

Geotechnical Engineering

LAB MANUAL



William A. Kitch



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Acknowledgements

In the opening chapter of this manual, I write that no significant engineering project has ever been accomplished by an individual. That is certainly true of this work. I am indebted to many people who made this project possible. Several deserve special recognition.

First, I need to thank all the Cal Poly Pomona students whose interest in civil engineering and passion for learning motivate me daily. In particular, I would like to thank Robert Bateman and Jehu Johnson, two exemplary students who volunteered to be photographed performing field and lab tests for this manual.

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Table of Contents

Chapter 1. Course Organization: Guidance for Instructors and Students	1
Chapter 2. Site Exploration: Drilling and Sampling	5
Chapter 3. Moisture Determination: Moisture Content Test	21
Chapter 4. In Situ Unit Weight Determination: Boring Sample Measurements	29
Chapter 5. Soil Plasticity Determination: Atterberg Limits	37
Chapter 6. Particle Size Determination: Part I, Sieve Analysis.....	51
Chapter 7. Particle Size Determination: Part II, Hydrometer Analysis	63
Chapter 8. Laboratory Compaction Determination: Proctor Test	77
Chapter 9. In Situ Unit Weight Determination: Sand Cone Test	89
Chapter 10. Compressibility Determination: Consolidation Test	99
Chapter 11. Strength Determination: Direct Shear Test	121
Chapter 12. Reporting Results: The Geotechnical Report.....	139
Appendices	
A: Field Identification of Soils	143
B: Safe Practices Plans	149
References	153
Field Logging Guide	Laminated Sheet

C H A P T E R 1

Course Organization: Guidance for Instructors and Students

1.1: GUIDANCE FOR INSTRUCTORS

1.1.1: Setting up the Lab Course

I believe that deep student learning is facilitated by a good story. That is to say, there must be a plausible reason or motivation for a student to engage in a learning activity. Unfortunately, too many student lab experiences provide only procedures for measuring a given property or behavior, without any context to help the student understand why that particular property or behavior is important and should be measured. This is understandable because nearly all tests have standardized procedures that must be followed in order to make meaningful, repeatable measurements. However, merely following a standardized procedure generally leads to the *cookbook lab* in which students get caught up in following procedures, making measurements, and reporting results in an arcane report that focuses on what was done and how, without answering the question of why. It is my experience that little learning occurs in the cookbook lab.

This manual uses several techniques to avoid the pitfalls of the cookbook lab. The first is to have a good story. In the context of geotechnical lab testing, a good story is a practical engineering project requiring geotechnical data and analysis. I recommend beginning the class with a realistic project. The design of a shallow foundation is an excellent project for this purpose; it requires students to first measure basic index properties in order to classify soils and then to measure both the stress-strain and strength properties of soils in order to complete the design problem. Another suitable project might be a highway embankment design. Any project that requires determination of both the strength and compressibility properties of soils would be appropriate.

A second technique to provide a context and motivation for performing lab testing is to use real soils from the field and have students collect soil samples themselves. This obviously limits potential soils to those at a depth of 15 feet or less at a location on or close to campus. However, at many locations, this will include a significant variety of soils. The local soils can be supplemented by regional soils collected by commercial testing labs in the area. The additional motivation gained by having students test soils they have themselves collected significantly outweighs the additional cost and time in collecting such samples.

Finally, this manual presents a short description of the purpose of each test and the theory behind the test before presenting the test procedures. The purpose section is intended to tie the test to a specific applied engineering reason for performing the test. The theory section is intended to provide an overview of the theory needed to understand why the procedures work. The theory section is not intended to replace fundamentals covered in a soil mechanics class. It is assumed that the theory has been thoroughly covered in such a class. The theory sections in this manual are intended to reinforce material previously presented.

In summary, this manual is intended primarily for use in an undergraduate soil mechanics lab class. It is intended neither as a guide for researchers nor as a manual of practice for commercial labs. It is instead focused on engaging students in learning fundamental soil mechanics by applying standard lab tests to the solution of realistic, applied engineering problems.

1.1.2: Manual Organization

Because this manual encourages students to collect samples for testing, the first procedures described, in Chapter 2, are for drilling and sampling. This chapter is followed by a section (Chapters 3 through 7) that covers the basic index properties needed to classify soils. The tests in these chapters should be completed before starting the strength and compressibility tests in Chapters 8 through 11. However, there is no specific order for the tests outside of the general organization just presented. Each chapter is designed to stand alone as much as possible. References to earlier chapters have been kept to a minimum. The final chapter of the manual provides a guideline for preparing a geotechnical data report. This chapter assumes you have been performing lab tests for a specific project, as discussed above. The outline is intended to provide a reasonable organization for a typical report of site conditions and soil data for a geotechnical project. It is modeled after state-of-the-practice geotechnical reports.

1.1.3: Use of ASTM Standards

Each chapter contains a list of ASTM standards that apply to the material in that chapter. However, there are notable deviations from ASTM standards in this manual. Deviations from standards are made only when deviations will not detract from student learning. Most often, they are related to saving the precious commodity of class time. The use of non-standard procedures will, in some instances, introduce bias and reduced accuracy in results. Students should be reminded of the importance of following standard procedures in professional practice. Instructors should read and become familiar with ASTM (and other) standards; they should understand when and why we deviate from these procedures and make students aware of important deviations. Some of the most significant deviations are identified in the manual.

1.1.4: Student Teams

No significant engineering project has ever been accomplished by an individual. Practicing engineers, accreditation bodies, and our professional societies all tell us that universities need more and better opportunities to build students' teamwork and leadership skills. A geotechnical engineering lab is an excellent opportunity to learn and practice teamwork skills. Teams of three or four students work well. Each team should officially appoint a project manager who has the responsibility to see that the workload is properly distributed and that deliverables are on time and meet the instructor's standards. If the course is established around a project, as suggested in Section 1.1.1, the team should produce a single lab report.

1.1.5: Safety

Good safety practices are an essential element of professional practice; they save lives, reduce injuries and equipment damage, and increase profits. Good safety practice is about risk management. It starts by indentifying risks, and then managing those risks by taking proactive actions to mitigate risks. Every lab session can, and should, begin with a one- to two-minute safety presentation that identifies specific risks associated with the lab and actions that will be taken to manage those risks.

Appendix B provides example safe practice plans for a typical soils laboratory and field setting where hand augering and sampling is being performed. Instructors are encouraged to adapt and modify these plans for their specific site needs.

1.2: GUIDANCE FOR STUDENTS

1.2.1: Note-taking and Data Collection

One of the most tedious and difficult tasks in any lab setting is to take clear notes and record data so it will be understandable at a later date for analysis. Some good rules to follow are listed below:

- **Write it down once. Check it twice.** Write your data down directly on your data sheet. Do not write data on a scrap of paper and transcribe it later. Every time you transcribe data, you create an opportunity for error. Once you've recorded your data, have a second person check it. Every data sheet in the manual has a place for the data and a location for the signature of both the data-taker and data-checker. Use them.
- **Perform calculations as soon as possible. Now's not too soon!** If you perform your calculations immediately after taking your data and find an error, you can often find the source of the error and correct it on the spot. If you're performing calculations a week after the data was taken, there is little chance of finding the error source and less chance of correcting it.
- **Know what comes next.** If you know where the test procedure is going and why, you will understand what data you need to record now and why.

1.2.2: Care of Samples

Carefully label all samples immediately after collecting them. Few things are more frustrating than spending hours collecting samples only to get back to the lab and find that you don't know which sample is which.

Carefully bag and seal field samples so they don't lose moisture. There's not much value in determining the water content of a sample after it's dried out.

Reseal samples immediately after use. Ensure you are returning samples to the correct container or reattaching the proper tag to samples.

Treat undisturbed samples gently. You should have designated storage containers for your undisturbed samples and store them properly.

1.2.3: Proper Care of Equipment

Lab equipment is expensive. Properly maintained equipment that is properly operated can last for years. Poorly maintained or improperly operated equipment may last only minutes. The first step is to know the proper use of a tool or piece of equipment. If you don't know, ask! The vast majority of equipment damage and loss is due to improper usage.

Clean up after yourself. Never put dirty equipment away. Always clean equipment when you are done with it. Do not put equipment away when it is wet. Dry it or leave it on the counter to dry. Proper cleanup takes time. Leave 5 to 10 minutes at the end of class for cleanup.

If something is broken or malfunctioning, tell your instructor. Often, a malfunctioning piece of equipment can be easily fixed or adjusted. If the problem is not reported, more often than not, the equipment breaks down, which leads to greater expense and longer downtime. Keep your instructor and lab technician informed about the state of lab equipment.

1.2.4: Equipment Calibration

Calibration is the process of checking measuring equipment to ensure that it is producing accurate readings. All measuring equipment should be calibrated on a regular basis. This is particular-

ly true for digital devices. When mechanical devices are not properly functioning and out of calibration, it is often easy to see that there is a problem. However, digital devices seldom provide the same kind of feedback as mechanical devices. Your lab should have a regular program for calibrating measuring devices.

1.2.5: Housekeeping

There is no better indicator of how well a lab is run than its general state of tidiness and organization. Neat tidy operations generally indicate well-run labs with good safety practices. Disorderly, messy operations generally indicate poorly run labs with safety problems. Be part of the solution, not part of the problem. Clean up your working area constantly during the lab. After the lab is complete, put all tools and equipment back in the proper locations. Sweep all countertops and floors and properly dispose of all trash.

1.2.6: Have Fun

Sure, it's work. Yes, it can be confusing at times. But in the long run, if you're not having fun, what's the point? What could be more fun than playing with soils and water? It's what we all did on the playground growing up. Sometime around the end of grade school or in middle school, through peer pressure and some misguided adult influence, we learned that playing with soil was a bad thing. What nonsense! Where's it written that we shouldn't play with soil? Now, finally, for the first time since you were a kid, you have a bona fide excuse to play with soil. So, put on your jeans, roll up your sleeves, and let's get dirty!

C H A P T E R 2

Site Exploration: Drilling and Sampling

2.1: PURPOSE

Geotechnical projects require knowledge of the subsurface conditions at the project site. It is impossible to determine the subsurface conditions without performing some sort of site investigation. There are three main purposes of a site investigation program: identifying and delineating the soil layers, acquiring soil samples for laboratory testing, and performing in situ tests to determine soil properties. The primary means of site exploration is drilling and sampling of soils.

2.2: BACKGROUND

2.2.1: Preliminary Work

Before performing any site investigation, you should first complete a desk study for the project site. The purposes of the desk study are to determine the regional geological setting and identify the types and conditions of soils expected at the site. Resources for a desk study include geologic maps, surveys of historic groundwater level, aerial photographs, agricultural soil maps, and reports of previous investigations at nearby sites.

After completing your desk study, you should complete a *site walk* or *field reconnaissance* of the site. This consists of a simple walk around the site, looking for visual information that will help to confirm the information developed during your desk study. You should try to answer questions such as these:

- What type of soil or rock is exposed at the surface or in nearby road cuts or trenches?
- What are the drainage conditions at the site?
- What is the topography like?
- Is there evidence of previous landslides?
- Are there existing structures at or near the site? If so, do they appear to be performing well?
- Is there evidence of existing development at the site?
- Is the site accessible? Will you have difficulty getting equipment onto or around the site?
- Who manages or controls the site? Do you need special permission or permits to access the site?
- Is there evidence of underground utilities at the site?

A well-done desk study combined with a site walk should allow you to develop a site investigation plan. The goal of this plan is to determine the testing and sampling required to properly investigate the site and the equipment needed to complete the investigation on site.

There are many different types of drilling tools, methods, and equipment. The tools, methods, and equipment must be properly matched to the site conditions and project requirements. There is not much point in bringing a soil-drilling rig to a site if you expect to find bedrock 5 feet below the ground surface.

The plan must be prepared before drilling and sampling. It should include

- an estimate of the number and depth of borings
- the expected depth of the groundwater table
- the expected stability of drill holes
- an approximate number and type of samples required
- the type and number of in situ tests required

A good site investigation plan will save time and money.

2.2.2: Drilling and Sampling Methods

Drilling and sampling is a process of drilling a hole in the ground, called a *boring*, and occasionally stopping the boring to take a sample of soil or to perform an in situ test. The process is a continuous cycle of drilling, sampling, drilling, sampling, until you have investigated to the desired depth of interest.

Drilling Methods

There are two characteristics of the subsurface soils that control the type of drilling equipment used. The first is the ability of the soil to maintain an open bore hole after drilling. If the soil is firm and will hold the hole open after drilling, then the hole can be advanced with a solid-stem *flight auger* or *bucket auger*. These augers are twisted into the ground, and the soil is removed from the boring either through the screw action on a flight auger or in the bucket of the bucket auger. The soil removed from the boring is called *spoil* or *cuttings*. When you wish to take a sample from the bottom of the boring, the auger is simply removed, and the open hole allows access to the bottom of the boring. This is called the *open hole* method of drilling. It is the fastest and cheapest method, but it only works in situations where the soils are strong enough to keep the hole open.

There are conditions that will cause the hole to close up when drilling equipment is removed from the hole. These are *squeezing* and *caving* ground conditions. Squeezing conditions occur in deposits of soft or weak clay, where the strength of the soil is so low that it cannot resist the earth pressures and squeezes into the hole when drilling equipment is removed. Caving conditions occur in clean sands and gravels, especially below the water table. In this case, the lack of cohesion in the soil allows the soil to *ravel* or flow into the boring when drilling equipment is removed from the boring.

In cases of squeezing or caving soils, we must provide some sort of lateral support in the boring to keep the hole open for sampling. Three basic methods are used: casing, hollow stem augering, or drilling with slurry. In the casing method, a hollow pipe, or *casing*, is driven in the hole as the hole is advanced. The casing then holds the hole open for sampling. The *hollow-stem auger* method uses a flight auger similar to that used in the open hole method, but the center of the auger is hollow and provides access to the bottom of the boring. The auger remains in the boring during both drilling and sampling and supports the hole. When you want to take a sample, you place your sampling equipment down the hollow stem of the auger and sample soils at the bottom of the boring. In the *slurry* or *mud drilling* method, the hole is filled with a slurry of bentonite and water or a polymer compound and water. The fluid pressure of the slurry holds the hole open, and you have access through this slurry to the soil at the bottom of the boring.

The second characteristic that controls equipment type is the hardness or toughness of the material being drilled. Most soils can successfully be drilled with auger-type drill bits. However, if the soil is very dense or contains cobbles, it may be necessary to use a rotary bit to break up the soil at the bottom of the hole. Since rotary bits do not have a way to carry cuttings to the surface, these bits are always used for drilling mud. The mud is circulated down the drill rod and up from the bottom of the boring. The mud carries the cuttings to the surface. This system is called the *rotary wash* method. Finally, if soils are very hard, approaching rock-like, they may be drilled using a coring bit in the same fashion as rock coring.

Sample Types and Methods

Soil samples fall into two broad categories: *disturbed* and *undisturbed* samples. In reality, there are not two categories, but a continuum of sample types from completely disturbed to nearly undisturbed. There is no such thing as a totally undisturbed sample. Simply removing the soil from the ground changes the stresses on the soil and generates some sample disturbance. The highest-quality samples are taken with a *thin-walled sampler*, often called a *Shelby tube sampler*. The Shelby tube is pushed into the soil at a constant rate (rather than being driven in by a hammer). Because of the gentle pushing action and the thin wall of the sampler, it leaves the structure of the soil largely intact. Such samples are called undisturbed, even though there is some disturbance to the soil during sampling. The major disadvantage of thin-walled samplers is that they can be used only for soft to medium soils, where they can be pushed into the soil without buckling the wall of the sampler.

At the other end of the spectrum are disturbed, or *bulk samples*. Bulk samples may consist of cuttings removed from auger flights or the material from the bucket of a bucket auger. In taking disturbed samples, there is no attempt to preserve the structure of the in situ soil. However, it is important to preserve the makeup of the soil both in particle size distribution and in moisture content. Therefore, it is important that drillers carefully select bulk samples so that the samples are representative of the in situ soil, and bag or otherwise seal the samples so that they do not lose moisture after sampling.

In between the thin-walled Shelby tube samplers and bulk samples are *thick-walled samplers*. These samplers have relatively thick walls (approximately ¼" thick) and are lined with brass rings that retain the sample. The chief advantage of these samplers is that they can be driven into stiff and dense soils where thin-walled samplers cannot be used. Samples taken with thick-walled samplers are often called undisturbed, but they are significantly more disturbed than samples from thin-walled samplers.

Sample Disturbance and the Area Ratio

A good measure of the amount of disturbance of a sample is the *area ratio* of the sampler. The area ratio, R_a , is the ratio of the cross sectional area of the sampler walls to the area of the sample itself and is computed as

$$R_a = \frac{OD^2 - ID^2}{ID^2} \quad (2.1)$$

where

OD = the outside diameter of the sampler

ID = the inside diameter of the sampler

Table 2.1 presents the dimensions and area ratios for a number of common sizes of thin- and thick-walled samplers. Note that the thin-walled samplers have area ratios substantially less than the thick-

walled samplers. In general, the area-ratio should be less than 20% to be considered an undisturbed sample (Chapter F-2, EM 1110-1-1840, US Army, 2001). When thick-walled samplers are required, you should use a sampler with an area ratio of less than 50% (e.g., 3" diameter or larger). Samples from thick-walled samplers are often called undisturbed, even though the sampler is driven rather than pushed and the area ratio of these samplers is greater than 20%. Also notice that the area ratio for the standard split spoon sampler used for the Standard Penetration Test has an area ratio greater than 100%. Samples from this sampler are always considered disturbed samples.

It is important that boring logs record the type of sampler used. This eliminates confusion and allows anyone reviewing the data to make a personal assessment of the level of disturbance in a sample.

2.2.3: Recording Field Data

Field exploration is expensive and limited. Therefore, it is critical that you get as much information out of your site exploration program as possible. Key tools in maximizing the information gained in a site investigation are the *boring log* and *field notes*. You should start taking your field notes as soon as you arrive on site. The notes should include such information as weather, general site conditions, names and contact information for personnel on site, and types of equipment being used. You should maintain a boring log for each boring. Fill out the boring log as the boring is made; do not wait until after the boring is complete. You will forget or lose information if you do not record it immediately as it becomes available. Your boring log should include

- date and time of drilling
- depth at which each soil was encountered
- location and type of all samples taken
- a preliminary field classification of all soils
- information about the location of the water table
- information about the stability of the hole (stable, caving, squeezing, etc.)

You should also note any unusual problems or issues encountered.

Every sample taken should be properly stored and tagged for identification. The tag should include the date of sampling, boring number, sample number, depth of sample, and a preliminary classification of the soil.

Quality field notes, boring logs, and sample tags are essential to a successful site investigation. They can seem tedious and time consuming while in the field, but they will prove to be invaluable when you return to the office and perform lab tests on your field samples. Take quality notes.

2.3: APPLICABLE STANDARDS

- ASTM D420: Standard Guide to Site Characterization for Engineering, Design, and Construction Purposes
- ASTM D1452: Standard Practice for Soil Investigation and Sampling by Auger Borings
- ASTM D 1486: 08 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
- ASTM D1587: Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes
- ASTM D3550: Standard Practice for Thick Wall, Ring-Lined, Split Barrel, Drive Sampling of Soils
- ASTM D5434: Standard Guide for Field Logging of Subsurface Explorations of Soil and Rock

2.4: EQUIPMENT AND MATERIALS

Augering Equipment

- Sample bucket
- T-Handle
- Extension rods, 4
- Rod pins, 4
- Weeding tool
- Sample bags, small and large
- Sample tags



Thick-Wall Sampling Equipment

- Split barrel sampler
- Drive weight
- Drill rods, long and short with connector
- Anvil connector
- Pipe wrenches, 2
- Tape measure
- Chalk or carpenter's marker
- Wooden board
- Jack
- Soil knife
- Liner rings
- Sample storage tube
- Electrical tape



2.5: PROCEDURE

2.5.1: Preparation

For our field investigation, we will use lightweight hand-operated equipment. The hand-operated equipment will limit the depth you will be able to drill; however, it will also give you a better idea of the variety and variability of soil at the site than you would get using a traditional truck-mounted drilling rig.

