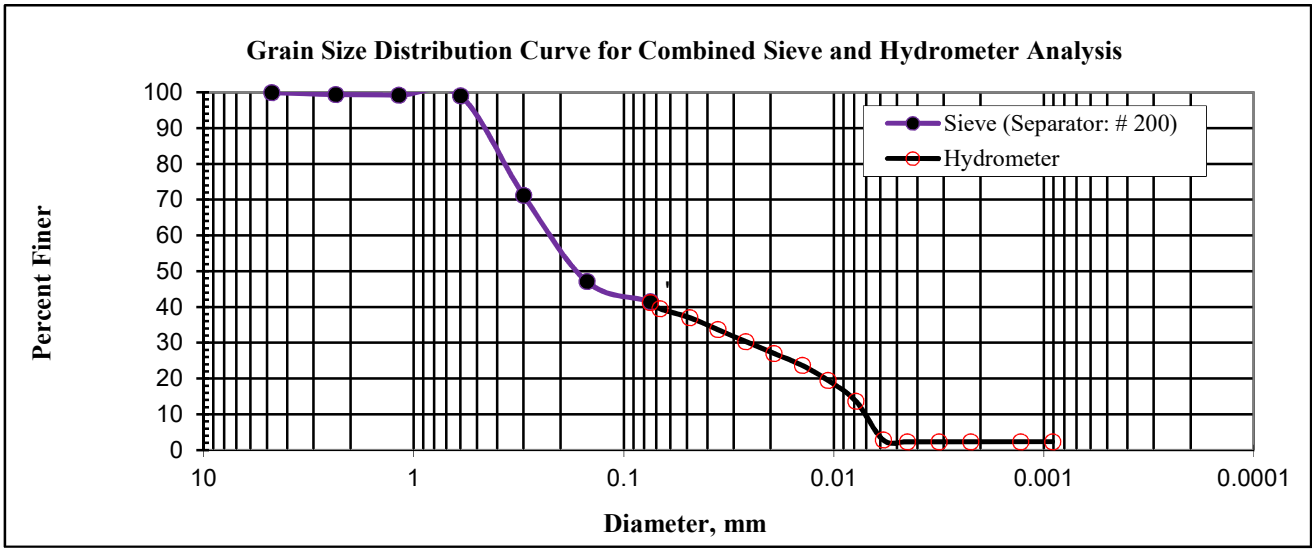
Percent Finer = (R_c x a x 100) / Ws = (46.9 x 0.99 x 100) / 50

= 92.862



Quiz

- Find out coefficient of uniformity (C_u) and coefficient of gradation (C_c) from the above graph



Report

1. For calculation use the G_s you obtained in your specific gravity test, if there was no gross error.
2. Calculate % finer (combined) assuming that 17% of the original soil pass #200 sieve.
3. Plot the gradation curve [% finer versus log(diameter)]. Plot both the curves as shown in Fig.

Note:

a. Students should not be confused seeing the trend line of both the plots shown in the worked out example. The trend line should always be the 'best fitting curve'.

b. To get the complete grain size distribution of any soil sample, especially for clay, it is required to do the sieve and hydrometer analysis both. Then the both curves are merged together to get the complete picture.

SECTION 5

SPECIFIC GRAVITY TEST

References

- ASTM D 854-14 – Standard Test for Specific Gravity of Soil Solids by Water Pycnometer.
- Bowles, Joseph E., Engineering Properties of Soils and Their Measurement (4th Edition)
- Lambe, T.W., Soil Testing for Engineers.
- ASTM D 6026 - 06 – using Significant Digits in Geotechnical Data

General Discussion

This test is performed to determine the specific gravity of soil by using a pycnometer. Specific gravity is the ratio of the mass of unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature.

In general the specific gravity G_S of a material represents the ratio of the mass of a given volume of that material at a temperature to the mass of an equal volume of distilled water at the same temperature.

$$G_s(\text{at } T^0C) = \frac{\text{Wt. of a given volume of the material at } T^0C}{\text{Wt. of the same volume of water at } T^0C}$$

However, for geotechnical Engineering purpose, the Sp. Gr. of soils are usually reported for water temperature at 20^0C.

$$\text{Thus, } G_s\,(\text{at } 20^0C) = \frac{\text{Wt. of a given volume of the material at } T^0C}{\text{Wt. of the same volume of water at } 20^0C}$$

From this basic definition, sp.gr. can be obtained from different formulae such as :

$$G_s\,(\text{at } 20^0C) = \frac{\text{Wt. of unit volume of soil at } T^0C}{\text{Wt. of unit volume of water at } 20^0C}$$

If the specific gravity of soil for water temperature T^0C is known then it can be converted to the specific gravity for water temperature 20^0C as follows :

$$G_s\,(\text{at } 20^0\text{C}) = \frac{\text{Wt. of unit volume of soil at T}^0\text{C}}{\text{Wt. of unit volume of water at } 20^0\text{C}}$$

$$= \frac{\text{Wt. of unit volume of soil at T}^0\text{C}}{\text{Wt. of unit volume of water at T}^0\text{C}} \text{ X } \frac{\text{Wt. of unit volume of water at T}^0\text{C}}{\text{Wt. of unit volume of water at } 20^0\text{C}}$$

$$= G_s\,(\text{at T}^0\text{C}) \text{ X } \frac{(\gamma_w)_{\text{at T}^0\text{C}}}{(\gamma_w)_{\text{at } 20^0\text{C}}}$$

$$= G_s\,(\text{at T}^0\text{C}) \text{ X } \frac{(G_w)_{\text{at T}^0\text{C}}}{(G_w)_{\text{at } 20^0\text{C}}}$$

It is usually determined by the principle of displacement of water by soil in a density bottle, volumetric flask, pycnometer or a gas jar. Typical values of specific gravity of soil are given in Table 5.1.

Table 5-1 Typical Values of Specific Gravity or Soil

Soil Type	**Specific Gravity**
Sand	2.65 – 2.67
Silty Sand	2.67 – 2.70
Inorganic clay	2.70 – 2.80
Soils with mica or iron	2.75 – 3.00
Organic soils	Quite variable, as low as 2.0

Equipment

The equipment that is used in this experiment are:

- Pycnometer of 250 ml.
- Balance of 0.01 gm sensitivity.
- Vacuum pump and hot water tub.
- Funnel.
- Spoon.
- Thermometer

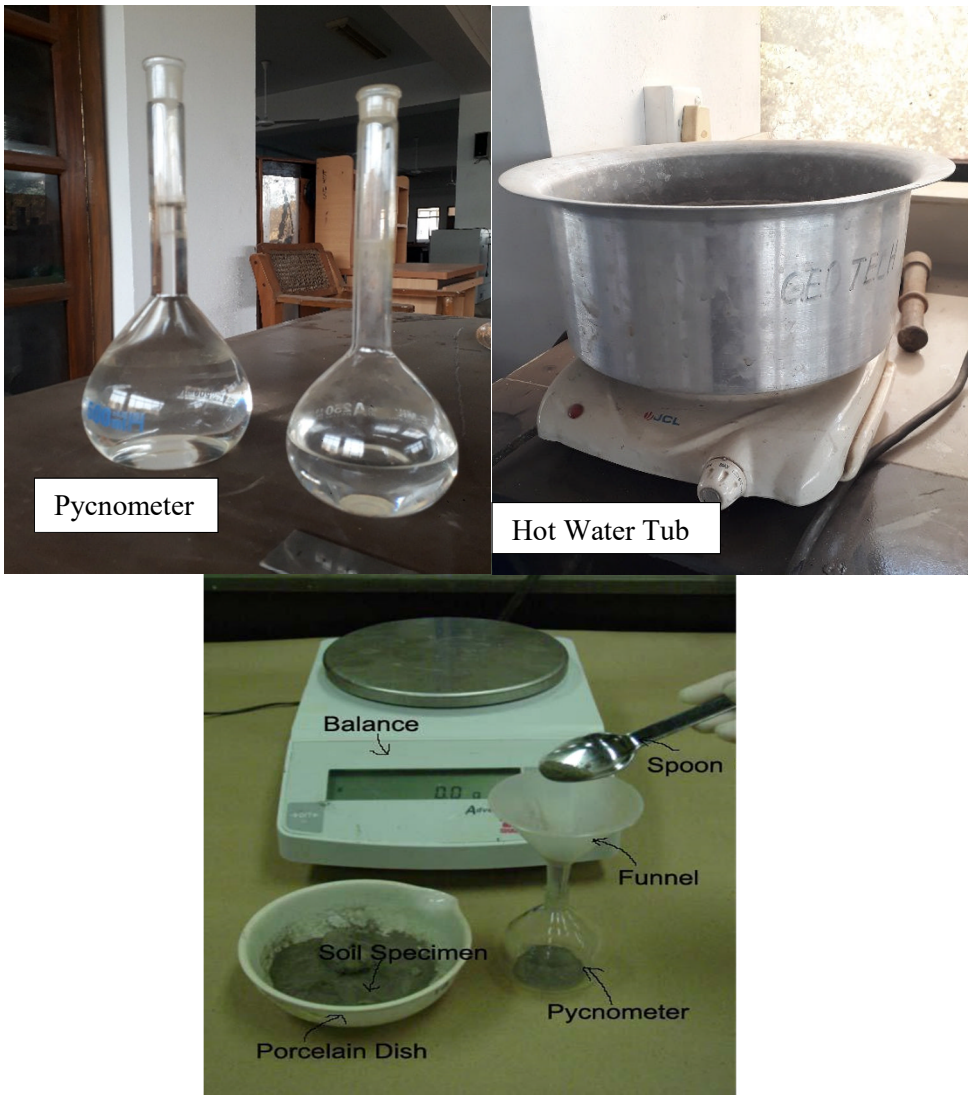


Figure 5-1: Equipment used for specific gravity test.

Test Procedure

The procedure is as follows:

a. First, one should determine and record the weight of the empty clean and dry pycnometer, W_P.

b. Then 50g of a dry soil sample (passed through the sieve No. 200) should be placed in the pycnometer with the help of a funnel. Again, the weight of the pycnometer containing the dry soil, W_{PS} should be determined and recorded.

c. Distilled water should be added to fill about half to three-fourth of the pycnometer. Then it should be put in the boiling water for 10 minutes applying agitation and partial vaccum at 2 minutes interval to remove the entrapped air.

d. The bottle should be cooled and suspension to room temperature. (One should keep it overnight to be cooled to room temperature).

(One will do steps e to h on the following day).

e. Water should be put at room temperature into the flask using wash bottle or pipette to bring the bottom of the meniscus to the calibration mark. Water should be added slowly and droped by drop when the water level comes near the graduation mark.

f. The outside of the bottle should be dried and weighed with water and soil in it nearest to 0.01 g. This weight, W_1 should be recorded in the data sheet.

g. The room temperature T^0C should be recorded from the thermometer kept in a jar filled with water.

h. The soil water mixture should be removed from the volumetric flask, should be washed and cleaned, then it should be mixed with water only (up to the graduaton mark). The outer surface and the portion above the meniscus should be wiped. Weight of the bottle with water, W_2 should be determined and recorded in the data sheet. [a]

Data Analysis

$$\text{Specific Gravity at } T^0C\ , G_T = \frac{W_s}{W_s - W_1 + W_2}$$

Where:

W_S = weight of sample of oven-dry soil, gm = W_{PS} - W_P

W_2 = weight of pycnometer filled with water

W_1 = weight of pycnometer filled with water and soil

Note that W_s-W_1+W_2 represents the weight of water at T^0C whose volume is equal to the volume of the soil sample at T^0C.

$$G_{20} = G_T * \frac{(G_w)_{\text{at } T^0C}}{(G_w)_{\text{at } 20^0C}}$$

The density of water at different temperature can be obtained from **Table 3-2**.

[a] The temperature of the bottle and its contents while taking weight in step f and h (i.e. weights W_1 and W_2) should be the same. As an alternative to taking weight in step h, a calibration curve of the flask may be used to obtain the weight of the bottle+water at T^0C.

Table 3-2: Specific Gravity of water at different temperature (from T.W.Lambe, Soil testing for Engineers)

0^0C	0	1	2	3	4	5	6	7	8	9
	0.9999	0.9999	1.0000	1.0000	1.0000	1.0000	1.0000	0.9999	0.9999	0.9998
10	0.9997	0.9996	0.9995	0.9994	0.9993	0.9991	0.9990	0.9988	0.9986	0.9984
20	0.9982	0.9980	0.9978	0.9976	0.9973	0.9971	0.9968	0.9965	0.9963	0.9960
30	0.9957	0.9954	0.9951	0.9947	0.9944	0.9941	0.9937	0.9934	0.9930	0.9926
40	0.9922	0.9919	0.9915	0.9911	0.9907	0.9902	0.9898	0.9894	0.9890	0.9885
50	0.9881	0.9876	0.9872	0.9867	0.9862	0.9857	0.9852	0.9848	0.9842	0.9838
60	0.9832	0.9827	0.9822	0.9817	0.9811	0.9806	0.9800	0.9795	0.9789	0.9784
70	0.9778	0.9772	0.9767	0.9761	0.9755	0.9749	0.9743	0.9737	0.9731	0.9724
80	0.9718	0.9712	0.9706	0.9699	0.9693	0.9686	0.9680	0.9673	0.9667	0.9660
90	0.9653	0.9647	0.9640	0.9633	0.9626	0.9619	0.9612	0.9605	0.9598	0.9591

Significant Figures

1. The result of Specific Gravity should be presented in 3 significant digits.

WORKED OUT EXAMPLE

Specific Gravity Determination Data Sheet

Test Serial No.: **05**

Date Tested: **February 7, 2009**

Tested By:

Project Name: **Airport Road – Rokeya Sharoni Link Road Project**

Sample Number: **B-3, Depth 3 ft**

Sample Description: **Reddish clay**

Determination No	**1**	**2**	**3**
Bottle No	**2**	**3**	**4**
Wt of Bottle, W_P gm	**94.11**	**92.79**	**92**
Wt of Bottle + Dry Soil, W_{PS} gm	**144.16**	**143.26**	**142.21**
Wt of Dry Soil, W_0 gm	**50.05**	**50.47**	**50.21**
Wt of Bottle + Soil + Water, W_B gm	**374.58**	**373.94**	**373.16**
Wt of Bottle + Water, W_A gm	**342.94**	**342.06**	**341.26**
Temperature T^0 C	**23**	**23**	**23**
Sp. Gravity of Water at T^0 C	**0.9976**	**0.9976**	**0.9976**
Sp. Gravity of Soil, G_S	**2.712106**	**2.708385**	**2.735636**

Sample Calculations

For Determination No 1:

W_P **= 94.11 gm**

W_{PS} **= 144.16 gm**

W_B **= 374.58 gm**

W_A **=342.94 gm**

W_0 **= 144.16 – 94.11 = 50.05 gm**

$$G_S = \frac{50.05 \times 0.9976}{50.05 + (342.94 - 374.58)} = 2.712$$

SECTION 6

ATTERBERG LIMITS TEST

References

- ASTM D4318 -17 "Standard Test Method for Liquid Limit, Plastic Limit and Plasticity Index of Soils.
- Bowles, Joseph E., Engineering Properties of Soils and Their Measurement (4th Edition).
- Lambe, T.W., Soil Testing for Engineers.
- U.S. Army Corps of Engineers, Laboratory Soils Testing, EM 110-2-1906.
- ASTM D 6026 - 06 – using Significant Digits in Geotechnical Data

General Discussion

These test methods cover the determination of the liquid limit, plastic limit, and the plasticity index of soils. These test methods are used as an integral part of several engineering classification systems to characterize the fine grained fractions of soils and to specify the fine-grained fraction of construction materials. The liquid limit, plastic limit, and plasticity index of soils are also used extensively, either individually or together, with other soil properties to correlate with engineering behavior such as compressibility, hydraulic conductivity (permeability), compactibility, shrink-swell, and shear strength. A fine-gained soil can exist in any of several states; which state depends on the amount of water in the soil system. When water is added to a dry soil, each particle is covered with a film of adsorbed water. If the addition of water is continued, the thickness of the water film on a particle increases. Increasing the thickness of the water films permits the particles to slide past one another more easily. The behavior of the soil, therefore, is related to the amount of water in the system. Approximately sixty years ago, A. Atterberg defined the boundaries of four states in terms of "limits" as follows:

a. **Liquid Limit**: The boundary between the liquid and plastic states.

b. **Plastic Limit**: The boundary between the plastic and semi-solid states.

c. **Shrinkage Limit**: The boundary between the semi-solid and solid states.

These limits have since been more definitely defined by A. Casagrande as the water contents which exist under the following conditions:

a. **Liquid Limit**: The water content at which the soil has such a small shear strength that it flows to close a groove of standard width when jarred in a specified manner.

b. **Plastic Limit**: The water content at which the soil begins to crumble when rolled into threads of specified size.

c. **Shrinkage Limit**: The water content that is just sufficient to fill the pores when the soil is at the minimum volume it will attain by drying.

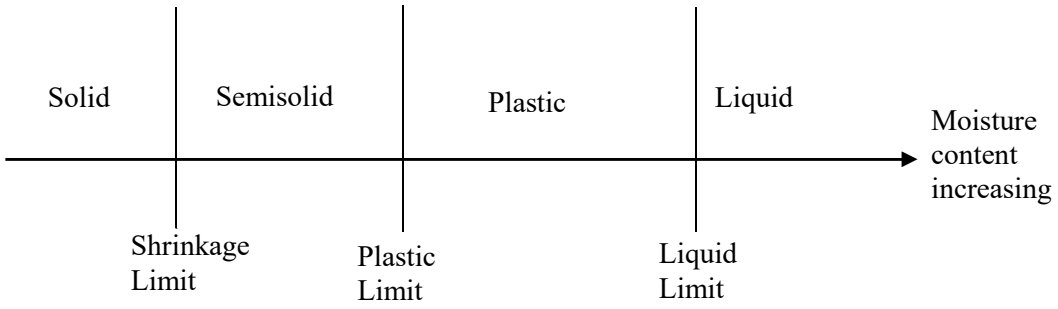


Fig 6-1: Qualitative position of Atterberg Limits on a moisture content scale.

The amount of water which must be added to change a soil from its plastic limit to its liquid limit is an indication of the plasticity of the soil. The plasticity is measured by the "plasticity index", which is equal to the liquid limit minus the plastic limit.

Although the liquid and plastic limits are necessarily determined on soils which have had their natural structure completely destroyed by kneading or "remolding", the shrinkage limit can be obtained on soils in either their undisturbed or their remolded states. The difference between the undisturbed and remolded shrinkage limits may be an indication of the amount of natural "structure" a soil possesses. Also the condition of an in situ soil is often partially revealed by its "water-plasticity ratio", which is the ratio of the difference between the natural water content and the plastic limit to the plasticity index. A high water plasticity ratio, which means that the natural water content is high relative to the liquid limit, indicates a very low remolded strength.

For example, if the ratio is greater than 100%, the soil exists at a water content greater than the liquid limit, and its remolded strength is thus less than that very small amount which it would possess at the liquid limit.

The chemical and mineral composition, size, and shape of the soil particles influence the adsorbed water films on the particles. Because such soil properties as compressibility, permeability, and strength, as well as the limits are dependent on the water films, approximate relationships exist between these properties and the limits.

For a given soil, we can often set up definite semi-empirical relationships between a property and the limits or indices. From such expressions, we can then make predictions of the properties of another sample of the same soil by knowing the limits. Such a procedure is often very helpful because the limits are usually more easily determined than the compressibility, permeability or strength. On construction jobs in which detailed studies of the underlying soil conditions are made, plots of the limits against depth can be made (along with other test results).

The limits furnish an excellent basis for the classification and identification of fine-grained soils. From Plasticity Chart of Aurther Casagrande the soil can be classified using the value of plasticity index and liquid limit and also from PI-LL chart of clay the clay mineral can be identified (see Figure 6 – 2). They are also often used directly in specifications for controlling soil for use in fills, and in semi-empirical methods of design. From the preceding discussion, we can see that even though the limits do not furnish numbers which can be substituted directly into scientifically derived formulas, they are extremely useful to the soil engineer.

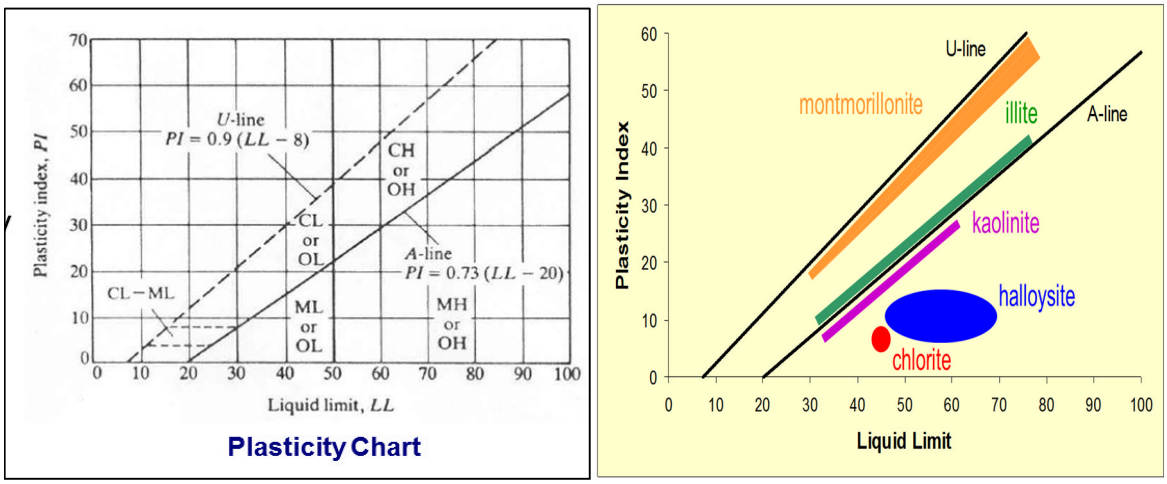


Figure 6 – 2: Aurther Casangrande's Plasticity Chart for Classification of fine-grained soil and Pl-LL chart for identifying Clay mineral by

Equipment

The following equipment are required for the test:

- Liquid limit device and grooving tool
- Mixing bowl
- Drying cans
- Spatula
- Large glass plate
- Drying oven.
- Distilled water
- Evaporating dishes
- Balance (0.01 g sensitivity)
- Large glass plate
- Drying oven



Figure 6 - 3: Equipment used for test.

Soil Preparation

The preparation procedure is as follows:

a. The soil should be dried in air and crushed to sieve through # 40 sieve.

b. Then approximately 200 gm of that soil has to be taken and mixed with water to make Paste

c. The paste should be covered with some container and put some wet cloth around the container to avoid evaporation.

d. The sample should be kept like this for 24 hours to ensure proper mixing of soil particles and water.

Test Procedure

Liquid Limit Test.

a. Each group will pulverize a sufficient quantity of air dried soil to obtain approximately 250 ± 10 g of material passing the #40 sieve. The residue remaining on the sieve should be discarded since it is no longer representative of the sample. It should be made sure that the lumps are broken up before sieving, since many of the fines are in these lumps. The soil must not be oven dried, since this will reduce the liquid and plastic limits of the soil.

b. The height of fall of their liquid-limit device should be checked. The device must be adjusted for a fall of exactly 1 cm (not over ±.0.1 mm). The 1 cm calibration block on the end of the grooving tool must be used for making the adjustment. The adjustment with respect to the worn spot should be made on the base of the cup, not the shortest fall. If the height of the fall is not calibrated within these limits, a water content error of several percent may be introduced, see Figure 6 - 4.

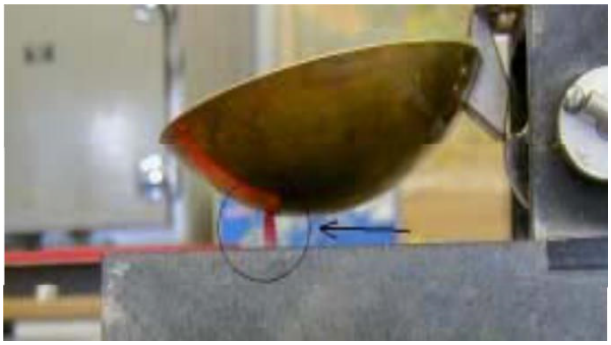


Figure 6 - 4: Use of calibration block. Block can be moved back to ensure a 1 cm drop.

c. The 250 g of soil should be placed in a mixing dish, then a small amount of water should be added, and the soil should be mixed carefully to a uniform color. Another major source of error is poor mixing of the soil-water mixture. When the color is uniform throughout and the mixture has a creamy appearance, proper mixing has generally been achieved. It should be continued to add small amounts of water and should be mixed the mass to a uniform color each time. When the soil reaches to the point of consistency (stickiness) where one can estimate (or by making a trial test) it will take about 50 blows to close the standard groove ½ in., then about 20 g of the thoroughly mixed soil should be removed from the dish. This sample should be set aside for the plastic limit test. Now more water should be added and mixed to a uniform color until the soil reaches to a consistency which will yield a blow count of from 30 to 40 blows to close the standard groove ½ inch.

d. The brass cup should be removed from the liquid-limit machine and a small amount of soil should be placed to the correct depth of the grooving tool, and well centered in the cup with respect to the hinge. The surface of the soil pat must be smoothed carefully, and using the grooving tool, a clean, straight groove should be cut that completely separates the soil pat into two parts. The tool should be held perpendicular to the instantaneous tangent to the cup surface so that the soil pat is not trimmed too shallow in depth. The soil depth should be just barely trimmed with the shoulders of the tool at the deepest part, see Figure 6-5. After making the groove, quickly the cup should be attached to the device and the blow count should be made. The rate of rotation of the crank on the Casagrande device should be about 2 revolutions per second. The test must be stopped after the groove has closed a length of 13 cm, see Figure 6 - 5. Unnecessary delays will affect the blow counts; if the laboratory humidity is low, dry the soil surface. The effect will show when plotting the data by giving an erratic locus of points. Other test errors will also do this, however.



Figure 6 - 5: Groove separating two halves of pat (left); Stop blow counts after this length of groove closes a length of 13 cm (right)

e. Step d should be repeated until 3 consistent blow counts are achieved for the water content. A moisture sample should be taken in the pre-weighed moisture cups (as large as possible and of about 40 grams being sure to take the water content sample from the closed part of the groove. This sample must be weighed immediately and the blow count and wet weight should be recorded on the data sheet. The remainder of the soil from the brass cup should be removed and returned to the mixing dish. Then the cup should be washed and dried up.

f. A small amount of water should be added to the mixing dish of soil, and carefully mixed to a uniform color and consistency to yield a blow count of between 25 and 30 blows. Steps d and e should be repeated.

g. The sequence for two additional tests for blow counts of between 20 and 25, and between 15 to 20, should be repeated for a total of at least four test determinations. It should be made sure to clean and dry the brass cup between tests, and should be tried to maintain a consistent time lapse for each test. The soil should not be allowed to sit in the brass cup for a long period of time before the test. This would allow adhesion to build up, which would result in higher blow counts. The blow count sequence is done in descending order to make sure the sample is well mixed. It is much easier to add water and mix to a uniform consistency (resulting in lower blow counts) than it is to add soil and mix to a uniform consistency. If too much water is added so that the sample is driven to the wet side of 25 blow counts before a sufficient spread of data points is collected on the dry side of 25 blow counts, the sample should be air dried rather than adding dry soil. Mixing or working the sample will help to quicken the drying time. The cups of soil should be placed in the oven (105 to 110°C) and dry 24 hours. When dry, the dry weight of the samples should be weighed and recorded. The water content of the samples should be determined, and a plot of water content versus blow count for this tests should be prepared.

Plastic Limit Test.

a. First 2 to 5 g of the material remaining from the liquid limit test should be taken. The material should be taken at any stage of the drying process at which the mass becomes plastic enough to be shaped into a ball easily without sticking to the fingers when squeezed.

b. Then the specimen should be shaped into an ellipsoidal mass and it should be rolled either under the root of the fingers, under the palm of the fingers, under the palm of the hand, or under the heel of the thumb and against the glass plate. It should be used just enough pressure to roll the soil mass into a thread that is 1/8 in. in diameter. When the diameter of the thread becomes 1/8 in. without crumbling, the thread should be folded and kneaded into a ball again and should be repeated the rolling

process. Kneading and rolling the specimen must be continued until the soil has dried to the point where the rolled thread will break into numerous pieces with a diameter of 1/8 in. and about 1/8 to 3/8 in. in length, see Figure 6 - 6.

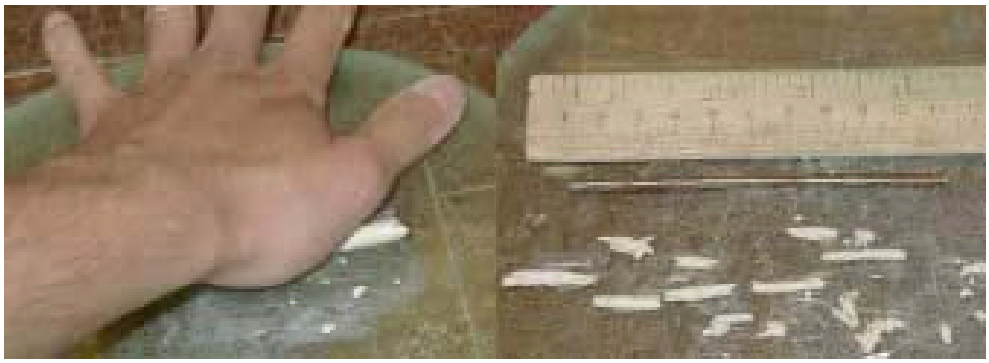


Figure 6 - 6: (a) rolling a thread and (b) rolled threads.

c. The pieces of the crumbled thread should be placed into a specimen container and the water content should be determined. Steps a and b should be repeated three to four times with different portions of the prepared material in order to check the plastic limit.

Both tests should agree within 1/20 of the plastic limit; otherwise the plastic limit test should be repeated.

Shrinkage Limit Test.

a. The shrinkage limit dish should be cleaned and the weigh should be taken. Also the inside diameter and height must be taken to determine the inside volume of the dish.

b. Then the petroleum jelly should be coated inside of the dish and the dish should be filled with wet soil.

c. Excess soil standing above the edge of the dish must be strucked off by a glass plate or straight edge.

d. Weight of the dish with wet soil is noted to record the weight of the wet soil pat.

e. The soil pat in the dish is the oven dried for 24 hours at 110^0C.

f. The oven dried soil should be weighed with container to record the weight of the dry soil pat.

g. The dry soil pat should be taken out of the dish and dipped inside a melted pot of wax and cool it.

h. The wax coated soil pat should be submerged in a measuring jar. Increase of the height of water indicates the volume of dry soil pat.