You will retrieve both disturbed bulk samples and relatively undisturbed samples with a thick-walled sampler. It is not possible to perform the Standard Penetration Test (SPT) using the available equipment, but you will be able to perform a non-standard penetration test while driving a 3-inch diameter thick-walled sampler. We will refer to this test as the *CPP test* (Cal Poly Pomona test). You can convert CPP N-values to SPT N-values using the equation

$$N_{60} = 0.15 N_{cpp} \quad (2.2)$$

where

N_{60} = standard SPT N-value at 60% hammer efficiency

N_{cpp} = N-value using equipment and procedure described below

This is an imprecise conversion and should be used only for educational purposes. We will call the samples retrieved from this test *thick-wall samples* to distinguish them from bulk samples or undisturbed samples taken using thin-walled samplers.

Before your first day of drilling, you should have completed your desk study and performed a site walk. Based on the information from the desk study and site walk, you should have prepared a site plan showing the desired locations for your two borings. You should also have read the Drilling and Sampling Safe Practices Plan in Appendix B. Ensure you have all the proper personal protective equipment (PPE) when you arrive on site.

2.5.2: Bulk Sample for Compaction Testing

Before you start augering, you will need to take a bulk sample of the surficial soil for laboratory compaction testing.

1. Using a shovel, place 30–40 lbs. of soil from the top 6" of the site into a large sample bag. Avoid getting any plant debris or trash in the sample.
2. Twist the bag closed. Fill out a sample tag with wire ties and attach it to the bag so the bag stays closed.

2.5.3: Augering and Taking Bulk Samples

During this process, you will be advancing the hole using the hand-augering equipment. Each time your boring reaches a new soil layer, you should take a small bulk sample. When you reach a layer where you want a higher-quality sample, stop augering and perform a CPP test to determine N_{cpp} and retrieve a thick-wall sample.

3. Before starting your boring, complete the upper portion of the boring log that contains the general information about the boring (client, project #, date, time, crew, equipment, etc.)
4. Using the shovel, clear the surface by removing any debris or loose soil that might fall into your boring.



5. Assemble the auger by placing the T-handle on top of a rod and the bucket auger on the bottom of the rod. Use the retaining pins to hold the sections together.



6. Turn the T-handle, advancing the auger through hole. Downward pressure may be required to start the boring. Once the boring is 8 to 12 inches deep, the auger will advance by simply turning it.



7. When the auger is filled, remove it from the hole and empty it on a clean plastic sheet. You may need to use the weeding tool to remove the soil from the bucket.



8. Whenever you reach a new soil layer, perform a field classification of the soil using your field logging guide and the visual classification procedures in Appendix A. Your description of the soil should include color; moisture level; cementation, if any; odor, if any; unified soil classification system (USCS) designation (e.g., SM); and the USCS description (e.g., silty sand). For coarse-grained soils, you should include particle shape and relative density. For fine-grained soils, you should include consistency (i.e., soft, medium, stiff, etc). Use standard classification nomenclature, as shown on your field logging guide.
9. For each new layer encountered, bag a bulk sample as soon as possible after the soil is removed from the boring. You want to preserve the in situ moisture content of the soil. For fine sands, silts, and clays, a small bag sample ($\frac{1}{2}$ to 1 lb.) should be sufficient. For medium sands or gravelly sands, you will probably need two small bags ($1\frac{1}{2}$ to 2 lbs.).



10. Immediately complete and attach a tag to your sample. The tag should include the date, project number, boring #, sample #, depth from which the sample was taken, and soil classification.
11. Keep your boring log up to date as you advance your boring. Take notes constantly. Do not wait until the boring is complete to update your boring log.



2.5.4: Performing the CPP Penetration Test and Acquiring Thick-Wall Samples

12. When you reach a layer where you wish take a higher-quality sample, stop advancing the boring. Carefully clean out the bottom of the boring with the auger bucket. Try to get all loose cuttings off the bottom of the boring.

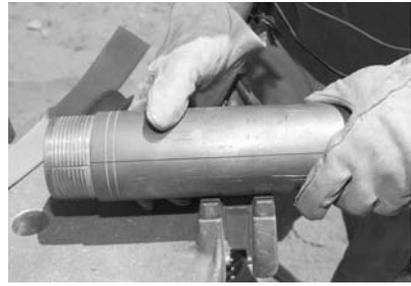
The first day of drilling, you may have a difficult time determining when and where to take an undisturbed sample. Some general rules of thumb are to try to take a sample approximately every 5 feet, and to take one sample from each different soil stratum identified. Your instructor can help you decide where to sample. Before the second day of drilling and sampling, review your boring log from the first day and identify soil layers you did not sample the first day; be sure to sample them the second day.

Assembling the Sampler

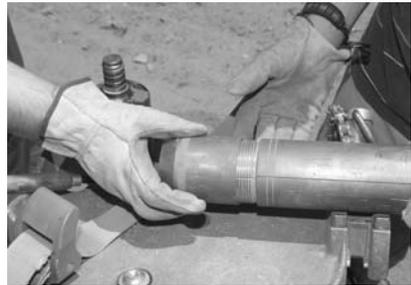
13. Remove the shoe and drive head from the sampler using the strap wrench. Use only the strap wrench when tightening or loosening the sampler shoe or sampler head. Open the sampler.
14. Place the rings on one-half of the sampler, remembering to use clean rings. If the rings are not clean, scrape the inside of the rings with the sample scraper before placing the rings in the sampler.



15. Place the second half of the sampler on the rings. The sampler halves have two circumferential lines scribed at one end. Be sure to line up the scribe lines.



16. Align the two halves, and screw on the drive head and shoe. The scribe lines should be at the bottom, adjacent to the shoe.
17. Clamp the sampler in the plumber's vice and carefully tighten the assembly using the vise and strap wrench. Do not overtighten.



Assembling Drive Rods

18. Inspect the threads on the rods and connectors for any stripping. Replace any damaged rods or connectors.



19. Attach the rods to the sampler and tighten using two pipe wrenches. Tighten the rods as tightly as you can, using the pipe wrenches. If the rods come loose, the threads will get damaged when driving. Attach additional rods as needed, being sure to tighten each connection securely, using both pipe wrenches.



20. Lower the rods into the hole. If the rod length is long, you may wish to use the pipe wrenches to hold the rod as you lower it into the hole.



21. Attach additional rods as needed to raise the rod to a comfortable height for using the drop hammer. Always tighten sections using pipe wrenches.
22. At the top of the rod set, screw on the anvil and a short rod extension above the anvil. Tighten using pipe wrenches.
23. Place the hammer on the top rod. Raise the hammer approximately 12", and drop it once to seat the sampler in the bottom of the hole.

Driving Sampler and Retrieving Sample

24. Place a wood board against the drill rod, and mark the drill rod at intervals of 6" and 12" from the top the board.



25. Mark the drill rod at a distance of 12" above the top of the hammer.



26. Drive the sampler, with one person holding the rod and one person dropping the hammer, from the 12" mark. Count the number of blows required to drive the sampler each 6" interval. Continue driving until you have driven the sampler a total of 12".
27. Using the boring log, record the depth of the sample and the number of blows required to drive the sampler. You should record the number of blows in the format n/m , where n is the number of blows required to drive the first 6", and m is the number of blows required to drive the second 6".



28. To retrieve the sampler, place the jack on top of the wooden board, with the head of the jack below the anvil. Jack the sampler up until it is loose.



29. Once the sampler is loose from the bottom of the boring, carefully lift the rods and sampler to the surface and remove them from the boring.
30. Remove the sampler assembly from the drill rod, using pipe wrenches as needed.



Retrieving Sample from Sampler

31. Place the sampler on a plumber's vise and loosen the drive head and shoe with a pipe wrench, carefully unscrewing the drive head and shoe.



32. Lay the sampler on its side to remove the top half and open the sampler.



33. Identify 6 adjacent rings with the best quality samples, and separate them from the rest of the sample using the knife. Carefully slide the 6-ring sample rings into a plastic storage tube.



34. Properly seal and tape the sample tube lid; then fill out the sample label with date, project number, boring #, sample #, and depth of sample.



35. Place the sample in the sample box.



2.6: PRESENTATION OF RESULTS

As soon as possible after finishing drilling and sampling, complete a draft boring log. The draft log should include all the header data. It should clearly identify each soil layer encountered and have your preliminary soil classification and description for each layer. The location and type of every sample should be annotated and include measured N_{cpp} values.

You will continue to add to this draft boring log as you conduct your laboratory tests. Through your laboratory testing, you may find that your field classifications are not exactly correct. If so, make corrections to the log. You will also gather additional data from your soil samples, such as moisture content, unit weight, cohesion, and friction angle. As you gather these additional data, you should add them to your boring log.

When you have completed all of your laboratory tests, prepare a final boring log containing full soil classifications for each layer. Include all pertinent data on the log. You should prepare your log in a neat professional manner, using a spreadsheet, CAD program, or a program designed specifically to generate boring logs.

2.7: COMMON ERROR SOURCES

By far, the most common error is failure to properly record field data. This leads to misidentification of samples, confusion of boring logs, and a loss of valuable data. To combat this problem you should do the following:

- Complete your desk study and site walk before drilling.
- Prepare a drilling and sampling plan.
- Come to the field prepared with all needed equipment and data sheets.
- Complete the header information on your boring log as soon as you reach the field.
- Take careful and complete notes on your boring log and only on your boring log.
- Tag each sample and fill in the entire sample tag immediately after taking the sample.
- Review your data as soon as possible after leaving the field.

When performing the CPP penetration test, common sources of error include not having a straight hole, driving with loose drill rods, miscounting the blows, and overdriving the sampler. If you do not start your boring vertically, you will have problems throughout the drilling and sampling. Be careful to start your hole vertically. If it is properly started, it will remain straight throughout the entire boring. The vibration of hammering the sampler into the ground will often loosen the drill rods. If the drill rods are loose, you will lose energy when driving, and you will likely damage threads on the rods. To combat this problem, tighten the rods as tightly as possible before starting. If the rods loosen during driving, stop and retighten them. Before you start driving, make sure you know who is counting blows. It helps to have two people count blows to reduce errors.

The inside of a heavy-walled sampler, including the shoe, is only about 15" long. You have to drive the sampler 12" into the ground during the CPP test. If there is too much debris in the bottom of the hole, or if the sample penetrates more than an inch or two during seating, the soil may fill up the sampler before you have driven it 12". To deal with this problem, you should first carefully clean out the bottom of the boring with the bucket auger to get all loose cuttings out of the boring before placing the thick-walled sampler in the boring. While driving, take careful notes. If the blow count rises dramatically during the last one or two inches of driving, it is an indication that you may have overdriven the sampler. Note this in your boring log. When you disassemble the sampler, observe how much soil is inside the sampler. If the sampler is totally full and there is soil pushed up into the sampler head, it is evidence that you have overdriven the sampler. In this case, you should not use the blow count data from this test.

Table 2.1: Dimensions and area ratio of typical soil samplers
(From ASTM D3550, ASTM D1587, and Geomatic® specifications).

Sampler Type	OD (in)	ID (in)	Area Ratio
Thin-Walled	2.00	1.90	11%
	3.00	2.88	9%
	5.00	4.76	10%
Thick-Walled	2.50	1.88	77%
	3.00	2.42	54%
	3.50	2.88	48%
Standard Split Spoon	2.00	1.38	110%

Project:		Project Number:		Client:		Boring No.			
Address, City, State				Drilling Contractor:		Drill Rig Type:			
Logged By:		Date	Started:		Bit Type:		Diameter:		
Drill Crew:			Completed:		Hammer Type:				
USA Ticket Number:			Backfilled:		Hammer Weight:		Hammer Drop:		
			Groundwater Depth:		Elevation:		Total Depth of Boring:		
Depth (feet)	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log	Lithology		Dry Density (pcf)	Moisture Content (%)	Additional Test
					Soil Group Name: modifier, color, moisture, density/consistency, grain size, other descriptors Rock Description: modifier color, hardness/degree of concentration, bedding and joint characteristics, solutions, void conditions.				
5									
10									

Boring Log: Sheet ___ of ___

- Standard Penetration Slit Spoon Sampler (SPT)
- California Sampler
- Shelby Tube
- CPP Sampler
- Bulk/ Bag Sample

- Stabilized Ground water
- Groundwater At time of Drilling

C H A P T E R 3

Moisture Determination: Moisture Content Test

3.1: PURPOSE

Soil behavior is dramatically affected by the amount of water the soil contains. This is particularly true for fine-grained soils. Moisture content is an important index property of a soil and can help you identify some of the basic characteristic of the soil. Fine-grained soils with low moisture contents are generally hard and stiff but may have high swelling potential. Fine-grained soils with high moisture contents are generally soft and weak and have high potential for consolidation.

3.2: THEORY AND DEFINITIONS

In order for the moisture content to be a useful measure, it must provide you with a quantity that describes the relative amount of moisture in the soil. It is of little value to know that a soil sample contains 16 grams of water. What you need to know is how much of the soil structure is made up of water. To accomplish this, we compute the moisture content as the ratio of the mass of the water to the mass of the solid material in a soil using the equation.

$$w = \frac{M_w}{M_s} \times 100\% \quad (3.1)$$

where

- w = moisture content (in percent)
- M_w = mass of the water in the soil sample
- M_s = mass of the soil solids in the soil sample

It may seem unusual that the denominator in moisture content is the mass of the soil solids rather than the total mass of the soil. With the definition shown in Equation 3.1, it is mathematically possible to have a moisture content greater than 100% if the mass of water in a soil exceeds the mass of soil solids. In fact, some highly plastic clays do have a moisture content greater than 100%. We use the definition in Equation 3.1 to determine moisture content because the moisture content of natural soils can vary over time. Water may be added from infiltration or lost through evapotranspiration. When the water table fluctuates, it can also change the moisture content of the soil. When the moisture content changes due to these environmental conditions, not only does the total mass of a given soil deposit change, but the volume of the soil may change also as the soil swells or shrinks. The only thing that remains constant for a given unit of soil is the mass of the solid material. Whether the soil shrinks or swells as water is added or lost, the amount of soil solids in a given unit of soil remains constant.

One benefit of the moisture content test is that it does not require an undisturbed sample. Since measurements are made only of the weight of soil constituents, accurate moisture contents can be measured from disturbed samples as long as the samples are sealed after sampling so no water loss occurs.

3.3: APPLICABLE STANDARDS

- ASTM D2216: Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass

3.4: EQUIPMENT AND MATERIAL

- Spatula
- Soil moisture content tin
- Scale with an accuracy of ± 0.01 g
- Drying oven maintained at $110^\circ \pm 5^\circ\text{C}$



3.5: PROCEDURE

This is a simple test that entails weighing a representative sample of moist soil, drying the sample, reweighing the soil, and then determining the original moisture content, assuming that all of the weight lost during drying is due to the evaporation of free water in the soil.

A metal water content tin will be used to hold the soil sample during testing. Most water content tins have lids. In general, lids are not necessary and their use is discouraged. The only time a lid is needed is when there will be more than a few minutes between when the sample is initially placed in the tin and when the tin is weighed. In these circumstances, a lid is required to ensure that no moisture is lost before the sample is weighed. If a lid is used, the same tin and lid must be kept together the entire test.

1. Check moisture content tin to be sure the tin is clean. If necessary, wipe the tin out. Do not wash the tin with water.
2. Obtain a clean data sheet and fill out the header information.
3. Weigh the empty moisture content tin (with lid, if required) on the laboratory scale and record the mass on the data sheet to the nearest 0.01g. Each tin should be permanently marked with a unique number. Do not put any other marks or place any tape on the tin. The tin should be identified only by the permanent mark. Sticking labels or tape on tins reduces the accuracy of the test (labels lose mass in the oven). Writing on the tin ruins it for future use.
4. Obtain a representative sample of the soil to be tested. To be representative, the sample must be made up of the same mixture of soil materials as the parent soil, must be at the same moisture content as the parent soil, and must be large enough that the water removed during drying can be accurately measured. See Table 3.1 for minimum sample sizes needed. Avoid taking samples from the



edges of soil containers, as these locations are most susceptible to loss or addition of water during sample storage. The most representative samples are taken from the middle of the parent soil sample.

5. Place the representative soil sample in the moisture content tin. Ensure that there is no loose soil hanging on the edges of the tin.



6. Immediately weigh the moist sample and moisture content tin on the scale. Record the mass to the nearest 0.01 g.



7. Place the moisture content tin(s) in a shallow pan and place that pan in the oven. You may label the pan containing the moisture content tin(s) to distinguish your sample(s) from others in the oven. Do not label or mark the individual moisture content tins. Use the unique number stamped on the tin(s) for identification. If any of the tins has a lid, take the lid off the top of the tin and place it on the bottom of the tin.



8. Let the soil dry overnight, approximately 12 to 16 hours.
9. Return after the drying period. Remove the moisture content tin(s) from the oven. Weigh the dried soil and moisture content tin on the scale and record the mass to the nearest 0.01 g.
10. Discard the soil. Clean the moisture content tin(s) without using water, if possible. Return the cleaned and dried moisture content tins to the proper storage location. Do not return the tins or soil to the oven.

3.6: DATA REDUCTION

Recalling that the moisture content is the ratio of the mass of water to the mass of soil solids, the moisture content, w , is computed as

$$w = \frac{M_1 - M_2}{M_2 - M_{tin}} \times 100\% \quad (3.2)$$

where

M_1 = mass of tin with moist soil sample

M_2 = mass of tin with dried soil sample

M_{tin} = mass of empty tin

3.7: PRESENTATION OF RESULTS

The moisture content, w , is normally presented to the nearest tenth of a percent. If the soil sample(s) came from an exploratory boring, the computed moisture content(s) is annotated on the final boring log. Generally, a soils report for a particular site contains a table that lists the boring number, sample number, sample depth, moisture content, and unit weight for all samples obtained at the site.

3.8: TYPICAL RESULTS

Nearly all soils, even those that appear dry, have a moisture content of a few percent. Coarse-grained soils generally have moisture contents less than 35% for saturated soils. Finer-grained soils can have moisture contents ranging from a few percent for desiccated stiff clays to over 100% for saturated highly plastic clays.

3.9: COMMON ERROR SOURCES

The most common source of error is improperly recorded data. Record data directly on data sheets. Avoid transcribing data from scratch sheets to datasheets. Every transcription is an opportunity for error. Carefully check moisture content tins' identification numbers when recording masses. Another common source of error is using a scale without the required accuracy. Check your scale before weighing samples to ensure it has the required accuracy (see Table 3.1).

Table 3.1: Required sample size to determine moisture content to the nearest 0.01% (based on ASTM D2216).

Maximum Particle Size of Sample		Required Sample Size (g)	Required Accuracy of Scale (g)
Diameter (mm)	Standard Sieve Size		
75.0	3 in	50,000	10
17.5	1-1/2 in	10,000	10
19.0	3/4 in	2,500	1
9.5	3/8 in	500	0.1
4.75	No. 4	100	0.1
2.00	No. 10	20	0.01

Moisture Content (Multiple Samples)

Client: _____
 Project Title: _____
 Test Date: _____

Project #: _____
 Test performed by: _____
 Checked by: _____

Boring No.					
Sample No.					
Sample Depth					
Moisture Can ID					
Mass of Can					g
Mass of Soil+Can (wet)					g
Mass of Soil+Can (dry)					g
Moisture Content					%

Boring No.					
Sample No.					
Sample Depth					
Moisture Can ID					
Mass of Can					g
Mass of Soil+Can (wet)					g
Mass of Soil+Can (dry)					g
Moisture Content					%

C H A P T E R 4

In Situ Unit Weight Determination: Boring Sample Measurements

4.1: PURPOSE

In order to compute the in situ state of stress, cut and fill quantities, and perform most basic geotechnical analyses, it is necessary to know the in situ unit weight of soils. Using undisturbed soil samples, it is possible to measure unit weight in the laboratory. These tests are commonly called “moisture-density tests” because they entail measuring both the moisture content and unit weight.

4.2: THEORY AND DEFINITIONS

In order to compute the unit weight of a soil, it is necessary to determine both the mass and the volume of a given in situ soil sample. Given these quantities, the unit weight can then be computed as

$$\gamma = \frac{W_t}{V_t} \quad (4.1)$$

where

$$\begin{aligned} \gamma &= \text{total unit weight of the soil} \\ W_t &= \text{total weight of the soil sample} \\ V_t &= \text{total volume of the soil sample} \end{aligned}$$

In addition to determining the total unit weight, γ , it is customary to also determine the dry unit weight, γ_d . The dry unit weight is not measured directly because drying out the sample would actually change its volume and give an inaccurate measure of the dry unit weight. The dry unit weight is computed by mathematically removing the water. First, you must measure the in situ moisture content of the soil and then compute the dry unit weight as

$$\gamma_d = \frac{\gamma}{1 + w} \quad (4.2)$$

where

$$\begin{aligned} \gamma_d &= \text{dry unit weight of the soil} \\ w &= \text{in situ moisture content of the soil sample, in decimal form} \end{aligned}$$

Therefore, the quantities that must be determined are the volume of the sample, the total weight of the soil in the sample, and the moisture content of the soil in the sample.

4.3: APPLICABLE STANDARDS

- ASTM D2937 Test Method for Density of Soil in Place by the Drive-Cylinder Method

4.4: EQUIPMENT AND MATERIALS

- Scale with an accuracy of ± 0.01 g
- Drying oven maintained at $110^\circ \pm 5^\circ\text{C}$
- Soil moisture content tin with lid
- Soil knife
- Wire saw
- Metal rule marked in mm
- Empty soil sampler rings
- Pan



4.5: PROCEDURE

This procedure assumes you have taken an undisturbed soil sample using a lined split spoon-type sampler. It is easily adapted for any undisturbed cylindrical soil sample. For determination of the density of surficial soils, see the sand cone test procedure in Chapter 9.

This test requires measurement of moisture content. If you have not previously performed a moisture content test, you should read the procedures for that test before starting this test.

4.5.1: Set Up

1. Obtain an undisturbed soil sample for which you wish to know the unit weight.
2. Obtain a clean data sheet and fill out the header information.
3. Obtain a soil moisture tin with lid. Ensure that it is clean. Weight the tin with lid and record the mass on the data sheet to the nearest 0.01 g.
4. Obtain 10 to 20 sample liner rings of the same type as used for your undisturbed sample. Ensure the sample rings are free of debris and soil.

4.5.2: Average Ring Weight Determination

5. Place the clean, empty sampler rings on the scale and record their mass on the data sheet to the nearest 0.01 g. Also record the number of rings weighed.
6. On the data sheet, compute the average mass of a single ring.



4.5.3: Unit Weight Determination

The objective in this test is to determine the unit weight while keeping as much of the undisturbed sample intact. In this manner, the sample may be used for further tests. Therefore, you should use as much of the original sample as possible without separating the sample rings.

7. Carefully remove the rings and soil from the plastic tube, being sure to keep the entire sample intact. Check the top and bottom ring to determine if the entire sample is of the same soil. If not, you will have to separate the sample into rings of the same soil.

8. If the soil extends beyond the ring on either end, carefully trim it back so it is flush with the ring. You may have to remove one or two rings from the end of the sample if there are holes at the ends of the sample, but try to keep as much of the sample intact as possible. If small voids exist below the end of the ring, you may fill them with recompacted soil. If large voids are present, then remove one ring and trim back to the next ring. As you trim the sample, collect your trimmings in your pre-weighed moisture content tin. Keep the lid on the tin so the trimmings don't dry out before you have a chance to weigh the moist soil.



9. Weigh the rings with the trimmed soil and record the combined mass on the data sheet.



10. Measure the inside diameter and height of the ring to the nearest 0.1 cm and record this information on the data sheet, along with the number of rings in your sample.



11. Return the undisturbed sample to its container. If you removed any rings of soil during testing, add the empty rings to the container so the sample doesn't slide around in the container. Reseal the container. If you have been careful in performing the test, you will be able to use the remaining sample for other lab tests.

4.5.4: Moisture Content Determination

In order to determine the dry unit weight, we must measure the moisture content of the sample. We want to use as little soil as possible in this step because we wish to retain the rest of the sample for other tests.

12. If not done earlier, obtain a moisture content tin, determine its mass, and record both the tin identification number and its mass on the data sheet. You should not use a lid on this tin.
13. Obtain a representative portion of the soil sample for use in a moisture content test. If you have collected a sufficient quantity of soil during previous trimming of the sample, you may use that as long as it has not been allowed to dry out. Otherwise, you will have to obtain a soil sample from either the top or bottom ring of the undisturbed sample. Refer to the moisture content test for required sample size and procedures in the moisture content test.
14. Record moisture content data on the data sheet.

4.6: DATA REDUCTION

Recalling that density of the soil is simply the mass of soil divided by the volume, compute the density, ρ , as

$$\rho = \frac{M_s - n \cdot M_r}{n \cdot \frac{\pi}{4} D^2 h} \quad (4.3)$$

where

- ρ = density of the soil sample (g/cm^3)
- M_s = mass of the soil sample and rings (g)
- M_r = average mass of a single ring (g)
- n = number of rings in the soil sample
- D = inside diameter of a sample ring (cm)
- h = height of a sample ring (cm)

Now it is necessary to convert the density of the soil in g/cm^3 to unit weight, γ , in either lb/ft^3 or kN/m^3 using the equation

$$\gamma = \rho \cdot \gamma_w \quad (4.4)$$

where

- γ_w = unit weight of water ($62.4 \text{ lb}/\text{ft}^3$ or $9.81 \text{ kN}/\text{m}^3$)

Once you have determined the moisture content of the sample, you may compute the dry unit weight, γ_d , using Equation 4.2.

These computations should be performed and recorded on your data sheet.

4.7: PRESENTATION OF RESULTS

If the soil sample(s) came from an exploratory boring, the computed moisture content(s) and unit weight(s) are annotated on the final boring log. Generally, a soils report for a particular site contains a table that lists the boring number, sample number, sample depth, moisture content, and unit weights for all samples obtained at the site.

4.8: TYPICAL RESULTS

The in situ unit weight for coarse-grained soils typically ranges from 100 to 140 lb/ft³ (17.5 to 22.0 kN/m³). For fine-grained soils, the typical range is from 80 to 130 lb/ft³ (12.5 to 20.5 kN/m³). See Table 4.1 of Coduto (1999) for a list of typical unit weights as a function of soil type. The dry unit weight will always be less than the total unit weight.

4.9: COMMON ERROR SOURCES

The most common errors in this test are caused by data reduction errors. Carefully check your units when reducing data, and compare your computed unit weights to published data. Other data error sources include not trimming the sample flush with the ends; leaving or creating holes in the sample during trimming; or performing the test on a sample that consists of two different soils.

Unit Weight

Client: _____ Project #: _____
 Project Title: _____ Test performed by: _____
 Test Date: _____ Checked by: _____
 Boring No. : _____ Sample No. : _____ Sample Depth: _____
 Soil Classification: _____

Moisture Content Test

Moisture Can ID Number	
Mass of Can	g
Mass of Soil+Can (wet)	g
Mass of Soil+Can (dry)	g
Moisture Content	%

Average Ring Mass

Number of rings	
Total mass	g
Average mass for one ring	g

Unit Weight

Total Number of Rings	
Inside Diameter of Ring	cm
Height of Ring	cm
Weight of Rings + Soil	g
Average Weight of Empty Rings	g

Density of Soil	g/cm ³	
Unit Weight of Soil	lb/ft ³	kN/m ³

Dry Unit Weight

	lb/ft ³	kN/m ³
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Soil Plasticity

Determination: Atterberg Limits

5.1: PURPOSE

Fine-grained soils (clays and silts) have a significant ability to absorb and retain moisture. As these soils absorb more moisture, they become softer and more compressible. Conversely, as these soils lose moisture, they shrink and become stiffer. The plasticity of a soil is its ability to hold moisture and remain in a deformable state. In order to compare the plasticity of different fine-grained soils, we must have a common way of measuring soils' ability to absorb and retain moisture. The Atterberg Limits tests provide this common framework.

5.2: THEORY AND DEFINITIONS

In 1911, the Swedish soil scientist, A. Atterberg, developed standardized tests to measure soil plasticity. The Atterberg Limits tests were initially developed for agricultural use, but were adopted by civil engineers particularly to help classify highway subgrade soils. Arthur Casagrande revised and standardized these tests in 1932. The current standard is described in ASTM D4318.

There are three Atterberg Limits: the liquid limit, the plastic limit, and the shrinkage limit, each with its own standardized test. However, only the first two are commonly used in engineering practice, so we will cover only the liquid limit and the plastic limit tests. These tests are important in the classification of fine-grained soils. In particular, the liquid limit and plastic limit are used to classify fine-grained soils.

The Atterberg Limits tests quantify the relationship between the consistency of a soil and its moisture content. ASTM defines *consistency* as “the relative ease with which a soil can be deformed.” Thus, a soil with a soft consistency can be easily deformed, while one with a stiff consistency cannot be deformed without cracking and falling apart.

Consider a sample of a clayey soil that is hard, having a very low moisture content (less than 10%). If you tried to mold or deform this soil, it would simply break into pieces. This soil would have a hard consistency and would be in a solid state.

If you added water to the soil in a solid state, it would move from a hard to a stiff consistency. At some point, you would be able to deform the soil into different shapes, but it would crack as you deformed it. This soil would be in a semi-solid state, as shown in Figure 5.1a.

If you added water to the soil in Figure 5.1a, the consistency of the soil would change from stiff to medium to soft. At some point, you would be able to mold the soil into different shapes without having the soil crack. This soil would then be in a plastic state, as shown in Figure 5.1b.

If you continued to add water to the soil, the consistency of the soil would change from soft to very soft. Eventually, the soil would act as a viscous liquid, as shown in Figure 5.1c. This soil would be in the liquid state.

The purpose of the Atterberg Limits tests is to determine the boundaries between solid, semi-solid, and liquid states. The moisture content separating the semi-solid and plastic states is called the *plastic limit* (PL, or w_p). The moisture content separating the plastic and liquid states is called the *liquid limit* (LL, or w_l). The Atterberg Limits tests provide a standardized method to determine the plastic and liquid limits.

Another way to think of the Atterberg Limits is as the upper and lower bounds of moisture contents between which the soil will act in a plastic state. For example, if the test indicates the soil is at its liquid limit when it has a moisture content of 61%, then its liquid limit (LL) is 61. Likewise, if the tests determine a soil is at the plastic limit when its moisture content is 22%, then the plastic limit (PL) is 22.

The *plasticity index* (PI) is the numerical difference between the liquid limit and the plastic limit and is a numerical measure of how much water a soil can absorb between the plastic and liquid states. In this example, the $PI = 61 - 22 = 39$.

The test procedures may appear to be crude and unscientific. However, they continue to be used for the following reasons:

- When performed by experienced soil technicians, these tests produce consistent and repeatable results.
- The tests have been used for many years, so we have a large base of experience founded on these test results.
- The test equipment is simple and inexpensive.

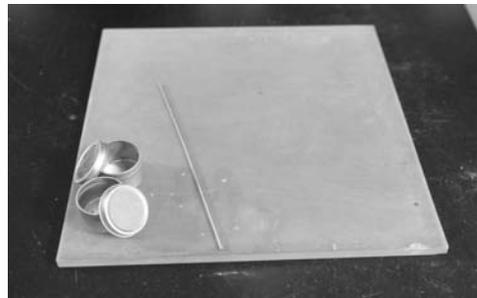
Be careful not to connect the reliability and value of a test with the cost or sophistication of the test equipment. Simple equipment often produces the best results.

5.3: APPLICABLE STANDARDS

- ASTM D4318: Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

5.4: EQUIPMENT AND MATERIALS

- Mortar and pestle
- #40 sieve
- Stainless steel bowl
- Evaporating dish
- 7 Moisture content tins
- Spatula
- Wash bottle filled with deionized water
- Liquid Limit device
- Grooving and calibrating tool
- Glass plate
- 1/8" diameter brass rod
- Scale with an accuracy of ± 0.01 g



5.5: PROCEDURE

You will need 150–300 g of soil for these tests. You do not need to use undisturbed samples for these tests. If you have a disturbed bag sample of the soil of interest, you should use that sample rather than destroying an undisturbed sample of the soil.

These tests are performed on only the portion of a soil that passes the #40 sieve (very fine sand, silt, and clay). Visually inspect your soil sample. If it contains little or no material greater than the #40 sieve, you may skip the sample preparation section below. However, if your sample contains significant amounts of material greater than the #40 sieve, follow the sample preparation procedures.

5.5.1: Sample Preparation

Before performing the tests, we must pass the soil through a #40 sieve to remove larger particles from the sample. However, in sieving the soil, it is important that we don't confuse silt and clay lumps with sand and gravel particles. We want to keep the sand and gravel out of the sample we test while ensuring that the silt and clay materials are in the sample we test. Therefore, we follow this procedure to prepare the sample.

1. Remove the soil from its container and place it in a stainless steel bowl. If you're using an undisturbed sample, remove the sample from the rings.
2. The soil must be dry enough that you can sieve it, but not so dry that you cannot break up the clay and silt lumps. Check the moisture of the removed soil; if needed, dry this soil by laying it out in the sun or placing it in the microwave oven until it can be pulverized by a mortar and pestle. Ask your instructor for help determining the proper amount of drying.

- Once the soil is properly dried, pulverize it using a mortar and pestle. When using the mortar and pestle, be sure to break down all of the silty and clayey clumps. A common mistake is to break down only the silty lumps, but not the harder clayey lumps. Therefore, be sure to break down the clay, too.



- Once you have properly pulverized the soil, place it on a #40 sieve and gently work through the sieve that portion of the sample that readily passes the sieve. Return that portion retained on the sieve to the mortar and pulverize it again. Resieve the portion you just pulverized. Any soil retained on the sieve is discarded. Save the portion passing through the sieve for the liquid and plastic limits tests. You will need approximately 125 g for the tests.

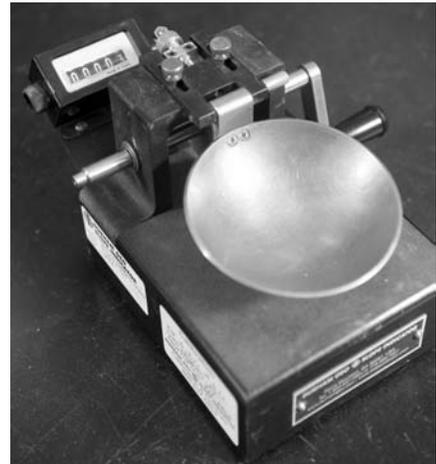


5.5.2 Liquid Limit Test

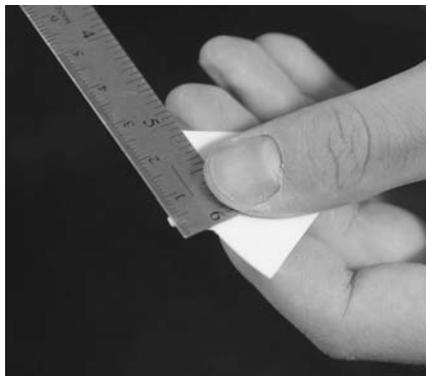
Calibration of the Liquid Limit Device

Before you can conduct the liquid limit test, it is necessary to adjust and calibrate the liquid limit device as follows:

- Inspect the liquid limit device for cleanliness. Turn the handle to see how the cup is lifted up and dropped. The cup should easily move up and down without binding, but it shouldn't wobble in its pivots. Check the screws that hold the cup to the pivot and tighten if needed.



- Check the grooving tool for cleanliness and inspect the tip of the tool. The tip should be 1.9–2.1 mm wide. If the tip of the tool is wider than 2.1 mm, the tool is worn out and should be discarded. While examining the grooving tool, measure the dimensions of the width and height of the calibration tab at the back of the tool handle.



7. Verify that the fall height of the cup is 10 mm. Raise the cup to its highest position; then slide the grooving tool under the cup with the calibration tab up. You should feel the tab touching the bottom of the cup as you slide it back and forth under the cup, but it should not raise the cup visibly. If necessary, adjust the cup by turning the adjustment screw at the back of the device. Be sure to tighten the set screws once you've adjusted the fall height. Recheck the fall height after tightening the set screws.



Obtaining the First Data Point

It is nearly impossible to prepare a soil sample that is exactly at the liquid limit. Rather than attempting to get a test point exactly at the liquid limit, we will acquire data for several data points close to, but above and below, the liquid limit and then interpolate between them to determine the liquid limit.

8. Obtain five moisture content cans, weigh each one, and record the masses and can identification numbers on the data sheet.
9. Place approximately 100 g of soil in an evaporating dish. Slowly add water to this soil and mix it using a spatula. Continue until the soil has a consistency slightly stiffer than that of toothpaste.



10. Place the soil in the lower portion of the cup in the liquid limit device and squeeze it down with a spatula. When you are finished, the soil should be free from large air pockets, have an approximately horizontal surface, and be about 10 mm thick at its thickest point. A common mistake is to put too much or too little soil in the cup.



11. Form a V-shaped groove in the soil by drawing the grooving tool through the soil. This groove should extend along the length of the sample from top to bottom and should penetrate through the sample. Use a single, smooth motion to cut the groove.



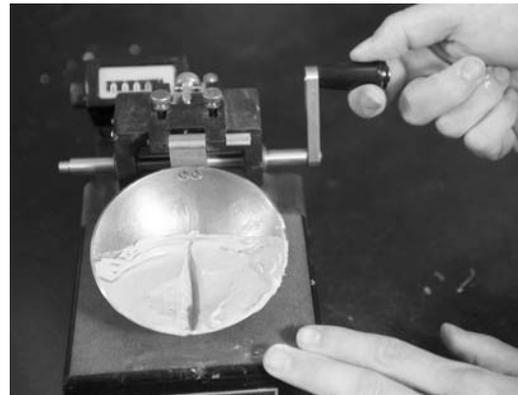
12. Examine the groove you have cut. There should be a clear brass line visible at the bottom of the groove and along its entire length. At the top of each side of the groove, a small amount of soil should have been removed by the shoulders of the grooving tool. This indicates that the depth of soil placed in the cup is correct. If no soil is removed by the shoulders, then there is not enough soil in the cup. More soil should be added and the groove recut. If, however, a second wide groove has been cut by the shoulders of the grooving tool, then some soil should be removed from the cup. If you have to add or remove soil, remold the soil in the bottom of the cup and cut a new groove.



13. Verify that the base of the liquid limit device is clean and that no soil scraps are present between the bottom of the cup and the base.

14. If the device has a counter, set it to zero.

15. Lift and drop the cup by rotating the crank counterclockwise. The cup should drop at a rate of about 2 blows per second. Count the number of blows and continue this operation until the two halves of the soil pat come in contact along the bottom of the groove for a distance of 13 mm (0.5 in). Use the 13 mm width of the grooving tool calibration tab to help you determine when the correct length of soil has been closed.



16. When you reach the point where a 13 mm (0.5 in) section of the groove is closed, stop cranking. If 25 blows were required to close the gap for a distance of 13 mm (0.5 in), then the soil is, by definition, at its liquid limit. If more than 25 blows were required, the soil is dryer than the liquid limit; if less than 25 were required, it is wetter than the liquid limit.

17. If the number of blows was less than 15, the soil is much too wet; if it was greater than 40, the soil is much too dry. If either is the case, do not record the data. Remove the soil from the cup. Remix the soil with the original sample. Adjust the moisture content of the sample, and try again, starting with Step 10.



18. If the number of blows was between 15 and 40, record the number on the data sheet under "1st try."

19. Without removing the soil sample from the cup, remold the soil in the cup to close the groove without changing its moisture content. Then, repeat Steps 10–15 and record the blow count on the data sheet under "2nd try."