Figure 6 - 7: Equipment needed for shrinkage limit test.

Discussion of the Procedure

The American Society for Testing Materials and the American Association of State Highway Officials permit the use of the "hand method" of determining the liquid limit in addition to that given in this lab procedure. In the hand method, a tool is used to cut a groove (same size as in the mechanical method) in a soil paste contained in an evaporating dish. The water content is adjusted until the groove is closed by jarring the dish lightly against the heel of the hand ten times. The liquid limit is this adjusted water content.

A simplified procedure for determining the liquid limit has been investigated. The method is based on the assumption that the slope of the plot of blows on log scale against water content on log scale is a straight line with a constant slope. If this assumption were correct, the liquid limit could be obtained from any point on the

curve. Based on 767 liquid limit tests, the discovery was made that for the soils investigated the liquid limit, w_L, could be found from:

$$w_L = w_N(N/25)^{0.121}$$

in which w_N = the water content of the soil which closes in N blows in the standard liquid limit device.

Natural soils used for liquid and plastic limit determinations should not be dried prior to testing because drying may alter the soil by causing the particles to subdivided or agglomerate, by driving off adsorbed water which is not completely regained on rewetting, or by effecting a chemical change in any organic matter in the soil. These effects can significantly change the limits, especially the liquid limit. The liquid limit of oven dried organic soils tends to be lower than that of the undried. On the other hand, the effect of drying on some clays (particularly those composed of minerals of the montmorillonite group) cannot be predicted. For example, the liquid limit of an air-dried sample of a soil was 20% greater than that determined on the soil not previously dried, whereas the liquid limit of an oven-dried sample of the same soil was 24% less than that determined on the undried soil.

Calculations

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

a. **Liquid Limit**. w_L is read from the flow curve as the water content at 25 blows.

b. **Plastic Limit**. w_p is the water content of the soil which crumbled when 1/8 in. threads were rolled (the average of the consistent determinations made)

c. **Shrinkage Limit** . w_s is found from

$$w_s = \frac{W_i - W_f}{W_f} \text{X } 100 - \frac{V_i - V_f}{W_f} \text{X } 100\, \gamma_w$$

In which γ_w = Unit weight of water,
W_i = Weight of wet soil pat,
V_i = Volume of wet soil pat,
W_f = Weight of dry soil pat,
V_f = Volume of dry soil pat

Atterberg indices can be calculated from the limits as follows:

a. Plasticity index, I_P = Liquid limit – plastic limit
b. Flow index, I_F = Slope of flow curve
c. Toughness index, $I_T = \frac{Plasticity\ Index}{Flow\ Index}$

Water – plasticity ratio, B is found from the equation:

$$B = \frac{w_N - w_P}{w_L - w_P}$$

in which w_N = natural water content

Errors in computing the liquid or plastic limits sometimes can be detected by plotting the values of liquid limit versus plasticity index with reference to the upper limit line on the plasticity chart (shown in Figure 3.2 in Holtz and Kovacs, 1981). The upper limit line starts from a liquid limit of 8 at a plasticity index of 0 and rises toward the right with a slope of 9 vertically on 10 horizontally; the equation of the upper limit line, therefore, is

$$w_i = 0.9\ (w_L - 8).$$

Significant Figures

1. All the Atterberg Limits should be expressed to their nearest whole number.

Possible Errors

General.

a. The specimen is not representative of the soil. The liquid and plastic limits must be determined using the same mixture of soil as that used for determination of natural water content or for other tests.

b. Specimen is improperly prepared. The specimens must be thoroughly mixed and be permitted to cure for a sufficient period before testing. Erroneous results may be caused by the loss of colloidal material when removing particles coarser than the No. 40 sieve or by testing air-dried or oven-dried soils.

c. Inaccurate determination of the water contents would greatly affect the computed liquid and plastic limits because of the small quantities of material available for the water content determinations.

d. Computational mistakes.

Liquid Limit Test.

a. Improperly constructed or adjusted liquid limit device.

b. Worn parts of liquid limit device especially at point of contact between the cup and the base, or worn tip of grooving tool.

c. Soil at the point of contact between the cup and the base. Removing the cup for shaping and grooving the sample will insure that the bottom of the cup and the top of the base are clean. Any soil that has dropped onto the base can be removed with one stroke of the back of the hand before replacing the cup.

d. Loss of moisture during the test. Erratic and erroneous results may be caused by drying of some soil mixtures unless the test is performed in a humid room.

Plastic Limit Test.

a. Rolling thread under fingers will break the thread prematurely.

b. Incorrect final thread diameter. A length of 1/8 in. diameter metal rod close at hand will help in estimating this diameter accurately.

c. Stopping the rolling process too soon. If there is any doubt as to whether the thread has crumbled sufficiently, it is better to roll the thread once more than to stop the process too soon.

Worked Out Example

Atterberg Limits

Test Serial No. **06**

Analyst Name: _____ Date: **04 March 09**

Project Name: **Airport Road – Rokeya Sharoni Link Road Project**

Boring No.: **03** Sample No.: **07** Depth: **1.5 m**

Source: **Shalby Tube from SPT**

Natural Water Content:

Sample No	**1**	**2**	**3**
Container #	**2**	**19**	**95**
Container Wt (gm)	**43.8**	**32.7**	**27.3**
Wet Wt + Container (gm)	**71.2**	**89.5**	**93.2**
Dry Wt + Container (gm)	**68.8**	**84.3**	**87.1**
Wt of Water (gm)	**2..4**	**5.2**	**6.1**
Dry Wt of Soil (gm)	**2.5.0**	**51.6**	**59.8**
Water Content (%)	**9.6**	**10.1**	**10.2**
Average (Natural Water Content)		**9.97**	

Sample Calculations (For Sample No 1)

Weight of container = 43.8 gm

Weight of wet soil + container = 71.2 gm

Weight of dry soil + container = 68.8 gm

Weight of water = (71.2 – 68.8) = 2.4 gm

Weight of dry soil = ((68.8 – 43.8) = 25.0 gm

Natural water content = (2.4 / 25.0) x 100 = 9.6%

Plastic Limit:

Sample No	**1**	**2**	**3**
Container #	**21**	**9**	**59**
Container Wt (gm)	**59.7**	**25.2**	**25.3**
Wet Wt + Container (gm)	**61.1**	**25.9**	**26.8**
Dry Wt + Container (gm)	**60.9**	**25.8**	**26.6**
Wt of Water (gm)	**0.2**	**0.1**	**0.2**
Dry Wt of Soil (gm)	**1.2**	**0.6**	**1.3**
Water Content (%)	**16.7**	**16.7**	**15.4**
Average (Plastic Limit)		**16.3**	

Sample Calculations (For Sample No 1)

Weight of container = 59.7 gm

Weight of wet soil + container = 61.1 gm

Weight of dry soil + container = 60.9 gm

Weight of water = (61.1 – 60.9) = 0.2 gm

Weight of dry soil = ((60.9 – 59.7) = 1.2 gm

Water content (Plastic limit) = (0.2 / 1.2) x 100 = 16.7%

Liquid Limit:

Container No	**4**	**6**	**22**	**1**	**17**
No. of Blows	**20**	**22**	**24**	**28**	**31**
Container Wt (gm)	**59.7**	**59.8**	**59.7**	**58.4**	**58.5**
Wet Wt + Container (gm)	**84.9**	**69.2**	**68.6**	**76.2**	**71.2**
Dry Wt + Container (gm)	**76.8**	**66.2**	**65.9**	**71**	**67.8**
Wt of Water (gm)	**8.1**	**3.0**	**2.7**	**5.2**	**3.4**
Dry Wt of Soil (gm)	**17.1**	**6.4**	**6.2**	**12.6**	**9.3**
Water Content (%)	**47.4**	**46.4**	**43.5**	**41.3**	**36.6**
Liquid Limit (from graph, for 25 blows)			**42.5**		

Sample Calculations (For Container No 4)

Number of blows = 20

Weight of container = 59.7 gm

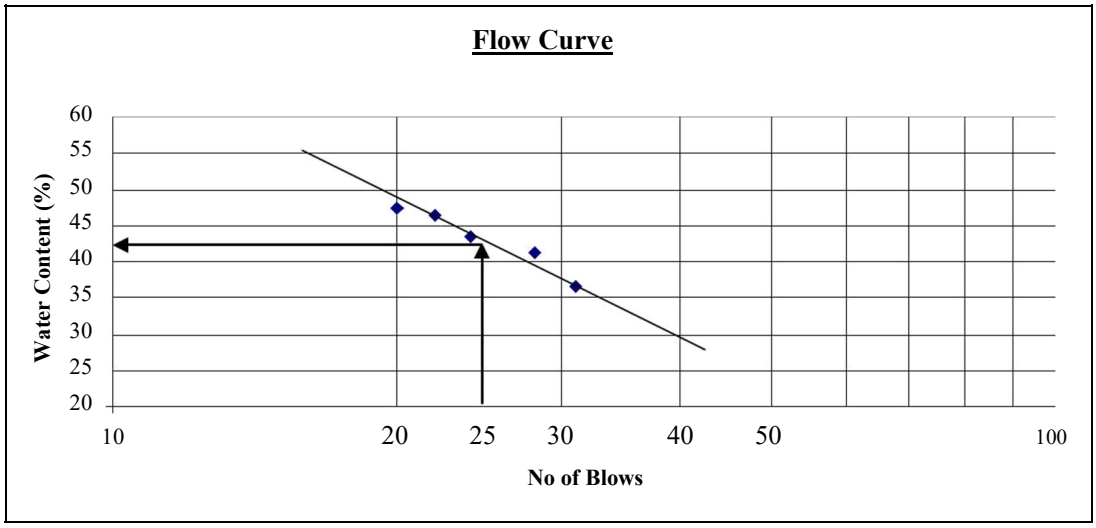
Weight of wet soil + container = 84.9 gm

Weight of dry soil + container = 76.8 gm

Weight of water = (84.9 – 76.8) = 8.1 gm

Weight of dry soil = ((76.8 – 59.7) = 17.1 gm

Water content (Liquid limit)= (8.1 / 17.1) x 100 = 47.4%



SECTION 7

RELATIVE DENSITY TEST

References

- ASTM D 4253 - Standard Test Method for Maximum Index Density and Unit Weights of Soils and Calculations of Relative Density.
- ASTM D 4254 - 16 - Standard Test Method for Minimum Index Density and Calculation of Relative Density
- Bowles, Joseph E., Engineering Properties of Soils and Their Measurement (4th Edition)
- Lambe, T.W., Soil Testing for Engineers.
- ASTM D 6026 - 06 – using Significant Digits in Geotechnical Data

General Discussion

This test is performed to determine the state of density of cohessionless, free-draining soils with respect to its maximum and minimum densities using a vibrating table. Relative Density and percent compaction are commonly used for evaluating the state of compactness of a given soil mass. The engineering properties, such as shear strength, compressibility, and permeability, of a given soil depend on the level of compaction.

The relative density of a soil is the ratio expressed as a percentage, of the difference between the maximum index void ratio of a cohessionless, free-draining soil; to the difference between its maximum and minimum index void ratios.

The necessary formula for relative density determination is as follows:

$$\text{Relative Density, } \quad D_r = \frac{e_{max} - e}{e_{max} - e_{min}}$$

Where, e_{max} = void ratio in loosest state

e_{min} = void ratio in densest state

e = void ratio in natural state

Since void ratios are difficult to obtain (they require specific gravity G_s of the soil) the equation form usually used for D_r is,

$$D_r = \left(\frac{\gamma_d - \gamma_{d\,min}}{\gamma_{d\,max} - \gamma_{d\,min}}\right)\left(\frac{\gamma_{d\,max}}{\gamma_{d\,min}}\right)$$

Where, γ_{dmax} = maximum laboratory dry density

γ_{dmin} = minimum laboratory dry density

γ_d = field or in situ soil density

Therefore, the following three components of relative density is to be determined:

a. Minimum dry unit weight, γ_{dmin}.
b. Maximum dry unit weight, γ_{dmax}.
c. Field dry unit weight, γ_d.

Equipment

Figure 7-1 shows the equipment used for relative density test. The list of the equipment is as following:

- Vibrating Table
- Mold Assembly consisting of standard mold
- Guide sleeves
- Surcharge base plate
- Surcharge weights
- Surcharge base plate hand
- Dial indicator gage
- Scoop
- Straightedge
- Balance

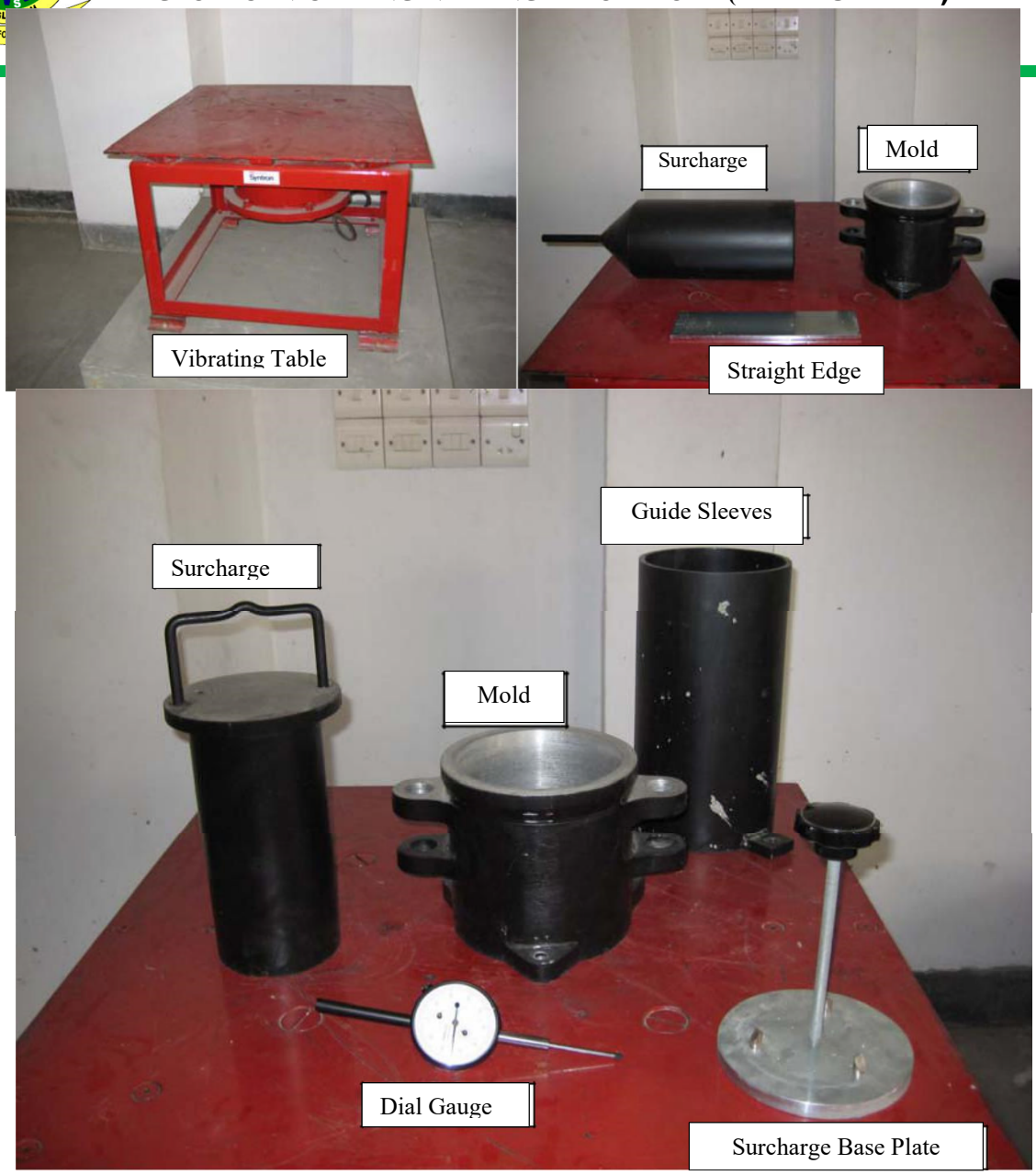


Figure 7-1: Equipment Used for Relative Density Test.

Test Procedure

Loosest Condition.

a. The weight of the empty mold (W_0) should be determined and recorded. Also the dimensions of the mold should be determined to calculate the calibrated volume of the mold, V_m.

b. The mold should be filled with the soil (approximately 0.5 inch to 1 inch above the top of the mold) as loosely as possible by pouring the soil using a scoop or pouring device (funnel). The diameter of the spout (outlet of funnel) should be ½ inch. Spiraling motion should be just sufficient to minimize particle segregation.

c. The excess soil should be trimmed off to level with the top by carefully trimming the soil surface with a straight edge.

d. The weight of the mold and soil (W_1) should be determined and recorded.

Densest Condition.

a. The mold should be filled with soil. The sides of the mold may be struck a few times using a metal bar or rubber hammer to settle the soil so that the surcharge base-plate can be easily placed into position and there is no surge of air from the mold when vibration is initiated.

b. The surcharge base plate should be placed on the surface of the soil and it must be twisted slightly several times so that it is placed firmly and uniformly in contact with the surface of the soil. Then the surcharge plate handle should be removed.

c. The mold should be attached to the vibrating table.

d. The initial dial reading should be determined by inserting the dial indicator gauge holder in each of the guide brackets with the dial gage stem in contact with the rim of the mold (at its center) on the both sides of the guide brackets. Six sets of dial indicator readings should be obtained, three on each side of each guide bracket. The average of these readings is the initial dial gauge reading, R_i. Record Ri to the nearest 0.001 in. (0.025 mm).

e. The guide sleeve should be firmly attached to the mold and lower the appropriate surcharge weight [14 kN/m^2 or 2 lb/in^2] onto the surcharge base plate.

f. The mold assembly and soil specimen should be vibrated for 8 minutes at a frequency of 3600 cycles/sec and amplitude of 1/40 inches.

g. After the vibration, the dial indicator gage readings should be determined and recorded as in Step (d). The average of these readings is the final dial gage reading, R_f.

h. The surcharge base-plate should be removed from the mold and the mold should be detached from the vibrating table.

j. The weight of the mold and soil (W_2) should be determined and recorded. Also, the thickness of the surcharge base-plate, T_p should be determined.

Data Analysis

Minimum Dry Unit Weight, γ_{dmin} or Loose Unit Weight.

a. Volume of the mold, V_m determination:

(1) Height of the mold = H_m inch.

(2) Diameter of the mold = d inch

(3) Volume of the empty mold, $V_m = \pi/4 \times d^2/144 \times H_m/12$ ft^3

b. Weight of soil, W_{s1} determination:

(1) Weight of empty mold, W_0 lb.

(2) Weight of loose soil + mold, W_1 lb.

(3) Weight of soil, $W_{s1} = (W_1 - W_0)$ lb.

c. Minimum Dry Unit Weight, $\gamma_{dmin} = W_{s1}/V_m$ pcf.

Maximum Dry Unit Weight, γ_{dmax} or Dense Unit Weight

a. Volume of the tested dry soil, V_s determination:

(1) Diameter of the mold = d inch.

(2) Height of the mold, H_m inch

(3) Initial dial reading, R_i

(4) Final dial reading, R_f

(5) Thickness of base plate, T_p

(6) Subsidence Height, $H = \{(R_f - R_i) \times 0.001 + T_p\}$ inch

(7) Volume of the soil sample, $Vs = \pi/4 \times d^2/144 \times H_m/12$ ft^3

b. Weight of soil, W_{s2} determination:

(1) Weight of empty mold, W_0 lb.

(2) Weight of dense soil + mold, W_2 lb.

(3) Weight of soil, $W_{s2} = (W_2 - W_0)$ lb.

c. Maximum Dry Unit Weight, $\gamma_{dmax} = W_{s2}/V_s$ pcf.

Field Dry Unit Weight, γ_d.

A sample collection container of specific shape (known dimension) whose weight is known is inserted into the soil. Measures to be taken to create least possible disturbances. Soil sample is collected to the full volume of the container. Additional soil has to be removed with the help of trimmer and care has to be taken not to give extra pressure to fill the container. The soil sample along with the container is put in the oven for drying. Oven dry sample is then weighted to get the dry weight of the sample. As the volume of the container represents the volume of soil, so the field dry unit weight γ_d can be determined.

Significant Figures

1. The value of relative density should be expressed to the nearest 0.1.

WORKED OUT EXAMPLE

Relative Density Test

Test Serial No.: **07**
Date Tested: **January 15, 2009**

Tested By: _____

Project Name: **Hatirjheel Project**

Location : **Behind Sonargaon Hotel**

Sample Number: **S-1, Embankment Top**.

Visual Classification of Soil: **Fine Sand to silty sand**.
Job No: **3** Sample Depth: **0 ft**.
Height of Mold: **6. 1″** Diameter of Mold: **6″** Thickness of Base Plate: **0.5″**
Area of Mold: **0.1963 ft$_2$** Volume of Mold: **0.0998 ft^3**

LOOSEST STATE

No. of Obsn	Wt. of mold + Soil, W_1 in lb	Wt. of mold, W_0 in lb	Wt. of Soil, W_{s1} in lb	Vol of Soil, V_m in ft^3	Min Dry Density, γ_{dmin} in pcf	Avg. Density, pcf	Min Dry γ_{dmin} in pcf
1.	16.4252	7.744	8.6812	0.0998	86.99		
2.	16.1	7.68	8.42	0. 0998	84.17		84.65
3.	16.07	7.67	8.4	0.0998	84.17		
4.	16.038	7.7264	8.3116	0.0998	83.28		

DENSEST STATE

No. of Obsn.	Wt. mold Soil, W_2 (lb)	of Wt. of + mold, W_0 (lb)	Wt. of Soil, W_{s2} (lb)	Initial Dial Reading, R_i (inch)	Final Dial Reading, R_f (inch)	Subsidence Height, $H = (R_f - R_i) * 0.001$ + Thickness of Base Plate, T_p (inch)	Vol of Soil, $V_s=(\pi d^2/4)*(H_m-H)$ (ft^3)	MaxDry Density (pcf)	Avg. Max Dry Density, γ_{dmax} (pcf)
1.	16.432	7.744	8.688	100	200	0.6	0.08997	96.57	96.28
2.	16.09	7.68	8.410	140	-110	0.75	0.0875	96.11	
3.	16.083	7.67	8.413	63	307	0.744	0.0876	96.04	
4.	16.083	7.726	8.357	100	400	0.8	0.0867	96.39	

If the field dry unit weight, γ_d is determined to be 92 pcf (say). Then,

The relative density,

$$D_r = \frac{\gamma_d - \gamma_{d\min}}{\gamma_{d\max} - \gamma_{d\min}} \times \frac{\gamma_{d\max}}{\gamma_d}$$

$$= \frac{92 - 84.65}{96.28 - 84.65} \times \frac{96.28}{92}$$

$$= \frac{7.35}{11.63} \times 1.04652$$

$$= 0.6614$$

$$= 66.1\%$$

SECTION 8

UNCONFINED COMPRESSION TEST

References

- ASTM D 2166 – 16 - Standard Test Method for Unconfined Compressive Strength of Cohesive Soil.
- Bowles, Joseph E., Engineering Properties of Soils and Their Measurement (4th Edition)
- Lambe, T.W., Soil Testing for Engineers.
- ASTM D 6026 - 06 – using Significant Digits in Geotechnical Data

General Discussion

The primary purpose of this test is to determine the unconfined compressive strength, which is then used to calculate the unconsolidated undrained shear strength of the clay under unconfined conditions. According to the ASTM standard, the unconfined compressive strength (q_u) is defined as the compressive stress at which an unconfined cylindrical specimen of soil will fail in a simple compression test. In addition, in this test method, the unconfined compressive strength is taken as the maximum load attained per unit area, or the load per unit area at 15% axial strain, whichever occurs first during the performance of a test.

For soils, the undrained shear strength (s_u) is necessary for the determination of the bearing capacity of foundations, dams, etc. The undrained shear strength (s_u) of clay is commonly determined from an unconfined compression test. The undrained shear strength (s_u) of a cohesive soil is equal to one-half the unconfined compressive strength (q_u) when the soil is under the $\varphi = 0$ condition (φ = the angle of internal friction). The Mohr's circle construction for this test assumes that, (1) σ_3, the minor principal stress, is zero or atmospheric (any surface tension confinement is ignored) and (2) the angle of internal friction φ is zero.

The most critical condition for the soil usually occurs immediately after construction, which represents undrained conditions, when the undrained shear strength is basically equal to the cohesion (c). This is expressed as:

$$s_u = c = \frac{q_u}{2}$$

Then, as time passes, the pore water in the soil slowly dissipates, and the intergranular or effective stress increases, so that the drained shear strength (τ), given by $\tau = c + \sigma' \tan \varphi$, must be used. Where σ' = intergranular or effective pressure acting perpendicular to the shear plane; and $\sigma' = (\sigma - u)$, σ = total pressure, and u = pore water pressure; c' and φ' are drained shear strength parameters.

The unconfined compression test may be either strain-controlled or stress-controlled . The strain-controlled test is almost universally used, as it is simply a matter of attaching the proper gear ratio to a motor to control the rate of advance of a loading head (no timing is needed for the test). The test has been found to be somewhat sensitive to the rate of strain, but a strain rate between ½ and 2 percent/min appears to yield satisfactory results. One should use the higher strain rate with softer soils (which may require failure definition at a strain of 15 percent) and the lower strain rate (½ percent/min) for dry or stiff soil.

A stress-controlled test requires changing of loads and may result in erratic strain response and/or the ultimate strength falling between two stress increments. The loads are applied through a dead-load apparatus yoke. Some persons suggests developing the loads by adding water to a container attached to the yoke. In most cases, however, the dead load is simply a hanger to which masses are periodically added. Both methods, produce “shock” loading of the sample and are difficult to apply. For these several reasons, stress-controlled tests are seldom used in this or in any similar soil test

Equipment

Figure 8-1 showing the equipment used for unconfined compression test. The list of the equipment is as following:

- Compression device.
- Load and deformation dial gauges.
- Sample trimming equipment.
- Balance.
- Moisture can.



Figure 8-1: Equipment Used for Unconfined Compression Test.

Test Procedure

The test procedure is as following:

a. The soil sample should be extruded from Shelby tube sampler. A soil specimen should be cut so that the ratio (L/d) is approximately between 2 and 2.5, where L and d are the length and diameter of soil specimen, respectively.

b. The exact diameter of the top of the specimen should be measurd at three locations 120° apart, and then the same measurements should be made on the bottom of the specimen. The measurements should be averaged and the average must be recorded as the diameter on the data sheet.

c. The exact length of the specimen should be measured at three locations 120° apart, and then the measurements should be averaged and the average should be recorded as the length on the data sheet.

d. The sample should be weighed and recorded on the data sheet.

e. The deformation (L) corresponding to 15% strain (ε) should be recorded.

$L = Strain\ (\varepsilon) \times L_0$

Where L_0 = Original specimen length (as measured in step c).

f. The specimen in the compression device should be placed carefully and it should be centered on the bottom plate. If the ends are not exactly perpendicular to the specimen axis, the early part of the stress-strain curve will be flat (until the full sample area contributes to stress resistance, strains will be too large for the nominal stresses computed). The device should be adjusted so that the upper plate just makes contact with the specimen and the load and deformation dials should be set to zero.

g. The load should be applied so that the device produces an axial strain at a rate of 0.5% to 2.0% per minute, and then the load and deformation dial readings should be recorded on the data sheet at every 20 to 50 divisions on deformation the dial.

h. The load should be kept applying until:

(1) The load (load dial) decreases on the specimen significantly,

(2) The load holds constant for at least four deformation dial readings, or

(3) The deformation is significantly past the 15% strain that was determined in step e.

j. A sketch should be drawn to depict the sample failure.

k. The sample from the compression device should be removed and then a sample should be obtained for water content determination.

Data Analysis

The data recording and computation procedure are as under:

a. The dial readings should be converted to the appropriate load and length units, and these values should be entered on the data sheet in the deformation and total load columns. (Confirm that the conversion is done correctly, particularly proving dial gage readings conversion into load).

b. The sample cross-sectional area A_0 should be computed as,

$$A_0 = \frac{\pi}{4} \times d^2$$

c. The strain ε should be computed,

$$\varepsilon = \frac{L}{L_0}$$

d. The corrected area should be computed,

$$A' = \frac{A_0}{1 - \varepsilon}$$

f. Using A′, the specimen stress σ should be computed,

$$\sigma = \frac{P}{A'}$$

(One must be careful with unit conversions and should use constant units).

g. The water content, w% should be computed.

h. The stress versus strain should be plotted. q_u should be shown as the peak stress (or at 15% strain) of the test. It must be made sure that the strain is plotted on the abscissa (See the worked out example).

j. Mohr's circle should be drawn using q_u from the last step and the undrained shear strength, s_u = c (or cohesion) = $q_u/2$ should be shown (See the worked out example).

Significant Figures

1. The value of water content should be expressed to the nearest 0.1.
2. The value of axial strain should be expressed to 3 significant digits.
3. The value of stress or pressure should be expressed to 3 significant digits.

WORKED OUT EXAMPLE

Unconfined Compression Test

Test Serial No. : **08**

Date Tested: **January 12, 2009**

Tested By: _____

Project Name: **Airport Road – Rokeya Sharoni Link Road Project**

Location : **Jahangir Gate**

Sample Number: **B-1, U - 2**.

Visual Classification of Soil: **Silty clay**.

Job No: **2** Sample Depth: **1.5 m**.

Initial diameter (d_0) = **1.5 inch**

Initial area (A_0) = **1.767 inch^2**

Initial length (L_0) = **3 inch**

Initial volume (V_0) = **5.3 inch^3**

Moisture Content Determination

Tare No	**402**
Wt of tare	**7.32 gm**
Wt of tare + wet sample	**176.81 gm**
Wt of tare + dry sample	**145.32 gm**
Wt of water	**31.49 gm**
Wt of dry soil	**138 gm**
Moisture content, W%	**22..8**

Unconfined Compression Test Data

(Load Dial: 1 unit = 0.3 lb; Deformation Dial: 1 unit = 0.001inch)

Load Dial	Axial Load, P (lb)	Displacement Dial	Total Displacement (in)	Unit Strain	Corrected Area (in^2)	Stress, σ (psi)	Shear Stress (psi)
0	**0**	**0**	**0**	**0**	**1.766**	**0.00**	**0.00**
1	**0.3**	**3**	**0.003**	**0.001**	**1.768**	**0.17**	**0.08**
1.5	**0.45**	**6**	**0.006**	**0.002**	**1.770**	**0.25**	**0.13**
2	**0.6**	**9**	**0.009**	**0.003**	**1.772**	**0.34**	**0.17**
2.5	**0.75**	**12**	**0.012**	**0.004**	**1.773**	**0.42**	**0.21**
5	**1.5**	**15**	**0.015**	**0.005**	**1.775**	**0.85**	**0.42**
9	**2.7**	**30**	**0.03**	**0.01**	**1.784**	**1.51**	**0.76**
15	**4.5**	**45**	**0.045**	**0.015**	**1.793**	**2.51**	**1.25**
22	**6.6**	**60**	**0.06**	**0.02**	**1.802**	**3.66**	**1.83**
25	**7.5**	**75**	**0.075**	**0.025**	**1.812**	**4.14**	**2.07**
35	**10.5**	**90**	**0.09**	**0.03**	**1.821**	**5.77**	**2.88**
45	**13.5**	**120**	**0.12**	**0.04**	**1.840**	**7.34**	**3.67**
52	**15.6**	**150**	**0.15**	**0.05**	**1.859**	**8.39**	**4.20**
55	**16.5**	**180**	**0.18**	**0.06**	**1.879**	**8.78**	**4.39**
58	**17.4**	**210**	**0.21**	**0.07**	**1.899**	**9.16**	**4.58**
60	**18**	**240**	**0.24**	**0.08**	**1.920**	**9.38**	**4.69**
60	**18**	**270**	**0.27**	**0.09**	**1.941**	**9.27**	**4.64**
59	**17.7**	**300**	**0.3**	**0.1**	**1.963**	**9.02**	**4.51**
58	**17.4**	**330**	**0.33**	**0.11**	**1.985**	**8.77**	**4.38**

Sample Calculations (For Last Reading)

Load dial reading = 58

Axial load = 58 x 0.3 (load factor) = 17.4 lb

Displacement dial reading = 330

Total Displacement or change in length, L= 330 x 0.001 = 0.33 inch Initial length, L0 = 3 inch

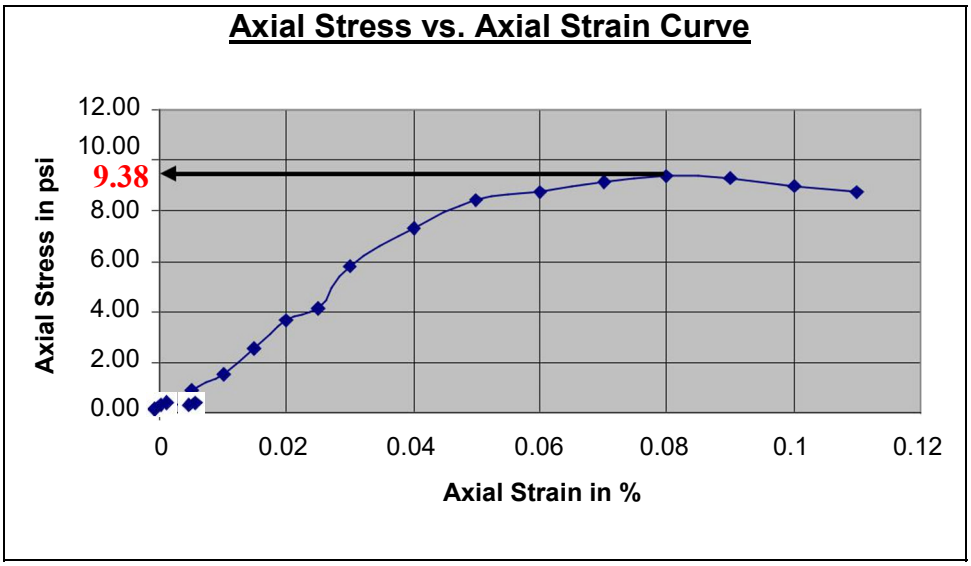
Unit strain, ε = L / L_0 = 0.33 / 3 = 0.11

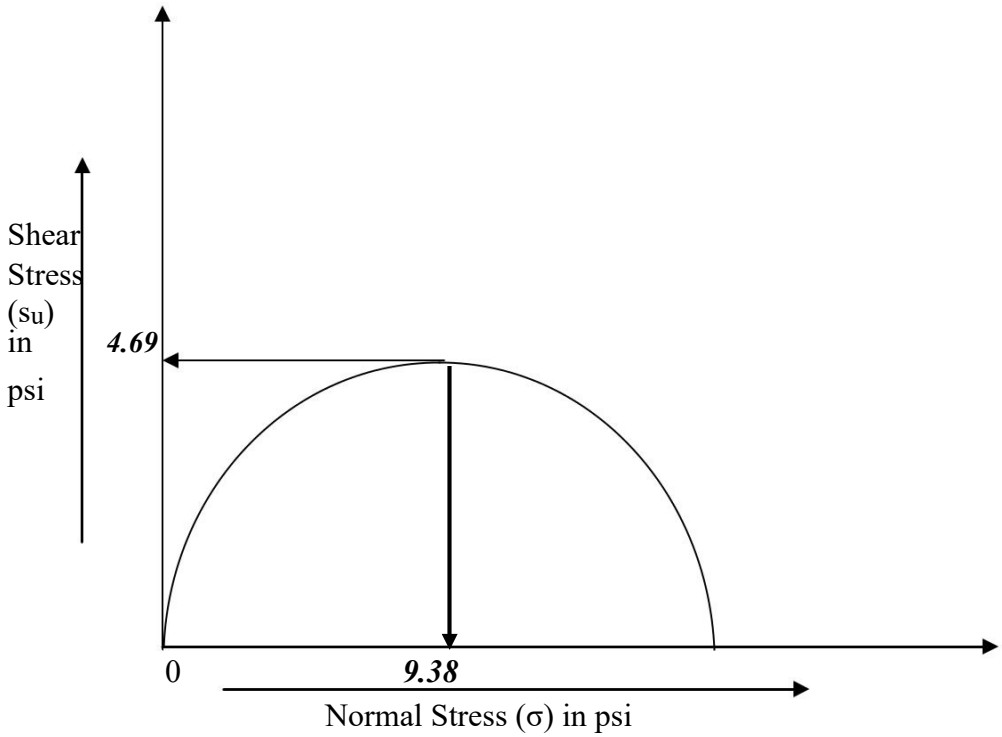
Initial area (A_0) = 1.767 inch^2

Corrected area, $A' = \frac{A_0}{1-\varepsilon}$ = 1.767 / (1 – 0.11) = 1.985 inch^2

Stress, σ = load / corrected area = 17.4 / 1.985 = 8.77 psi

Shear Stress, $\tau = \sigma / 2$ = 8.77 / 2 = 4.38 psi





From the stress-strain curve and Mohr's circle:

Unconfined compressive strength (q_u) = 9.38 psi Cohesion (c) = 4.69 psi

SECTION 9

COMPACTION TEST

References

- ASTM D 698 - Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbs/ft^3 (600 KN-m/m^3)).
- ASTM D 1557 - 12 - Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbs/ft^3 (2,700 KN-m/m^3)).
- Bowles, Joseph E., Engineering Properties of Soils and Their Measurement (4th Edition)
- Lambe, T.W., Soil Testing for Engineers.
- ASTM D 6026 - 06 – using Significant Digits in Geotechnical Data

General Discussion

This laboratory test is performed to determine the relationship between the moisture content and the dry density of a soil for a specified compaction effort. Laboratory compaction tests provide the basis for determining the percent compaction and molding water content needed to achieve the required engineering properties, and for controlling construction to assure that the required compaction and water contents are achieved.

Mechanical compaction is one of the most common and cost effective means of stabilizing soils. An extremely important task of geotechnical engineers is the performance and analysis of field control tests to assure that compacted fills are meeting the prescribed design specifications. Design specifications usually state the required density (as a percentage of the “maximum” density measured in a standard laboratory test), and the water content. Compaction achieves soil stabilization through the input of energy to produce a more dense soil structure than previously existed. The laboratory test supplies compaction energy to the soil in the mold by the impact of rammer. In the field the energy comes from rolling with sheeps-foot, wobble-wheel, or smooth-wheel rollers.

Generally, the compaction effort imparts to the soil:

1. An increase in shear strength since this is a function of unit weight
2. An increase in swell potential
3. An increase in unit weight
4. A decrease in shrinkage potential
5. A decrease in permeability
6. A decrease in compressibility

It has also been found that whether the field compaction water content is on the dry or wet side of the optimum Moisture Content also affects the end result – in particular items 2,4 and 6.

The optimum water content is the water content that results in the greatest density for a specified compaction effort. Compacting at water contents higher than (wet of) the optimum water content results in a relatively dispersed soil structure (parallel particle orientations) that is weaker, more ductile, less pervious, softer, more susceptible to shrinking, and less susceptible to swelling than soil compacted dry of optimum to the same density. The soil compacted lower than (dry of) the optimum water content typically results in a flocculated soil structure (random particle orientations) that has the opposite characteristics of the soil compacted wet of the optimum water content to the same density. The ultimate strength of soils compacted on the dry side of the OMC is higher at low strains and the soil usually fails abruptly by fracture (also called brittle failure). If compacted on the wet side of OMC the failure is gradual and often not clearly defined (a plastic failure) and strength is ultimately lower. The residual strength of the soil compacted on the dry side of optimum is nearly the same as the ultimate strength of soil compacted on the wet side of optimum.

The soil mass involved in the compaction process starts as a three-phase system of soil, water and air. During the initial trials considerable air is present, but the process results in a change of state with more soil and water being present. Even at the OMC there is considerably air present. On the wet side of optimum, the principal effect is to displace more and more air with water. If the process were completely efficient, it might be possible to replace all the air in the voids with water to produce a two-phase system (a condition of zero air voids). Since it is never possible to get all the air out of the voids, which would result in a condition of saturation, S = 100%, any compaction curve will always fall below the zero-air-voids curve.

For any given water content w the zero-air-voids dry unit weight γ_{ZAV} is computed as

$$\gamma_{ZAV} = G_s \gamma_w / (1 + w G_s)$$

Where, G_s = Specific gravity of soil solids

w = water content

γ_w = unit weight of water

The compaction effort is the amount of mechanical energy that is applied to the soil mass. Several different methods are used to compact soil in the field, and some examples include tamping, kneading, vibration, and static load compaction. This laboratory will employ the tamping or impact compaction method using the type of equipment and methodology developed by R. R. Proctor in 1933, therefore, the test is also known as the Proctor test.

Two types of compaction tests are routinely performed: (1) The Standard Proctor Test, and (2) The Modified Proctor Test. Each of these tests can be performed in three different methods as outlined in the attached Table 9-1. In the Standard Proctor Test, the soil is compacted by a 5.5 lb hammer falling a distance of one foot into a soil filled mould. The mould is filled with three equal layers of soil, and each layer is subjected to 25 drops of the hammer. The Modified Proctor Test is identical to the Standard Proctor Test except it employs, a 10 lb hammer falling a distance of 18 inches, and uses five equal layers of soil instead of three. There are two types of compaction moulds used for testing. The smaller type is 4 inches in diameter and has a volume of about 1/30 ft^3 (944 cm^3), and the larger type is 6 inches in diameter and has a volume of about 1/13.333 ft^3 (2123 cm^3). If the larger mould is used each soil layer must receive 56 blows instead of 25 (See Table 9-1)

Table 9-1: Alternative Proctor Test Methods

	Standard Proctor ASTM 698			Modified Proctor ASTM 1557		
	Method A	Method B	Method C	Method A	Method B	Method C
Material	≤20% Retained on No.4 Sieve	>20% Retained on No.4 ≤20% Retained on 3/8" Sieve	>20% Retained on No.3/8" <30% Retained on 3/4" Sieve	≤20% Retained on No.4 Sieve	>20% Retained on No.4 ≤20% Retained on 3/8" Sieve	>20% Retained on No.3/8" <30% Retained on 3/4" Sieve
For test sample, use soil passing	Sieve No.4	3/8" Sieve	¾" Sieve	Sieve No.4	3/8" Sieve	¾" Sieve
Mold	4" DIA	4" DIA	6" DIA	4" DIA	4" DIA	6" DIA
No. of Layers	3	3	3	5	5	5
No. of blows/layer	25	25	56	25	25	56

Note: Volume of 4" diameter mold = 944 cm^3, Volume of 6" diameter mold = 2123 cm^3 (verify these values prior to testing).

Equipment

The equipment used for this test are listed below (See Figure 9-1):

- Molds.
- Manual or mechanical rammer.
- Extruder.
- Balance.
- Drying oven.
- Mixing pan.
- Trowel.
- Sieve #4.
- Moisture cans.
- Graduated cylinder and
- Straight Edge

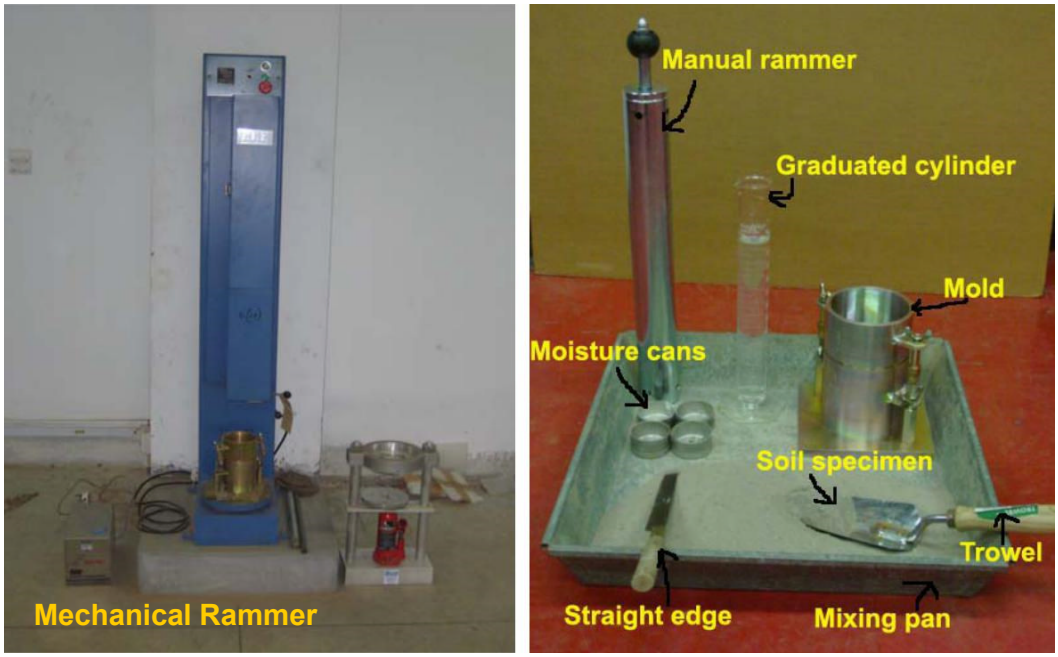


Figure 9-1: Equipment Used for Compaction Test.

Test Procedure

Test procedure for the compaction test is as under:

a. Depending on the type of mold a sufficient quantity of air-dried soil should be taken in large mixing pan. For the 4-inch mold approximately 10 lbs, and for the 6-inch mold thoroughly 15 lbs should be taken. The soil should be pulverized and run through the # 4 sieve.

b. The weight of the soil sample should be determined as well as the weight of the compaction mold with its base (without the collar) by using the balance and the weights should be recorded.

c. The amount of initial water should be computed to add by the following method:

(a) Water content has to be assumed for the first test is 8 percent.

(b) Water can be computed to add from the following equation:

$$Water\ to\ add\ (ml) = \frac{Soil\ mass\ in\ gm \times 8}{100}$$

It should be remembered that a gm of water is equal to approximately one ml of water.

d. The water should be measured out, added to the soil, and then mixed thoroughly into the soil using the trowel until the soil gets a uniform color (See Figure 9-2).



Figure 9-2: Mixing of Water with Soil.

e. The compaction mold should be assembled to the base, some soil should be placed in the mold and the soil must be compacted in the number of equal layers specified by the type of compaction method employed (See Figure 9-3). The number of drops of the rammer per layer is also dependent upon the type of moud used (See Table 9-1). The drops should be applied at a uniform rate not exceeding around 1.5 seconds per drop, and the rammer should provide uniform coverage of the specimen surface. It should be tried to avoid rebound of the rammer from the top of the guide sleeve



Figure 9-3: Assembly and Compaction of Soil in the Mold.

f. The soil should completely fill the cylinder and the last compacted layer must extend slightly above the collar joint. If the soil is below the collar joint at the completion of the drops, the test point must be ***repeated***. (Note: For the last layer, it should be watched carefully, and added more soil after about 10 drops if it appears that the soil will be compacted below the collar joint.)

g. The collar should be removed carefully and the compacted soil should be trimmed off so that it is completely even with the top of the mold using the trowel. Small bits of soil should be replaced that may fall out during the trimming process (See Figure 9-4).

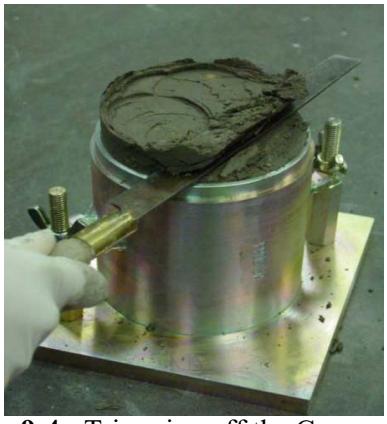


Figure 9-4: Trimming off the Compacted Soil

h. The compacted soil should be weighed while it's in the mold and to the base, and the mass should be recorded (See Figure 9-5). Then the wet mass of the can be determined by subtracting the weight of the mold and base.

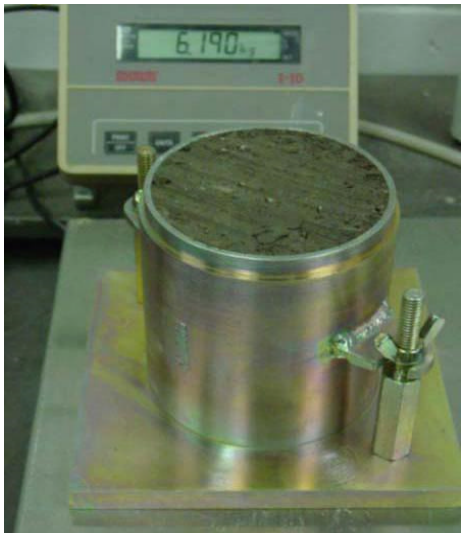


Figure 9-5: Weighing the Compacted Soil with Mold.

j. The soil should be removed from the mold using a mechanical extruder and soil moisture content samples must be taken from the top and bottom of the specimen (See Figure 9-6). The moisture cans should be filled with soil and the water content can be determined.



Figure 9-6: Use of Extruder to take the Compacted Soil Out and Collection of Sample for Moisture Content Determination.

k. The soil specimen should be placed in the large tray and the soil should be broken up until it appears visually as if it will pass through the # 4 sieve, add 3% more water based on the original

sample mass, and re-mix as in step d. Steps e through j has to be repeated until, based on wet mass, a peak value is reached followed by two slightly lesser compacted soil mass. This sequence is repeated a sufficient number of times that a curve of dry unit weight vs. water content can be drawn which has a zero slope (a maximum value) with sufficient points on either side of the maximum unit weight point to accurately define its location.

Data Analysis

Analysis of test data is done following the after mentioned procedure:

a. The moisture content of each compacted soil specimen can be calculated by using the average of the two water contents.

b. The wet density in grams per cm^3 of the compacted soil sample can be computed by dividing the wet mass by the volume of the mold used.

c. The dry density can be computed using the wet density and the water content determined in step 1. The following formula should be used:

$$\rho_d = \frac{\rho}{1 + w}$$

Where, w = moisture content in percent divided by 100, and ρ = wet density in grams per cm^3.=

d. The dry density values on the y-axis and the moisture contents on the x-axis should be plotted. A 'best fitting' smooth curve should be drawn connecting the plotted points.

e. On the same graph a curve of complete saturation or "zero air voids curve" should be drawn. The values of dry density and corresponding moisture contents for plotting the curve can be computed from the following equation:

$$w_{sat} \quad \frac{\rho_w}{\rho_d} - \frac{1}{G_s} \times 100$$

or,

$$\rho_d = \frac{\rho_w}{\frac{w}{100} + \frac{1}{G_s}}$$

Where,

ρ_d = dry density of soil grams per cm^3.

G_s = specific gravity of the soil being tested (assume 2.70 if not given).

ρ_w = density of water in gm per cm^3 (approximately1 g/cm^3).

w_{sat} = moisture content in percent for complete saturation.

Example Calculations:

G_s=2.7 (assumed)

ρ_w=1.0 g/cm^3

Assumed w_{sat}%	Calculated ρ_d (g/cm^3)
8	2.22
10	2.13
12	2.04
14	1.96
16	1.89
18	1.82

f. The optimum moisture content and the maximum dry density should be identified and recorded. It should be made sure that the method of compaction used (e.g., Standard Proctor, Method A) on data sheet has been recorded.

Significant Figures

1. The value of dry density should be expressed to 3 or 4 significant digits.
2. The value of water content should be expressed to nearest 0.1.

WORKED OUT EXAMPLE

Compaction Test

Test Serial No.: **09**
Test Method: **Standard Proctor, Method A (ASTM 698)**

Date Tested: **October 05, 2008**

Tested By:

Project Name: **Airport Road – Rokeya Sharoni Link Road Project**

Sample Number: **Bag-1, 2'-6″**

Visual Classification of Soil: **Gray silty clay, trace fine sand, low plasticity, moist, CL**

Density Determination:
Mould Volume=944 cm^3

Compacted Soil - Sample No	**1**	**2**	**3**	**4**	**5**
Assumed water content, w_a%	**10**	**12**	**14**	**16**	**18**
Actual average water content, w%	**8.0**	**11.0**	**12.8**	**15.65**	**17**
Mass of compacted soil and mould (gm)	**3457.2**	**3721.2**	**3909.0**	**3782.5**	**3715.2**
Mass of mould (gm)	**1933**	**1933**	**1976.0**	**1849.5**	**1782.2**
Wet mass of soil in mould (gm)	**1524.2**	**1788.2**	**2176**	**2149**	**2082**
Wet density, ρ , (g/cm^3)	**1.615**	**1.894**	**2.093**	**1.959**	**1.888**
Dry density, ρ_d , (g/cm^3)	**1.50**	**1.71**	**1.86**	**1.69**	**1.61**

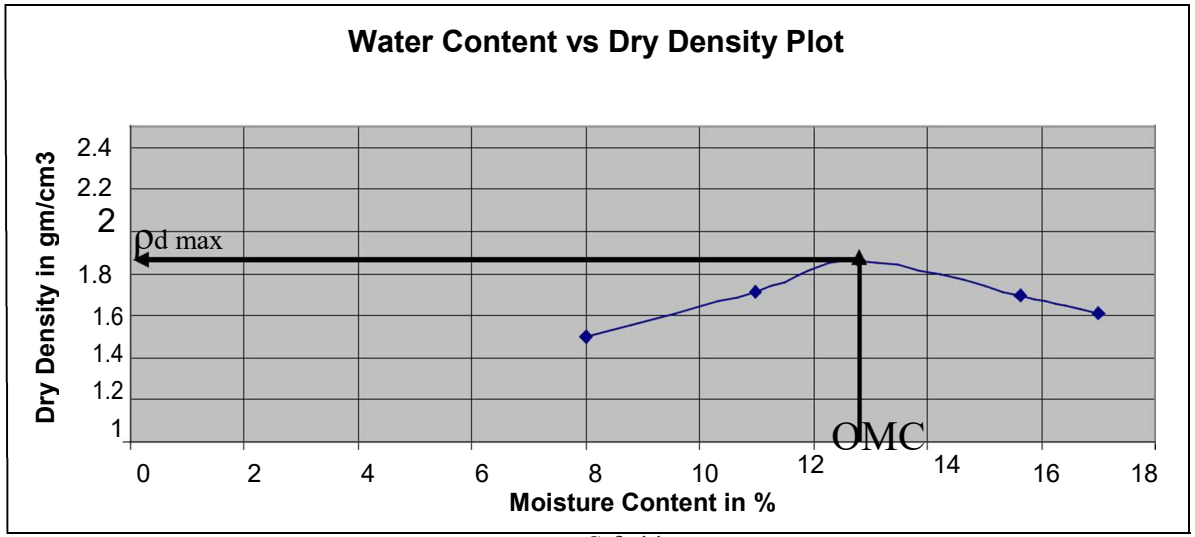
Sample Calculations (For Sample No 1)

Wet density, ρ = 1524.2 / 944 = 1.615

Dry density, ρ_d = 1.615 / (1 + 0.08) = 1.495 ≈ 1.5

Water Content Determination:

Compacted Soil - Sample no.	1		2		3		4		5	
Water content - Sample no.	1A	1B	2A	2B	3A	3B	4A	4B	5A	5B
Moisture can number	1	2	3	4	5	6	7	8	9	10
M_C = Mass of empty, clean can (gm)	7.78	7.83	7.71	7.9	7.5	7.9	8.1	7.6	7.7	7.65
M_{CMS} = Mass of can + moist soil (gm)	11.78	11.05	10.71	11.1	10.7	12 .0	11. 1	10.2	10.3	10.33
M_{CDS} = Mass of can + dry soil (gm)	11.48	10.81	10.41	10.75	10.3	11.52	10.70	9.84	10.02	9.92
M_S = Mass of soil solids (gm)	3.70	2.98	2.7	2.85	2.84	3.62	2.60	2.24	2.32	2.27
M_W = Mass of pore water (gm)	0.29	0.24	0.30	0.35	0.40	0.53	0.40	0.35	0.4	0.39
w = Water content, w%	7.9	8.1	11.1	10.9	12.5	13.1	15.3	16.0	17. 1	17.6



SECTION 10

PERMEABILITY TEST

References

- ASTM D 2434 - Standard Test Methods for Permeability of Granular Soil (Constant Head).
- Bardet, Jean-Pierre (1997), Experimental Soil Mechanics. Prentice-Hall, Inc.
- Lambe, T.W., Soil Testing for Engineers.
- Bowles, Joseph E., Engineering Properties of Soils and Their Measurement (4th Edition)
- ASTM D 6026 - 06 – using Significant Digits in Geotechnical Data

General Discussion

This test method is for the determination of the coefficient of permeability of soil which is a constant of proportionality to the ease with which fluid passes through a porous medium. Soils are permeable (water may flow through them) because they consist not only of solid particles, but a network of interconnected pores. The degree to which soils are permeable depends upon a number of factors, such as soil type, grain size distribution and soil history. This degree of permeability is characterized by the coefficient of permeability.

In the design of geotechnical engineering projects, one of the most important soil properties of interest to the soils engineer is permeability. To some degree, permeability will play a role in the design of almost any structure. For example, the durability of concrete is related to its permeability. In designs that make use of earthen materials (soils and rock, etc.) the permeability of these materials will usually be of great importance.

To illustrate the importance of permeability in geotechnical design, consider the following applications where knowledge of permeability is required:

a. Permeability influences the rate of settlement of a saturated soil under load.

b. The stability of slopes and retaining structures can be greatly affected by the permeability of the soils involved.

c. The design of earth dams is very much based upon the permeability of the soils used.

d. Filters to prevent piping and erosion are designed based upon their permeability.

e. The rate of flow to wells from an aquifer is dependent on permeability.

f. The performance of landfill liners is based upon their permeability.

The coefficient of permeability, *k*, is a product of Darcy's Law. In 1856, Darcy established an empirical relationship for the flow of water through porous media. His relationship has evolved into Darcy's Law, which states:

$$Q = kiA$$

Where:

Q = flow rate (volume/time)

i = hydraulic gradient (unit less)

A = cross-sectional area of flow (area)

k = coefficient of permeability (length/time)

It should be noted that the coefficient of permeability is often referred to as hydraulic conductivity by hydrologists and environmental scientists. In their notation, permeability has a entirely different definition. Some typical values of the coefficient of permeability are shown in Table 10-1:

Table 10-1: Some Typical Values of the Coefficient of Permeability.

Soil	**Coeff. of Permea., k cm/sec**	**Degree of Permeability**
Gravel	$>10^{-1}$	Very high
Sandy gravel, clean sand, fine sand	$10^{-1} > k > 10^{-3}$	High to Medium
Sand, dirty sand, silty sand	$_{10}$-3 $> k > 10^{-5}$	Low
Silt, silty clay	$_{10}$-5 $> k > 10^{-7}$	Very low
Clay	$<10^{-7}$	Virtually impermeable

The coefficient of permeability of a homogeneous, isotropic soil mass depends primarily on the following factors:

a. The viscosity of the pore fluid (usually water). The viscosity of water decreases with the increasing temperature which increases the coefficient of permeability. The coefficient of permeability is standardized at 20°C, and the coefficient of permeability at any temperature T is related to k_{20} by the following ratio

$$k_{20} = k_T \frac{\eta_T}{\eta_{20}} .$$

Where, η_T = Viscosity at temperature T^{0}C

η_{20} = Viscosity at temperature 20°C

b. The void ratio (e)of the soil. The relation of void ratio and coefficient of permeability can be given as:

$$k_2 = k_1 \frac{e_2^3 (1+e_1)}{e_1^3 (1+e_2)}$$

c. The size and shape of the soil particles. Angular and platy particles tend to reduce k more than when rounded and spherical particles predominate in the soil. According to Allan Hazen for clean sand and gravels coefficient of permeability could be expressed using the D_{10} grain size approximately as

$$k = 100\ D_{10}^2 \text{ cm/s} \qquad (0.01 < D_{10} < 0.3\text{cm})$$

d. The degree of Saturation S. As the degree of saturation increases, the apparent coefficient of permeability also increases.

A number of different methods for determining the coefficient of permeability for soils exist, including in-situ (field) methods and laboratory methods. In the laboratory, two common tests are generally used to determine this soil property. These two tests are the falling head permeability test and the constant head permeability test. Which test is used depends upon the type of soil to be tested. For soils of high permeability (sands and gravels) a constant head test is used. For soils of intermediate to low permeability, a falling head test is used.

In the constant head test, a constant total head difference is applied to the soil specimen, and the resulting quantity of seepage can then be measured. This works very well for coarse-grained soils, but with clays and silts, the quantity of seepage is much too small to be accurately measured. The falling head test is different in that it does not fix the total head difference across the specimen. Instead, a standpipe is connected to the inflow, and the water level in this standpipe is then allowed to drop as water flows through the specimen. This will not work well for coarse-grained soils, because they are so permeable that the head drops too rapidly to be accurately measured.