20. If the difference between the first and second blow counts is less than 3, remove the sample and conduct a moisture content test. If not, repack the soil and repeat Steps 3-7 again until you obtain a suitably consistent blow count.
21. When you have a consistent pair of blow counts, perform a moisture content test. Take the moisture content sample from the center of the cup in the location where the soil is the thickest and where the groove was closed. Don't take the sample near the edges where it might not be representative of the average moisture content of the soil.



When plotting the data, use the final blow count and moisture content from the sample taken after the last trial. Don't average the blow counts. You are comparing the different trials to ensure you have a consistent technique and there are not large errors. However, the moisture content of the soil will change as you move through several trials. So always use the blow count and moisture content from the final trial.

22. Empty and thoroughly clean the cup so that no soil remains. This is important: soil left in the cup can dry and form a rough crust that will adversely affect subsequent tests.

Obtaining the Remaining Data Points

We wish to have a total of at least three data points: one requiring 15 to 25 blows for closure, one requiring 20 to 30 blows for closure, and one requiring 25 to 35 blows for closure. Repeat the process described above to obtain these points. It may take 4 or 5 trials to get three points properly spread across the range from 15 to 35.

If the blow count of the previous point was less than 25, the soil was wetter than the liquid limit. Therefore, try testing a slightly drier sample. Likewise, if the blow count of the previous point was greater than 25, the soil was drier than the liquid limit, and you should try a slightly wetter sample.

Don't worry if you never obtain exactly 25 blows. During the data analysis, we will determine the liquid limit by interpolation. However, you want most of your data points to be between 15 and 35. If you have points lying outside this range, continue until you have three data points spread between 15 and 35, as outlined above.

5.5.3: Plastic Limit Test

In the liquid limit test, we intentionally tested the soil at various moisture contents, none of which were likely to be exactly at the liquid limit (i.e., exactly 25 blows). We then determined the liquid limit by interpolation. However, the plastic limit test is performed differently. The idea here is to start with a sample slightly above (moister than) the plastic limit, and gradually dry the sample until it reaches the plastic limit. When the sample reaches the plastic limit, it is saved for a moisture content measurement. This test uses a small amount of soil, so it will be repeated several times in order to get a suitable-sized sample and to get an average moisture content from many tests. The average moisture content will be used to determine the plastic limit (PL).

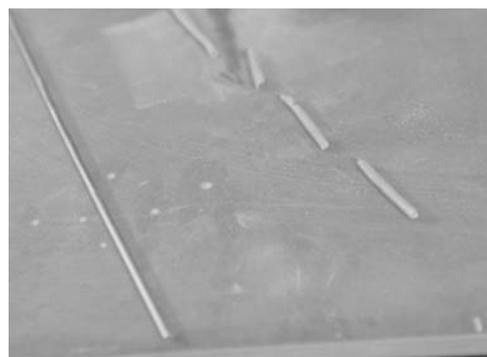
The test procedure is as follows:

23. Obtain two moisture content cans with lids, weigh them with lids, and record the masses and can identification numbers on the data sheet.

24. Obtain approximately 20 g of the prepared soil. Then, select enough of the sample to form a $\frac{1}{4}$ - to $\frac{3}{8}$ -inch diameter ball. Roll this ball between the palm of your hand and the glass plate, using just enough pressure to form a thread of uniform diameter throughout its length. Continue rolling the thread into a smaller diameter, while trying to keep it from breaking. If the thread becomes more than 2 inches long, you may discard the excess and continue rolling the 2-inch section.



25. When the thread finally does break, compare its diameter with that of the $\frac{1}{8}$ -inch diameter rod. If the soil diameter is greater than $\frac{1}{8}$ inch, then it is drier than the plastic limit. Conversely, if it is smaller than $\frac{1}{8}$ inch, then it is too wet. However, if it breaks at a diameter of exactly $\frac{1}{8}$ inch, it is, by definition, at the plastic limit.



26. If the soil is too wet, dry it using any of the following methods:
- Spread it out on the glass plate and allow it to air dry.
 - Roll it in your hands.
 - Blot it with paper towels.
- If it is too dry, add a small amount of water and thoroughly mix. Then repeat the rolling operation.
27. When you have achieved the moisture content that corresponds to the plastic limit (i.e., the soil thread breaks when $\frac{1}{8}$ -inch in diameter), then place it into a moisture content can and cover it with a lid.
28. Repeat Steps 22–24 until a total of five soil threads are in the moisture content tin.
29. Perform a moisture content test on each sample. Be sure to remove the lid and place it underneath the tin before placing it into the oven.
30. Repeat Steps 22–26 to obtain a second moisture content can with five samples. Obtain its moisture content in the same way.

5.6: DATA REDUCTION

5.6.1: Liquid Limit Test

Plot your data points on a semi-log plot, with moisture content on an arithmetic vertical axis and blow count on a logarithmic horizontal axis. Connect these points with a best-fit straight line. The moisture content that corresponds to the intersection of this line and the 25 blows coordinate is the liquid limit. Express it as a whole number with the percent sign dropped (e.g., if 25 blows corresponds to $w = 43.7\%$, the liquid limit is 44.)

5.6.2: Plastic Limit Test

Use the average of the moisture contents from the two moisture content tests.

5.6.3: Plasticity Index

Compute the plasticity index (PI), which is the numerical difference between the liquid limit and the plastic limit.

$$PI = LL - PL \quad (5.1)$$

5.6.4: Unified Soil Classification System

Using the test results and an estimated (or measured) percentage passing the #200 sieve, determine the unified group symbol and group name.

5.7: PRESENTATION OF RESULTS

The Atterberg Limits are most often presented using the liquid limit (LL) and plasticity index (PI). If the soil sample(s) came from an exploratory boring, the liquid limit (LL) is often annotated on the final boring log, along with the natural moisture content of the sample. Generally, a soils report for a particular site contains a table that lists the boring number, sample number, sample depth, natural moisture content, liquid limit, and plasticity index for all fine-grained soil samples obtained at the site.

5.8: TYPICAL RESULTS

The plastic and liquid limits (and, therefore, the plasticity index) will vary with clay content and mineralogy. Table 5.1 presents typical values of liquid limit and plasticity index for various soil types.

5.9: COMMON ERROR SOURCES

The most common source of error in Atterberg Limits testing is the inexperience of the technician; this can lead to difficulty in determining when the soil has reached its plastic or liquid limit. Another error source is incomplete absorption of water, particularly for highly plastic soils. The higher the plasticity of the soil, the longer it will take for it to absorb moisture. If tests are run too quickly, the liquid limit may be underestimated. Finally, there are some soils that undergo irreversible dehydration. For these soils, it is critical that tests be performed starting at high moisture contents and systematically decreasing the moisture content. Drying these soils and then rehydrating them will generate erroneous results.

Table 5.1: Typical Atterberg Limits for various soils.

Soil Type	Typical Clay Mineral	Liquid Limit (LL)	Plasticity Index (PI)
Non-plastic silt	Kaolinite or silts	< 50	< 10
Low-plasticity clay	Kaolinite or Illite	< 50	10 – 15
High-plasticity clay	Bentonite, montmorillonite, smectite	> 50	> 20

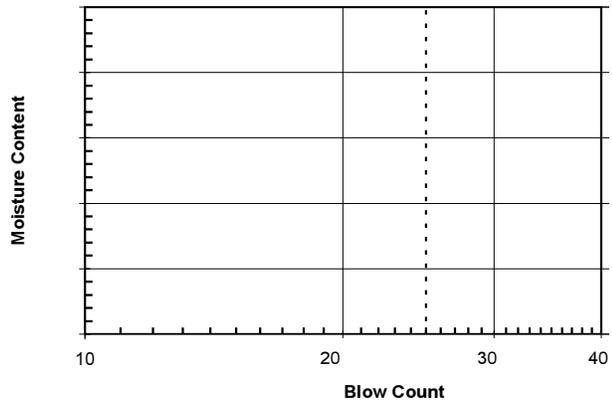
Atterberg Limits

Client: _____ Project #: _____
 Project Title: _____ Test performed by: _____
 Test Date: _____ Checked by: _____
 Boring No. : _____ Sample No. : _____ Sample Depth: _____
 Soil Classification: _____

Liquid Limit Test

Trial No.	1	2	3	4	5
No. of blows (1st try)					
No. of blows (2nd try)					
No. of blows (3rd try)					
No. of blows (4th try)					
Moisture can number					
mass of can (g)					
mass of can + moist soil (g)					
Mass of can + dry soil (g)					
Moisture content (%)					

LL



Plastic Limit Test

Trial No.	1	2
Moisture can number		
mass of can (g)		
mass of can + moist soil (g)		
Mass of can + dry soil (g)		
Moisture content (%)		

PL (average)

Plasticity Index

PI = ^{LL} - ^{PL} = _____

Particle Size Determination: Part I, Sieve Analysis

6.1: PURPOSE

You will need to know the soil particle size distribution, often called *grain size distribution*, for a number of reasons. Perhaps the most obvious use of grain size distribution is for soil classification. It is impossible to determine the proper classification of a soil without knowing what percentage of the soil is in the gravel versus sand versus fine-grained category. In addition to classification, grain size distribution is useful in estimating such soil properties as hydraulic conductivity.

6.2: THEORY AND DEFINITIONS

Determining the grain size distribution is conceptually a simple process. A soil sample is separated into sub-samples of similar-sized particles from large to small. The mass of each sub-sample is determined. Knowing the total mass of the sample and the mass of each sub-sample, it is possible to generate a graph of the cumulative percent of the sample that is finer than a given size. This plot of percent finer versus particle diameter is called a *grain size distribution curve*. The grain size distribution curve is plotted on a semi-logarithmic scale, as shown in Figure 6.1. The percent smaller than a given grain size is also referred to as the *percent passing* a given grain size. The terms *percent finer* and *percent passing* are equivalent; both are used interchangeably in practice. The grain size is always measured in millimeters.

The relationship between percent finer and a given size is expressed by the *D-size* of the soil, using the following nomenclature:

D_{xx} = the size of particle where $xx\%$ of the soil is smaller

For example, from Figure 1.1, the following D-sizes can be determined:

$D_{60} = 2.0$ mm

$D_{30} = 0.61$ mm

$D_{10} = 0.17$ mm

6.2.1: Methods for Measuring Grain Size Distribution

There are two methods for determining the grain size distribution of a soil. For coarse-grained soils, gravels and sands, the grain size distribution can be measured simply by passing the sample of the soil through a series of decreasing-sized sieves and determining the mass of soil retained on each sieve. This is the sieve analysis method.

For fine-grained soils, silts, and clays, sieving is not an effective method for determining grain size distribution for several reasons. Particles this small have significant static surface charges and tend

to stick to surfaces of the sieves rather than passing through them. Clay and silt particles clump together forming clods of soil that are much larger than the individual particles themselves. Finally, the clay and silt particles tend to stick to the larger sand and gravel particles rather than passing through the sieves.

Size distribution for fine-grained soils is determined through a sedimentation process called the hydrometer analysis. This method uses the fact that larger soil particles settle out of suspension faster than smaller particles to determine the distribution of grain sizes.

The effective ranges of both the sieve and hydrometer analyses are shown in Figure 6.1. The choice of which method to use will depend on the size of material in the soil to be tested. For coarse-grained soils (> 0.075 mm), the sieve analysis will be most effective. For fine-grained soils (< 0.075 mm), the hydrometer analysis will be required.

6.2.2: Selecting the Proper Test Method

Depending on the relative amount of coarse-grained (sand and gravel) and fine-grained (clay and silt) material in a soil, the grain size distribution may be determined by the sieve analysis, the hydrometer analysis, or by a combination of both methods. The following general guidelines apply:

- If the sample is essentially all coarse-grained (> 0.075 mm), then a sieve analysis is all that is required. As a rule of thumb, if less than 5% of a sample, by mass, is finer than 0.075 mm (#200 sieve), then a sieve analysis is sufficient.
- If the sample is essentially all fine sand, silt, and clay, then a hydrometer analysis is all that is required. As a rule of thumb, if 80% or more of a sample, by mass, is finer than 0.075 mm (#200 sieve), then a hydrometer analysis is sufficient.
- If the soil is a mix of fine- and coarse-grained materials, then both a sieve analysis and hydrometer analysis will be needed to properly characterize the grain size distribution.

Use these descriptions only as a guideline. If you are unsure which test methods will be required, discuss the situation with your instructor before proceeding.

The procedure for hydrometer analysis is covered in Chapter 7. The remainder of this chapter will deal with the sieve analysis.

6.2.3: Wet-Washing Samples

As discussed above, fine-grain materials tend to clump together and stick to larger soil particles. This can cause erroneous results in a sieve analysis. To eliminate these problems, we will use a procedure called wet-washing to prepare samples for the sieve analysis.

In wet-washing, the fine-grained materials are removed from the sample before sieving by washing the sample with water over a #200 sieve. This procedure flushes all of the fine material out of the sample, leaving only the coarse material for sieving. The amount of fine material must be accounted for in the analysis. Therefore, it is necessary to determine the dry mass of the sample before and after washing. The difference between these two masses is material finer than the #200 sieve.

In the wet-washing process, it is not practical to capture the fine material. It is simply washed down the drain. Therefore, this process does not allow for determination of grain size distribution finer than the #200 sieve.

6.3: EQUIPMENT AND MATERIALS

6.3.1: Sample Preparation

- Sample splitter (if available)
- Large ceramic evaporating dish
- Moisture content tin
- Deep #200 sieve with reinforcement under the #200 screen (for wet-washing)
- Water bottle
- Spatula



6.3.2: Sieve Analysis

- Nest of standard sieves
 - Lid, 3", 1½", ¾", ⅜", #4, #8, #16, #30, #50, #100, #200, pan. Not all these sieve sizes will be needed. Required sieve sizes will depend upon the size of the particles in the soil sample.
- Motorized sieve shaker
- Scale with an accuracy of ± 0.01 g
- Drying oven maintained at $110^\circ \pm 5^\circ\text{C}$



6.4: APPLICABLE STANDARDS

- ~~ASTM D421: Practice for Dry Preparation of Soil Samples for Particle Size Analysis and Determination of Soil Constants~~
- ~~ASTM D422: Standard Test Method for Particle Size Analysis of Soils~~
- ASTM D1140: Standard Test Methods for Amount of Material in Soils Finer than No. 200 (75- μm) Sieve

6.5: PROCEDURE

6.5.1: Selecting a Representative Sample

1. Determine the size of the sample needed based upon the size of the maximum soil particle in the sample, according to Table 6.1. If your source sample is significantly larger than the required minimum sample, you will have to reduce your sample either by quartering or using a sample splitter.

Quartering a Sample

2. Place a plastic sheet large enough to hold the entire sample on the floor or on a table. Dump the entire sample into a pile in the middle of the sheet.
3. Divide the pile into equal quarters using a straightedge, spatula, or trowel.
4. Retain one-quarter of the sample for testing. Return the rest of the sample to the original container for future testing.
5. If one-quarter is still too large, split the sample again using the same procedure.



Using a Sample Splitter

6. Place the sample splitter on a level location on the floor and place two sample pans under the splitter, one on each side. Ensure the sample pans are completely under the splitter.
7. Dump or scoop the entire sample into the top of the splitter. Use care to reduce dust during this process. Besides causing a breathing hazard, excessive dust creates a loss of fine material from the sample.
8. Remove the pans from under the sample. Return the material in one pan for testing. Return the material in the second pan to the original container for future testing.
9. If the material in one pan is still too large, split the sample again using the same procedure.



6.5.2: Measuring Moisture Content of the Test Sample

10. Select a representative sub-sample of the test sample and determine its moisture content using standard moisture content procedures.
11. Obtain a large ceramic evaporating dish, determine its mass, and record it on the data sheet.
12. Weigh the remainder of the sample to a precision of 1 g.

6.5.3: Wet-Washing the Sample

If your sample contains less than approximately 20% fines, you can wash the entire sample at one time. If there are significant fines in the sample, then it will be faster to wash the sample in 4 to 6 sub-samples.

13. Obtain a “deep dish” #200 sieve. Ensure you are using a sieve with reinforcement under the #200 screen.
14. Carefully place a portion of the test sample inside the “deep-dish” #200 sieve. Do not lose any of the soil.
15. Wet-wash the sample using the hose or the faucet. Using the hose, spray water into the top of the sieve, thus washing the soil particles finer than the #200 sieve through the sieve and down the drain. Carefully break up any clumps of fine-grained material by simply touching the clumps with your finger. Do not attempt to push the clumps through the #200 sieve, as this will damage the sieve.



16. Wash the sample until the wash water is clear and you are satisfied that only the portion greater than the #200 sieve remains. It is helpful to wash the sample into a light-colored dish to help determine when the wash water is clear.
17. Using a water bottle, carefully transfer the contents of the sieve into the pre-weighed ceramic evaporating dish. The objective is to save all of the material greater than the #200 sieve; so be careful to wash all of the material remaining on the sieve into the dish.



18. Let the sample sit on the counter for a few minutes to allow the material to settle to the bottom. Pour out any excess water, being careful not to lose any of the remaining material.



19. Place the evaporating dish and soil into the microwave oven. Heat it until the soil is dry, using heating periods of no longer than 2 minutes each. At the end of each 2-minute period, carefully stir the soil with a clean spatula to allow water vapor to escape.
20. When the soil appears dry, reduce the heating time to periods of 1 minute. Between each heating period, stir the soil with a clean spatula and weigh the soil. When the soil mass changes by less than 0.1 g between successive heating periods, the soil is dry. Take the final mass to be the mass of the dried soil and evaporating dish. Record this quantity on the data sheet.



6.5.4: Sieving the Soil

21. Select the necessary sieve stack using the following process: Select the largest sieve to be one sieve size larger than the largest apparent particle diameter in the sample. For the remaining sieves, each sieve should be approximately half the size (or twice the sieve #) of the previous (larger) sieve. Your sieve stack must include the #4 and #200 sieves, as well as a pan below the #200 sieve.

22. Weigh each sieve to a precision of 0.1g. Record the data on your data sheet.
23. Assemble your sieve stack. Make sure that the sieves are in the proper order: those with the largest openings on top of the stack. Make sure you have a lid for the top and a pan on the bottom of the stack.



24. Pour the oven-dried soil from Step 20 into the top sieve. Be careful not to lose any of this soil.
25. Place the sieve cover on top of the stack and shake the stack by hand until the soil has passed through the upper few sieves.
26. Inspect the upper few sieves to see if they have retained any soil. Remove the uppermost sieves that have not retained any soil and replace the lid.



27. Place the stack in the sieve shaker. Carefully secure the stack in the shaker using the clamping device on the shaker. Shake the stack for approximately 10 minutes.
28. Remove the sieve stack intact from the sieve shaker and place it next to the scale.



29. Carefully remove one sieve at a time from the stack. Use a screwdriver to pry the sieves apart. If you simply pull the sieves apart, they are likely to pop off suddenly, spilling your sample.



30. Weigh each sieve with its contents to a precision of 0.1g. Record the data on your data sheet.



6.6: DATA REDUCTION

Data reduction for the sieve analysis is relatively simple if you apply the following two concepts. First, the grain size distribution curve is based on dry soil mass. The initial sample mass was measured with the soil at its existing moisture content. It is necessary to convert this total mass to dry mass based on the moisture content of the sample. Second, the mass of material finer than the #200 sieve is the sum of two different measurements in the tests: the dry mass of soil washed out of the sample during wet-washing and the mass of material in the pan during sieving. Be sure to combine these two values when computing both the total sample mass and the mass of soil passing the #200 sieve.