Neither the constant-head nor the falling-head laboratory method provides completely reliable value for the coefficient of permeability of a soil as conditions at the boundary are not the same as the laboratory and orientation of the in situ stratum to the flow of water is probably not duplicated. Laboratory tests often require the use of deaerated water. It has some merit, since the influence of a bubble of air coming

out of solution in the small laboratory sample may be very important compared with the effect of a bubble of the same size in the field soil.

Constant Head Test

Equipment.

a. **Special – Permeameter**. The equipment (permeameter) consist of the items as shown in Figure 10-1. It is generally used for soil samples with maximum particles diameter lower than 7.5 mm. The perspex cylinder has a perforated plate, placed in the lower part and a bayonet clutch, for keeping the sample. The piezometric glass tubes are placed in vertical position on a proper wooden panel. Connections between permeability cell and piezometric glass tubes are made by rubber tubes. The equipment has a aspirator tank, 2 litres capacity, for applying the hydraulic head.

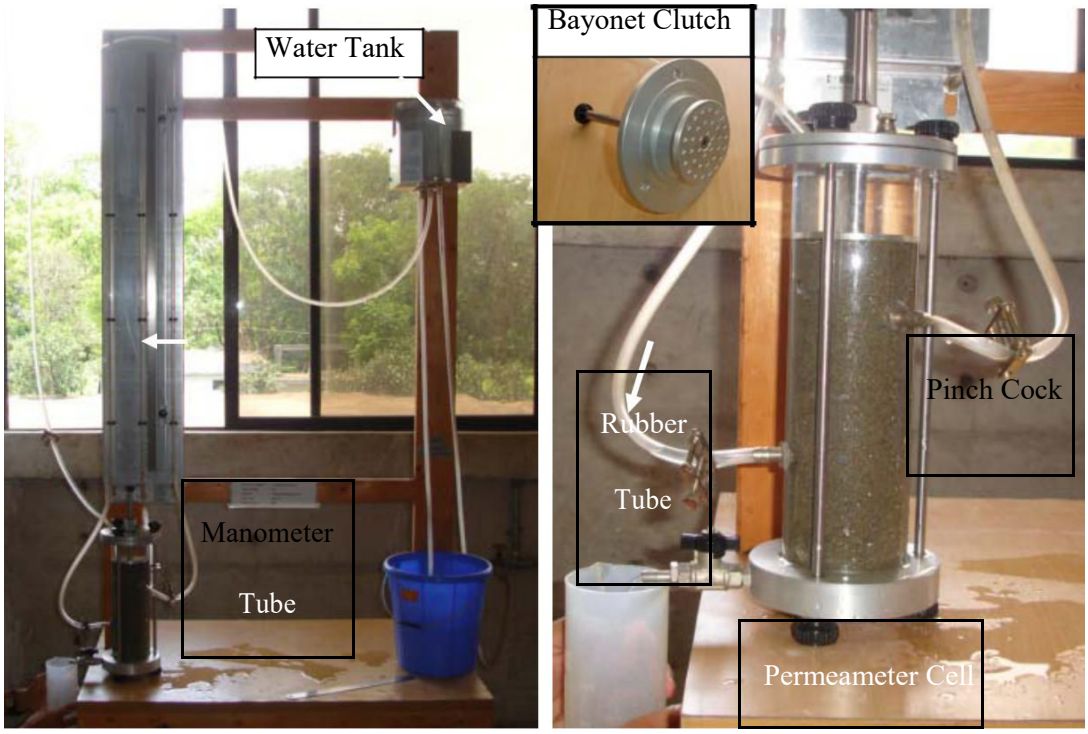


Figure 10-1: Equipment for Constant Head Permeability Test.

b. **Other Equipment**

- Measuring cylinders with capacity from 100 to 1000 ml.
- De-airing tank for water 7 litre capacity (optional**).**
- Nylon tubing 10 m coil (optional).

- Water trap (Optional).
- Portable vacuum pump (Optional).
- Rubber tube 6.5 x 16.5 mm dia 2 m long for vacuum (optional).
- Vacuum control panel for permeameter(optional).

c. **General**

- Balance (0.1 g sensitivity).
- Drying oven.
- Desiccators.
- Scale.
- Thermometer (0.1º sensitivity).
- Stop clock.
- Evaporating dish.
- Funnel.
- Stop watch.

Assembly of Apparatus.

The apparatus should be arranged as shown in Figure 10-2.

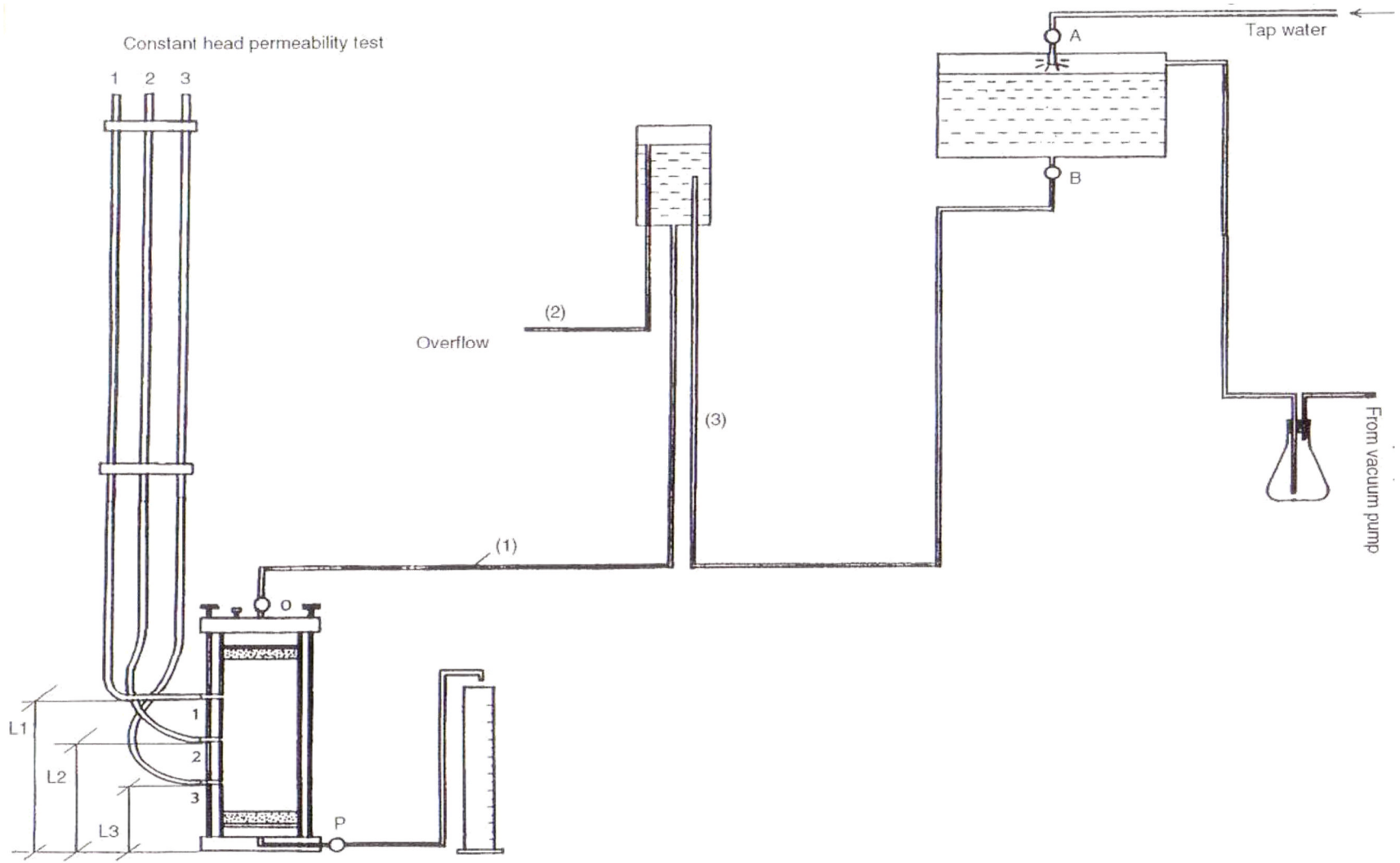


Figure 10-2: Schematic Diagram Showing Arrangement of Equipments for Constant Head Permeability TesT

Preparation of Water for the Test

Generally, when a permeability test is performed in the laboratory for use in the design of a civil engineering project, the water used should be identical to the native water, that in the field. For all but the most important projects, such water can be practically unattainable. Therefore, de-aired tap water is generally used. Tap water contains dissolved oxygen, and when flowing through a porous media, such as soil, this dissolved oxygen can actually come out of solution, and deposit air bubbles within the pore structure of the soil specimen. These tiny air bubbles would impede the flow of water through the specimen, thereby giving erroneous results. In order to circumvent this problem, water used in the permeability test should be freed of all (or most) of its dissolved oxygen content. This is done by spraying water into a sealed tank kept under vacuum pressure. ***If vacuum pump is not available, water may be boiled for a long duration, stirred and cooled, before supplying into the system.***

Preparation of Sample for the Test

A representative sample should be obtained from original soil. Sieving should be done the 6.25 mm sieve to eliminate the particles larger than 1/12 of the diameter of the permeameter cell. From the obtained sample a portion is to be extracted to determine the moisture content and calculate the dry weight. The test sample should not be dried so as to cause difficulty during the saturation stage. A large plastic funnel connected to a flexible tube about 35 cm long should be used. The stored sample should be taken and with a scoop the material should be poured into the funnel. The tube should be kept vertical and the permeameter should be started to fill checking that the distance from the end of the tube to the surface of the placed material is maintained constant. The tube should be moved around in a spiral motion from the periphery towards the center, to avoid the segregation of fine particles. Different densities can be obtained by varying the height of drop and tube diameter.

Saturation of Dry Sample

a. When preparation of dry samples either by pouring or compacting has been completed, the upper filler, the top plate should be fitted and the position of the pistons should be adjusted described above for samples poured into water. The connection lines between glands should be checked and manometer tubes are properly fitted.

b. The pitch cocks of the tubes near the glands should be tighten, to avoid sucking of air.

c. The vacuum pump should be run to apply vacuum to the sample. At the same time the lower valve of the cell should be opened to flow de-aired water within the sample.

d. When the level of water is about 1/3 of the sample height, the lower valve should be closed so that vacuum can operate to suck any air bubbles. When no air bubbles are observed, the lower

valve should be re-opened and let the water raise up to 2/3 of the sample height. The procedure as above should be repeated with the lower valve closed.

e. Flow of water should be continued upwards to reach the upper filter. The procedure should be repeated with the vacuum until no air bubbles are observed within the cylinder.

f. Vacuum should be lowered to zero and the pinch cocks should be released from the manometer tubes, so that water can saturate the lines. To obtain a complete saturation it may be necessary to remove the manometer tubes from the panel operating on the clips that fasten the tubes.

g. Lower valve of the permeameter cell and lower valve of the de-airing tank should be closed. The piece of tube should be removed and the connection should be checked from constant head water tank to the top of the cylinder is properly fitted.

h. It should be remembered:

(1) In the permeability tests the presence of air bubbles inside the sample can affect the precision of measurements.

(2) During the test formation of air bubbles reduces the hydraulic flow and gives lower values of lower permeability.

(3) To eliminate this type of error use of de-aired water and saturation of the sample before the test are recommended.

Test Performance Test and Data Recording

a. It must be checked that the valve at the base of the cell is open and the pinch cock of the flexible tube connected to the top outlet should be released, to the flow water downwards.

b. When water flows from the valve at the base of the cell and the water level in the monometer tubes becomes stable, test measurements should be started. A measuring cylinder of suitable capacity should be placed under the outlet valve and at the same time the stop watch should be started. A proper time interval should be selected for data recording so that an amount of an about 50-100cm^3 of water is measured. If necessary the distance from the parameter to the tank to obtain the required hydraulic head.

c. For each time interval record the following data:

(1) Elapsed time from the beginning of measurement.

(2) Amount and temperature of water within the graduated cylinder.

(3) Level of water in the manometer tubes.

d. The amount of water should be divided by the time interval to calculate the rate of flow "q" in cm/sec and this data vs. time should be plotted to evaluate when the flow becomes steady. A better evaluation is also obtained by plotting the rate of

flow "q" vs. $\frac{1}{\sqrt{time}}$.

e. When the rate of flow is steady, the test can be stopped. If a series of tests with different hydraulic gradients is required, the procedure herein described should be repeated by increasing the height of the inlet reservoir.

f. When a sufficient number of readings have been obtained, the valve at the base of the cell should be closed and the average height of the sample should be measured.

g. From the sample a certain amount should be taken for moisture content measurement and sample final unit weight calculation.

Data Analysis.

a. From the measurements carried out the following data are available:

M_i	= Initial wet mass of the soil used to prepare the sample (gm)
M_f	= Mass of the dried soil remaining after setting up the sample (gm)
W_0	= Moisture content of the initial sample (%)
A_c	= Cross section of the sample (cm^2)
L_c	= Mean height of the sample after the test (cm)
L_1,L_2,L_3	= Distance of the gland points from the base of the permeameter (cm)

b. For a given time interval (min) measured in the conditions of steady flow the following data are recorded:

Q	= Volume of water collected in the graduated cylinder (cm^3)
H_1	= Level of water within the manometer tube connected to the lower gland (cm)
H_2	= Level of water within the manometer tube connected to the intermediate gland (cm)
H_3	= Level of water within the manometer tube connected to the tube upper gland (cm)
T	= Temperature of water (^oC)

c. The coefficient of permeability (m/sec) can be calculated from the following:

$$k = \frac{Q \times 10^{-2}}{60 \times i \times t \times A_c}$$

d. The hydraulic gradient "i" is the average of i_1, i_2 and i_3.

$$i_1 = \frac{H_1 - H_2}{L_1 - L_2}\ ;\ i_2 = \frac{H_2 - H_3}{L_2 - L_3}\ ;\ i_3 = \frac{H_3 - H_1}{L_3 - L_1}$$

e. Theoretically the 3 values of "i" should be similar. During the test some discrepancies are also possible for different reasons:

(1) The sample has not been prepared uniformly.

(2) A manometer tuber does not work properly.

(3) Within the sample a flow of the finest particles or a preferential flow of water has been installed.

f. To standardize the permeability to 20^0 C, the calculated values can be corrected as follows:

(1) Ratio of the viscosity of water for different temperatures:

$$k_{20} = k_T \frac{\eta_T}{\eta_{20}}\ .$$

(2) The dry density of the sample can be calculated as follows:

$$\gamma_d = \frac{\frac{M_i}{(1 + \frac{W_0}{100})} - M_f}{A_c * L_c}$$

Variable Head Test

Equipment

a. **Special – Permeameter** The equipment (Figure 10-3) allow operating with both undisturbed and remolded samples. The range of permeability that can be investigated by variable head tests can vary from 10^{-2} to 10^{-6} cm/sec. Lower permeability coefficients are generally measured by indirect methods with undisturbed specimens (generally saturated) in the consolidation tests. Figure 10-3 shows the permeameter used in variable head test.

(1) Cutting edge compaction permeameter, 4" in diameter, suitable for undisturbed sampling or for preparing test samples from block samples. The cell is with 75 microns circular filter, perforated base plate, water container with overflow tube, valves and connection lines.

(2) Measuring panel consisting of 4 glass standpipe tubes with different diameters, placed vertically on a wooden panel, used apply and measure the hydraulic falling head.

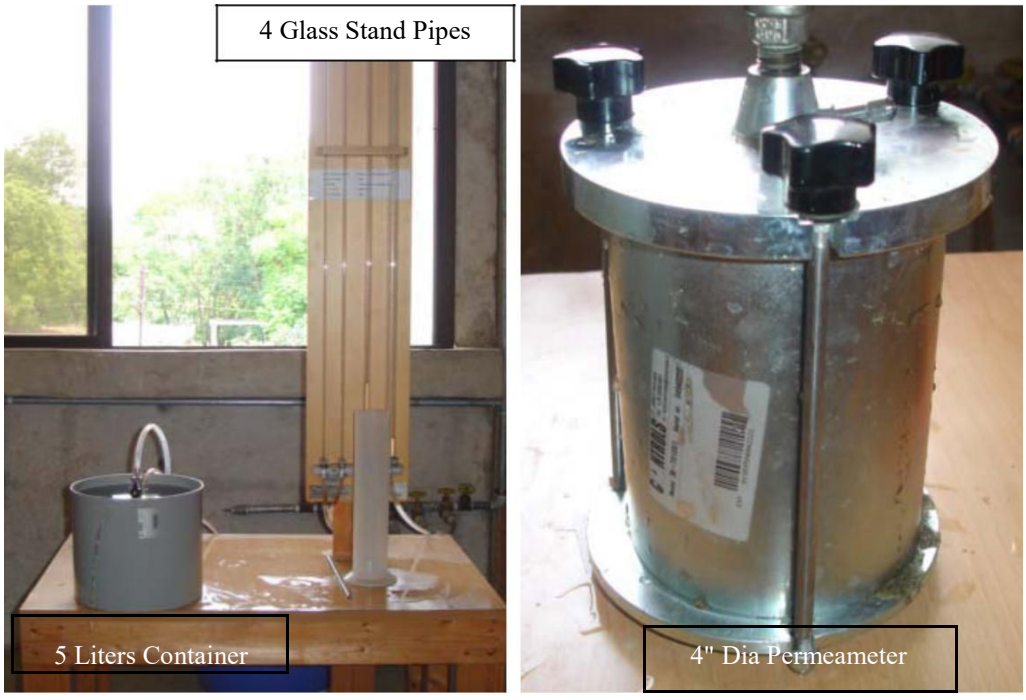


Figure 10-3: Equipments for Variable Head Permeability Test.

b. **General**.

- Balance (0.1 g sensitivity).
- Drying oven.

- Desiccators.
- Scale.
- Thermometer (0.1º sensitivity).
- Stop clock.
- Evaporating dish.
- Funnel.
- Stop watch.

c. Observations in Equipment

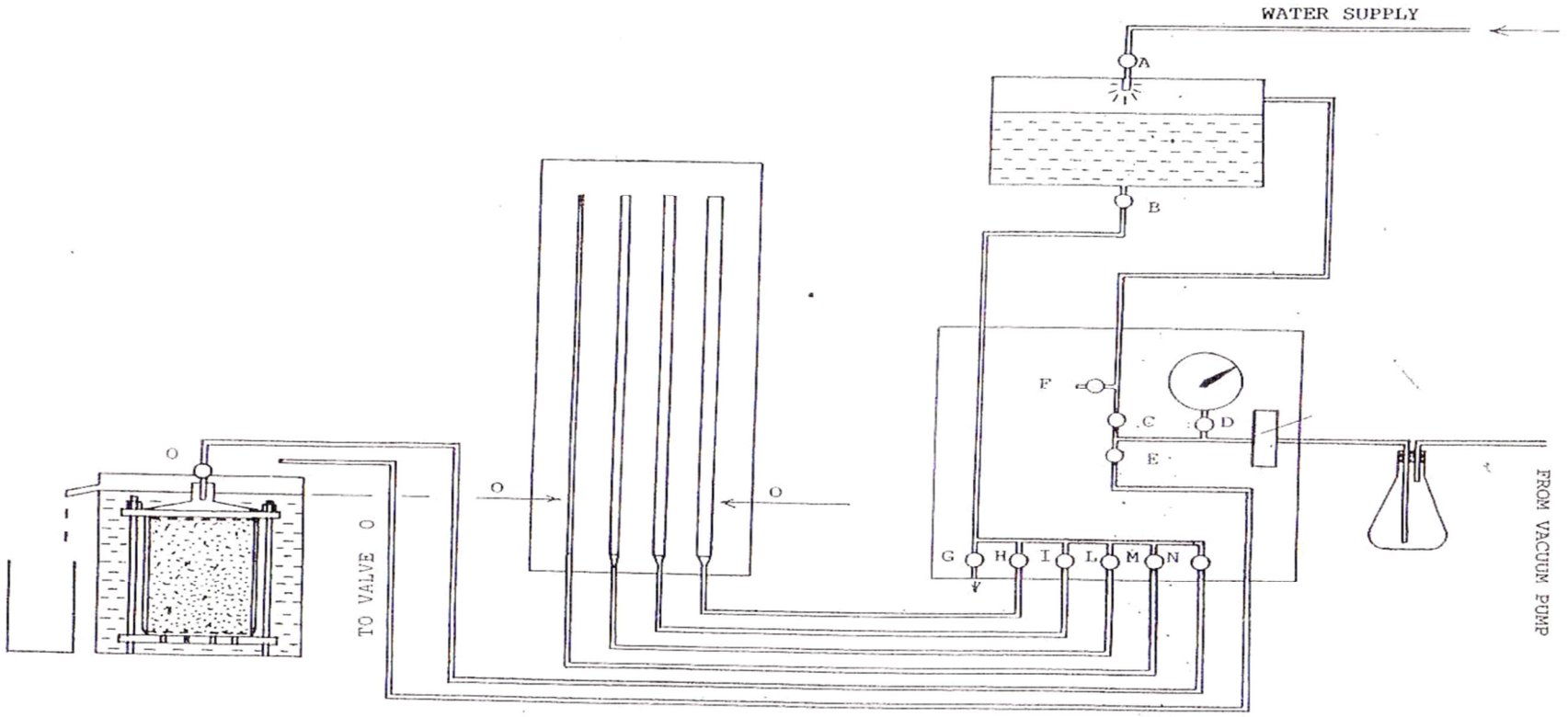
(1) As indicated before for constant head permeability tests, the apparatus is completed by the same equipment to produce de-aired water and to saturate the sample.

(2) Unlike the scheme of assembly for constant head test, the control panel is connected to the permeability cell and the measuring panel with glass standpipes.

(3) The control and panel has been designed in order to be used for constant and falling head

Assembly of Equipment

The apparatus should be arranged as shown in Figure 10-4.



PROCEDURE	A	B	C	D	E	F	G	H	I	L	M	N	O
Saturating the sample	X	O	X	O	O	O	X	X	X	X	X	O	O
Filling pipelines	X	O	X	X	X	O	X	O	O	O	O	X	X
Performing the test	X	X	X	X	X	O	X	X	O	X	X	O	O

O open valve, X closed valve

Figure 10-4: Schematic Diagram Showing Arrangement of Equipment for Variable Head Permeability Test

Preparation of Sample.

a. For taking the undisturbed sample, use of the proper cutting edge cylinder should be made, by pushing it directly into the soil; it should be taken care to prepare the surface of the sample so that it is reasonably plane and undisturbed.

b. During this procedure, it should be made sure that the sample is a tight fit in the body and that there are no cavities around the perimeter through which water could pass compromising the test results. If necessary, the finer particles of the samples should be used for packing the cavities.

c. The cylinder should be pushed into the soil, by taking whichever device to avoid bending movements, which could disturb the soil and create undesired micro cracks along the side surfaces.

d. When the cylinder contains the required length of sample plus a small excess, the sample should be cut off along the base surfaces by means of wire saw and straightedge and continue till the two surfaces are perfectly plane. During this operation sleeking the surface should be avoided, which can close the micro pore of the soil and reduce the measured permeability.

e. Finally, it can be pointed out that, if the soil sampling is performed with its axis horizontal (or parallel to bedding) it is possible to measure the horizontal permeability.

Saturating Sample.

a. The cell should be prepared by placing the cell body, cutting edge downwards, on the base plate, separated by the filter disc and a strip of filter paper.

b. Then another strip of filter paper should be placed on the upper surface of the sample, the filter disc and the permeameter head should be assembeled, taking care that the sealing ring is in the correct position. In this procedure, the line from valve N to O is not connected.

c. The cell should be placed the immersion tank and filled it with de-aired water up to the overflow level. It should be made sure that during these operations and after during the test the temperature of water is equal to room temperature.

d. Valve O should be opened and checked water level in the tank falls owing to water absorption by the sample; this process should be allowed to continue overnight for at least 12 hours or longer for low permeability soils.

e. Valve O should be connected to the line from valve E, both valves should be opened. The saturation process should be continued by applying a low suction by vacuum pump and increasing it if necessary, until the water is drawn up into the line from valve O to valve E. Before turning the vacuum pump off, it must be checked that no air bubbles are coming up through the vacuum line.

f. Vacuum should be reduced to a minimum, valves C and F should be opened for establishing pressure equilibrium. Valve O should be opened and the tube coming from valve E should be disconnected. Then valve C and F should be closed.

g. The hydraulic line should be connected from valve N to valve O, checking the lines are completely filled with water. During these operations air bubbles entrapped over valve O must be avoided.

h. Subsequently the manometer tubes should be filled by opening valves H, I, L, M. At the same time the sealing of each line must be checked.

Test Performance Test and Data Recording.

a. It should be checked valves B and O are closed (Figure 10-4) and the valve corresponding to the glass standpipe to be used for the test is open.

b. It should be checked that the water level in the water container is up to the overflow and a thermometer should be placed in it. Then a glass container of suitable capacity should be placed for pouring water coming out from overflow tube.

c. The lower part of the paper should be filled in, indicating the sample height L, the level of water H_0 in the standpipe used for the test and the temperature T measured in the outflow tank.

d. It should be checked that valve G is closed, N is opened and only one valve of H, I, L, M is opened.

e. Valve O should be opened and at the same time the stop watch should be started. At present intervals of time (generally the procedure suggests to follow a geometrical progression of times: 1min, 2, 3, 4, 8, 15 etc.) the level of water in the manometer tube should be recorded and indicated it as H_1.

f. When a sufficient number of readings have been recorded, the test should be stopped and calculations should be made according to the procedure indicated in the following paragraph.

g. Before dismantling the apparatus the mold with sample should be taken from the water tank and the final height of the sample should be measured in several points to obtain an average value.

h. From the soil sample a certain amount should be taken for moisture content measurement and sample final unit weight calculation.

Data Analysis.

a. The coefficient of k can be computed from

$$k = 2.3 \frac{aL}{A * (t_2 - t_o)} \log_{10} \frac{H_o}{H_1}$$

where, a = cross sectional area of standpipe

L = Length of the soil sample in permeameter

A = cross sectional of permeameter

L = Length of soil of soil sample in permeameter t_0 = time when water in stand pipe is at h_0

t_1 = time when water in stand pipe is at h_1

H_0, H_1 = heads between which the permeability is desired

b. To standardize the permeability to 20^0 C, the calculated values can be corrected as follows:

$$k_{20} = k_T \frac{\eta_T}{\eta_{20}}$$

Method of Presentation of Results and Report Writing

The results of permeability test are usually presented in the form of a plot of some function of void ratio, e, against some function of permeability, k_{20} .

Often two plots are made:

a. k vs. e^2, $e^2/(1+e)$, $e^3/(1+e)$.

b. e vs. log k

Significant Figures

1. The value hydraulic head should be expressed to 3 significant digits.
2. The value hydraulic conductivity should be expressed to 2 or 3 significant digits.
3. The value of velocity should be expressed to 3 significant digits.

WORKED OUT EXAMPLE

Permeability Test (Constant Head Method)

Test Serial No: 10

Date Tested: **October 25, 2008**

Tested By: Specific Gravity of Soil, G_S: **2.58**

Project Name: **Hatirjheel Project** Sample Number: **S-1, 25'-6″**

Visual Classification of Soil: **Coarse Sand**

Initial Wet Mass of the Soil Used, M_i

Weight of empty permeameter = 2709.5 gm

Weight of permeameter with filled wet soil = 3973.2 gm

Weight of wet soil, M_i = (3973.2 - 2709.5) = 1263.7 gm

Moisture Content of the Sample, W_0:

Sample No	**1**	**2**	**3**
Container #	**5**	**7**	**24**
Container Wt (gm)	**60.1**	**59.6**	**60.1**
Wet Wt + Container (gm)	**148.0**	**158.6**	**145.9**
Dry Wt + Container (gm)	**138.5**	**147.8**	**136.7**
Wt of Water (gm)	**9.5**	**10.8**	**9.2**
Dry Wt of Soil (gm)	**78.4**	**88.2**	**76.6**
Water Content (%)	**12.11**	**12.24**	**12.01**
Average (Water Content)		**12.12%**	

Mass of Dried Soil Remaining after Setting-up the Sample, M_f

Weight of dry soil used, M_f = 1263.7 / (1+w)

= 1263.7 / (1+0.1212) = 1127.1 gm

Permeameter Diameter: 7.5 cm

Permeameter Area, A_c: 44.18 cm^2

Determination of Hydraylic Gradient

Obsn No	L_1 cm	L_2 cm	L_3 cm	H_1 cm	H_2 cm	H_3 cm	i_1	i_2	i_3	Average i
1.	7.5	12.5	17.5	45.5	18.5	0	5.4	3.7	4.55	4.55
2.	7.5	12.5	17.5	43.0	19.0	0	4.8	3.8	4.3	4.3
3.	7.5	12.5	17.5	51.5	23	0	5.7	4.6	5.15	5.15

Sample Calculations (For Observation # 1)

$$i_1 = \frac{H_1 - H_2}{L_1 - L_2} = \frac{45.5 - 18.5}{7.5 - 12.5} = \frac{27}{-5} = -5.4 \qquad i_2 = \frac{H_2 - H_3}{L_2 - L_3} = \frac{18.5 - 0}{12.5 - 17.5} = \frac{18.5}{-5} = -3.7 \qquad i_3 = \frac{H_3 - H_1}{L_3 - L_1} = \frac{0 - 45.5}{17.5 - 7.5} = -\frac{45.5}{10} = -4.55$$

Obsn No	Sample Length, L_c (cm)	Temp. (˚C)	Amount of Water Collected, Q (cm^3)	Collection Time, t (sec)	Hydraulic Gradient, i	Permeability at T˚C, K_T (cm/sec)	Viscosity Ratio, η_T/η_{20}	Permeability at 20˚C, K_{20} (cm/sec)	Void Ratio, e
1.	20.7	24	500	42	4.55	0.0592	0.908	0.0538	1.093
2.	19.8	24	700	71	4.3	0.0519	0.908	0.047	1.002
3.	19.3	24	1000	92	5.15	0.048	0.908	0.0436	0.952

Note: Weight of wet soil was kept constant for each test.

Sample Calculations (For Observation # 1)

Coefficient of permeability at T°C, K_T

$$= \frac{Q}{i \times t \times A_c} = \frac{500}{4.55 \times 42 \times 44.18} = 0.0592\ cm/\sec$$

Permeability at 20°C, K_{20} $= k \dfrac{\eta_T}{\eta_{20}} = 0.0592 \times 0.908 = 0.0538 cm/\sec$.

Volume of Soil Used, V_s = Weight of dry soil used / (Sp. Gravity of soil x Density of water) = 1127.1 / (2.58 x 1) = 436.86 cm^3

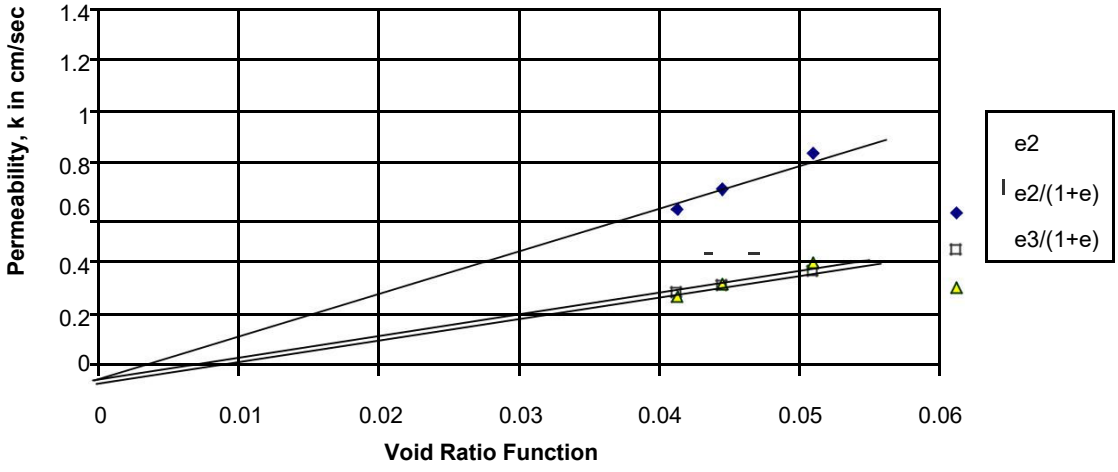
Total volume of soil, V = π x $(d/2)^2$ x L_c = π x $(7.5/2)^2$ x 20.7 = 914.5 cm^3

Volume of soils, V_v = (V – V_s) = (914.5 – 436.86) = 477.64 cm^3

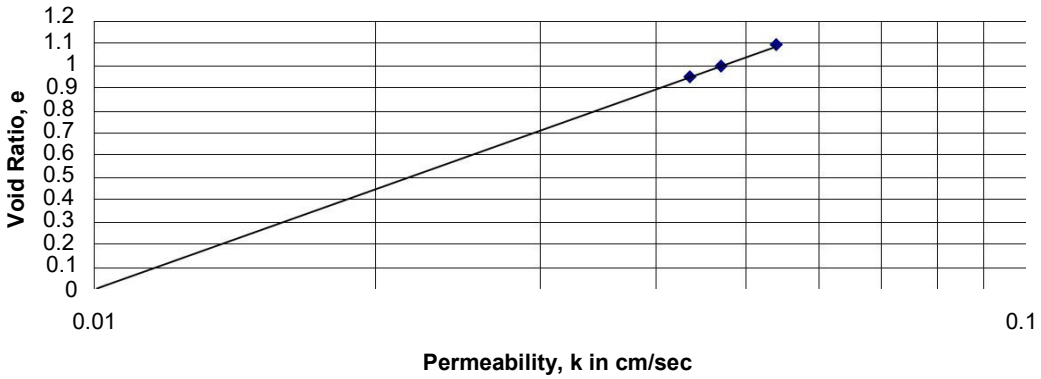
Void ratio, e = V_v / V_s = (477.64 / 436.86) = 1.093

Observation No	Void Ratio, e	e^2	$e^2/(1+e)$	$e^3/(1+e)$
1.	1.093	1.195	0.571	0.624
2.	1.002	1.004	0.501	0.503
3.	0.952	0.906	0.464	0.442

Permeability vs. Void Ratio Function Plot



Void Ratio vs. Permeability Plot



WORKED OUT EXAMPLE

Permeability Test (Variable Head Method)

Date Tested: **October 29, 2008**

Tested By: _____ Specific Gravity of Soil, G_S: **2.61**

Project Name: **Hatirjheel Project** Sample Number: **S-2, 35'-0″**

Visual Classification of Soil: **Sandy Silt, SM**

Obsn No	Weight of Empty Permeameter in gm	Weight of Permeameter with Wet Soil in gm	Weight of Wet Soil in gm	Moisture Content of Sample in %	Weight of Dry Soil Used in gm
1.	2480.0	4062.1	1582.1	7.5	1471.7
2.	2480.0	4011.1	1531.1	7.5	1424.2 8
3.	2480.0	4232.9	1752.9	7.5	1630.6

Sample Calculations (For

Observation # 1)

Weight of empty permeameter = 2480.0 gm

Weight of permeameter with filled wet soil = 4062.1 gm

Weight of wet soil, M_i = (4062.1 – 2480.0) = 1582.1 gm

Moisture Content of the Sample, W_0:

Sample No	1	2	3
Container #	20	19	1
Container Wt (gm)	60	60	60
Wet Wt + Container (gm)	112.8	119.4	115.2
Dry Wt + Container (gm)	109.1	115.2	111.4
Wt of Water (gm)	3.7	4.2	3.8
Dry Wt of Soil (gm)	49.1	55.2	51.4
Water Content (%)	7.5	7.6	7.4
Average (Water Content)		7.5%	

Weight of dry soil used, M_r = 1582.1 / (1+0.075) = 1471.7 gm

Other Measured Data

Permeameter Diameter = 10 cm

Permeameter Area, A = 78.54 cm_2

Stand Pipe Diameter = 2.1 cm

Stand Pipe Area, a = 3.46 cm_2

Obsn No	Sample Length, L (cm)	Temp (°C)	Initial Head, H_0 (cm)	Final Head, H_1 (cm)	Time Difference for Head Fall, (t_0-t_1) sec	Permeability at T°C, K_T (cm/sec)	Viscosity Ratio, η_T/η_{20}	Permeability at 20°C, K_{20} (cm/sec)	Void Ratio, e
1.	14	24	125	50	80	0.00706	0.908	0.0064	0.95
2.	14	24	100	25	105	0.00813	0.908	0.00738	1.002
3.	14	24	125	25	200	0.00496	0.908	0.0045	0.76

Sample Calculations (For Observation # 1)

Coefficient of permeability at T°C, K_T $= 2.3 \dfrac{aL}{A \times (t_1 - t_o)} \log_{10} \dfrac{H_o}{H_1} = 2.3 \times \dfrac{3.46 \times 14}{78.54 \times 80} \log_{10} \dfrac{125}{50} = 0.00706 cm/\sec$

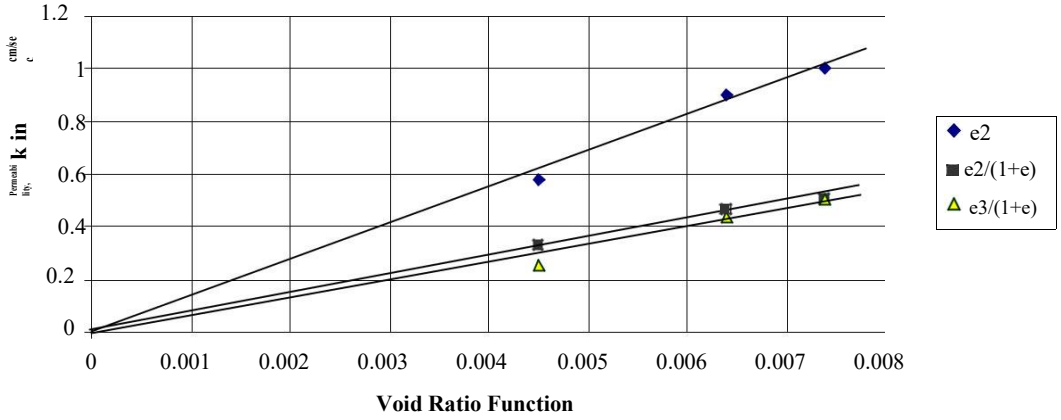
Permeability at 20°C, K_{20} $= k \dfrac{\eta_T}{\eta_{20}} = 0.00706 \times 0.908 = 0.0064 cm/\sec$ **.**

Volume of Soil Used, V_s = Weight of dry soil used / (Sp. Gravity of soil x Density of water) = 1471.7 / (2.61 x 1) = 563.88 cm^3 Total volume of soil, V = π x $(d/2)^2$ x L_c = π x $(10/2)^2$ x 14 = 1099.56 cm^3

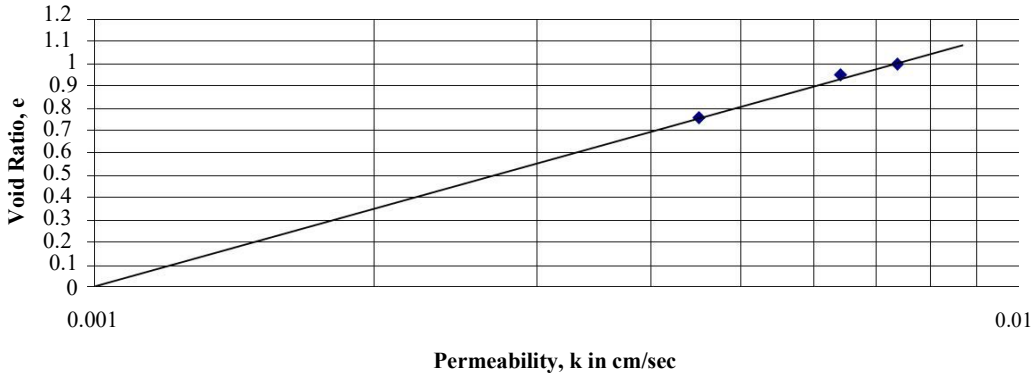
Volume of soils, V_v = (V – V_s) = (1099.56 – 564.88) = 535.68 cm^3 Void ratio, e = V_v / V_s = (535.68 / 563.88) = 0.95

Obsn No	Void Ratio, e	e^2	$e^2/(1+e)$	$e^3/(1+e)$
1.	0.95	0.9025	0.4628	0.4397
2.	1.002	1.004	0.5015	0.5025
3.	0.76	0.5776	0.3282	0.2494

Permeability vs. Void Ratio Function Plot



Void Ratio vs. Permeability Plot



SECTION 11

DIRECT SHEAR TEST

References

- ASTM D3080 – 11 – Standard Test Method for Direct Shear Test of Soils under Consolidation Drained Conditions.
- BS 1377: part 7 Soils for Civil Engineering purpose; Shear Strength Tests.
- Lambe, T.W., Soil Testing for Engineers.
- Bowles, Joseph E., Engineering Properties of Soils and Their Measurement (4th Edition)
- ASTM D 6026 - 06 – using Significant Digits in Geotechnical Data

General Discussion

This test is performed to determine the consolidated-drained shear strength of a sandy to silty soil. The shear strength is one of the most important engineering properties of a soil, because it is required whenever a structure is dependent on the soil's shearing resistance. The shear strength is needed for engineering situations such as determining the stability of slopes or cuts, finding the bearing capacity for foundations, and calculating the pressure exerted by a soil on a retaining wall.

The direct shear test is a strain-controlled test - the rate at which the soil will be strained is controlled. A specimen of soil will be placed into a *shear box*, and consolidated under an applied normal load. The shear box is made of two separate halves - an upper and a lower. After the application of the normal load, these two halves are moved relative to one another, shearing the soil specimen on the plane that is the separation of the two halves. The direct-shear test imposes stress conditions on the soil that force the failure plane to occur at a predetermined location (on the plane that separates the two halves of the box). On this plane there are two forces (or stresses) acting - a normal stress, σ_n, due to an applied vertical load P_v and a shearing stress, τ, due to the applied horizontal load P_h. These stresses are simply computed as:

$$\sigma_n = P_v/A$$

$$\tau = P_h/A$$

where A is the nominal area of the specimen (or of the shear box).

It is usually not corrected for the change in sample area caused by the lateral displacement of the sample under the shear load P_h. These stresses should satisfy Coulomb's equation:

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

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

Since the shear stress and normal stress have the same significance as when used in a Mohr's circle construction, rather than solving a series of simultaneous equations for c and tan φ, one may plot on a set of coordinate axes the values of t versus σ_n from several tests (generally with τ on the ordinate), draw a line through the resulting locus of points, or the average locus of points, and establish the slope of the line as tan φ and the τ-axis intercept as the c parameter. This is commonly known as the *Mohr-Coulomb Failure Envelope*. For cohesion less soils, the intercept is usually negligible, and Coulomb's equation becomes

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

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

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

a. Cheap, fast and simple - especially for sands.
b. Failure occurs along a single surface, which approximates observed slips or shear type failures in natural soils.

Disadvantages of the test include:

a. Difficult or impossible to control drainage, especially for fine-grained soils.
b. Failure plane is forced--may not be the weakest or most critical plane in the field
c. Non-uniform stress conditions exist in the specimen.
d. The principal stresses rotate during shear, and the rotation cannot be controlled.

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

Equipment

Special – Digital Direct Shear Machine The shear machine is designed to operate with lever arm assembly arranged to multiply a given normal load by 10.i.e. 1 kg on the load hanger applies a load of 10 kg to the sample. The lever arm should be balanced in the horizontal position by adjusting the position of the balance weight on the arm.

Figure 11-1 showing the machine used for direct shear test. The main components of this machine are as following:

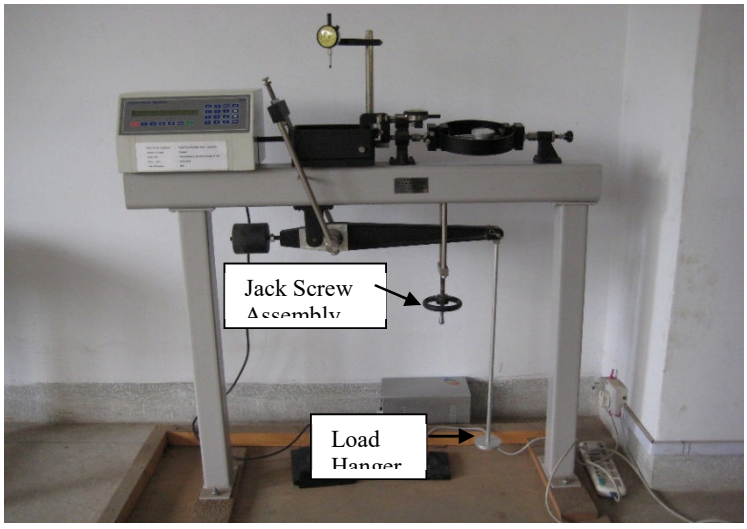


Figure 11-1: Digital Direct Shear Machine

a. **Shear Box**. Shear box is manufactured throughout in non ferrous metal. A range of shear boxes are available to suit the specimens of various sizes. It has two main part, upper and lower shear box, which slide over one another due to application of horizontal force. The floor of the box has retches for ease of drainage – it is lower pressure pad. The sample is placed within the box covered by upper and lower porous plates for draining out of water. At the top, upper pressure pad is placed. Figure 11-2 and 11-3 show shear box and its components

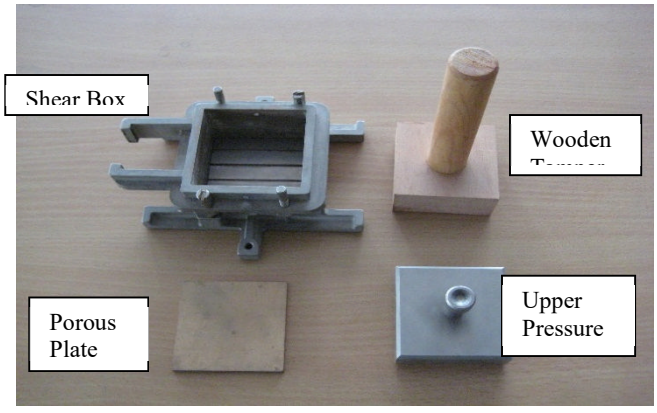


Figure 11-2: Components of Shear Box.

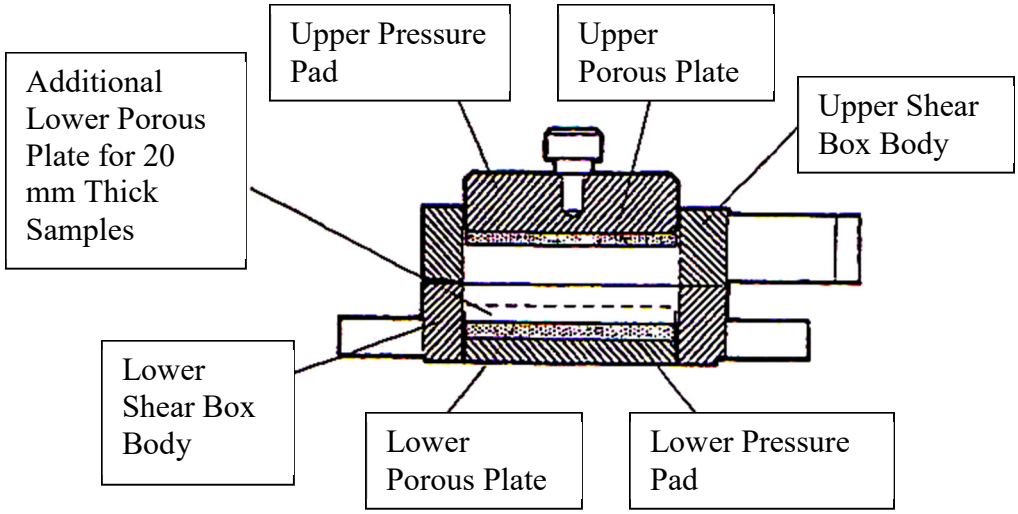


Figure 11-3: Schematic Diagram of Shear Box.

b. **Dial Gauges** There are three dial gauges fitted with the machine for taking various readings. Vertical dial gauge is attached directly to the up stand arm and shows the vertical displacement due to shearing. This reading indicates the volume change of sample due to application of stress. Horizontal dial gauge displays the displacement in sample due to shearing. This reading is used to measure the strain in the sample. Proving ring dial gauge displays the displacement in proving ring due to application of horizontal force. This reading is multiplied by a factor to determine the applied load. Figures 11-4 and 11-5 show different types of dial gauges.

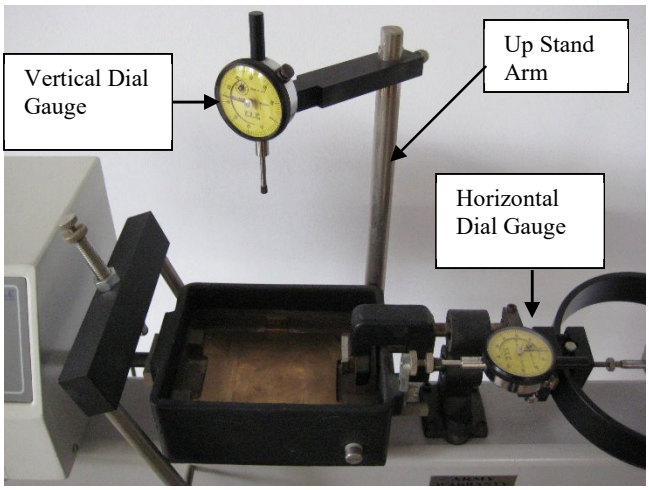


Figure 11-4: Vertical and Horizontal Dial Gauge.

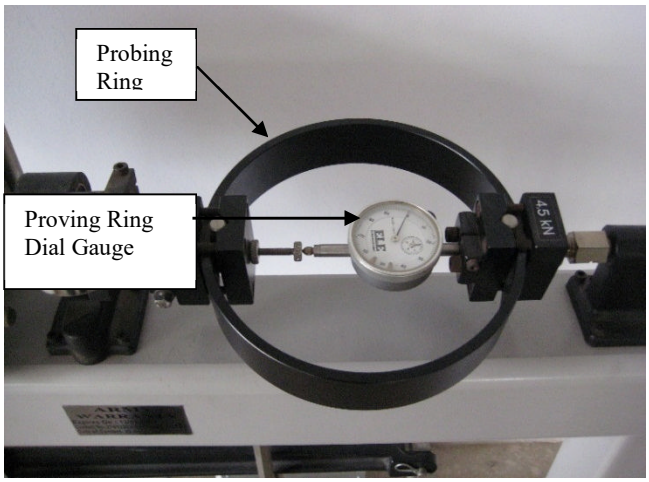


Figure 11-5: Probing Ring Dial Gauge.

c. **Shear Apparatus** It is a digital machine fitted with motor to control the strain. Figure 11-6 shows the apparatus.



Figure 11-6: Digital Shear Apparatus.

General.

- Tamper for compacting soil
- Balance (0.1 g sensitivity)
- Drying oven
- Calipers
- Straight edge
- Large evaporating dish
- Spoons, etc.
- Timer

Test Procedure.

First It must be needed to be familiar with the apparatus including the direction of travel of all parts of the apparatus, electric motor (on/off switch, forward/reverse, speed) etc. One can take help from laboratory technicians. First time users should also know - how to apply normal loads, function of each screw in the shear box, measurement of the dial indicators including the units. After that the following procedure should be continued:

a. The volume of the specimen to be constructed should be determined. The cross-sectional area of the interior of the shear box should be measured. The depth of the shear box should be measured. The thickness of the bottom frictional plate and the top porous stone should be determined.

b. When the specimen is setup, the top of the porous stone should be flush with the top of the shear box. Knowing this, and with the above dimensions measured, the volume of the prepared specimen may be calculated.

c. Knowing the density desired and the volume of the specimen, the mass of sand required to set up the specimen can be calculated. The required mass of the sample should be obtained.

d. **Setting up the Specimen**.

(1) The shear box should be placed in the direct shear device. It should be made sure the fixing screws (screws that locks the shear box together) are locked in place and the set screws (screws that adjust the gap space between the shear box halves) are at the bottom of their travel.

(2) The bottom frictional plate should be placed into the bottom of the shear box with the rough side up. The lower porous stone and a filter paper above it should be placed. See Figure 11-7.

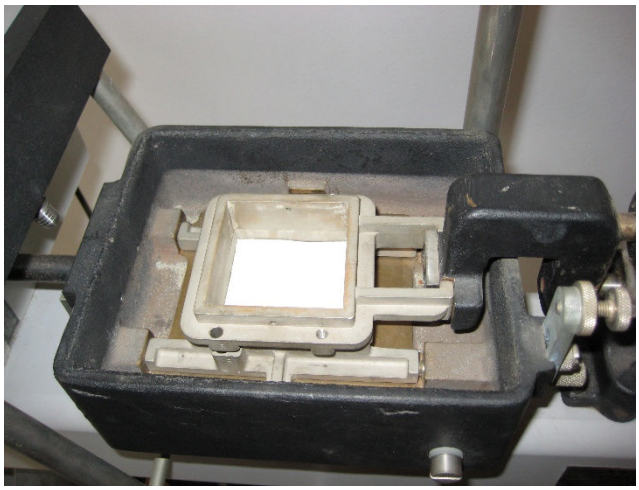


Figure 11-7: Placement of Lower Porous Plate and Filter Paper.

(3) The cutter above the soil sample should be placed and with a push; the cutter into the soil should be inserted to collect the specimen. In case of sandy sample, the sand should be poured in rather loosely.

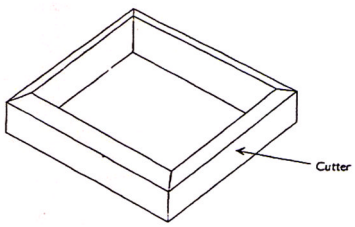


Figure 11-8: Cutter for Sample

(4) The cutter filled sample on the shear box should be placed and with the help of wooden tamper, the sample into the shear box should be inserted. See Figures 11-9 and 11-10 for the schematic diagram and placement of sample into the shear box.

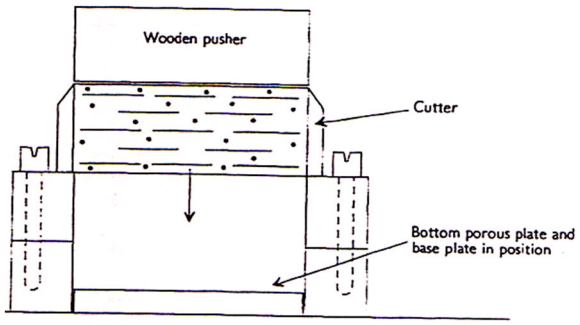


Figure 11-9: Schematic Diagram of Placement of Sample into the

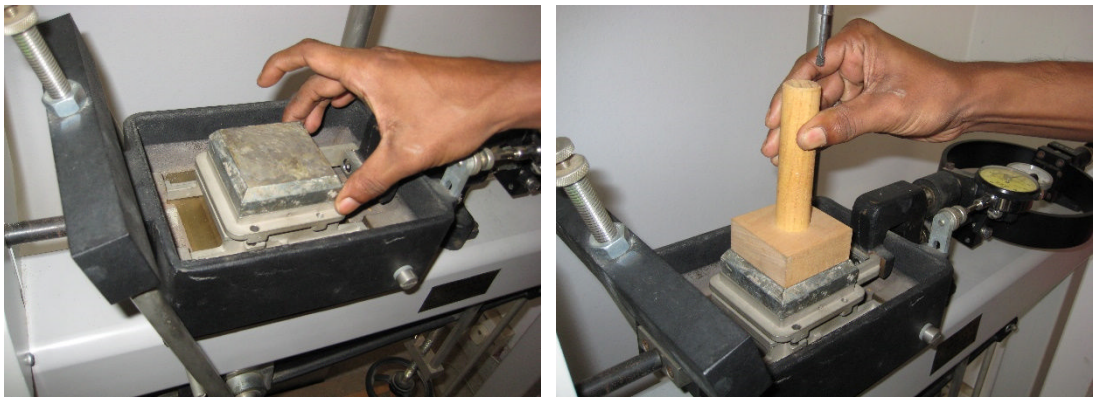


Figure 11-10: Placement of Sample into the Shear Box.



Figure 11-11: Placement of Sample is Completed and Cutter Removed.

(5) A filter paper on the top of the sample and the upper porous stone above it should be placed. It should be sticking up out of the shear box at this point, see Figure 11-12.

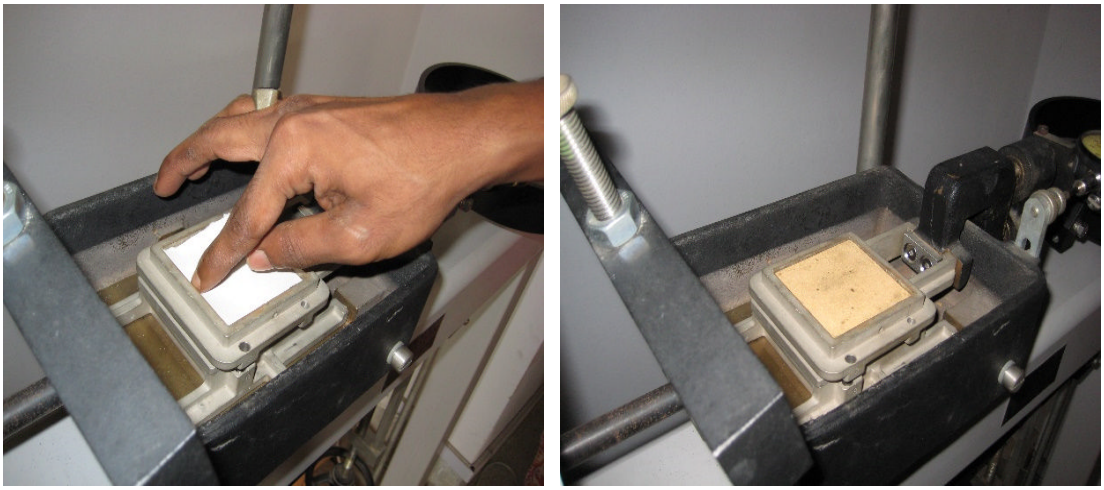


Figure 11-12: Placement of Filter Paper and Upper Porous Plate.

(6) The loading cap and ball bearing should be placed on top of the porous stone. The reaction bar of the loading frame should be lowered until it just touches the ball bearing. The nuts should be used to lock the reaction bar into place. Finally, the fixing screws should be removed. See Figure 11-13.



Figure 11-13: Placement of Loading Cap and Ball Bearing.

e. The extensometers should be attached which measure shear and normal displacements. The dial should be adjusted which measures normal displacement to read either expansions or contractions. The initial readings should be recorded on all dials. Before proceeding with the test, It should be checked carefully to see that there is no connection between the two parts of the soil container except the soil.

f. The new vertical displacement dial reading should be recorded. It will have moved, and the specimen will thus have densified. This combined with the initial dial reading that already obtained will allow to calculate the actual density of the specimen (it will be slightly denser than planned).

g. The desired normal load should be applied. In direct shear test, the test procedure is repeated using at least three identical specimens with variable normal stresses. It is sometimes convenient to interpret the results, if the normal stresses are so chosen one is closer to the existing overburden and one of each of the other two on the either side of the overburden.

(1) The normal stress required on the specimen must be related to a mass comprising the load yoke and set weights W.

(2) $W=(\sigma_n * L^2/9810)$kg, Where L represents length of box in mm and σ_n stress kN/m^2.

(3) The shear machine is fitted with a balanced lever loading device, fine adjustment of which is achieved by the balance weight.

(4) The initial position of lever loading device and hence loading yoke can be achieved by adjusting the jacking handle and load pin.

(5) To apply the load to the sample the weights may either be added to the weight hanger with the jacking handle fully down, or with the jacking handle supporting the lever loading arm then slowing winding the jacking handle down to transfer the load to the sample.

h. **Pre-Test Check**.

(1) Ring of required capacity/sensitivity should be loaded and should be set correctly.

(2) The 'train' of components are all in contact from load ring adjuster to strain lead screw.

(3) Shear box is set tight in carriage.

(4) Loading yoke correctly positioned with vertical displacement measuring device set.

(5) Alignment screws are removed. Shear box separator screws screwed in to just lift the upper box away from the lower box then retracted.

(6) Strain rate is set (0.00001 to 9.99999 mm/min).

i. **Shearing the Specimen**.

(1) The horizontal (shear) loading should be started and readings of the load, shear displacement, and vertical displacement dials should be taken. Readings should be taken every 0.01 inches of shear (horizontal) deflection—this corresponds to every 10 units on the dial.

(2) For a strain- controlled test, a set of readings should be taken every 15 seconds for the first 2 minutes, and then a set every 0.03 in. of horizontal displacement. The test should be continued to a horizontal displacement of approximately 15 % of the length of the specimen unless a constant shear force is obtained first.

(3) For a stress- controlled test a set of readings should be taken before each new load is added. The test should be continued to failure.

(4) The shearing is complete when load is determined to be dropping (the residual strength of the soil has been reached).

j. The applied normal pressure should be backed off, dismantled and the shear box should be cleaned.