31. Compute the moisture content of the sample from the moisture content sample taken. Use the standard procedures from Chapter 3.
32. Compute the dry mass (mass of solids) of the entire soil sample (before fines were washed out of the sample) based on the measured total mass and moisture content as

$$M_d = \frac{M_t}{(1 + w)} \quad (6.1)$$

where

M_d = dry mass of sample

M_t = total mass of sample

w = moisture content of sample expressed in decimal form

33. Compute the mass of solids washed down the drain and add that mass to the mass of solids in the sieve pan. This will be the total mass of solids passing the #200 sieve.
34. Compute the mass of soil retained on each sieve.
35. Compute the cumulative mass of soil retained on each sieve as the summation of the individual mass retained on that sieve and all sieves above that sieve in the stack.
36. For each sieve size, compute the percent coarser as the cumulative mass of soil retained divided by the total dry mass of the sample.
37. Compute the percent passing each sieve as 100% minus the percent coarser. The table on the data sheet gives a convenient format for these computations.
38. Verify that the sum of the mass of soil retained on all the sieves and the pan plus the dry mass of the soil washed out is nearly equal to the dry mass of the sample before wet-washing. If there is a significant difference, there are errors in your data.
39. Plot the grain size distribution curve using the percentages computed in Step 36 and the sieve sizes listed in Table 6.2.

40. If the soil is a sand or gravel, it is necessary to compute the *coefficient of uniformity*, C_u , and the *coefficient of concavity*, C_c , in order to classify the soil using the Unified Soil Classification System (USCS). C_u and C_c are computed as

$$C_u = \frac{D_{60}}{D_{10}} \quad (6.2)$$

$$C_c = \frac{D_{30}^2}{D_{10}D_{60}} \quad (6.3)$$

where

D_{60} = is the diameter at which 60% of the soil passes

D_{30} = is the diameter at which 30% of the soil passes

D_{10} = is the diameter at which 10% of the soil passes

If the soil contains more than 10% fines (passing the #200 sieve), it will be necessary to perform a hydrometer analysis in addition to the sieve analysis in order to compute C_u and C_c .

6.7: PRESENTATION OF RESULTS

Grain size distribution is generally presented in a graphical form as a plot of percent passing (or percent finer) versus the log of mean grain diameter, similar to that shown in Figure 6.1. You may prefer to plot the grain size distribution curve with the x-axis plotted in the opposite direction of that shown in Figure 6.1. That method plots the larger grain size on the left and decreases to the right. Either method is acceptable.

6.8: TYPICAL RESULTS

Grain size distribution curves can vary dramatically based on the type of soil being tested. Mathematically, the curves must always increase from left to right when plotted as shown in Figure 6.1. The steeper the plot, the more uniform the soil. The shallower the plot, the more well-graded the soil.

The best check of the grain size distribution curve is to compare the results to a visual classification of the soil. If the soil appears to be a sand, then most of the soil should be retained between the #200 and #4 sieves. If the soil appears to be very uniform, then the plot should be steep, whereas a shallow plot should indicate a well-graded soil. Such visual checks of the soil will often identify errors in the data or the data reduction.

6.9: COMMON ERROR SOURCES

During data collection, the two most common error sources are holes in the sieves and excessive loss of soil during sieving. All sieves should be carefully inspected before use. Damaged sieves should not be used. If the loss of soil during the test is more than a few percent, the test should be repeated.

During data reduction, the most common error is failure to include the dry mass of the soil washed out of the sample during wet-washing. Remember that the soil washed out during wet-washing is part of the original soil sample. The dry mass of the soil washed out must be computed and included as part of the mass of the soil less than the #200 sieve.

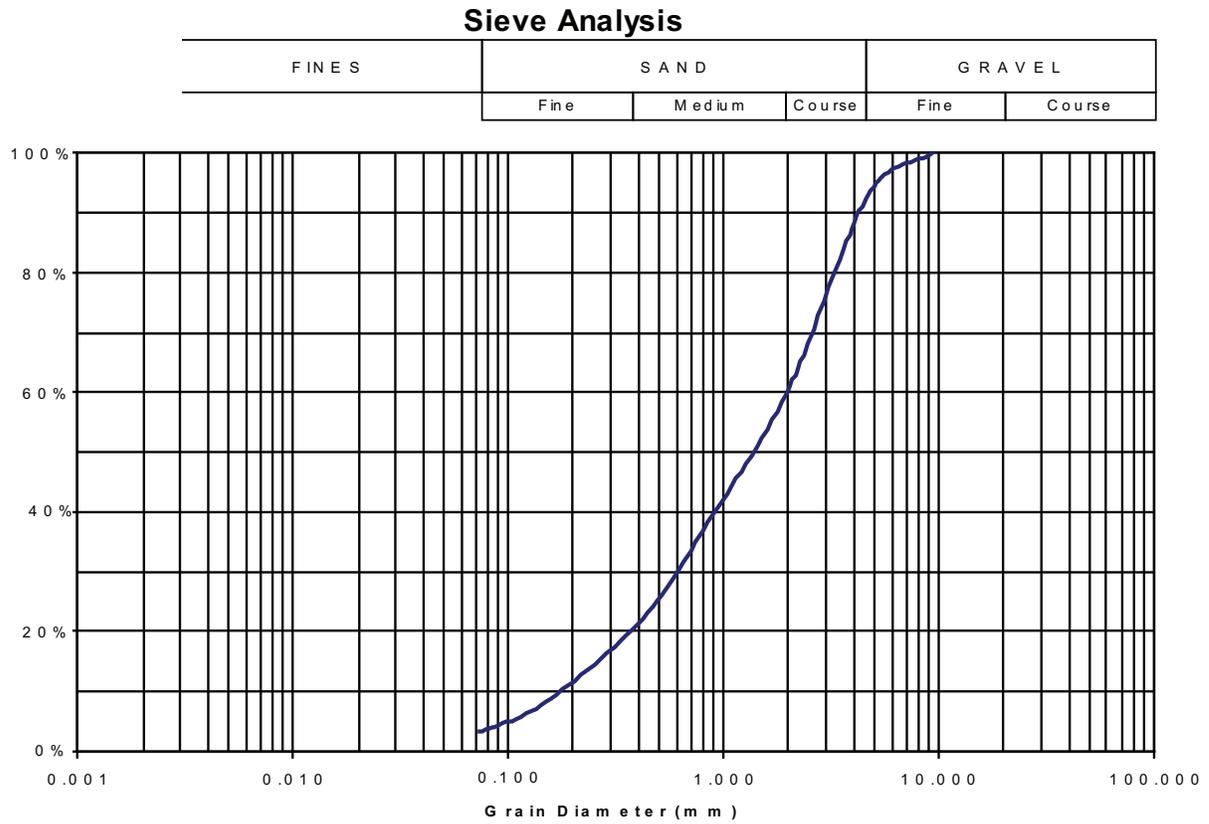


Figure 6.1: Example of a Grain Size Distribution Curve for a Coarse-Grained Soil.

Table 6.1: Minimum sample size for grain size determination based on maximum grain size (based on ASTM D 422).

Nominal Diameter of Largest Particles, in. (mm)	Approximate Minimum Mass of Sample Required, g
$\frac{3}{8}$ (9.5)	500
$\frac{3}{4}$ (19.0)	1000
1 (25.4)	2000
$1\frac{1}{2}$ (38.1)	3000
2 (50.8)	4000

Table 6.2: Opening size of standard sieves.

Sieve Number	Opening Diameter (mm)
3 in.	75
1½ in.	37.5
¾ in.	19.0
⅜ in.	9.5
#4	4.75
#8	2.36
#10	2.00
#16	1.18
#20	0.850
#30	0.600
#40	0.425
#50	0.300
#60	0.250
#100	0.150
#140	0.106
#200	0.075

Particle Size Determination: Part II, Hydrometer Analysis

7.1: PURPOSE

The hydrometer analysis is a companion to the sieve analysis presented in Chapter 6. As discussed in Chapter 6, the hydrometer analysis is used to determine the distribution of grain size for fine-grained soils. It can either be used alone for determining grain size distribution of silts and clays or in combination with the sieve analysis for determining grain size distribution of mixed coarse- and fine-grained soils.

7.2: THEORY AND DEFINITIONS

The hydrometer analysis takes advantage of the observation that large-sized particles fall out of suspension in water (sediment) faster than small-sized particles. Because fine-grained soils (silts and clays) cannot be effectively sieved, this sedimentation property of soils is useful in determining the grain size distribution of fine-grained soils.

In its simplest form, the hydrometer analysis consists of making a slurry of soil and water; mixing the slurry up so all the soil particles are suspended in the slurry; and letting the slurry rest after mixing to observe the rate at which the soil sediments settle out of the slurry to the bottom of the container. The faster the slurry settles out, the coarser the soil. The slower the slurry settles out, the finer the soil.

7.2.1: Stokes Law and the Sedimentation Process

Sir George Stokes studied the sedimentation of spheres in a fluid, and published his results in 1891. The formula that describes this phenomenon, known as Stokes Law, is

$$v = \frac{D^2 g (\rho_s - \rho_L)}{18 \eta} \quad (7.1)$$

where:

- v = terminal velocity of a spherical particle falling freely in a liquid
- D = diameter of the particle
- g = acceleration due to gravity
- ρ_s = mass density of the particle
- ρ_L = mass density of the fluid
- η = dynamic viscosity of the fluid

Clay particles are far from being spherical, so a hydrometer analysis based on Stokes Law produces the diameter of equivalent spheres, which is suitable for practical design problems.

Assuming the particle accelerates to its terminal velocity very quickly, we can write the equation:

$$v = \frac{H}{T} \quad (7.2)$$

where:

H = vertical distance a particle falls

T = time required to fall a distance, H

The hydrometer test is based on these two formulas. The test determines the amount of soil in suspension by measuring the specific gravity of the slurry at various time intervals and the rate of settlement by comparing these measurements.

7.2.2: Soil Dispersion

In order for the hydrometer analysis to accurately measure grain size, we must ensure that the individual soil particles are dispersed and not flocculated. This is especially true with clay minerals, which can be highly flocculated due to charges on the surfaces of the clay particles. We will add sodium hexametaphosphate to the soil slurry, which acts as a dispersing agent by neutralizing the surface charges on the soil particles, breaking up the flocs, and dispersing the individual clay particles.

7.2.3: Measuring Specific Gravity of the Soil Slurry

The hydrometer analysis requires that we measure the rate of sedimentation in order to determine grain size distribution. It is difficult to measure the amount of soil that has settled out of the soil slurry. However, it is relatively simple to determine how much of the soil remains suspended in the slurry at any given time. This is accomplished by measuring the specific gravity of the slurry. The higher the specific gravity, the more soil is suspended in the slurry.

The specific gravity is measured by use of a hydrometer (thus, the name of this technique.) The hydrometer is a weighted glass bulb with a stem on the top. It floats in the slurry somewhat like a fishing bobber, with the stem sticking out of the slurry. The greater the specific gravity of the slurry, the higher the hydrometer floats, and the more of the stem that sticks out above the slurry.

The stem of the hydrometer contains a graduated scale in units of specific gravity. Hydrometers are calibrated such that when placed in pure water at 20°C, the bottom of the meniscus formed between the water and the stem reads 1.000. When we read the hydrometer floating in the soil slurry mixture, we must make corrections to the reading for three reasons: 1) we can't read the bottom of the meniscus because the soil slurry is not clear, so we must read the hydrometer at the top of the meniscus; 2) measurements are not generally made at 20°C; and 3) we're adding sodium hexametaphosphate to the water, which increases the specific gravity of suspension fluid. We account for all three of these corrections by preparing a control sample of the suspension fluid that has the same temperature and concentration of sodium hexametaphosphate as that used for the test slurry. We then measure the specific gravity of this control sample, reading from the top of the meniscus, and use this reading as a correction to the reading from the actual sample.

7.3: APPLICABLE STANDARDS

- [ASTM D421: Practice for Dry Preparation of Soil Samples for Particle Size Analysis and Determination of Soil Constants](#)
- [ASTM D422: Standard Test Method for Particle Size Analysis of Soils](#)
- ASTM D1140: Standard Test Methods for Amount of Material in Soils Finer than No. 200 (75- μm) Sieve

7.4: EQUIPMENT AND MATERIALS

7.4.1: Sample Preparation

- Mortar and rubber-tipped pestle
- #10 sieve, sieve top, and pan
- Small evaporating dish
- 500 ml beaker
- Soil hydrometer, type 151H



7.4.2 Hydrometer Analysis

- Two 1000 ml hydrometer cylinders
- Electric mixer
- Mixer cup
- Water bottle
- Thermometer
- Spatula
- Sodium hexametaphosphate
- Scale with an accuracy of ± 0.01 g
- Drying oven maintained at $110^\circ \pm 5^\circ\text{C}$



7.5: PROCEDURE

The hydrometer analysis is performed using only that portion of the soil finer than the #10 sieve. Therefore, the first step of the hydrometer analysis is to separate that portion of your sample greater than the #10 sieve from that portion finer.

When you are done with the separation process, you must have at least 65 grams of soil finer than the #10 sieve, if your soil is predominately silt and clay. If there is a significant amount of sand in your sample, then you will need at least 115 grams of soil finer than the #10 sieve. You will have to estimate the size of your initial soil sample such that you end up with a large enough portion of the sample less than the #10 sieve.

If your sample has no materials coarser than the #10 sieve, you may skip the following sample preparation procedures.

7.5.1: Sample Preparation

1. Weigh and record the tare mass of an empty evaporating dish.
2. Visually estimate the percentage of soil coarser than the #10 sieve and weigh out a sample of soil such that you will have the appropriate amount of material finer than the #10 sieve after sieving.

Example: Your sample has a significant amount of sand. Therefore, you will need at least 115 grams of material finer than the #10 sieve. You estimate that 20% of your sample is greater than the #10 sieve. Therefore, you need to weigh out at least $115 \text{ g} \times 1.20 = 138$ grams of soil sample.

3. Use the quartering technique or a sample splitter as needed to obtain the appropriately-sized representative sample.
4. If the sample is too moist to sieve, dry the sample out by setting it in the sun or placing it in the drying oven for a few minutes. The sample does not have to be completely dried. It simply needs to be dry enough that the clods of silt and clay can be broken up with the mortar and pestle such that they can pass through the #10 sieve.
5. Once the soil is sufficiently dried, record the mass of the sample on the data sheet.
6. Place the soil sample in the mortar and break up the clumps of clay and silt with the pestle. Use just enough force to break up the silt and clay clumps. You do not want to break up the larger soil pieces, just the silt and clay clumps.



7. Create a sieve stack consisting of a pan, a #10 sieve, and a lid. Place the sample on the #10 sieve and sieve the soil, collecting the material finer than the #10 sieve in the sieve pan.
8. Return the material retained on the #10 sieve to the mortar. Break up any clay and silt lumps that were unable to pass through the #10 sieve.
9. Return the material from the mortar to the #10 sieve and sieve again, collecting the material finer than the #10 sieve in the sieve pan.
10. Repeat Steps 8 and 9 until all the clay and silt clumps have broken up and passed through the #10 sieve.



11. Wet-wash the material retained on the #10 sieve. Use the same procedure as described in Chapter 6, except wash the soil on the #10 sieve rather than the #200 sieve.
12. Transfer the washed soil to an evaporating dish, dry it in the microwave, weigh it, and record the dry mass of the soil retained on the #10 sieve.
13. Compute percent of sample passing the #10 sieve.



7.5.2: Hydrometer Analysis

The hydrometer analysis will be performed using the soil passing the #10 sieve. It is important to remember that this is only the finer portion of the total sample. When reducing the data, you will have to account for that portion of the sample that was retained on the #10 sieve. This is explained further in the data reduction section.

14. From the portion of the sample passing the #10 sieve, take a moisture content sample of about 15-20 grams. Use the standard moisture content procedures.
15. Weigh and record the tare mass of a 500 ml beaker to the nearest 0.01 g.
16. From the remaining portion of the sample passing the #10 sieve, weigh out approximately 50 grams of soil into the beaker. If the sample contains significant amounts of sand, weigh out approximately 100 grams. Weigh the sample to the nearest 0.01 g.



17. Add 5 g of sodium hexametaphosphate to the sample in the 500 ml beaker and add approximately 125 ml of deionized water. Mix together with a spatula until all of the lumps are worked out and a homogeneous mixture is obtained. According to ASTM D422, the sample should soak for 16 hours before proceeding, but we will forgo this step and proceed with the test.



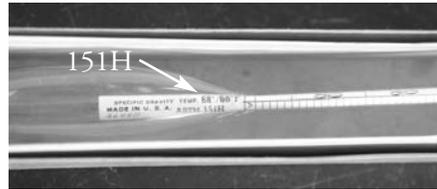
18. Transfer the mixture into the electric mixer cup. Wash all of the soil into the cup, using a squirt bottle filled with deionized water.



19. Add more deionized water into the mixer cup until it is one-half to two-thirds full; then mix the slurry for about 10 minutes.
20. While the mixer is running, prepare two 1000 ml graduated cylinders:
 - a test cylinder containing approximately 100 ml of deionized water
 - a control cylinder containing 5 g of sodium hexametaphosphate and 1000 ml of deionized water



21. Obtain an ASTM type 151H hydrometer. Do not use a type 152H. Examine the graduated scale on the stem of the hydrometer. This scale reads in units of specific gravity to the nearest one-thousandth (0.001). If you are unsure how to read the hydrometer scale, ask your instructor for help before proceeding.



22. Carefully place the hydrometer into the control cylinder and practice reading the scale at the top of the meniscus.
23. When the mixing process is complete, immediately transfer the slurry to the test cylinder. Use a water bottle as necessary to wash all of the soil into the cylinder.



24. Fill the test cylinder to the 1000 ml mark using deionized water.



25. Place a rubber stopper or the palm of your hand over the end of the test cylinder and turn the cylinder upside down and back for about 1 minute to thoroughly mix the sample. Be sure that none of the soil remains stuck to the bottom of the cylinder. When you are finished, set the cylinder down on the work table next to the control cylinder.



The soil sedimentation process starts as soon as you stop agitating the sample. Therefore, you must set the cylinder down immediately after agitating and note the clock time to the nearest second. All of the elapsed times will be measured from this point.

26. If necessary, insert a paper towel to soak up any foam that may be present on top of the slurry; then carefully insert the hydrometer and a thermometer. You will need to do this quickly because your first reading will be taken 30 seconds after the cylinder is placed on the table.
27. Read the hydrometer at times of 30, 60, and 120 seconds after the cylinder is placed on the table. These readings should be made at the top of the meniscus. Do not remove the hydrometer between these readings.
28. After the 120-second reading, remove the hydrometer and place it back into the control cylinder, using a spinning motion to remove any sediment attached to the hydrometer.
29. Repeat Steps 25-28. If the first and second readings are fairly consistent, then you may proceed. If not, continue to repeat these steps until the readings are consistent. This indicates that the slurry has been satisfactorily mixed.
30. For all further readings, the hydrometer is inserted into the slurry 5 to 10 seconds before the reading and removed immediately after the reading. At all other times, the hydrometer should be in the control cylinder.
31. Continue to take readings at the following times (all of which are measured from the beginning of the test): 5, 10, 15, 30, and 60 minutes. Take another reading at the end of the lab session. Ideally, you would take a final 24-hour reading. Discuss the necessity of taking a 24-hour reading with your instructor.
32. Whenever you take a hydrometer reading, measure the temperature in both the control and test cylinders.
33. While the hydrometer is in the control cylinder between readings, read and record the hydrometer control value, r_c . Read from the top of the meniscus when determining r_c , just as you read the hydrometer in the test cylinder.
34. At the end of the test, discard the entire slurry and the control solution. Rinse and clean the graduated cylinders.

7.6: DATA REDUCTION

There are three distinct parts to the data reduction for this test: 1) determination of the percent of sample passing the #10 sieve; 2) determination of the equivalent dry mass of total sample; and 3) determination of the percent finer using hydrometer data.

Determination of Percent Passing #10 sieve

35. Compute the mass of sample greater than the #10 sieve as

$$M_{+10} = M_1 - M_2 \quad (7.3)$$

where

M_{+10} = mass of sample greater than the #10 sieve

M_1 = mass of washed and dried soil retained on #10 sieve plus mass of dish containing soil

M_2 = mass of empty dish containing soil

36. Compute percent greater than the #10 sieve as

$$P_{+10} = \frac{M_{+10}}{M_t} \times 100\% \quad (7.4)$$

where

P_{+10} = percent of sample greater than the #10 sieve

M_t = total mass of sample that was sieved

37. The percent passing #10 sieve, P_{-10} , is then

$$P_{-10} = 100 - P_{+10} \quad (7.5)$$

There is a slight error in this computation because we have not accounted for the hygroscopic moisture content of the soil. However, because the sample has already been air-dried, the error is generally negligible.

Determination of the Equivalent Dry Mass of Total Sample

In order to compute the percent of soil in suspension at any time, we need to know the equivalent dry total sample mass, M_e . This will not be equal to the dry mass of the original sample because you did not use the entire portion of the sample less than the #10 sieve in your hydrometer analysis. You used only approximately 50 g of soil for the hydrometer analysis (or 100 g, if your sample contains significant sand). Therefore, we must compute the equivalent dry total sample mass, M_e , based on the dry mass of material less than #10 sieve, which was actually used in the hydrometer analysis.

38. Compute the moisture content of the sample, w , using the standard procedure and the sample taken in Step 14.

39. Compute the dry mass of soil in the hydrometer slurry, M_d , as

$$M_d = \frac{M_3}{1 + w} \quad (7.6)$$

where

M_3 = total mass of soil in the hydrometer slurry

w = moisture content of soil in decimal form

40. Compute the equivalent dry total sample mass, M_e , as

$$M_e = \frac{M_d}{P_{-#10}} \cdot 100\% \quad (7.7)$$

where

$P_{-#10}$ = percent passing #10 sieve, computed in Equation 7.5 above.

Determination of the Particle Diameter and Percent Passing

For each reading, you need to compute the diameter, D , of the soil particles that have settled to the bottom, and the percent of the sample, P_f that is finer than this diameter. Compute these values as follows:

41. Compute the particle diameter, D , for each reading using

$$D = \sqrt{\frac{30 \eta}{981 (G_s - 1) \rho_w t} L} \quad (7.8)$$

where:

D = particle diameter (mm)

η = dynamic viscosity of water (in poise from Table 7.3)

L = effective depth (in cm from Table 7.4)

G_s = specific gravity of soil solids (typically about 2.70)

ρ_w = density of water = 1 cm^3

t = elapsed time since beginning of test (min.)

The constant 30 in the numerator of Equation 7.8 accounts for a number of unit conversions. Therefore, Equation 7.8 is valid only when the units are as shown above.

42. Compute the percent finer, P_f for each reading, using the following formula:

$$P_f = \frac{G_s V \rho_w (r - r_c)}{(G_s - 1) M_e} \times 100\% \quad (7.9)$$

where

P_f = percent of the soil sample finer than the diameter, D , computed from Equation 7.8

V = volume of test cylinder = 1000 cm^3

ρ_w = density of water = 1.00 g/cm^3

r = uncorrected hydrometer reading

r_c = hydrometer reading in control cylinder

M_e = equivalent dry total sample mass computed from Equation 7.7 (g)

7.7: PRESENTATION OF RESULTS

The percent finer and grain size are presented in the same graphical form as the sieve analysis (see Figure 6.1). If the grain size analysis included a hydrometer and sieve analysis, then the two sets of data are combined on a single curve. There will likely be a discontinuity at the location where the data from the sieve analysis joins the data from the hydrometer analysis. This discontinuity is artificial and is the result of having performed the two tests on different soil samples. It is appropriate to smooth out the data at this point to remove the artificial discontinuity. The figure should be annotated to indicate which portion was derived from sieve analysis and which from hydrometer analysis.

7.8: TYPICAL RESULTS

The sieve analysis should always generate results for grain sizes smaller than 0.1 mm. In general, silt particles range from 0.075–0.002 mm. Clay particles are smaller than 0.002 mm. You should check the grain size distribution computed with visual observation of the soil. If the grain size distribution curve does not correspond to your visual observation of the soil composition, check your computations.

7.9: COMMON ERROR SOURCES

During data collection, the most common errors include loss of soil slurry during sample preparation, inadequate dispersion of the soil, leaving the hydrometer in the slurry between readings, dropping the hydrometer into the slurry too quickly, and incorrectly reading the hydrometer.

Preparing the soil sample requires transferring the sample between containers several times. Ensure that you transfer all of the sample between containers. Use a water bottle to rinse the sample from one container to another. Do not overfill the stainless steel mixing cup or your sample will splash out of the cup during mixing.

To ensure proper dispersion of the soil sample, ASTM D422 requires the sample to sit for 16 hours when first mixed with sodium hexametaphosphate and water (Step 17). In our procedure, this time has been reduced to a few minutes. This change will have the largest effect on highly plastic clays, such as bentonite, and will result in overestimated grain size, as individual clay particles will still be clumped together.

You must be careful when inserting and removing the hydrometer from the slurry to minimize agitating the slurry. If you drop the hydrometer into the slurry such that it goes to the bottom of the cylinder and bobs back to the top, you are remixing the slurry and disturbing the sedimentation process. You must also remove the hydrometer between readings (after the 2-minute reading). In between readings, soil will be deposited on the outside of the hydrometer and will change both the mass and the effective volume of the hydrometer. Also ensure you know how to read the hydrometer scale before starting the test.

By far the most common error is in reduction of the data. The most common data reduction error is basing the percent passing computation (Equation 7.9) on the mass of soil placed in the hydrometer rather than the equivalent dry total sample mass, M_e . Carefully check your computations.

Table 7.1: Dynamic viscosity of water.

Temperature (°C)	Viscosity, μ (Poise)	Temperature (°C)	Viscosity, μ (Poise)
12	0.01239	24	0.00916
13	0.01206	25	0.00895
14	0.01175	26	0.00875
15	0.01145	27	0.00855
16	0.01116	28	0.00836
17	0.01088	29	0.00818
18	0.01060	30	0.00800
19	0.01034	31	0.00783
20	0.01009	32	0.00767
21	0.00984	33	0.00751
22	0.00961	34	0.00736
23	0.00938	35	0.00731

Table 7.2: Effective depth of 151H hydrometer.

Hydrometer Reading, r	Effective Depth, L (cm)	Hydrometer Reading, r	Effective Depth, L (cm)
1.000	16.3	1.020	11.0
1.001	16.0	1.021	10.7
1.002	15.8	1.022	10.5
1.003	15.5	1.023	10.2
1.004	15.2	1.024	10.0
1.005	15.0	1.025	9.7
1.006	14.7	1.026	9.4
1.007	14.4	1.027	9.2
1.008	14.2	1.028	8.9
1.009	13.9	1.029	8.6
1.010	13.7	1.030	8.4
1.011	13.4	1.031	8.1
1.012	13.1	1.032	7.8
1.013	12.9	1.033	7.6
1.014	12.6	1.034	7.3
1.015	12.3	1.035	7.0
1.016	12.1	1.036	6.8
1.017	11.8	1.037	6.5
1.018	11.5	1.038	6.2
1.019	11.3		

Laboratory Compaction

Determination: Proctor Test

8.1: PURPOSE

Compaction of soil is one of the most common civil engineering construction operations. The purpose of soil compaction is to improve the soil's properties by increasing its strength, reducing its compressibility, and, sometimes, by reducing its hydraulic conductivity.

In order to control compaction operations in the field, you need a standard definition of the relative compactness of a soil. In addition, the compactability of a soil is greatly affected by the moisture content at which the soil is compacted. The Proctor compaction test provides both a standard measure of the compactability of soil and the relationship between soil compaction and compaction moisture content. It was first developed in the early 1930s by Mr. R. R. Proctor, an engineer with Los Angeles County, and is now one of the most common soil tests.

8.2: THEORY AND DEFINITIONS

The degree of compaction achieved when compacting a soil is dependant on a number of factors including

- amount of energy used per unit volume of soil
- the type of compaction: kneading, vibratory, static
- the moisture content at which the soil is compacted

The Proctor compaction test fixes the energy per unit volume and the type of compaction. The only variable then is the compaction moisture content. The net result of the Proctor compaction test is a plot of the compactability of the soil versus compaction moisture content for a fixed compaction energy.

The degree of compaction achieved in the Proctor test is measured by the dry unit weight of the compacted soil, γ_d . The dry unit weight is used to determine the degree of compaction because it allows us to compare two different samples that have the same material composition but are compacted at different moisture contents. Using the dry unit weight allows us to compare the compactness of the solid portion of the soil. Therefore, the result of a Proctor compaction test is a plot of dry unit weight, γ_d , versus compaction moisture content, as shown in Figure 8.1.

The compaction has a convex shape, as shown in Figure 8.1. Starting from a dry soil, increasing the compaction moisture content initially increases the compactability of a soil, leading to a peak in the dry unit weight. After this peak is reached, the dry unit weight of the soil decreases with further increases in the compaction moisture content. Initially, the water added to the soil helps to lubricate the contacts

between soil particles and to soften clay lumps in the soil. This leads to increased compactability and the initial upward climb of the compaction curve. At higher moisture contents, the water gets in the way of compaction because it must be squeezed out of the pores to allow the soil to be compacted further. This leads to decreased compactability and the decline in the compaction curve at higher compaction moisture contents. The dry unit weight at the peak of the compaction curve is called the *maximum dry unit weight*, γ_d -max. The compaction moisture content at the peak of the compaction curve is called the *optimum moisture content*, w_{opt} . These values are shown in Figure 8.1.

In addition to plotting the compaction curve, it is customary to plot two other curves: theoretical curves of the dry unit weight that correspond to 100% and 80% degrees of saturation. The curve of dry unit weight versus compaction moisture content at 100% saturation is sometimes called the *zero air voids curve*, because a soil compacted to this curve would have all the voids filled with water and would, therefore, contain no air. This curve represents the theoretical maximum compaction that can be achieved for any given compaction moisture content, since it is impossible to compact a soil beyond the point where all the air has been removed. From a practical standpoint, compacted soils never reach the 100% saturation curve. This provides a useful check on test data since any points on the compaction curve that lie to the right of the 100% saturation curve must be erroneous. The curve representing the dry unit weight at a saturation of 80% is useful to plot because, in practice, we find that most soils reach a maximum compaction at a degree of saturation near 80%. Together, these two curves provide a useful check on the validity of the test data and should always be included.

There are two different standards for the Proctor test. The two standards are essentially the same except for the compaction energy used.

- The *Standard Proctor Test* uses 12,400 ft-lb/ft³ (600 kN-m/m³) to compact 1/30 ft³ (9.44x10⁻⁴ m³) of soil.
- The *Modified Proctor Test* uses 56,000 ft-lb/ft³ (2,700 kN-m/m³) to compact 1/30 ft³ (9.44x10⁻⁴ m³) of soil.

The standard test was developed by Proctor for the types of equipment typically used to compact soil in the 1930s. Since that time, compaction equipment has dramatically increased in size and energy. The modified test was developed to address this change in field equipment. The modified Proctor test is today the most common compaction test used.

8.3: APPLICABLE STANDARDS

- ASTM D698: Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort
- ASTM D1557: Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort

8.4: EQUIPMENT AND MATERIALS

- A standard $1/30 \text{ ft}^3$ compaction mold (4" diameter) with base and collar
- A modified Proctor compaction hammer (10 lb. falling 18 in.)
- A 24 in. x 24 in. metal tray
- 5 Moisture content cans
- A #4 sieve (large tray sieve is best)
- Steel straightedge
- Scoop, spoon, or other mixing tool
- 250 ml graduated cylinder
- Spray bottle
- Stainless steel mixing bowls
- Hydraulic or mechanical soil extruder



8.5: PROCEDURE

As discussed above, the original standard Proctor test has largely been replaced by the modified Proctor test. Therefore, we will conduct the modified test (ASTM D1557).

The ASTM test method includes three alternative procedures, as follows:

- Method A is for soils with 80% or more passing the #4 sieve.
- Method B is for soils with less than 80% passing the #4 sieve and more than 80% of that portion greater than #4 sieve passing the $\frac{3}{8}$ -inch sieve.
- Method C is for soils with more than 80% passing $\frac{3}{8}$ -inch sieve and at least 70% passing the $\frac{3}{16}$ -inch sieve.

Most soils used as compacted fills fit in the first category, and we will use Method A in this lab.

To properly compute the compaction test results, you will need approximately five data points of γ_d vs. w , presented in the form shown in Figure 8.1. These data points are obtained by preparing samples at five different moisture contents, compacting each sample using the standard procedure, and determining γ_d for each. Ideally, one point should be near optimum moisture content and two points each should be above and below optimum.

Sample Preparation

The compaction test is performed using only that portion of the soil finer than the #4 sieve. Approximately 5 lbs. of soil is needed for each test point. According to ASTM standards, you cannot reuse soil after it has been compacted for one test. This requires approximately 25 lbs. of soil to complete the entire test. If you do not have this large a sample, or if you make errors and need to repeat a given compaction test, it is acceptable for teaching purposes to reuse some of the soil for the additional tests.

1. Spread the sample out in one of the large pans. If the sample is too moist to sieve, dry the sample in the sun, under the heat lamps, or using a blow dryer. The objective is just to get the sample dry enough to sieve. It does not have to be oven-dried.

2. Break up any large clods with a mallet or hammer.



3. Sieve the sample through a #4 sieve. It is convenient to use a large tray-type sieve for this step. Discard any soil retained on the #4 sieve.



Obtaining the First Data Point

Before compacting the first sample, you should attempt to identify the moisture content corresponding to the optimum condition. When the soil is at optimum moisture content, it should be possible to compact a sample of the soil into a ball the size of a golf ball using your hand. When this golf ball-sized sample is squeezed between your thumb and forefinger, it should break apart in a brittle fashion. If the sample cannot be formed into a compact ball, it is dry of optimum. If it can be formed into a compact ball, but the ball deforms when squeezed rather than breaking apart, it is wet of optimum.

We will attempt to obtain the first data point at the optimum moisture content. The other data points will be taken at intervals of 2% change in moisture content. That is, the five samples should be at -4% optimum; -2% optimum; $+2\%$ optimum; and $+4\%$ optimum. It is not critical that the sample be at exactly these moisture contents, just that they be spaced at intervals of approximately 2% moisture content. These procedures assume the current moisture content of the sample is below optimum.

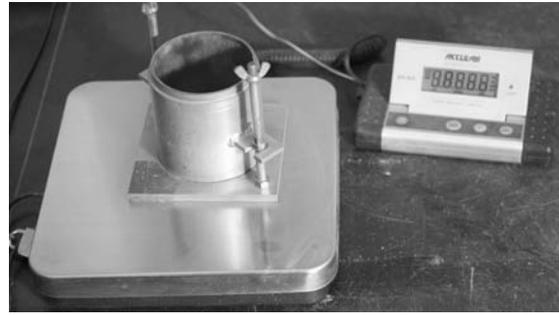
According to the ASTM standard, compaction samples containing fine-grain materials must sit for 3 to 16 hours at the mixed moisture content before compaction. This is to ensure the soil is uniformly hydrated. We will skip this step during our lab procedure.

4. Take a handful of soil and add water using a spray bottle until it is near optimum, using the above description. Observe and remember how the soil feels and behaves at this moisture content. You will need to repeat this process for a larger sample next.
5. Refill your spray bottle with water, weigh it, and record the weight of the full bottle on a scrap note page.
6. Weigh out approximately 5 lbs. of soil in a bowl, noting the amount of soil on your scrap note page.
7. Mix the water with the sample using the spray bottle until the sample appears to be at optimum moisture content. Weigh the water bottle, record its



weight on your scrap note page, and compute the quantity of water added.

8. Obtain the combined weight of the Proctor mold and its base. Do not include the collar in this measurement. Record your data.
9. Obtain a moisture content can without a lid. Record the can number on the data sheet and weigh the can.



10. Assemble the Proctor mold on its base and attach the collar.



11. Place some of the prepared soil into the mold until the top of the loose soil is approximately one-third of the way from the bottom of the mold to the joint between the mold and the collar.



12. Compact this soil by applying 25 blows from a modified Proctor hammer. Be sure to distribute these blows across the top of the sample.
13. Place approximately the same quantity of soil into the mold to form the second layer, and compact it in the same fashion. Continue this process until five layers have been placed and compacted. Thus, there will be a total of 125 hammer blows on the sample (25 blows on each of 5 layers). Do this in such a way that the top of the last compacted layer is slightly (i.e., about $\frac{1}{4}$ inch) above the joint between the mold and the collar.



14. Carefully remove the collar with a twisting motion. Be careful not to shear off the top of the sample when removing the collar.
15. Use the steel straightedge to trim the excess soil from the top of the mold. The top of the soil should then be flush with the top of the mold. If any depression exists, you will have to start the entire process over again.
16. Determine the combined weight of the Proctor mold with its base and the soil compacted inside the mold. Record your data.
17. Remove the mold (with soil) from the base. Press the soil out of the mold using the hydraulic jack.
18. Break open the soil and obtain a representative sample from the center for a moisture content test. Follow the standard moisture content procedures with this sample.



Obtaining the Remaining Data Points

19. From your notes, determine the approximate moisture content of the first sample compacted and then compute the weight of water you will need to add to each of the remaining soil samples in order to achieve moisture contents at optimum $\pm 2\%$ and optimum $\pm 4\%$, assuming 5 lbs. of the original soil sample are used for each test.
20. Obtain approximately 5 lbs. of the prepared soil and fill a spray bottle with the amount of water you have determined will be necessary to bring the soil to the target moisture content. Thoroughly mix the water and the soil.
21. Follow Steps 10–17, as described above, to obtain the second data point.
22. Repeat this process with different moisture contents until you have obtained data for all five points.

The goal is to have data points distributed from well dry of optimum to wet of optimum, with one or two points close to optimum. You can tell when the soil is wet of optimum when the dry density is lower than that of a drier point. When the wet density also is lower, you are definitely on the wet side of optimum. Therefore, it is good to compute the wet density of each sample as you perform the test.

When wet of optimum, the soil also will tend to move laterally when struck by the hammer, and it will bulge up around the hammer. The impact sound is also duller.

Avoid testing samples that are significantly above optimum moisture content, especially if they have a large clay content. Such samples can clog up the hammer, thus generating additional and needless cleanup work.

8.6: DATA REDUCTION

23. Compute the moisture content for each data point using the normal procedure.
24. Compute the unit weight of each data point using the weights obtained in the lab. Remember the mold has a volume of $1/30 \text{ ft}^3$.

$$\gamma = \frac{W_s}{V_m} \quad (8.1)$$

where

W_s = weight of soil in mold

V_m = volume of mold = $1/30 \text{ ft}^3$

25. Compute the dry density (dry unit weight) for each data point as

$$\gamma_d = \frac{\gamma}{1 + w} \quad (8.2)$$

8.7: PRESENTATION OF RESULTS

The compaction data are presented as an arithmetic plot of compaction moisture content versus dry unit weight, similar to Figure 8.1.

The compaction data are presented as an arithmetic plot with moisture content, w , on the horizontal axis, similar to Figure 8.1. The range of values for the plot should be selected so that they clearly show the curvature of the compaction curve. In general, the x axis (moisture content) should range from 0 to slightly above the maximum value obtained in the lab. The y axis (dry unit weight) should range from a value slightly less than the minimum value obtained in the lab to a value slightly higher than the maximum.

Plot the compaction curve using the following procedure.

26. Plot the γ_d vs. w data points obtained from the lab. However, do not connect them with a curve yet.
27. Develop curves for $S = 100\%$ and $S = 80\%$ using the following formula, and plot these curves on the γ_d vs. w plot. The $S = 100\%$ curve also is known as the *zero air voids curve*.

$$w = S \left[\frac{\gamma_w}{\gamma_d} - \frac{1}{G_s} \right] \times 100\% \quad (8.3)$$

Develop these two curves through the range of γ_d values obtained from the Proctor tests. For example, if the Proctor test results produced γ_d values between 102 and 123 lb/ft^3 , then generate the curves for the range 100 to 125 lb/ft^3 . You may use an estimated value of G_s in this formula (typically 2.65 for sands, or 2.70 for clays).

None of the data points from the lab should be to the right of the $S=100\%$ curve (This would indicate a degree of saturation greater than 100%, which is impossible).