Data Analysis

1. The shear stress can be computed as $\tau = Ph / A$, where P_h is the horizontal load and A is the area of specimen in the shear box.
2. The normal stress can be computed as $\sigma n = Pv / A$, P_v is the vertical load.
3. The angle of internal friction, φ, can be computed from

$$\varphi = \tan^{-1}\frac{\tau}{\sigma}$$

where,

τ = shear stress = {(Proving ring reading - initial proving ring reading) × ring calibration factor}/ area

σ = normal stress = Normal force / area.

However, since τ/σ is a ratio of stresses on the same surface, it can be determined by dividing the shear force by the normal force.

Results

Method of Presentation.

a. The results of a direct shear test on a cohesionless/cohesive soil can be presented in a summary table and/or by a stress-strain curve (or stress ratio-strain curve).

b. A stress- strain curve usually consists of a plot of τ/σ and normal displacement versus shear displacement. The two plots should be on the same page using the same scale of shear displacement.

A stress- strain curve usually consists of a plot of shear stress versus shear displacement (cohesive soil).

c. Normal stress and peak/maximum shear stress thus estimated and plotted in a peak shear stress-normal stress curve. A best possible straight line is fitted to the observed point to represent the failure envelope. The shear strength parameters c and φ is thus obtained from the plot. It is to be noted that usually we get the drained strength parameters from direct shear test. For granular soil as the cohesion is zero and the available strength parameter would be only φ. At least three test are carried out at different normal stresses and a straight line passing through origin and the respective points defines the failure envelope for sand soil (as c=0). The slope of the line gives the value of φ.

Significant Figures

1. The angle of shear resistance should be expressed to the nearest 0.1.
2. The axial strain should be expressed to 3 significant digits.
3. The value of stress or pressure should be expressed to 3 significant digits

Typical Values

1. Peak friction angles in dense, well- graded, coarse sand usually range from 37º to 60º for dense, uniform, fine sand they are usually between 33º and 45º.
2. There is less variation in the values of ultimate friction angle; a typical value is 30º.
3. The volume changes that occur during shear depend too much on the initial void ratio to give any typical values. Normally, only the loosest of sands tend to show a net negative volume change, or contraction, at failure.

Table 11-1: Typical Values of Effective Angle of Friction for Granular Soil (After Terzaghi and Peck, 1967)

Soil type	**Effective angle of internal friction, φ (Degrees)**	
	Loose	**Dense**
Nonplastic silt	27-30	30-34
Silty sand	27-33	30-35
Uniform sand	28	34
Well graded sand	33	45
Sandy gravel	35	50

WORKED OUT EXAMPLE

Direct Shear Test

Test Serial No. : 11

Date Tested: October 15, 2008

Tested By:

Project Name: Airport Road – Rokeya Sharoni Link Road Project

Sample Number: U-1, 3.0 m

Visual Classification of Soil: Gray silty clay, trace fine sand, low plasticity, moist, CL

Initial Data

Shear Box Length = 60 mm

Shear Box Width = 60 mm

Shear Box Area = 60 x 60 = 3600 mm^2

For Normal Stress, σ_1 = 0.0028 Kg/mm² (Load on hanger= 1 Kg)

(Shear and Normal Displacement Dial: 1 unit = 0.01 mm and Proving Ring Dial: 1 unit = 0.31 Kg)

Elapsed Time (sec)	Shear Displacement Dial Reading	Shear Displacement (mm)	Normal Displacement Dial Reading	Normal Displacement (mm)	Proving Ring Dial Reading	Shear Force (Kg)	Shear Stress (Kg/mm)
0	0	0	0	0	0	0	0
15	48	0.48	-8	-0.08	31	9.61	0.0027
30	98	0.98	-6	-0.06	45	13.95	0.0038
45	142	1.42	-3	-0.03	55	17.05	0.0047
60	205	2.05	4	0.04	62	19.22	0.0053
75	266	2.66	8	0.08	66	20.46	0.0057
90	330	3.30	15	0.15	67.5	20.92	0.0058
105	389	3.89	22	0.22	69.5	21.55	0.006
120	453	4.53	28	0.28	72	22.32	0.0062
135	513	5.13	32	0.32	73	22.63	0.0063
150	579	5.79	36	0.36	73.5	22.78	0.0063
165	638	6.38	38	0.38	73.5	22.78	0.0063
180	740	7.40	41	0.41	72	22.32	0.0062
195	770	7.70	44	0.44	68	21.08	0.0059

Sample Calculations (For last data)

Shear displacement = Shear displacement dial reading x 0.01

= 770 x 0.01 = 7.70 mm

Normal displacement = Normal displacement dial reading x 0.01

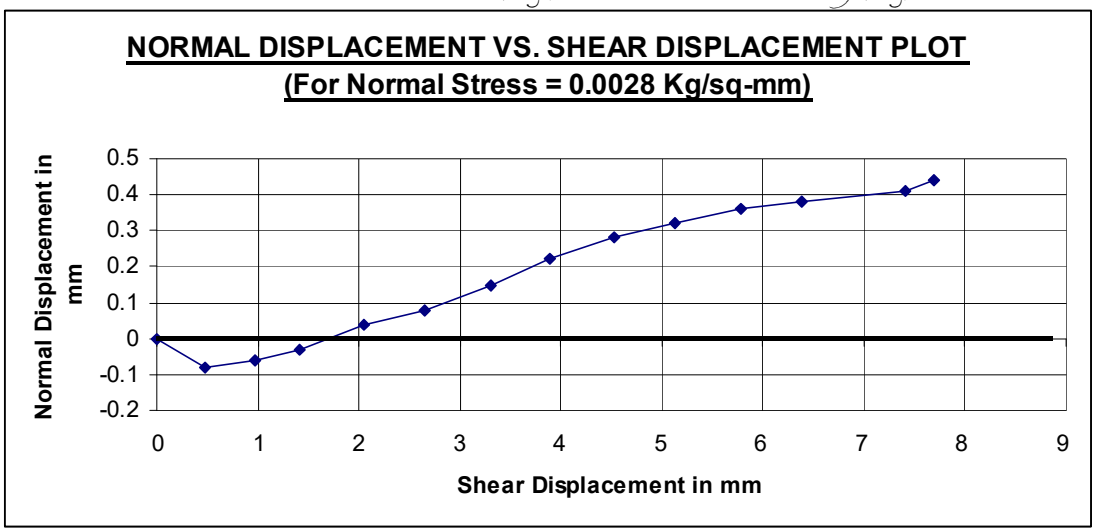
= 44 x 0.01 = 0.44 mm

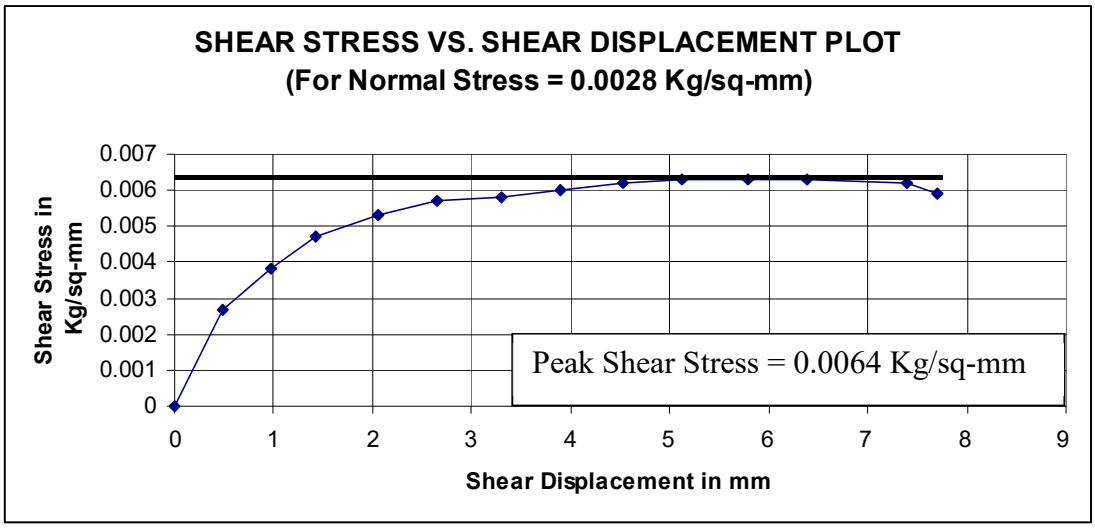
Shear force = Proving dial reading x 0.31 (load factor)

= 68 x 0.31 = 21.08 Kg

Shear stress = Shear force / sample area

= 21.08 Kg / 3600 mm^2 = 0.0059 Kg/mm^2

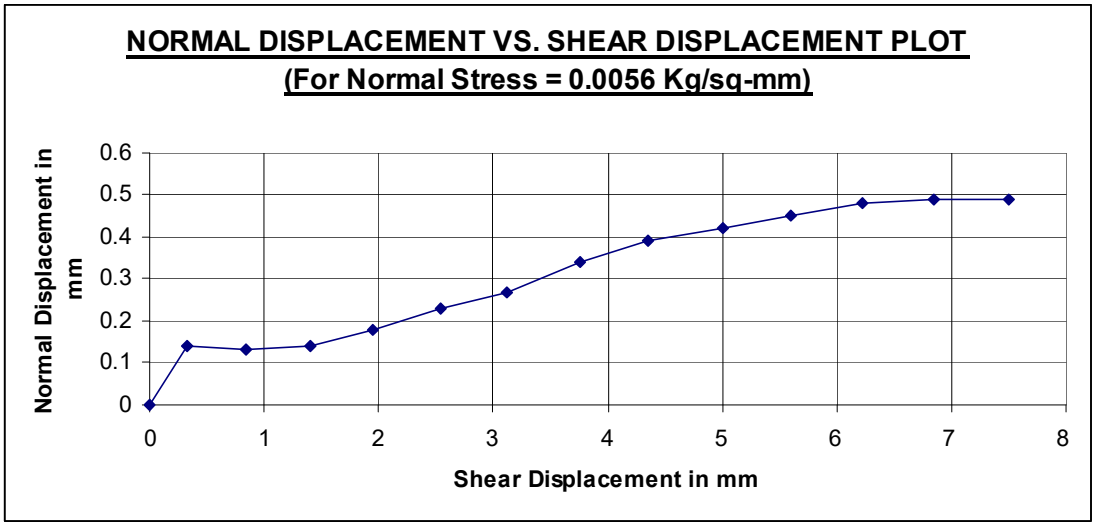


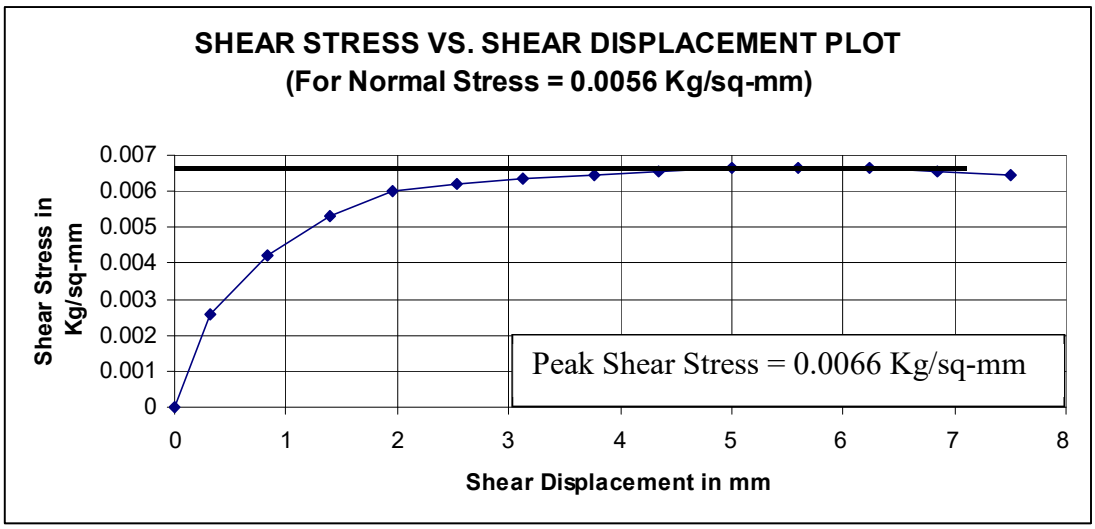


For Normal Stress, σ_2 = 0.0056 Kg/mm² (Load on hanger= 2 Kg)

(Shear and Normal Displacement Dial: 1 unit = 0.01 mm and Proving Ring Dial: 1 unit = 0.31 Kg)

Elapsed Time (sec)	Shear Displacement Dial Reading	Shear Displacement (mm)	Normal Displacement Dial Reading	Normal Displacement (mm)	Proving Ring Dial Reading	Shear Force (Kg)	Shear Stress (Kg/mm)
0	0	0	0	0	0	0	0
15	32	0.32	14	0.14	30	9.3	0.00258
30	84	0.84	13	0.13	49	15.19	0.00422
45	140	1.40	14	0.14	61.5	19.065	0.0053
60	195	1.95	18	0.18	70	21.7	0.00603
75	254	2.54	23	0.23	72	22.32	0.0062
90	312	3.12	27	0.27	74	22.94	0.00637
105	376	3.76	34	0.34	75	23.25	0.00646
120	435	4.35	39	0.39	76	23.56	0.00654
135	500	5.00	42	0.42	77	23.87	0.00663
150	560	5.60	45	0.45	77	23.87	0.00663
165	623	6.23	48	0.48	77	23.87	0.00663
180	684	6.84	49	0.49	76	23.56	0.00654
195	750	7.50	49	0.49	75	23.25	0.00646

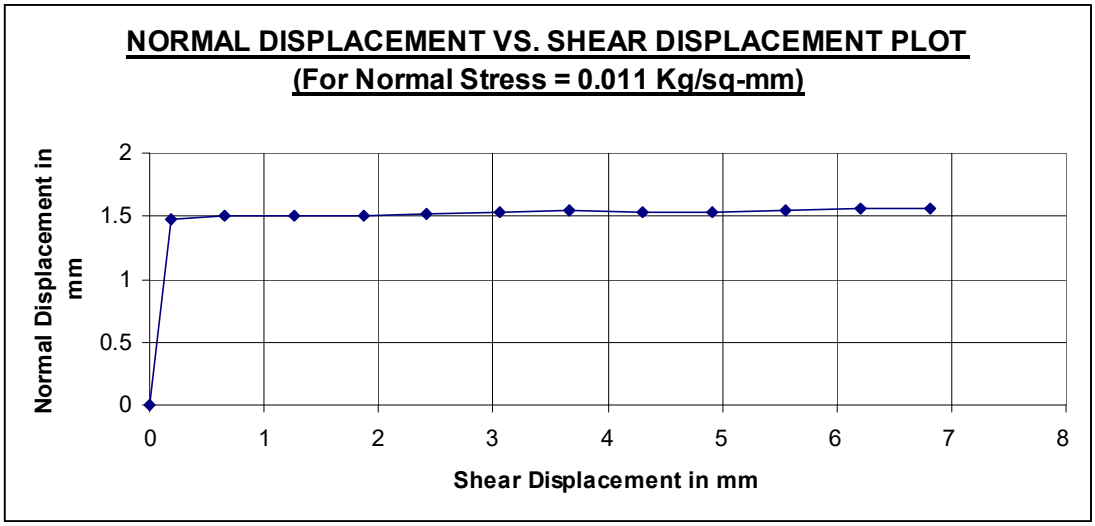

NORMAL DISPLACEMENT VS. SHEAR DISPLACEMENT PLOT
(For Normal Stress = 0.0056 Kg/sq-mm)
Normal Displacement in mm
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
5
6
7
8
Shear Displacement in mm

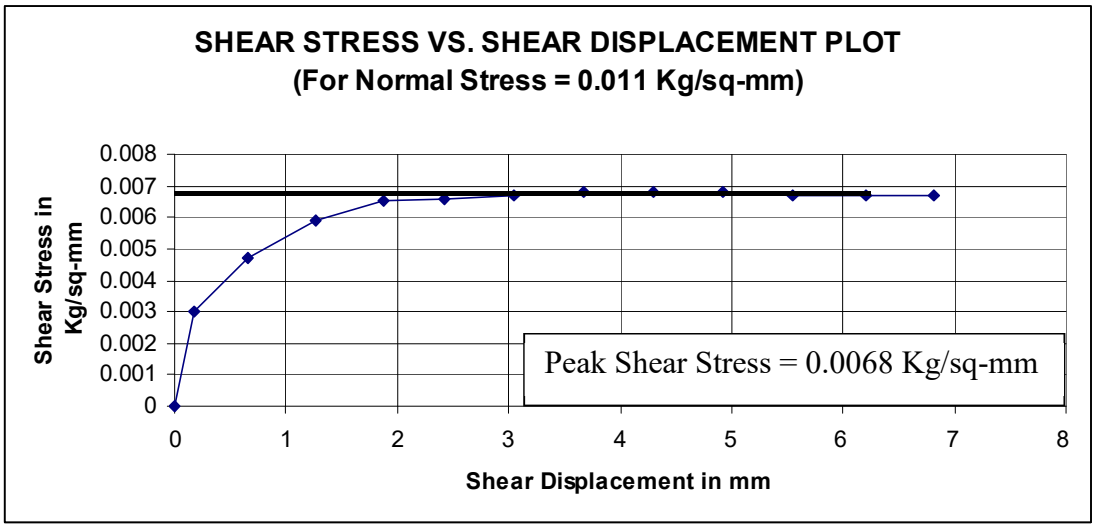

SHEAR STRESS VS. SHEAR DISPLACEMENT PLOT
(For Normal Stress = 0.0056 Kg/sq-mm)
Shear Stress in Kg/sq-mm
0.007
0.006
0.005
0.004
0.003
0.002
0.001
0
0
1
2
3
4
5
6
7
8
Peak Shear Stress = 0.0066 Kg/sq-mm
Shear Displacement in mm

For Normal Stress, σ_3 = 0.011 Kg/mm² (Load on hanger= 4 Kg)

(Shear and Normal Displacement Dial: 1 unit = 0.01 mm and Proving Ring Dial: 1 unit = 0.31 Kg)

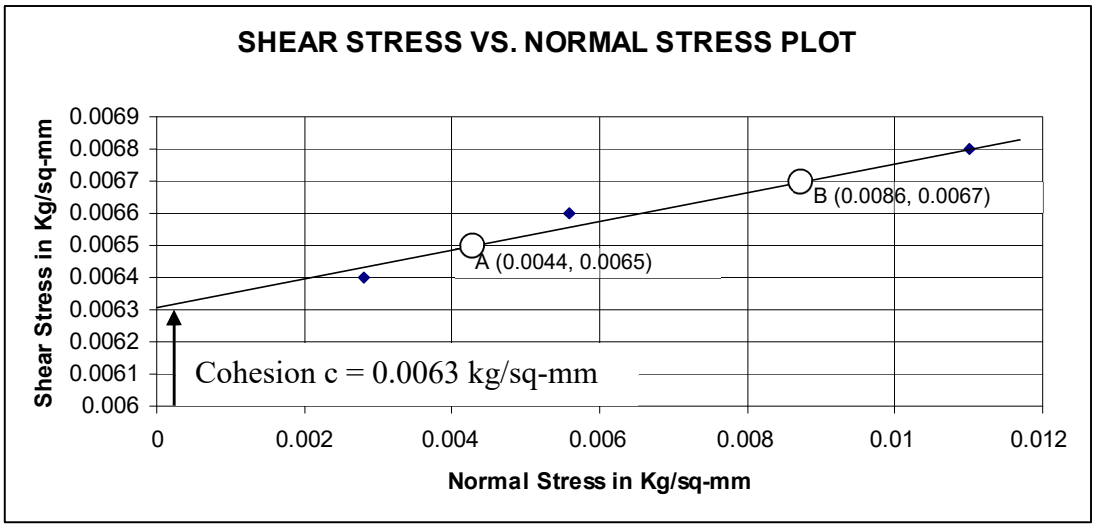
Elapsed Time (sec)	Shear Displacement Dial Reading	Shear Displacement (mm)	Normal Displacement Dial Reading	Normal Displacement (mm)	Proving Ring Dial Reading	Shear Force (Kg)	Shear Stress (Kg/mm)
0	0	0	0	0	0	0	0
15	18	0.18	148	1.48	35	10.85	0.003
30	65	0.65	150	1.50	55	17.05	0.0047
45	127	1.27	150.5	1.505	69	21.39	0.0059
60	187	1.87	151	1.51	76	23.56	0.0065
75	242	2.42	151.5	1.515	77	23.87	0.0066
90	305	3.05	153	1.53	78	24.18	0.0067
105	367	3.67	154	1.54	78.5	24.335	0.0068
120	430	4.30	152.5	1.525	78.5	24.335	0.0068
135	492	4.92	153	1.53	78.5	24.335	0.0068
150	555	5.55	154	1.54	78	24.18	0.0067
165	620	6.20	155.5	1.555	78	24.18	0.0067
180	682	6.82	156	1.56	78	24.18	0.0067





At different applied normal stress corresponding peak or maximum shear stress are:

Observation	Normal Stress in kg/mm^2	Shear Stress in kg/mm^2
1.	0.0028	0.0064
2.	0.0056	0.0066
3.	0.011	0.0068



RESULT

From graph,

Cohesion c of the soil sample = 0.0063 kg/sq-mm

Again, taking two points on the line A (0.0044, 0.0065) and B (0.0086, 0.0067), the slope angle of the line or angle of internal friction,

$$\begin{aligned}\phi &= \tan^{-1}\left(\frac{0.0067-0.0065}{0.0086-0.0044}\right)\\ &= \tan^{-1}\left(\frac{0.0002}{0.0042}\right)\\ &= 2.73^0\end{aligned}$$

Comment

The determined value of angle of internal friction φ is very less than expected.

Most probably, the readings of third observation are misleading.

SECTION 12

CONSOLIDATION TEST

References

- ASTM D4186 - 12 -" Standard Test Method for One-Dimensional Consolidation Properties of Soils".
- Holtz and Kovacs. (1981), An Introduction to Geotechnical Engineering.
- Lambe, T.W., Soil Testing for Engineers.
- Bowles, Joseph E., Engineering Properties of Soils and Their Measurement (4th Edition)
- ASTM D 6026 - 06 – using Significant Digits in Geotechnical Data

General Discussion

This test method is for the determination of the magnitude and rate-of-consolidation of saturated cohesive soils using continuous controlled-strain axial compression. Information concerning magnitude of compression and rate-of-consolidation of soil is essential in the design of earth structures and earth supported structures.

When any soil is subjected to an increase in pressure (or load), a readjustment in the soil structure occurs that may be considered as consisting primarily of plastic deformation with a corresponding reduction in void ratio *e*. The load is usually carried initially by the water in the pores because the water is incompressible when compared with the soil "skeleton". The pressure which results in the water because of the loading is called ***hydrostatic excess pressure***, because it is in excess of that pressure due to the weight of water. As the water drains from the soil pores, the load increment is gradually shifted to the soil structure. This transfer of load is accompanied by a change in the total volume of soil equal to the volume of water drained. This process is known as *consolidation.*

For dry and partially saturated fine-grained soil the pore fluid (being primarily air) has almost no viscosity or flow resistance. Similarly, if the soil is coarse-grained the coefficient of permeability *k* is large and the pore water can quickly flow out. A consolidation test is not required in these cases. When a load is applied to a fine-grained soil that is either nearly or completely saturated the time for all the plastic deformation and void – ratio reduction to take place is much longer.

The consolidation process can be understood by looking at the spring analogy shown in Figure 12 - 1. The saturated soil element is represented by Figure 12 -1a, in which the spring corresponds to the soil structure and the water to the soil pore water. If a weight W is placed on the water and the spring with the valve y closed (Figure 12-1a), the weight is almost entirely carried by the water, since it is

incompressible as compared to the spring. If the valve is opened and water is allowed to escape, the load will eventually be carried entirely by the spring (Figure 12-1c). The elapsed time required to transfer the load from the water to the spring depends upon how rapidly water is allowed to escape from the valve.

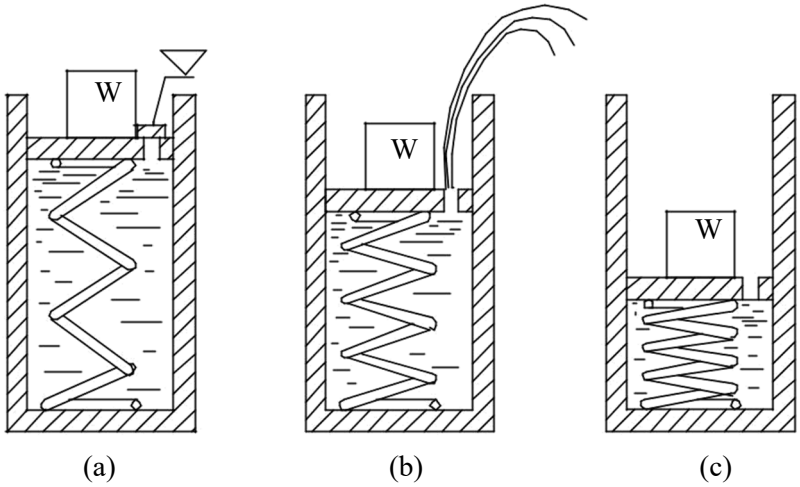


Figure 12-1: The Spring Analogy to Consolidation

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

The frictional component of the shear strength of a soil depends directly upon the effective stress (which can be thought of as intergranular stress for granular soils). In geotechnical engineering, the effective stress is inferred or calculated directly from the difference between the total stress and the pore water pressure (excess hydrostatic pressure), or

$$\sigma' = \sigma - u$$

In order to determine the shear strength of a soil, therefore, the stage of consolidation of the soil must be known.

The main purpose of consolidation tests is to obtain soil data which is used 12-2 in predicting the rate and amount of settlement of structures founded on clay. Although some of the settlement of a structure on clay may be caused by shear strain, most of it is normally due to volumetric changes.

This is particularly true if the clay stratum is thin compared to the width of the loaded area or the stratum is located at a significant depth below the structure.

The four most important soil properties furnished by a consolidation test are:

a. **Preconsolidation Stress, P′**. This is the maximum stress that the soil has "felt" in the past.

b. **Compression Index, C_c**. It indicates the compressibility of a *normally-consolidated* soil.

c. **Recompression Index, C_r**. It indicates the compressibility of an *over-consolidated* soil.

d. **Coefficient of Consolidation, C_v**. It indicates the rate of compression under a load increment.

A normally-consolidated soil is defined as a soil which, at the present time, is undergoing the application of a stress that is *larger* than any stress it has undergone in its history. That is,

$$\sigma_{present} > \sigma'_p$$

Conversely, an over-consolidated soil is defined as a soil which has experienced higher stresses in the past,

$$\sigma_{present} < \sigma'_p$$

In order to predict the settlements of structures in the field, a method of extrapolating laboratory test results in the settlement analysis is needed. Total settlement is computed from knowledge of the preconsolidation pressure, and the coefficients of compression and recompression. For predicting the rate of settlement, the Terzaghi One-Dimensional Theory of Consolidation is commonly used.

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

Equipment

The equipment used for this test are listed below (See Figure 12-2):

a. Consolidometer and dial indicator.

b. Loading frame (mechanically controlled).

c. Timing device.



Figure 12-2(a): Consolidometer.

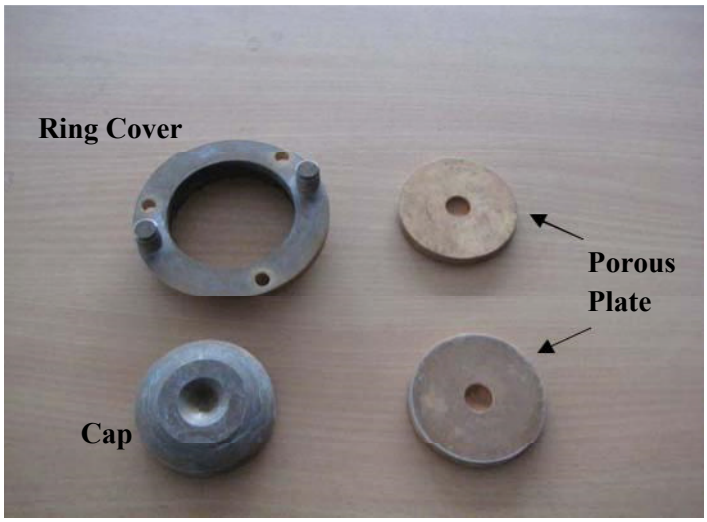


Figure 12-2(b): Accessories of Consolidometer.



Figure 12-2(c): Water Bath of Consolidometer.

Some notes on the apparatus:

a. The specimen is contained by a relatively rigid ring; thus compression and consolidation are *one-dimensional.*

b. There are porous stones at the top and bottom of the specimen so that water can drain, and thereby dissipate excess hydrostatic pressure.

c. The specimen is surrounded by a water bath at all times, to prevent the specimen from desiccating (drying out).

d. The dial indicator, accuarte to 0.0001 in. or 0.0025 mm, measures the change in height of the specimen after the load has been applied.

e. Because of the very large loads required for typical consolidation tests, some type of load multiplying system is commonly employed . Some systems use air pressure applied to a piston through a rolling diaphragm, while others, such as the equipment of MIST, use mechanical levers and fulcrums. In any case, students should be familiar with how the loading system works, and how to obtain the desired stress increment on the test specimen.

Procedure

The following procedure needs to be performed by the students prior to the test. This includes the apparatus setup and the specimen preparation. Specimen preparation is an extremely important part of the consolidation test: slight flaws introduced in the specimen during its preparation can severely affect

the remainder of the test. Proper specimen preparation takes much time and usually quite a bit of practice.