28. Using the $S=80\%$ and $S=100\%$ curves as guides, draw a concave-down curve connecting the data points from the lab. The high point on this curve represents the maximum dry density and optimum moisture content.

The optimum moisture content is usually somewhere near the 80% saturation curve. The right leg of the compaction curve should be between the 80% and 100% saturation curve and roughly parallel to them.

Express the maximum dry density to the nearest whole lb/ft^3 (e.g., 123 lb/ft^3) and the optimum moisture content to the nearest 0.1% (e.g., 12.2%).

8.8: TYPICAL RESULTS

For most soils, the optimum moisture content, w_{opt} is between 10–15% for the modified Proctor test. The maximum dry unit weight, γ_{d-max} typically ranges from 118–130 lb/ft^3 for the modified Proctor test. As discussed above, the degree of saturation at optimum moisture content should be around 80%, and the right-hand tail of the curve should be roughly parallel to the $S=100\%$ line. Any points plotting to the right of this line indicate an error in the test.

8.9: COMMON ERROR SOURCES

ASTM standards require that the samples sit for 3–16 hours before testing to ensure the samples are fully hydrated. Failure to fully hydrate the soil tends to skew the results to the left in the compaction curve. However, inadequate hydration affects the moister samples more than the drier samples; thus, the entire shape of the curve may be affected. The greater the amount of fines in the sample and the higher their plasticity, the longer the hydration process takes. For silty sand (SM) material with fine contents less than 20%, the errors caused by not allowing the full hydration period may be acceptable. Note that the procedure outlined above does not allow for a complete hydration period. This procedure should be acceptable for academic purposes when compacting silty sands. If you are compacting clayey soils, the samples should be prepared ahead of time and allowed to hydrate.

During data collection, two common errors are underfilling the Proctor mold and mixing up the moisture content samples. It is not uncommon, particularly the first time performing the test, to underestimate the amount of soil needed to fill the mold after compaction. When you remove the collar on the mold after completing the compaction, the soil should extend about $\frac{1}{4}$ inch above the top of the mold. If, when you strike off the mold with the straightedge, there are divots or holes in the top of the sample, the mold is underfilled, and you must repeat the test. This test requires five different moisture content samples: one for each compaction test. Be careful when recording your data that you properly record the moisture content samples for each test. If they are mixed up, all of your data will become worthless.

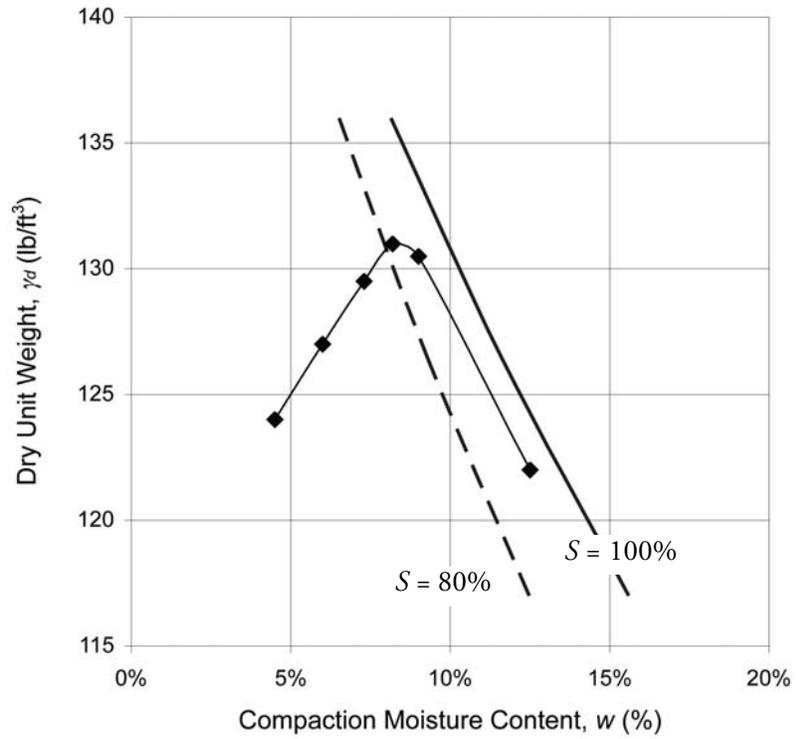


Figure 8.1 Typical Proctor Compaction Curve.

Proctor Compaction Test

Client: _____	Project #: _____
Project Title: _____	Test performed by: _____
Test Date: _____	Checked by: _____
Boring No. : _____	Sample No. : _____
Soil Classification: _____	
Sample Depth: _____	

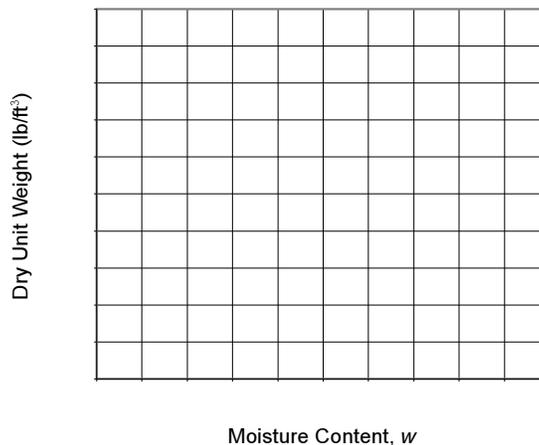
Moisture Content Data

Trial No.	1	2	3	4	5
Can ID Number					
Mass of tin (g)					
Mass of tin and moist soil (g)					
Mass of tin and dry soil (g)					
Moisture content (%)					

Compaction Data

Weight of mold+base (without collar):		lb
Volume of mold:		ft ³
Specific Gravity of Solids, G _s		

Trial No.	1	2	3	4	5
Weight of mold, base, and soil (lb)					
Weight of soil (lb)					
Unit Weight (lb/ft ³)					
Dry Unit Weight (lb/ft ³)					
Data for Check Lines	S=80%				
	S=100%				



Maximum Dry Unit Weight		lb/ft ³
Optimum Moisture Content		%

In Situ Unit Weight Determination: Sand Cone Test

9.1: PURPOSE

When monitoring earthwork operations, you need to be able to measure the in situ unit weight and moisture content of soils. These measurements must be made in the upper 6 to 8 inches of soil. A number of methods have been developed to perform these measurements, including the *sand cone*, *rubber balloon*, and *drive sample* methods. In this lab, we will use the sand cone method, which is one of the oldest, simplest, and most accurate. By comparing the in situ unit weight to the maximum dry unit weight computed in a Proctor test, the relative degree of compaction of the in situ material can be determined.

9.2: THEORY AND DEFINITIONS

This test is very simple in concept. In order to determine the in situ unit weight, a small hole is dug in the ground and all the soil is removed from the hole and stored in a plastic bag or other closed container to prevent moisture loss. The volume of the hole is measured by filling the hole with a sand of known unit weight and measuring the amount of sand required to fill the hole. Knowing the weight of sand filling the hole and the unit weight of the sand, we can easily compute the volume of the hole.

The weight and moisture content of in situ soil removed from the hole is then measured. Knowing the volume of the hole, the weight of the soil removed, and the moisture content of the removed soil, we can then compute both the total and dry unit weights of the in situ soil.

Another common method for measuring in situ unit weight and moisture content near the surface is the *nuclear densitometer*. This method uses completely different principles to determine unit weight and moisture content. This test depends on the radiation absorption characteristics of the soil. The theory of this test is beyond the scope of this manual. In spite of its high-tech nature, the nuclear densitometer is actually less accurate than the sand cone test.

9.3: APPLICABLE STANDARDS

- ASTM D1556 Standard Test Method for Density and Unit Weight of Soil in Place by the Sand-Cone Method

9.4: EQUIPMENT AND MATERIALS

- Sand cone assembly (plastic jar and metal cone section that screws on jar)
- Base plate
- 1/30 ft³ Proctor mold (4" diameter mold)
- Steel straightedge
- Washed clean #30 sand
- Square-tipped shovel
- Plastic bucket
- 2 Large sample bags with ties and sample tags
- Strainer
- Large spoon
- Weeding tool
- Laboratory scale



9.5: PROCEDURE

Before proceeding with the test, you should become familiar with the equipment.

1. Place the sand cone assembly on the end of an empty sand cone jar.
2. Place the base plate on the countertop, and place the sand cone assembly on the base plate.
3. Turn the valve on the sand cone assembly back and forth and imagine how the sand will flow out of the jar, into the cone, and into a hole in the ground below the cone.

Note that all of the sand that flows out of the jar does not go into the hole in the ground. Some of the sand will be filling up the cone and the space between the cone and the hole created by the base plate. In order to determine how much sand flows into the hole, we must know how much sand it takes to fill the cone, since we will be measuring only the total amount of sand that leaves the jar. We will also need to know the unit weight of the sand to compute the volume of the hole. Therefore, before taking any field measurements, we need to calibrate the equipment to determine the unit weight of the sand and the weight of sand that fills the cone.

Sand Cone Calibration

4. Remove the cone assembly from the jar and fill the jar with #30 washed sand.
5. Open and close the valve several times to verify that it moves freely. This repeated movement will help dislodge any sand particles that may be lodged in the valve. Apply some WD-40 to the valve, if necessary.
6. Close the valve and attach the cone assembly to the jar.
7. Weigh the sand cone assembly (cone, jar, and sand) and record this weight on the data sheet.



8. Set the base plate on the countertop and place the sand cone on top of the plate. Ensure the cone is properly seated in the base plate recess.
9. Open the valve and allow the sand to fill the cone. Do not shake or vibrate the sand cone while the valve is open.
10. When the sand stops flowing (30 seconds to 1 minute), close the valve.
11. Clean off the countertop. Repeat Steps 8–10 three or four times, but make sure the jar never completely empties.
12. Weigh the sand cone assembly again. Record the weight and number of times the cone was filled. You will use these data to compute the average weight of sand required to fill the cone.



Sand Unit Weight Calibration

13. Obtain a Proctor compaction mold and remove its collar. Leave the base attached to the mold.
14. Weigh the empty Proctor mold without the collar. Record the data.
15. Fill the jar with washed #30 sand, close the valve, and replace the cone assembly on the jar.
16. Hold the sand cone assembly over the Proctor mold and open the valve. Allow the sand to fill the mold while being careful not to shake or vibrate it. When the mold is filled, close the valve and remove the sand cone.



17. Carefully strike off the top of the mold with a steel straightedge to obtain exactly $1/30 \text{ ft}^3$ of sand. Clean any sand off the outside of the mold; then weigh the sand and mold. Record the data.
18. Repeat Steps 15–17 three times to obtain an average mass of sand filling the mold.



Field Measurements

19. Fill the jar with washed sand, place the cone assembly on the jar, and ensure the valve is working properly.
20. Weigh the filled jar and cone assembly. Record the data.
21. Bring the following equipment to the field:
 - The prepared and weighed sand cone assembly
 - Base plate
 - Square-tipped shovel
 - Bucket
 - 2 Large sample bags with ties and sample tags
 - Strainer
 - Large spoon
 - Weeding tool



22. Using the shovel and base plate, prepare a flat surface on which to conduct the test. This surface should be reasonably level, but it is especially important that it be planar. Care in preparing a good surface will help you avoid introducing needless error into the test.



23. Place the base plate over the prepared surface, ensuring that it fits flush to the ground. Dig the test hole through the hole in the plate. Be careful to recover all of the soil that comes out of the test hole and place it in one of the sample bags. Seal and label the bag.

If the soil is quite hard, you may find that it is helpful to first loosen the soil using the weeding tool. Then, remove it using the large spoon.



24. Continue digging the hole until its volume plus the volume of the cone is slightly less than the volume of the sand in the jar. Remember, the larger the hole, the more precise the test—as long as you don't run out of sand!
25. Be sure the rim of the base plate is clean. Place the sand cone on the base plate. Be sure the sand cone is set completely into the base plate recess.



26. Open the valve and allow the sand to flow into the hole. If you can't open the valve by hand, you may use pliers. **Do not shake, move, or vibrate the jar while the sand is flowing.** When the sand stops flowing (about 2 minutes), close the valve and remove the sand cone.



27. Using the spoon, strainer, and the second sample bag, recover as much of the washed sand as possible without recovering a significant amount of in situ soil. This is to be brought back to the lab for reuse.
28. Return to the lab and weigh the sand cone assembly. Then weigh the sample bag of in situ soil. Record the data.



Moisture Content Sample

29. From the in situ soil, take a moisture content sample and process it using the normal procedures.

9.6: DATA REDUCTION

Sand Unit Weight Determination

30. Compute the unit weight of the washed sand γ_{sand} as

$$\gamma_{sand} = \frac{W_1}{V_{mold}} \quad (9.1)$$

where

W_1 = average weight of sand filling Proctor mold

V_{mold} = volume of Proctor mold (1/30 ft³)

In Situ Unit Weight Determination

31. Compute the volume of the hole in the field, V_{hole} as

$$V_{hole} = \frac{W_1 - W_2}{\gamma_{sand}} \quad (9.2)$$

where

W_1 = total weight of sand used in field test

W_2 = average weight of sand required to fill cone

32. Compute the in situ total unit weight, γ , as

$$\gamma = \frac{W_3 - W_4}{V_{hole}} \quad (9.3)$$

where

W_3 = weight of in situ sample, sample bag, and tag

W_4 = weight of sample bag and tag

33. Compute the in situ moisture content, $w_{in-situ}$, using the normal procedures.

34. Compute the in situ dry unit weight, γ_d , as

$$\gamma_d = \frac{\gamma}{1 + w_{in-situ}} \quad (9.4)$$

The weight, $W_{in-situ}$, and moisture content, $w_{in-situ}$, of the soil removed from the hole are determined, and the total unit weight, γ , and dry unit weight, γ_d , of the in situ soil are computed as

$$\gamma_d = \frac{\gamma}{1 + w_{in-situ}} \quad (9.5)$$

35. If Proctor test data is available for the same soil, then the degree of compaction or relative compaction, C_r , can be computed as

$$C_r = \frac{\gamma_d}{\gamma_{d-max}} \cdot 100\% \quad (9.6)$$

9.7: PRESENTATION OF RESULTS

The presentation of the sand cone data depends on the purpose for which it was taken. In general, the results will be reported using the dry unit weight, γ_d , and the moisture content, w . If the data were taken to determine compliance with a compaction standard, then the location and time of sampling of each sample would be carefully noted. The computed dry unit weight would be compared with the dry unit weight required by the compaction standard. In general, these data are best presented in tabular form. If the data were used in a borrow to determine fill quantities, then both the dry unit weight and moisture content would be critical. In this instance, it is useful to report average values for a given borrow area in addition to the individual results.

9.8: TYPICAL RESULTS

For compacted soils, typical results are similar to those for the Proctor test (see Chapter 8) since the goal of field compaction is generally to achieve a dry unit weight that is 85% or more of the maximum Proctor value.

The in situ dry unit weight for coarse-grained soils typically ranges from 95 to 125 lb/ft³ (15.0 to 19.5 kN/m³). For fine-grained soils, the typical range is from 75 to 110 lb/ft³ (12.5 to 17.5 kN/m³).

9.9: COMMON ERROR SOURCES

During calibration, the most common error is to vibrate the cone or the Proctor mold during testing. This causes the sand to become denser and will overestimate the dry unit weight of the sand. This, in turn, will underestimate the unit weight of the in situ soil.

During field testing, common errors include placing the base plate on an uneven surface, failing to transfer all the soil from the hole into the sample bag, losing moisture in the in situ sample due to poor storage, and failing to allow enough time for the sand cone to completely fill the hole. Properly prepare for the test by creating a smooth, flat surface for the base plate. When digging the hole, you must be careful to transfer all of the soil from the hole into the sample bag. Any soil that is dropped on the base plate must be transferred to the sample bag also. Keep the sample bag sealed to prevent moisture loss, and test the sample as soon as possible after sampling. Be patient when the sand is filling the hole. It can take up to 2 minutes for the sand to flow into the hole. Do not tap or vibrate the sand cone as that will lead to erroneous results.

Sand Cone

(1 of 2)

Client: _____ Project #: _____
 Project Title: _____ Test performed by: _____
 Test Date: _____ Checked by: _____
 Boring No. : _____ Sample No. : _____ Sample Depth: _____
 Soil Classification: _____

Sand Cone Calibration

Weight of cone and sand before test		lb
Weight of cone and sand after test		lb
Number of trials		
Weight of sand to fill cone and plate		lb

Unit Weight of Washed Sand

Weight of empty Proctor mold		lb
Volume of Proctor mold		ft ³

Weight of Proctor mold and Washed Sand	Trial 1		lb
	Trial 2		lb
	Trial 3		lb
	Trial 4		lb
	Average		lb
Average weight of washed sand			lb

Unit Weight of washed sand		lb/ft ³
----------------------------	--	--------------------

Sand Cone

(2 of 2)

Client: _____ Project #: _____
 Project Title: _____ Test performed by: _____
 Test Date: _____ Checked by: _____
 Boring No. : _____ Sample No. : _____ Sample Depth: _____
 Soil Classification: _____

Field Sand Cone Measurements

Weight of cone and washed sand before test		lb
Weight of cone and washed sand after test		lb
Weight of washed sand used in test		lb
Volume of Hole in Ground		ft ³
Weight of sample bag, tag, and soil		lb
Weight of sample bag and tag		lb
Weight of soil		lb

Moisture Content

Weight of tin		g
Weight of moist soil and tin		g
Weight of dry soil and tin		g
Moisture Content		%

Proctor Compaction Test Results (from previous compaction test)

Maximum Dry Density		lb/ft ³
Optimum Moisture Content		%

Final Results

Field Unit Weight		lb/ft ³
Field Dry Unit Weight		lb/ft ³
Relative Compaction		%

C H A P T E R 10

Compressibility Determination: Consolidation Test

10.1: PURPOSE

You often will have to determine how much a soil will deform when loaded. One of the most common examples is computing the settlement of a building constructed on a shallow foundation. In order to make such a computation, we must know the stress-strain properties of the soil. The consolidation test, sometimes called the *oedometer test*, measures the stress-strain properties of soil under simple one-dimensional confined compression. The stress-strain data we get from this test allows us to answer the question, “How much will the building settle?”

When the soils being loaded are saturated silts and clays, the consolidation process can take a long time because pore water must flow out of the soils during consolidation. In addition to measuring stress-strain properties, the consolidation test also measures the strain-time behavior of a soil. The test gives us the data we need to answer the question, “How long will it take for the building to settle?”

10.2: THEORY AND DEFINITIONS

10.2.1: Stress-Strain Relationship: Magnitude of Deformation

The consolidation test applies a known vertical load to a soil sample. The sample is allowed to deform in the vertical direction, but is constrained in the horizontal direction. Thus, the test measures one-dimensional deformation under conditions of no lateral strain. We measure the vertical deformation in the soil sample, δ , under a series of increasing loads. Figure 10.1 shows the loading conditions for the one-dimensional consolidation test. We can compute the vertical strain, ϵ_z , in the soils as

$$\epsilon_z = \frac{\delta}{h_0} \quad (10.1)$$

where

δ = measured vertical deformation
 h_0 = original sample height

The load is maintained until any excess pore pressures have dissipated and, therefore, the effective vertical stress, σ'_z , is equal to the total vertical stress, σ_z

$$\sigma'_z = \frac{P}{A} \quad (10.2)$$

where

$$\begin{aligned} P &= \text{total vertical applied load} \\ A &= \text{cross sectional area of the sample} \end{aligned}$$

Using the applied vertical effective stress, σ_z' , and the computed vertical strain, ϵ_z , we can plot the stress-strain curve for the soil. In soil mechanics, the stress-strain curve is presented as strain versus the log of effective stress (ϵ_z versus σ_z'), with stress and strain defined as compression being positive. Figure 10.2 shows a typical strain versus log effective stress curve for soil.