Specimen Preparation and Consolidometer Set-Up.

a. First, a sample of the soil to be tested must be prepared. This is usually taken from a thin-walled Shelby tube. For the CE 342, a Shelby tube sample (an intact soil sample from the field) will be used.

b. The consolidometer ring should be measured and massed at this point.

c. The soil sample is next extruded from the Shelby tube a certain length. It should be extruded a little more than the height of the ring into which it will eventually be placed.

d. Using a thin wire saw, this disk of soil is cut off. Remember which side of the soil which is (up and down). Proper orientation of the specimen in the consolidometer will be required later.

e. At this point, one should have a disk of soil with a diameter greater than the inside diameter of the ring, and a height greater than that of the ring. Now, the soil must be placed into the ring.

f. The soil on a large glass plate should be set, or cleaned level surface. The *top* of the soil specimen should be *down.*

g. The ring on top of the specimen should be oriented such that it is approximately centered, with the cutting edge down (into the soil).

h. A small glass or acrylic plate should be placed on top of the ring, and then slowly and evenly start to push the ring into the soil specimen. The ring will begin to cut away excess soil around the outside of the specimen.

j. After the ring has been pushed in about ¼" to ½", the wire saw can be used to trim away excess soil being cut by the ring. If this is not done, there is a tendency for soil inside the ring to be ripped out with the excess soil being cut away.

k. The ring should be kept pushing, slowly and evenly into the soil, and cutting away the excess. The key here is patience, precision and carefulness. This step often takes much time and concentration, especially for beginners.

l. When the ring has been pushed far enough down that the *bottom* (which is now facing up!) of the soil specimen touches the small top glass or acrylic plate, one should stop at this point.

m. Using a ring with the same interior diameter, but a larger exterior diameter, pushing the ring downwards should be finished. This should be stopped so that the cutting edge of the ring

(facing down) is *not* flush with the *top* of the soil specimen (now facing down). There needs to be, at this point, a little bit of soil (about ¼") sticking out of either side of the ring.

n. Now, using the thin wire saw, the top and bottom of the specimen should be trimmed such that they are both flush with the top and bottom of the ring. A sawing motion should be used to cut *down* into the soil until the ring is reached (can't go any farther down). Then, using a sawing motion, it should be cut back keeping the saw's cutting wire in contact with the ring. Small bits should be taken off in this manner until the entire surface is flush with the ring. Then the specimen should be flipped over, and repeated on the other side.

p. Some of the fresh trimmings should be taken and obtained the initial water content.

q. Two pieces of filter paper should be massed, and these should be placed on either side of the specimen. They should stick somewhat.

r. The mass of the ring + specimen + paper should be measured.

s. The ring + specimen + paper should be placed into the consolidometer.

t. The mass of a porous stone should be massed and placed on top of the specimen.

u. A rigid cap and steel ball bearing of known mass should be placed on top of this top porous stone.

v. The consolidometer should be filled with water. Arrangement is shown in Figure 12-3.

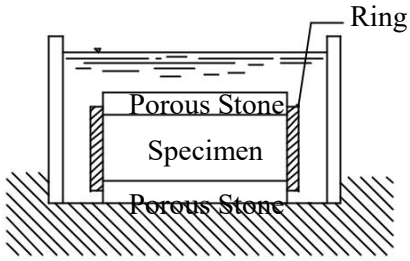


Figure 12-3: Arrangement of the Specimen after Set-up.

w. The reaction beam should be adjusted on the loading frame such that it barely contacts the seat.

x. One should be familiar with the apparatus. Specifically,

(1) How the load will be applied.

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

y. The vertical displacement dial gage should be set is so that its full range available in compression. The dial gage is to be zero. direction should be Units and travel adjusted, if digital dial gage.

z. A small seating load should be applied to the specimen (~30 kPa) and appropriate time- deformation readings should be taken. The seating load should be kept on for 24 hrs.

Consolidation of the Test Specimen.

a. One should understand the loading schedule. A load increment ratio (LIR) of 1 is normally used. This means that the load is doubled from the previous increment (1, 2, 4, 8, etc.). Other load increments may also be used, however. Sometimes, unload-reload cycles are performed.

b. A Data Sheet 2 for each loading increment should be prepared. Vertical deformation dia l readings are taken at elapsed time increments of 0, 6, 15, 30, 45, 60 sec., 2, 4, 8, 15, 30, 60 minutes or until 90% consolidation has been reached. The time required for 90% consolidation can be determined from the square root time plots that are to be plotted for every stage.

c. The load should be applied. Immediately one should begin to observe the vertical deformation, and take readings at the aforementioned elapsed time increments.

d. Vertical deformation dial readings (DR) v. the square root of time should be plotted. This will help in identifying recording errors, as well as one should determine when primary consolidation is complete. This second part is determined by finding t_{90} from the plot. When t_{90} is reached, 90% of primary consolidation is complete; it is now satisfactory to proceed with the next loading increment.

e. When primary consolidation is complete, or as instructed, the next load increment should be applied, and recorded and plotted vertical deformation readings as before.

f. The incremental loading process outlined before should be continued. It is a good idea to plot vertical deformation dial readings v. applied stress (log scale). This also helps prevent mistakes, and gives a picture of how the loading process is proceeding

g. After the final (maximum) load increment has been applied, and primary consolidation is complete for this stage, the specimen is unloaded (rebounded) using alternate loading increments. When unloading, the lower load increment is applied to the specimen, and it must be allowed to rebound in this state for a certain period of time. This time is t_{90} for the corresponding load increment. For example: if we're unloading from 1200 kPa to 800 kPa, we should let the specimen rebound for a period of time equal to t_{90} for the loading stage from 800 kPa to 1200 kPa

h. After the final (maximum) load increment has been applied, and primary consolidation is complete for this stage, the specimen is unloaded (rebounded) using alternate loading increments. When unloading, the lower load increment is applied to the specimen, and it must be allowed to rebound in this state for a certain period of time. This time is t_{90} for the corresponding load increment. For example: if we're unloading from 1200 kPa to 800 kPa, we should let the specimen rebound for a period of time equal to t_{90} for the loading stage from 800 kPa to 1200 kPa.

i. The specimen should be rebounded until only a very small seating load is applied to the specimen. This load should be left on overnight. This is necessary to be able to determine accurately the final height and void ratio of the specimen and correlate these values to the final strain dial reading.

j. The specimen should be unloaded completely the final dial reading should be recorded and carefully disassembled the apparatus. Quickly the mass of the ring plus the wet specimen should be measured , and then the ring and specimen in the oven should be placed. This will allow calculation of the final water content of the specimen. The equipment and work area should be cleaned

Analysis

Analysis of test data is done following the after mentioned procedure:

a. All data for the entire test should be gathered.

b. The height of solids can be calculated:

$$H_s = \frac{m_S}{G_s \times \rho_w \times A_r}$$

Where:

m_s = Mass of solids (dry mass of specimen)

Gs = Specific gravity

ρ_w = Density of water

Ar = Cross-sectional area of the rin

An idealized figure, showing void ratio/height relationships is shown in Figure 12-4.

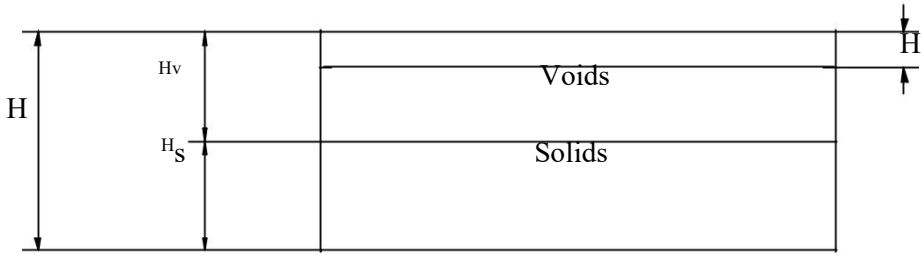


Figure 12-4: Void Ratio / Change in Height Relationship.

c. The initial height of voids should be calculated:
$H_{v0} = H_0 - H_s$
Where, H_0 = Initial height of the specimen

d. The initial void ratio can be calculated:
$e_0 = H_{v0} / H_s$

e. The change in height of the specimen should be calculated, H_I, due to an applied load. This is equal to the Final Dial Reading minus the correction factor due to the compression of the filter paper. This should be done for each applied loading increment.

f. The final specimen height should be determined for each loading increment.
$H_{f,I} = H_0 - H_I$

g. The height of voids should be determined for each loading increment:
$H_{v,I} = H_{f,I} - H_s$

h. The final void ratio should be determined for each incremental loading: $e_I = H_{v,I} / H_s$

j. The final height of voids can be calculated:
$Hv,f = mw,f / Ar$
Where, $m_{w,f}$ = Final mass of water in soil specimen

k. The final void ratio should be calculated :
$e_f = H_{v,f} / H_s$

l. The average specimen height should be compared during consolidation for previous loading increment added to the current increment's final specimen height. Then this should be divided by two.

m. The coefficient of consolidation should be calculated for each loading increment:

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

The above equation is a generic time rate consolidation equation. For any time t that we know the degree of consolidation that has taken place, we can look up the empirically derived time factor, T_v. In this generic equation, H is simply the maximum drainage path distance that water could take within the specimen. Since we've already computed the time for 90% consolidation at each increment, t_{90}, we can use this time, with the corresponding time factor, T_v, for 90% consolidation (0.848). The maximum drainage path is simply the average specimen height, Ht,av divided by two (since the specimen is drained on both ends). Putting these into the above equation yields:

$$C_v = \frac{0.848 H_{t,av}^2}{4 t_{90}}$$

n. A semi-log plot of pressure vs. final void ratio should be plotted. Pressure is plotted on the horizontal axis, at log scale. Void ratio is plotted on the vertical axis, at an arithmetic scale. See Figure 12-5.

o. The compression index, C_c, and the recompression index, Cr, should be determined from the consolidation curve. These are both just slopes of certain parts of the curve, and their locations are shown on the example consolidation curve above. The compression index is the slope of what's termed the "virgin compression curve" because it represents a state at which the soil has never existed before (the applied stress is greater than the preconsolidation pressure). It should be noted that the recompression index may also be taken as the slope of the unload-reload portion of the curve

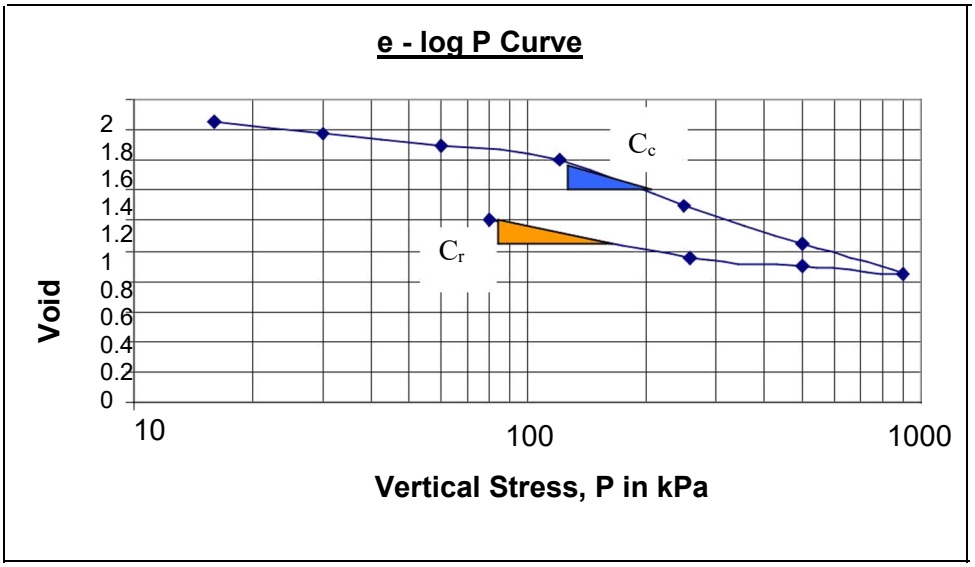


Figure 12-5: e – log p Curve

p. On a semi-logarithmic plot with the same horizontal scale as the consolidation curve, the coefficient of consolidation vs. pressure should be plotted. Again, pressure should be on the horizontal axis, at log scale, while the coefficient of consolidation should be plotted at arithmetic scale on the vertical axis. This plot should be located immediately below the consolidation curve in report. It should be noted that an average applied pressure will need to be computed in order to produce this plot.

q. The preconsolidation pressure should be determined for the specimen. This should be done using the Casagrande construction method referred in the textbook.

Contents of the Report

Report should include the following:

a. Visual classification of the soil. Cuttings from the trimming process will be saved for this purpose.

b. Include plots of:

(1) Void ratio vs. pressure (e – log p or consolidation curve)

(2) Coefficient of consolidation vs. pressure.

(3) Dial reading v. square root of time (for each loading increment).

c. The following should be determined:

(1) Coefficient of consolidation for each increment.

(2) Compression index.

(3) Recompression index from as many places on the curve as feasible (compute it for each unload-reload cycle, as well as the final unloading of the specimen). How do these indices compare to one another?

(4) Preconsolidation pressure.

Significant Figures

1. The void ratio should be expressed to 2 or 3 significant digits
2. The coefficient of consolidation should be expressed to significant digits.
3. The value of compression indexes should be expressed to 2 or 3 significant digits
4. The preconsolidation stress should be expressed to2 significant digits

WORKED OUT EXAMPLE

Consolidation Test

Test No : **12**
Date Tested: **January 29, 2008**
Tested By: _____
Project Name: **Airport Road – Rokeya Sharoni Link Road Project**
Location : **Jahangir Gate** Sample Numbe **B-2,U - 3**.
Visual Classification of Soil: **Silty clay**. Sample Depths: **4.5 m**.
Drainage Condition: **No impermeable layer (both side drainage).**

***Ring Dimensions*:**
Diameter: **2.5 inch** Area: **4.91 inch$_2$** Height: **0.78 inch**

Initial Data:
Specimen Height: **0.78 inch** Specific Gravity of Soil: **2. 65**

Initial Specimen Data:
Mass of ring: **65.4 gm**
Mass of paper: **0.6 gm**
Mass of ring + paper + wet soil: **197.1 gm**
Mass of wet soil: **(197.1 – 25.2 – 0.6) = 171.3 gm**
Initial water content: **18%**
Mass of dry soil, ms, from initial w/c: **145.17 gm**

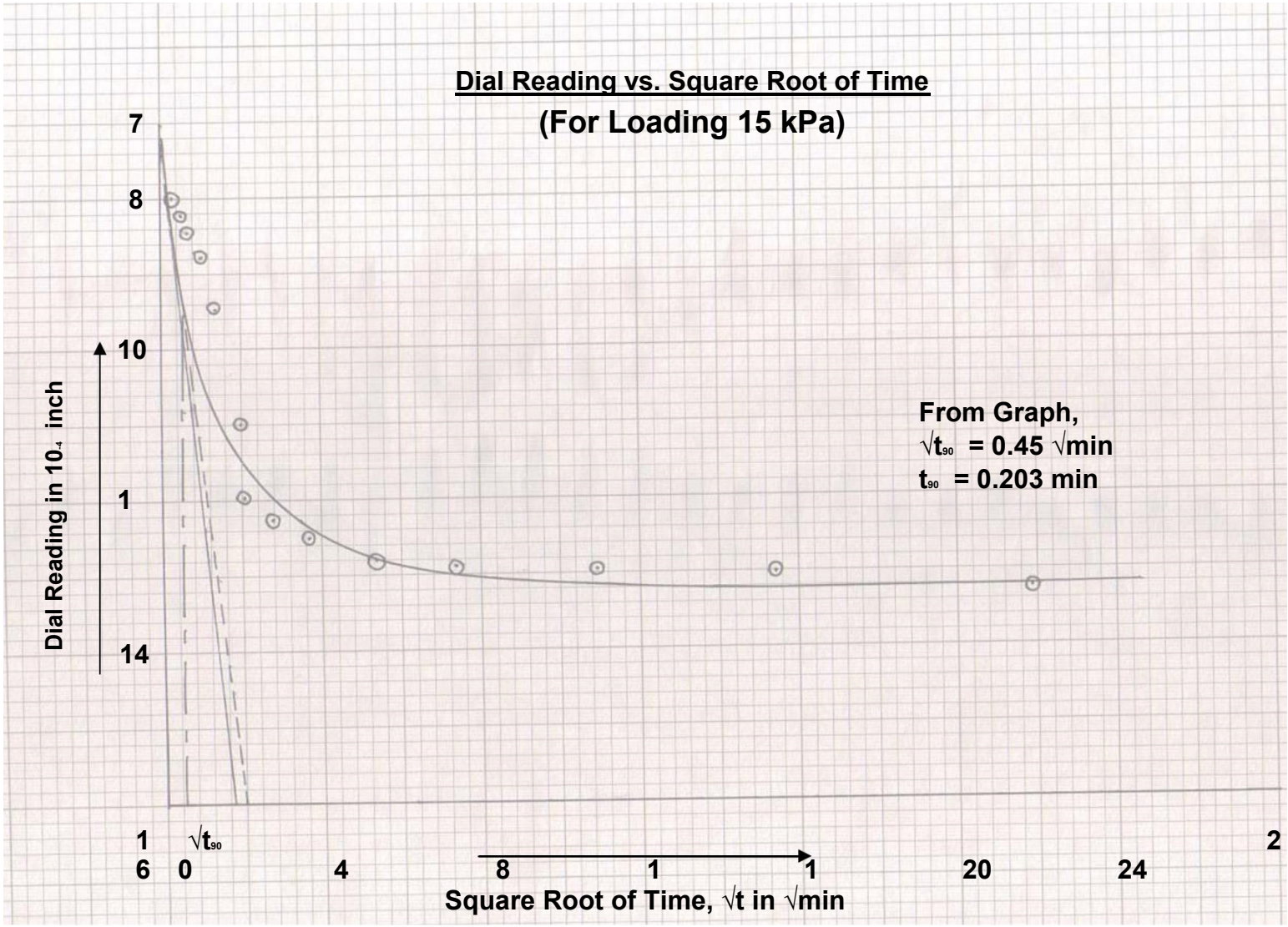
Final Water Content Determination:
Mass of dish: **25.2 gm**
Mass of ring + dish + wet soil: **224.2 gm**
Mass of ring + dish + dry soil: **201.4 gm**
Mass of dry soil, ms: **(201.4 – 65.4 – 25.2) = 110.6 gm**
Mass of water in soil, m_{wf}: **(224.2 – 201.4) = 22.8 gm**
Final water content, w_f: **(22.8 / 201.4) x 100 = 11.32%**

Load Increment Data:

Target Pressure: **15 kPa**

Actual Applied Pressure: **500 gm**

Date	Time	Elapsed Time (min)	Square Root of Time (√min)	Dial Reading x 10^{-4} (inch)
29 Jan 08	0800	0	0	0
29 Jan08	0800	0.1	0.316	8
29 Jan 08	0800	0.25	0.5	8.25
29 Jan 08	0800	0.5	0.707	8.5
29 Jan 08	0801	1	1	8.8
29 Jan 08	0802	2	1.414	9.5
29 Jan 08	0804	4	2	11
29 Jan08	0808	8	2.828	12.25
29 Jan 08	0815	15	3.872	12.5
29 Jan 08	0830	30	5.477	12.8
29 Jan 08	0900	60	7.746	12.9
29 Jan 08	1000	120	10.954	12.95
29 Jan	1200	240	15.492	13.0
29 Jan	1600	480	21.902	13.25
30 Jan	0800	1440	37.947	13.25


Dial Reading vs. Square Root of Time
(For Loading 15 kPa)
7
8
10
1
14
Dial Reading in 10⁻⁴ inch
From Graph,
√t₉₀ = 0.45 √min
t₉₀ = 0.203 min
1
6
√t₉₀
0
4
8
1
1
20
24
2
Square Root of Time, √t in √min

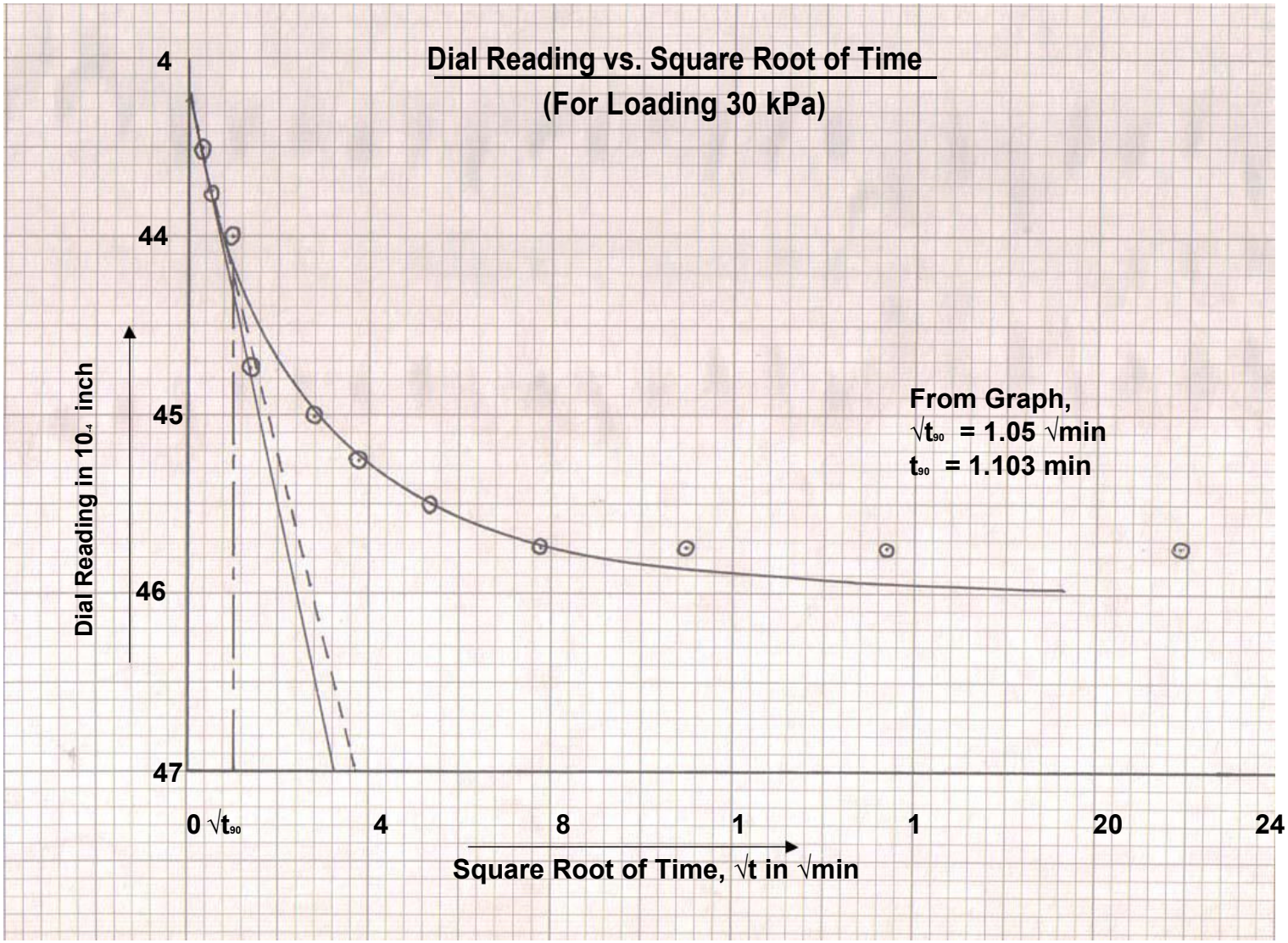
12 - 17

Load Increment Data:

Target Pressure: **30 kPa**

Actual Applied Pressure: **1 Kg**

Date	Time	Elapsed Time (min)	Square Root of Time (√min)	Dial Reading x 10^{-4} (inch)
30 Jan 08	0800	0	0	13.25
30 Jan 08	0800	0.1	0.316	43.5
30 Jan 08	0800	0.25	0.5	43.75
30 Jan 08	0800	0.5	0.707	44
30 Jan 08	0801	1	1	44
30 Jan 08	0802	2	1.414	44.75
30 Jan 08	0804	4	2	44.75
30 Jan 08	0808	8	2.828	45
30 Jan 08	0815	15	3.872	45.25
30 Jan 08	0830	30	5.477	45.5
30 Jan 08	0900	60	7.746	45.75
30 Jan 08	1000	120	10.954	45.75
30 Jan 08	1200	240	15.492	45.75
30 Jan 08	1600	480	21.902	45.75
31 Jan 08	0800	1440	37.947	47

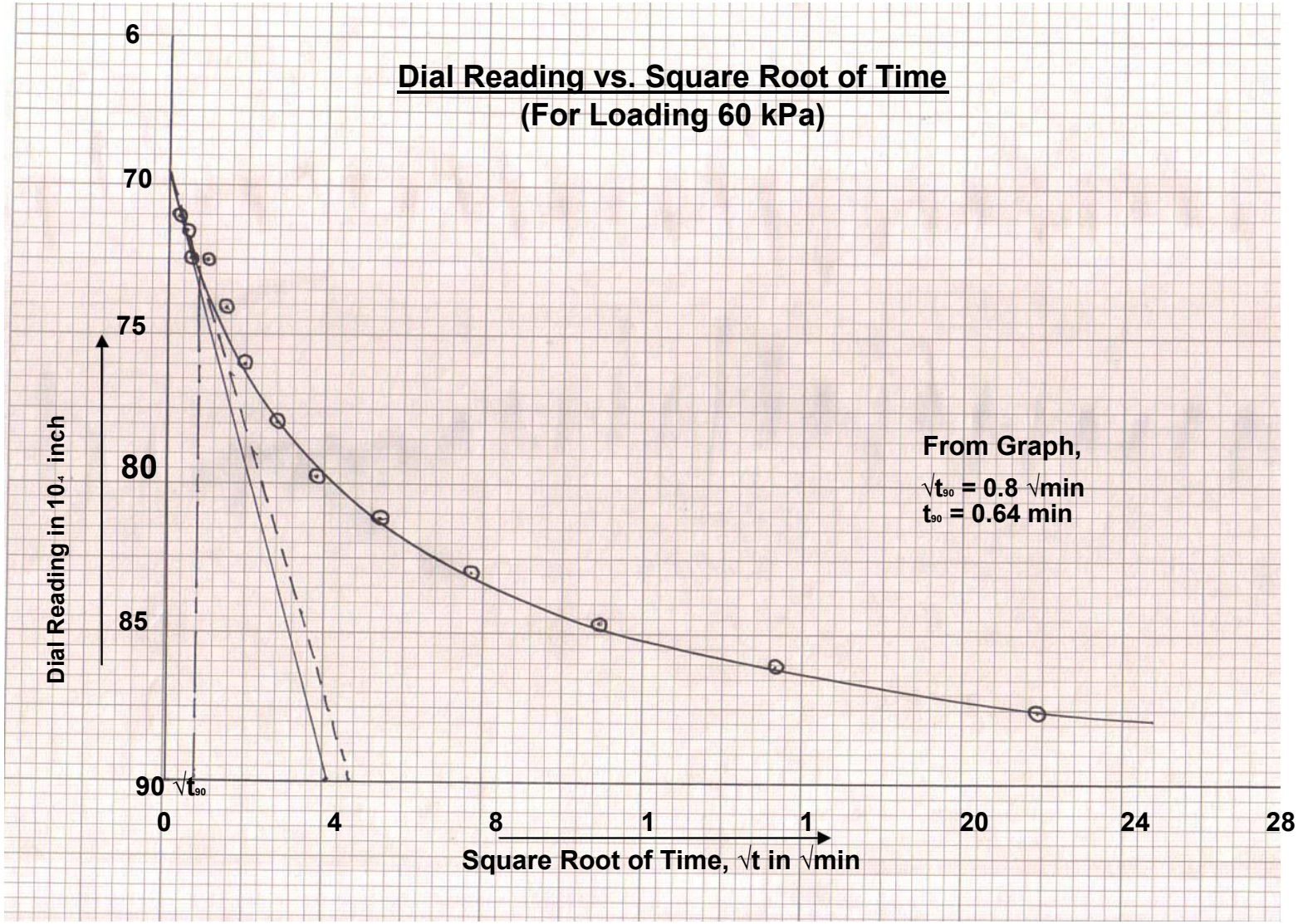

Dial Reading vs. Square Root of Time
(For Loading 30 kPa)
Dial Reading in 10⁻⁴ inch
4
44
45
46
47
From Graph,
√t90 = 1.05 √min
t90 = 1.103 min
0 √t90
4
8
1
1
20
24
Square Root of Time, √t in √min

Load Increment Data:

Target Pressure: **60 kPa**

Actual Applied Pressure: **2 Kg**

Date	Time	Elapsed Time (min)	Square Root of Time (√min)	Dial Reading x 10^{-4} (inch)
31 Jan 08	0800	0	0	47
31 Jan 08	0800	0.1	0.316	71
31 Jan 08	0800	0.25	0.5	71.5
31 Jan 08	0800	0.5	0.707	72.5
31 Jan 08	0801	1	1	72.5
31 Jan 08	0802	2	1.414	74
31 Jan 08	0804	4	2	76
31 Jan 08	0808	8	2.828	77.8
31 Jan 08	0815	15	3.872	79.8
31 Jan 08	0830	30	5.477	81
31 Jan 08	0900	60	7.746	82.8
31 Jan	1000	120	10.954	84.5


Dial Reading vs. Square Root of Time
(For Loading 60 kPa)
6
70
75
80
85
90
Dial Reading in 10⁻⁴ inch
√t₉₀
From Graph,
√t₉₀ = 0.8 √min
t₉₀ = 0.64 min
0
4
8
1
1
20
24
28
Square Root of Time, √t in √min

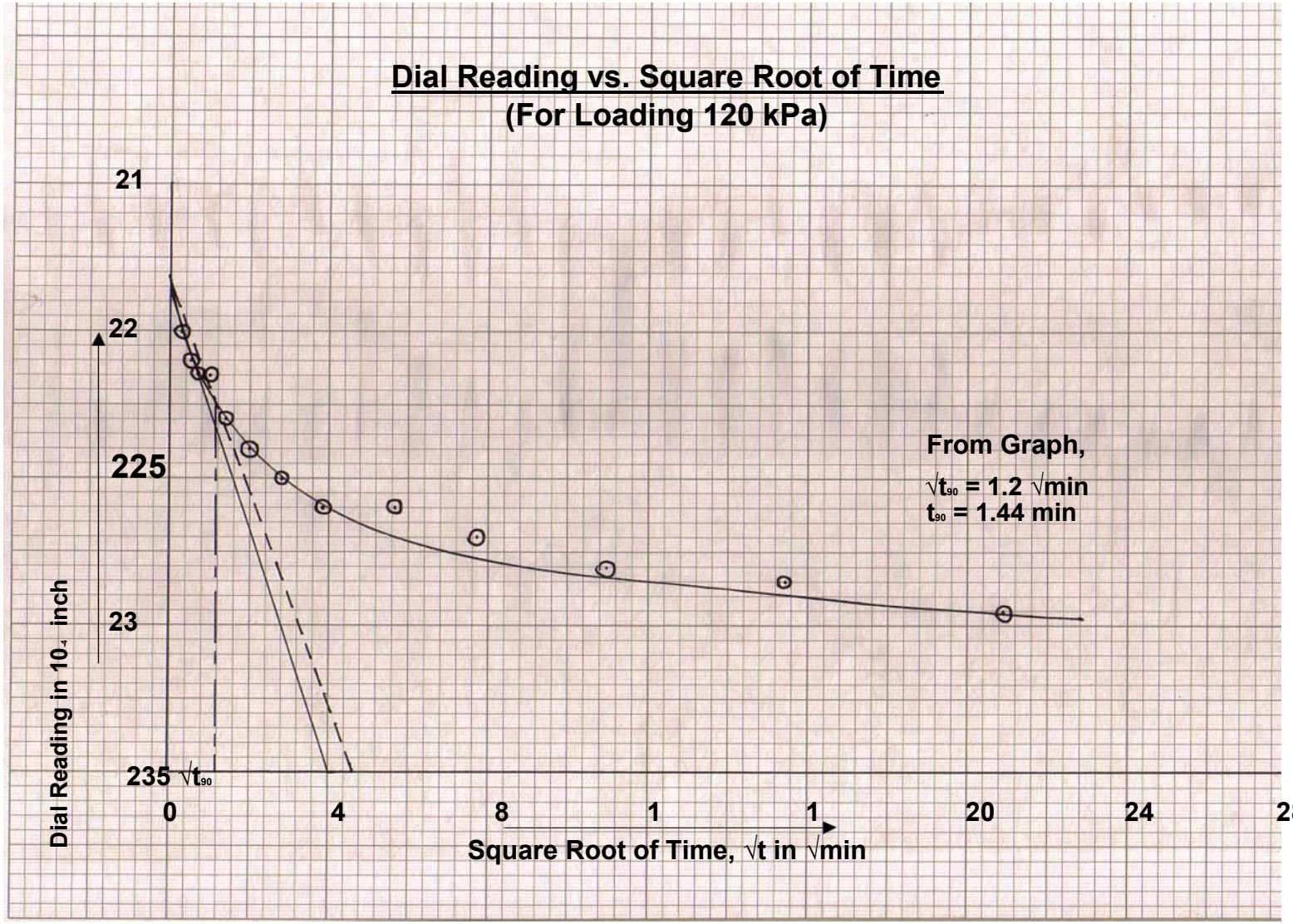
1
2
-
2
3

Load Increment Data:

Target Pressure: **120 kPa**

Actual Applied Pressure: **4 Kg**

Date	Time	Elapsed Time (min)	Square Root of Time (√min)	Dial Reading x 10^{-4} (inch)
01 Feb 08	0800	0	0	92.125
01 Feb 08	0800	0.1	0.316	220
01 Feb 08	0800	0.25	0.5	221
01 Feb 08	0800	0.5	0.707	221.5
01 Feb 08	0801	1	1	221.5
01 Feb 08	0802	2	1.414	223
01 Feb 08	0804	4	2	224
01 Feb 08	0808	8	2.828	225
01 Feb 08	0815	15	3.872	226
01 Feb 08	0830	30	5.477	226
01 Feb 08	0900	60	7.746	227
01 Feb 08	1000	120	10.954	228
01 Feb 08	1200	240	15.492	228.5
01 Feb 08	1600	480	21.902	229.5
02 Feb 08	0800	1440	37.947	231

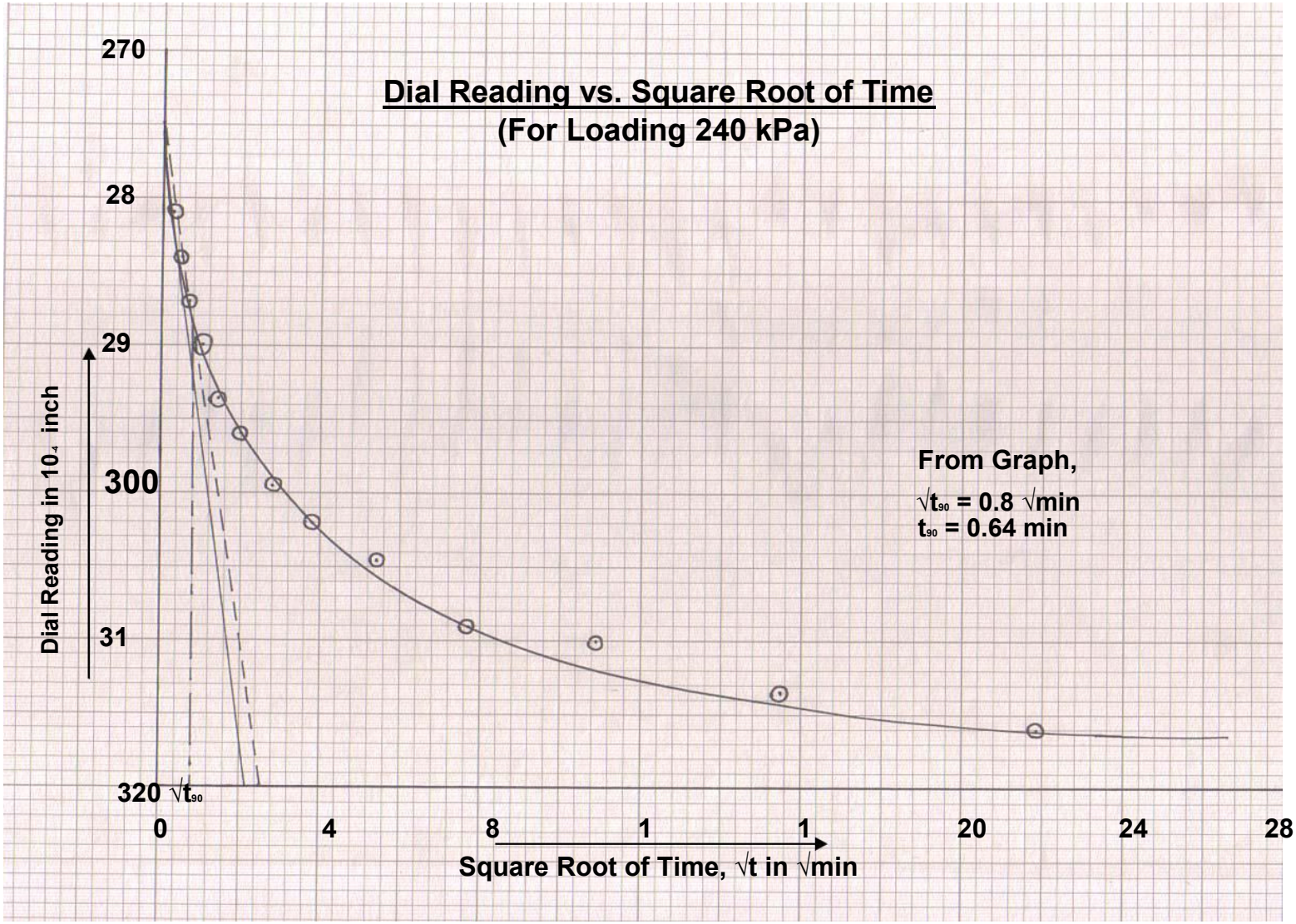

Dial Reading vs. Square Root of Time
(For Loading 120 kPa)
21
22
225
23
Dial Reading in 10⁻⁴ inch
235 √t90
From Graph,
√t90 = 1.2 √min
t90 = 1.44 min
0
4
8
1
1
20
24
28
Square Root of Time, √t in √min

Load Increment Data:

Target Pressure: **240 kPa**

Actual Applied Pressure: **8 Kg**

Date	Time	Elapsed Time (min)	Square Root of Time (√min)	Dial Reading x 10^{-4} (inch)
02 Feb 08	0800	0	0	231
02 Feb 08	0800	0.1	0.316	281
02 Feb 08	0800	0.25	0.5	284
02 Feb 08	0800	0.5	0.707	287
02 Feb 08	0801	1	1	290
02 Feb 08	0802	2	1.414	293.5
02 Feb 08	0804	4	2	296.5
02 Feb 08	0808	8	2.828	299.5
02 Feb 08	0815	15	3.872	302
02 Feb 08	0830	30	5.477	304.5
02 Feb 08	0900	60	7.746	309
02 Feb	1000	120	10.954	310

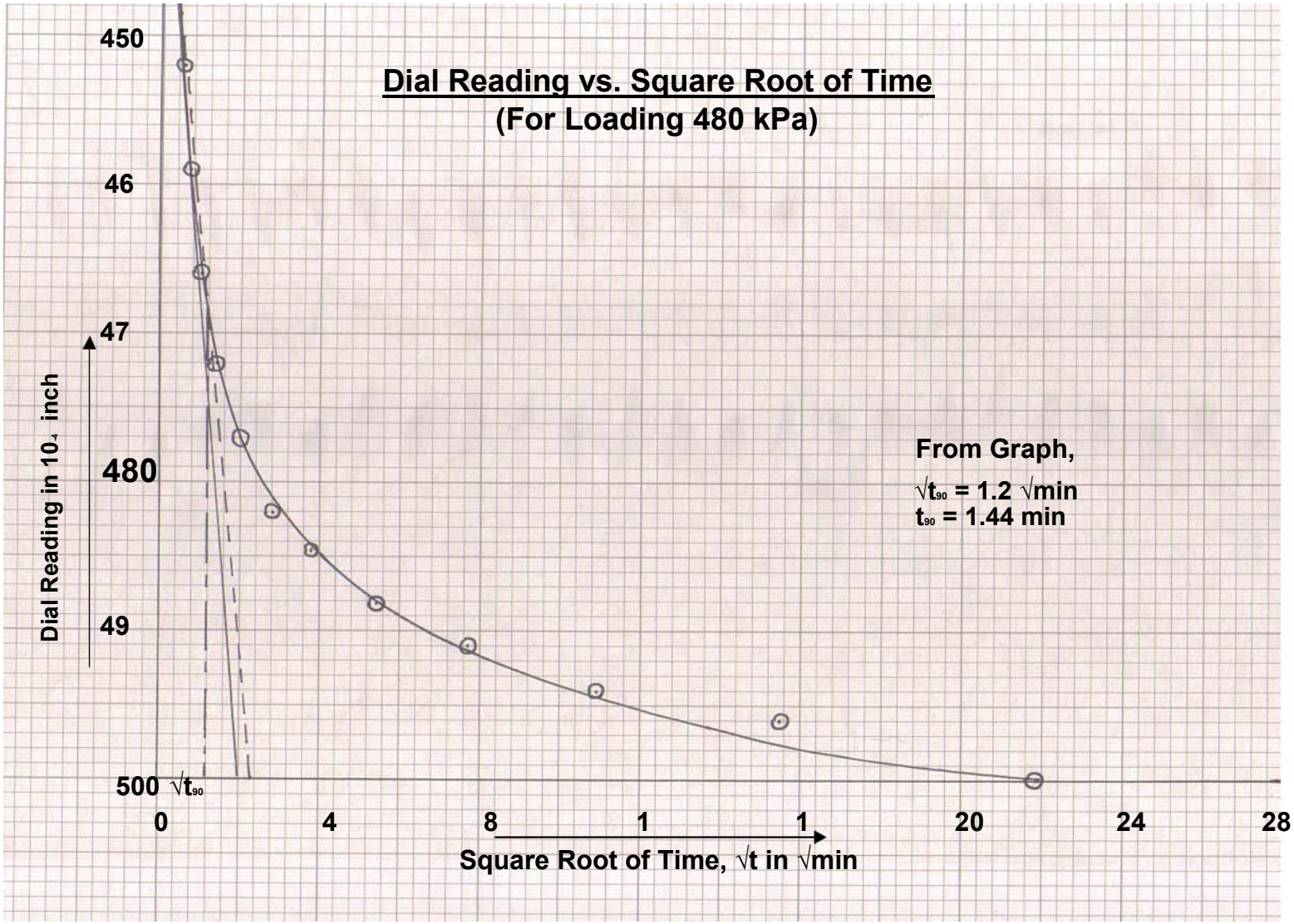

Dial Reading vs. Square Root of Time
(For Loading 240 kPa)
270
28
29
300
31
320 √t90
Dial Reading in 10⁻⁴ inch
From Graph,
√t90 = 0.8 √min
t90 = 0.64 min
0
4
8
1
1
20
24
28
Square Root of Time, √t in √min

Load Increment Data:

Target Pressure: **480 kPa**

Actual Applied Pressure: **16 Kg**

Date	Time	Elapsed Time (min)	Square Root of Time (√min)	Dial Reading x 10^{-4} (inch)
03 Feb 08	0800	0	0	318.5
03 Feb 08	0800	0.1	0.316	440
03 Feb 08	0800	0.25	0.5	452
03 Feb 08	0800	0.5	0.707	459
03 Feb 08	0801	1	1	466
03 Feb 08	0802	2	1.414	472
03 Feb 08	0804	4	2	477
03 Feb 08	0808	8	2.828	482
03 Feb 08	0815	15	3.872	484.8
03 Feb 08	0830	30	5.477	488.1
03 Feb 08	0900	60	7.746	491
03 Feb	1000	120	10.954	494

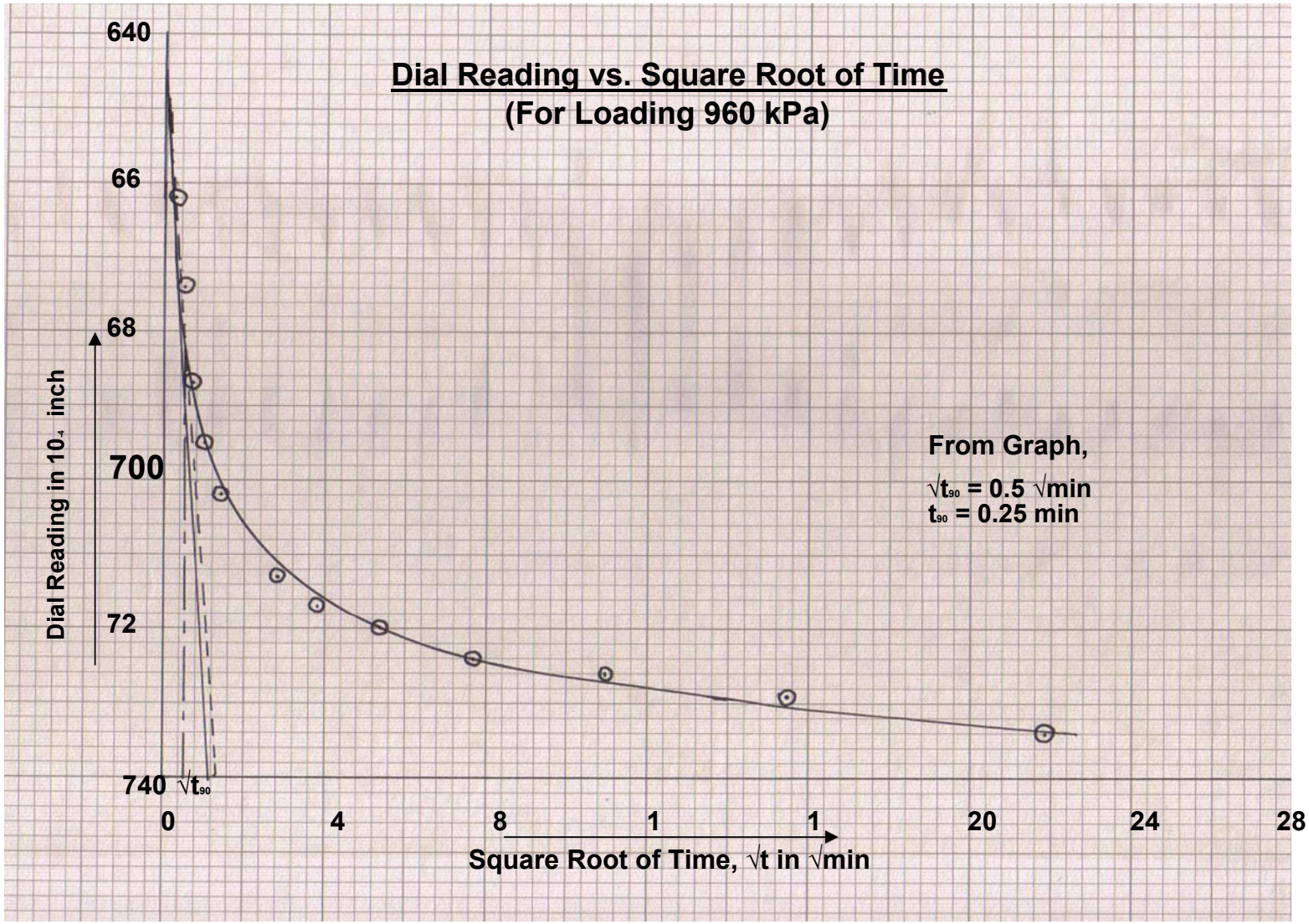

Dial Reading vs. Square Root of Time
(For Loading 480 kPa)
450
46
47
480
49
500 √t90
Dial Reading in 10-4 inch
From Graph,
√t90 = 1.2 √min
t90 = 1.44 min
0
4
8
1
1
20
24
28
Square Root of Time, √t in √min

Load Increment Data:

Target Pressure: **960 kPa**

Actual Applied Pressure: **32 Kg**

Date	Time	Elapsed Time (min)	Square Root of Time (√min)	Dial Reading x 10^{-4} (inch)
04 Feb 08	0800	0	0	503
04 Feb 08	0800	0.1	0.316	662
04 Feb 08	0800	0.25	0.5	674
04 Feb 08	0800	0.5	0.707	687
04 Feb 08	0801	1	1	695
04 Feb 08	0802	2	1.414	702
04 Feb 08	0804	4	2	709
04 Feb 08	0808	8	2.828	713
04 Feb 08	0815	15	3.872	717
04 Feb 08	0830	30	5.477	720
04 Feb 08	0900	60	7.746	724
04 Feb	1000	120	10.954	726

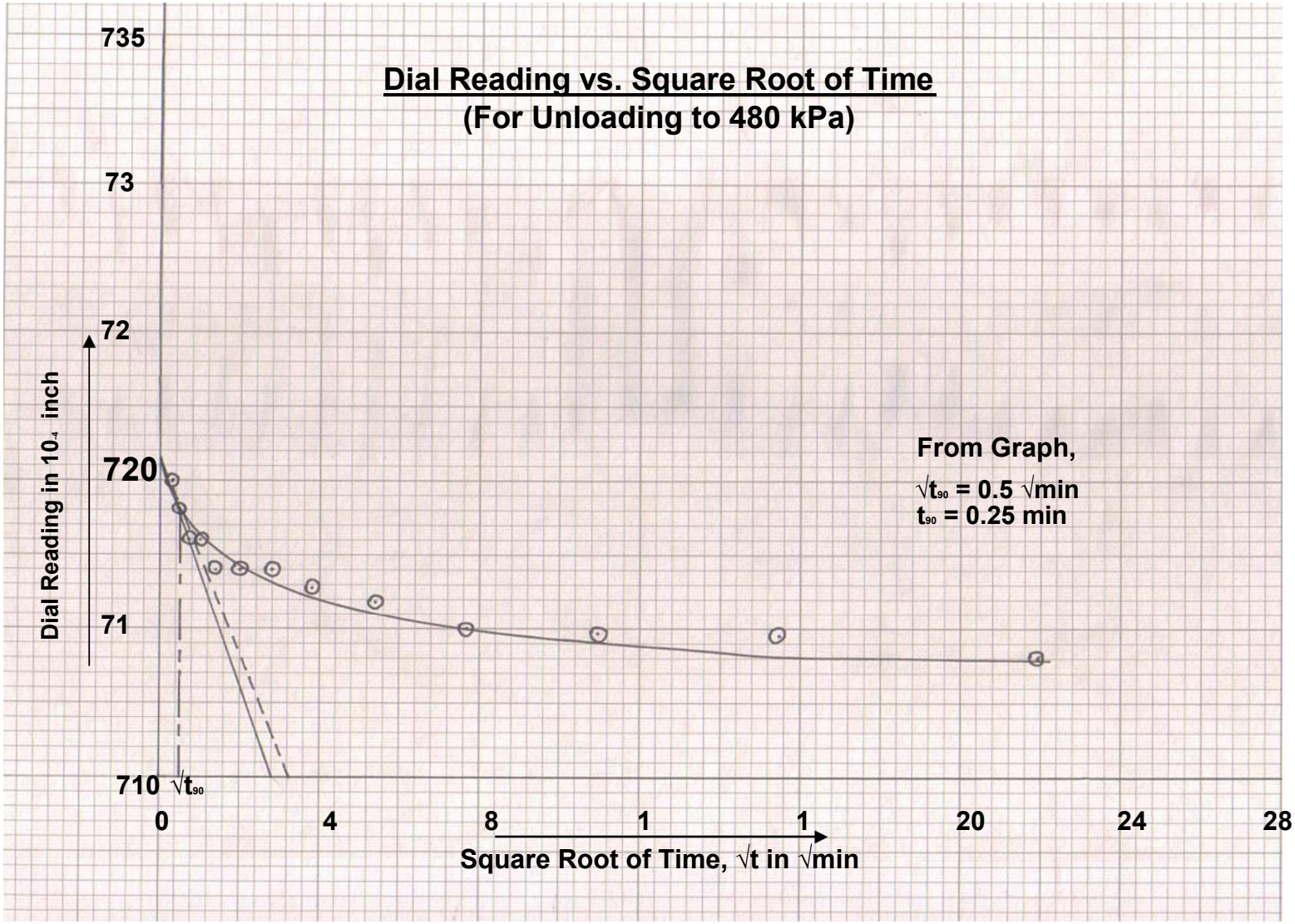

Dial Reading vs. Square Root of Time
(For Loading 960 kPa)
640
66
68
700
72
740
Dial Reading in 10⁻⁴ inch
√t₉₀
From Graph,
√t₉₀ = 0.5 √min
t₉₀ = 0.25 min
0
4
8
1
1
20
24
28
Square Root of Time, √t in √min

Load Decrement Data:

Target Pressure: **480 kPa**

Actual Applied Pressure: **16 Kg**

Date	Time	Elapsed Time (min)	Square Root of Time (√min)	Dial Reading x 10^{-4} (inch)
05 Feb 08	0800	0	0	737.5
05 Feb 08	0800	0.1	0.316	720
05 Feb 08	0800	0.25	0.5	719
05 Feb 08	0800	0.5	0.707	718
05 Feb 08	0801	1	1	718
05 Feb 08	0802	2	1.414	717
05 Feb 08	0804	4	2	717
05 Feb 08	0808	8	2.828	717
05 Feb 08	0815	15	3.872	716.8
05 Feb 08	0830	30	5.477	715.9
05 Feb 08	0900	60	7.746	715
05 Feb	1000	120	10.954	714.9


Dial Reading vs. Square Root of Time
(For Unloading to 480 kPa)
735
73
72
720
71
710
Dial Reading in 10₄ inch
From Graph,
√t₉₀ = 0.5 √min
t₉₀ = 0.25 min
0
4
8
1
1
20
24
28
Square Root of Time, √t in √min

Calculation of C_V for All Load Increment

Applied Pressure, P (kPa)	Final Dial Reading (inch)	Change in Specimen Height (inch)	Final Specimen Height (inch)	Void Height, H_V	Void Ratio, e	Average Height During Consolidation (inch)	Fitting Time, t_{90} (min)	Coefficient of Consolidation, C_V ($inch^2$/min)
0	0	0	0.78	0.26	0.5	-	-	-
15	0.00133	0.00133 (-)	0.7787	0.2587	0.498	0.7794	0.203	0.1586
30	0.00470	0.00337 (-)	0.7753	0.2553	0.491	0.7770	1.103	0.029
60	0.00921	0.00451 (-)	0.7708	0.2508	0.482	0.7731	0.64	0.0495
120	0.0231	0.01389 (-)	0.7569	0.2319	0.446	0.7639	1.44	0.0215
240	0.03185	0.00875 (-)	0.7482	0.2231	0.429	0.7526	0.64	0.0469
480	0.0503	0.01845 (-)	0.7297	0.2047	0.394	0.7390	1.44	0.0201
960	0.07375	0.02345 (-)	0.7063	0.1812	0.348	0.7180	0.25	0.1093
480 (unloading)	0.0713	0.00245 (+)	0.7087	0.1837	0.353	0.7075	0.25	0.1061

Sample Calculations (For Applied Pressure 480 kPa)

Height of solids,

$$H_s = \frac{m_s}{G_s \times \rho_w \times A_r} = \frac{110.6}{2.65 \times \left[1 \times (2.54)^3\right] \times 4.91} = 0.52 inch$$

Initial void height = (0.78 – 0.52) = 0.26 inch

Final dial reading = 0.0503 inch

Change in specimen height = (0.0503 – 0.03185) = 0.01845 inch

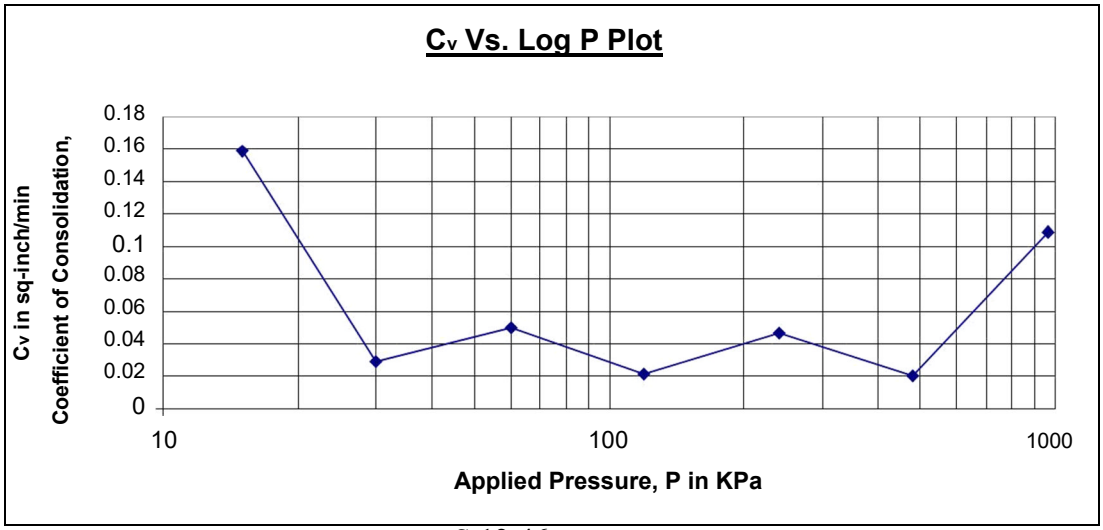
Final specimen height = (0.74815 – 0.01845) = 0.7297 inch

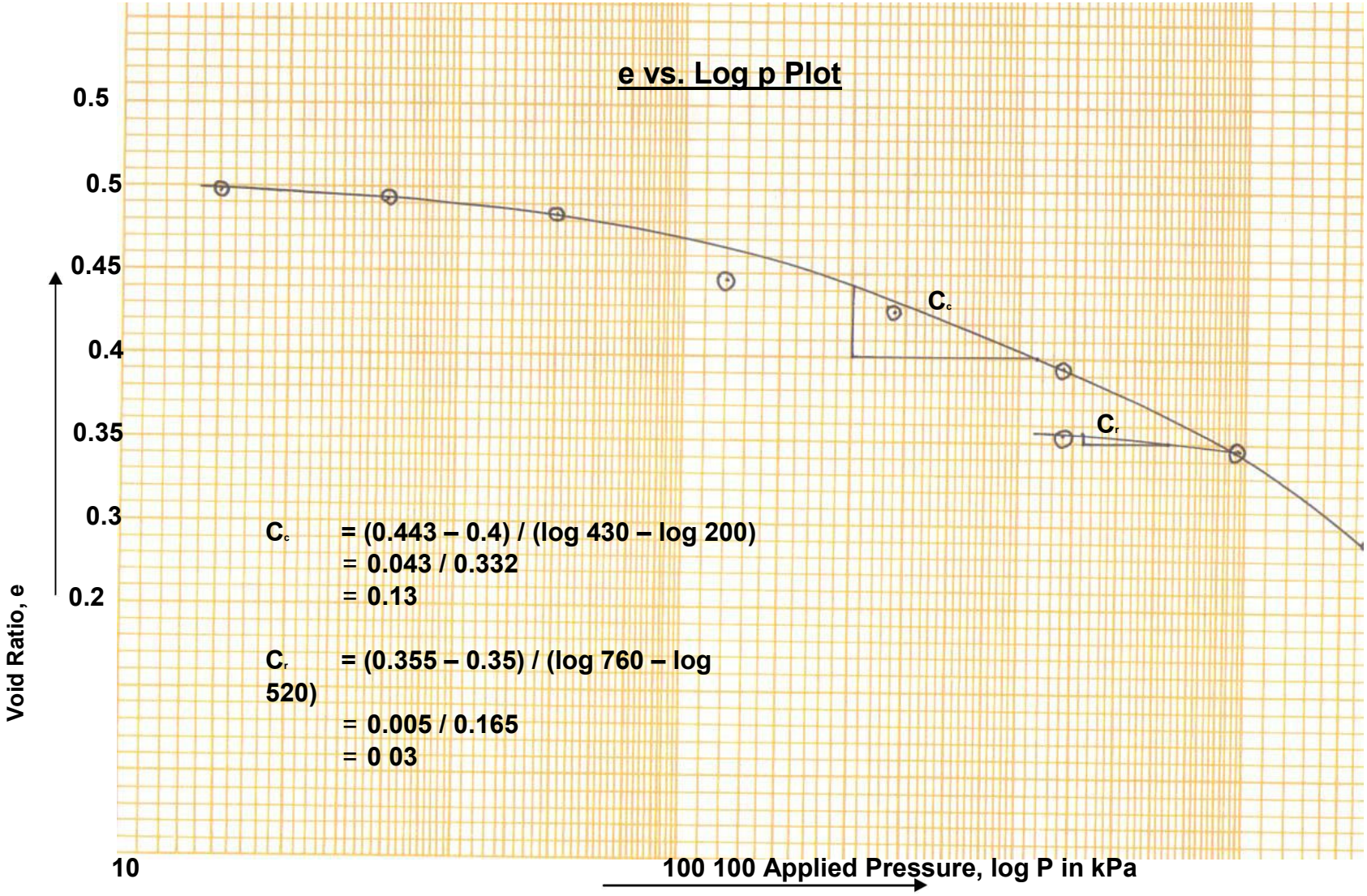
Void height, H_v = (0.2231 – 0.01845) = 0.2047 inch

Void ratio, e = (H_v / H_s) = (0.2047 / 0.52) = 0.394

Average height during consolidation, H_{av} = (0.7297 + 0.7482) = 0.7390 inch For both side drainage condition,

$$C_v = \frac{0.848 \times \frac{Hav^2}{2}}{4 \times t_{90}} = \frac{0.848 \times \frac{0.739^2}{2}}{4 \times 1.44} = 0.0201\ inch^2/\min$$





C_c = (0.443 – 0.4) / (log 430 – log 200)
= 0.043 / 0.332
= 0.13

C_r = (0.355 – 0.35) / (log 760 – log 520)
= 0.005 / 0.165
= 0 03

1
2
-
4
0