The stress strain curve for soil is clearly non-linear. However, note from Figure 10.2 that it can be readily simplified as a bi-linear curve in semi-log space. This is the main reason for plotting strain as a function of the log of effective stress. The two linear segments of the stress-strain curve intersect at a stress defined as the *preconsolidation stress*, σ_c' . The preconsolidation stress is the highest effective vertical stress the soil has ever been subject to. At stresses below the preconsolidation stress, soil deformation is defined by the so-called *recompression curve*. At stresses higher than the preconsolidation stress, soil deformation is defined by the so-called *virgin curve*.

Historically, stress-strain curves for soil have been plotted as the void ratio, e , versus the log of effective stress, as shown in Figure 10.3. The void ratio and the vertical strain are related by

$$\epsilon_z = \frac{\Delta e}{1 + e_0} \quad (10.3)$$

where

$$\begin{aligned} \Delta e &= \text{change in void ratio} \\ e_0 &= \text{initial void ratio} \end{aligned}$$

Using the traditional method of presenting the stress-strain curve as e versus $\log \sigma'$, the slope of the reload curve is called the *recompression index*, C_r , and the slope of the virgin curve is called the *compression index*, C_c , as shown in Figure 10.3. The definition of C_r is

$$C_r = \frac{\Delta e}{\Delta(\log \sigma')} = \frac{e_2 - e_1}{\log \sigma_2' - \log \sigma_1'} \quad (10.4)$$

Using the identity

$$\log(a) - \log(b) = \log\left(\frac{a}{b}\right)$$

we can write Equation 10.4 as

$$C_r = \frac{e_2 - e_1}{\log\left(\frac{\sigma_2'}{\sigma_1'}\right)} \quad (10.5)$$

where points 1 and 2 lie on the recompression curve, as shown in Figure 10.3.

Similarly, we can define C_c as

$$C_c = \frac{e_4 - e_3}{\log\left(\frac{\sigma_4'}{\sigma_3'}\right)} \quad (10.6)$$

where points 3 and 4 lie on the virgin curve, as shown in Figure 10.3.

Although soil stress-strain curves have traditionally been presented as e versus $\log \sigma'$ (Figure 10.3), there is no particular value in plotting in this format. In fact, it is easier to use the format of ε versus $\log \sigma'$, as shown in Figure 10.2, because the deformation measurements are more easily converted into strain than into void ratio. We will use the form shown in Figure 10.2, ε versus $\log \sigma'$, throughout the remainder of this manual.

When we plot the consolidation curve as ε versus $\log \sigma'$, it is important to understand that slopes of the recompression and virgin curve are no longer C_r and C_c . Instead, the slope of the recompression curve is

$$\frac{C_r}{1 + e_0}$$

And the slope of the virgin curve is

$$\frac{C_c}{1 + e_0}$$

After the soil is loaded to the peak stress, it is unloaded. During unloading, the soil will rebound on a curve that is roughly parallel to the reload curve, as shown in Figure 10.2.

10.2.2: Developing the Stress-Strain Curve

To develop the stress-strain curve, as shown in Figure 10.2, we must perform a series of loads on a single soil sample. For each point on the stress-strain curve, the soil sample is loaded and allowed to consolidate until all excess pore pressure is dissipated and consolidation is complete. When consolidation is complete, the vertical strain, ε_z , and the effective vertical stress, σ_z' , are computed, and a point can be plotted on the stress-strain curve. The soil is then loaded to a higher stress and the processes repeated until a complete stress-strain curve is developed. Therefore, the consolidation test actually consists of a series of time-dependent loads performed incrementally on the same soil sample. Figure 10.5 illustrates this concept.

10.2.3: Deformation Time Relationship: Time Rate of Consolidation

As discuss earlier, the soil sample does consolidate and deform to its final state immediately after loading. Consolidation can take a long time, particularly for a saturated, clayey soil. When a saturated soil is loaded, initially the additional stress will be carried by an increase in the pore pressure. This increased pore pressure is called *excess pore pressure*, u_e , because it will eventually dissipate and return to its equilibrium level.

The creation of excess pore pressure will cause water to flow out of the soil sample. As the water flows out of the sample, the excess pore pressure decreases, the stress is transferred from pore pressure to effective stress, and the sample will deform or consolidate. The rate at which this consolidation occurs is controlled by the hydraulic conductivity and compressibility of the soil. The soil property used to characterize the rate at which a soil will consolidate is called the *coefficient of consolidation*, c_v , and has the units of ft^2/day or cm^2/s .

Terzaghi's Theory of One-Dimensional Consolidation

Terzaghi's theory of consolidation can be used to predict and analyze this time-dependent process. Terzaghi's theory can be presented as a relationship between the degree of consolidation, U , and a dimensionless measure of time called the *time factor*, T_v . The time factor is defined as

$$T_v = \frac{c_v t}{(H_{dr})^2} \quad (10.7)$$

where

- t = time since application of the load
- H_{dr} = length of the maximum drainage path. For a sample drained at the top and the bottom (double drainage), this is equal to one-half the thickness of the sample. For a sample drained only at the top or the bottom (single drainage), this is equal to the thickness of the sample.
- c_v = the *coefficient of consolidation*

The relationship between the degree of consolidation, U , and the time factor, T_v , for Terzaghi's theory is shown in Figure 10.4.

Using Equation 10.7 and Figure 10.4, you can determine the degree of consolidation at any time if you know the maximum drainage path, H_{dr} , and the coefficient of consolidation, c_v . The maximum drainage path is generally known (either through boring logs in the case of field problems, or by measuring the sample thickness in laboratory tests). The coefficient of consolidation, c_v , is a soil material property that must be measured. The coefficient of consolidation is one of the soil properties that can be measured during the consolidation test.

Determination of the Coefficient of Consolidation, c_v

If we have laboratory data measuring the deformation of a soil sample versus time, we can use these data to determine c_v . Equation 10.7 can be rearranged to

$$c_v = \frac{T_v H_{dr}^2}{t} \quad (10.8)$$

Using Equation 10.8, we can compute c_v if we know the time at which a soil sample reaches a given degree of consolidation and we know the maximum drainage path.

For convenience, we normally compute c_v when a soil reaches 50% consolidation. We use laboratory data to estimate the time to reach 50% consolidation, t_{50} . From Figure 10.4, we know the time factor at 50% consolidation, T_{50} . Knowing t_{50} , T_{50} , and the drainage path of the sample, H_{dr} , we can compute c_v with Equation 10.8. Details are presented in the procedure section.

Primary versus Secondary Consolidation

Primary consolidation is consolidation due to the dissipation of excess pore pressure. According to Terzaghi's theory, when the excess pore pressure has completely dissipated, primary consolidation stops. However, in practice, we observe that settlement often continues after excess pore pressure has reached zero. Figure 10.6 is a plot of deformation versus log-time and illustrates the difference between Terzaghi's theory and observed behavior. The linear tail at the end of the curve is a form of creep deformation called *secondary consolidation*. At some point where the curve in Figure 10.6 starts to flatten out, primary consolidation ends and secondary consolidation begins. The slope of

the deformation versus log-time curve in the area of secondary consolidation is called the *secondary compression index*, C_{α} , defined as

$$C_{\alpha} = \frac{\Delta e}{\Delta \log(t)} \quad (10.9)$$

10.3: APPLICABLE STANDARDS

- ASTM D2435 Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading

10.4: EQUIPMENT AND MATERIALS

- Consolidation cell base w/water bath
- Confining ring
- Small porous stone
- Large porous stone
- Loading cap
- Steel ball
- Filter paper
- Allen wrench
- Screwdriver
- Loading frame
- Dial gauge
- Clipboard w/datasheet
- Calibrated weights



10.5: PROCEDURE

These procedures are written assuming you have an undisturbed sample taken with a California type split spoon sampler using 2.5-in OD by 1-in-high liner rings. The procedures also assume you are using a Geomatic® dead load consolidometer. If you are using a different consolidometer, you will have to adjust the procedures accordingly.

Sample Preparation

1. Select an undisturbed sample for consolidation testing. If you are uncertain as to which sample to choose, discuss sample selection with your instructor.
2. The porous stones that will be on the top and bottom of the sample should be prepared so that they do not significantly change the moisture content of the sample as the sample is prepared. If the sample is a saturated material from below the water table, the stones should be saturated before the test. For other samples, the stones should be air-dried before the test.
3. Remove one ring of soil from the sample such that there is excess soil extending on both sides of the sample. You will probably have to remove two or more rings of soil in order to get one good undisturbed ring.

4. Carefully trim the top and bottom of the soil so that it is flush with the top and bottom of the brass ring. When you're done trimming, you should have a 1-inch-tall cylindrical sample contained within the brass sample ring.



5. Examine the circular brass confining ring. The top of the ring is flat, and the bottom of the ring has a lip on it. Loosen the machine screw on the side of the confining ring and place the confining ring on top of the large porous stone. The smooth side of the ring should be facing upward, and the stone should fit inside the lip on the bottom of the confining ring.



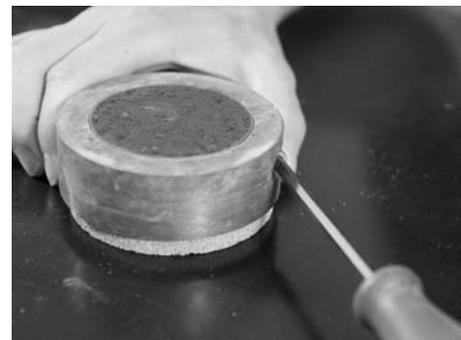
6. Place a filter paper on top of the porous stone inside of the confining ring. If you're preparing a saturated sample, saturate the filter paper before placing it on the stone.



7. Push your sample ring containing your trimmed sample into the confining ring. Push the sample ring all the way in so that it is flush with the top of the confining ring. If the sample ring does not slide easily into the confining ring, pry the confining ring open a bit by placing a screwdriver in the gap next to the machine screw.



8. Tighten the machine screw on the side of the confining ring using the Allen wrench. The machine screw should be tight enough to hold the sample ring firmly inside the confining ring, but it should not be excessively tight.



9. Place a filter paper on top of the sample. This paper should have a diameter about 2 mm smaller than the sample. Carefully center the filter paper on top of the sample.



10. Place the smaller porous stone on the filter paper and center it over the sample. We will refer to the sample in the confining ring sandwiched between the two porous stones as the *sample assembly*.



In order to get the sample assembly into the consolidometer cell base, we must invert the sample assembly and place the base on top of the inverted sample assembly. The entire consolidometer cell can then be inverted one more time and loaded into the loading frame. You will need a cylinder 4 to 5 inches high and $1\frac{1}{2}$ to 2 inches in diameter for this purpose. If you are using the Geomatic® consolidometer, the 800 lb/ft² weight works well.

11. Place the cylinder upright on the table; then grasp the sample assembly with both hands and place it upside-down on top of the cylinder. The large porous stone should now be on top.



12. Invert the consolidometer cell base and place it on the sample assembly. Be sure it seats properly on the large porous stone.

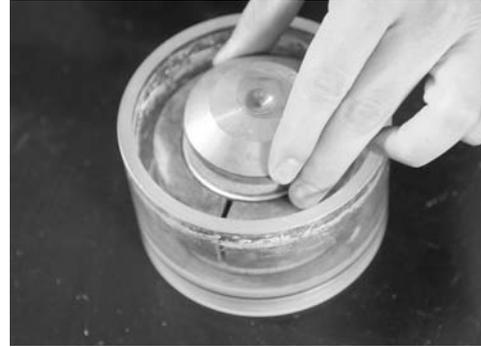


13. Using both hands, grasp the consolidometer cell and sample assembly. Lift the assembly off the cylinder, turn it right-side up, and place it on the table. Examine the assembly to be sure it is properly seated in the consolidometer cell base.

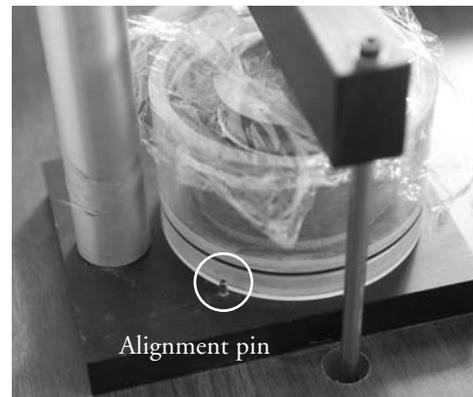


Placing the Sample Assembly in the Loading Frame

14. Place the brass loading cap on top of the small porous stone. Carefully inspect the porous stone to ensure that it is centered inside the brass sample ring.



15. Have one person hold up the cross bar on the loading frame while a second person places the sample assembly in the load frame. Be sure it touches the two pins at the rear of the load frame.



16. Place the steel ball on top of the loading cap and carefully lower the cross bar onto the steel ball. Inspect the porous stone to ensure that it is centered inside of the brass sample ring. If the stone is not perfectly centered inside the brass ring, the load will be applied to the brass ring rather than the sample.



17. Set the dial gauge through the center of the hole in the loading bar so that it rests on the ball. Compress the plunger about 0.3 inches, and then tighten the dial gauge mount.

18. The faceplate of the dial gauge can be rotated to change the location of the zero reading. If the zero is not at the 12:00 position, loosen the faceplate screw, rotate the faceplate so the zero is at the 12:00 position, and retighten the screw. Do not attempt to line up the zero on the faceplate with the needle on the dial gauge. This can actually make the dial gauge harder to read in the long run.



Reading the dial gauge is one of the most tedious and difficult tasks in this test. Before proceeding, be sure you know how to read the dial gauge. You should borrow an unused dial gauge and practice taking readings until you are certain that you can properly read the dial gauge. Do not skip this practice. This test can take a week or more to run. If you make an error in reading your dial gauge, the entire week can be a wasted effort.

Loading the Sample

The soil will be loaded such that each successive load is approximately twice the preceding load: e.g., 100 lb/ft², 200 lb/ft², 400 lb/ft², etc. For each load increment, you must allow the load to remain on the sample long enough for primary consolidation to be complete before you can add the next load increment. Ideally, you would determine when primary consolidation was complete for each increment and then apply the next load. For practical purposes, this procedure is used only when testing highly sensitive clays that exhibit large secondary consolidation. The general procedure is to leave each load on the sample for 24 hours, and apply a new load each day. We will follow this procedure. The recommended load sequence is shown in Table 10.1.

When the first load is applied to the soil, much of the initial deflection measured is due to phenomena other than compression of the soil sample. The initial compression includes things such as compression of filter papers, slack in bearings or connections in the loading mechanism, and seating of the porous stones on the sample. The initial load is called a *seating load*. The seating load is small, and the initial deformation observed when it is applied is ignored when computing deformation of the sample.

Recording Deformation versus Time

If you are going to determine the time rate of consolidation and compute the coefficient of consolidation, c_v , you must take deformation readings throughout the 24-hour period of loading. These interval readings should be taken such that each reading is taken at an elapsed time of approximately twice the preceding reading: e.g., 6 sec, 15 sec, 30 sec, 1 min, 2 min, 4 min, 8 min, 15 min, etc. The readings do not have to be taken at exactly twice the time of the preceding reading, but the exact time of the reading must be recorded. Table 10.2 presents a recommended schedule for deformation readings.

If you are performing the test to determine only the magnitude of settlement and are not concerned with the time rate of settlement, you need only take readings at 6 seconds and 24 hours after the load is applied.

Before proceeding with the test, determine what type of test you will be performing, what loads are to be applied, and when readings are to be taken. Prepare your data sheet for the appropriate loading and reading schedule. Have your weights for the seating load and first load increment prepared, as these two loads are applied in quick succession.

19. Apply a seating load of 100 lb/ft² by placing the appropriate weight on the load hanger. Record to the nearest second the time the load was applied. This is most easily accomplished by applying the load at an even minute. Have one student count down seconds to zero, and have another student place the load on the hanger when the countdown reaches zero.
20. Six seconds after the load is applied, read and record the dial gauge reading. This is your seating load reading and is taken to be the dial gauge reading corresponding to zero strain in the sample. You will subtract this reading from all future readings to determine the deformation in the sample.
21. If you are performing the test on a saturated soil, fill the water bath with deionized water at this time.



22. One minute after the seating load is applied, add a second increment of 100 lb/ft² to the sample (for a total of 200 lb/ft²) and commence your 24-hour reading cycle for this load increment.
23. At the time for each scheduled reading, record the wall clock time, the elapsed time, the total stress applied to the sample, and the dial gauge reading. Always use the time on the clock in the lab to avoid discrepancies between different timepieces.
24. At the end of each 24-hour loading period, plot the data on a graph of deformation versus log-applied stress. The data point should form a smooth curve. If there are jumps or discontinuities in the plot, recheck your dial gauge reading before proceeding. This is an important check. Plot the data and check its validity before applying a new load increment!
25. When you have finished plotting data, add the next load increment. You should add 200 lb/ft² (for a total of 400 lb/ft²). Record the dial gauge readings over 24 hours using your proscribed schedule.
26. If you are testing an unsaturated soil, instead of adding a load for the next increment, fill the water bath with water and record the dial gauge readings over 24 hours using your proscribed schedule. This will allow you to determine if your soil has significant expansion or collapse potential when wetted. If you are testing a saturated soil, your sample is already inundated with water, and you will skip this step.
27. Continue with the rest of the load sequence, recording the dial gauge readings over 24 hours using your proscribed schedule for each load increment. Be sure to plot your deformation versus log-applied stress at the end of each load increment before applying the load for the next increment.

Disassembly and Final Measurement

After your final unloading increment, you must completely unload the soils, determine the final moisture content of the sample, and clean up the consolidometer.

28. Measure the tare weight of a moisture content tin large enough to hold the entire soil sample.
29. Loosen the dial gauge mount, raise the dial gauge, and rotate it 90° so it is out of the way of the loading arm. Then tighten down the dial gauge mount.
30. Remove any remaining weights from the load hanger.
31. Raise the loading arm and remove the consolidometer cell from the load frame.
32. Empty the water from the consolidometer cell without disturbing the sample.
33. Remove the confining ring and the sample from the cell. Using a paper towel, daub any excess water from the surface of the sample.
34. Remove the filter paper from the bottom of the sample and extrude the sample into the moisture content tin by pushing on top of the sample with the small porous stone.
35. Remove the filter porous stone and filter paper from the top of the sample.
36. Determine the moisture content and mass of the sample using the normal procedures. You will use these measurements to determine the final void ratio of the sample.



10.6: DATA REDUCTION

10.6.1: Determination of the Final Void Ratio

37. Assuming the sample was saturated at the end of the test, compute the final void ratio, e_f as

$$e_f = G_s w_f \quad (10.10)$$

where

G_s = specific gravity of the soil solids

w_f = final water content of the sample in decimal form

You will have to assume a reasonable value for G_s .

10.6.2: Computation of the Stress-Strain Curve (Strain versus Log-effective Stress)

For each load increment, you must determine the strain at the end of the loading period using the deformation measured at 24 hours after loading. You can then plot the strain as a function of the applied vertical effective stress.

38. Determine the deformation reading as zero strain, δ_0 , using the dial gauge reading 6 seconds after the seating load was applied (Step 20).
39. For each load increment, compute the strain as

$$\epsilon_i = \frac{\delta_{24-i} - \delta_0}{l_0} \quad (10.11)$$

where

ϵ_i = the vertical strain for the i^{th} load increment

δ_{24-i} = the measured deflection at 24 hours for the i^{th} load increment

l_0 = the initial thickness of the sample (usually 1")

40. Use Casagrande's method for reconstructing the bi-linear strain versus log-stress curve. See Coduto (Section 11.4, 1999) or another soil mechanics text for presentation of Casagrande's method.
41. Using the reconstructed bi-linear strain versus log-stress curve, compute:
 σ'_c , the preconsolidation stress
 $\frac{C_r}{1 + e_0}$, the slope of the recompression
 $\frac{C_r}{1 + e_0}$, the slope of the virgin curve
42. Using the reconstructed bi-linear strain versus log-stress curve, compute the *overconsolidation ratio, OCR*, as

$$OCR = \frac{\sigma'_z}{\sigma'_c} \quad (10.12)$$

and the overconsolidation margin, σ_m , as

$$\sigma_m = \sigma'_c - \sigma'_z \quad (10.13)$$

where

σ'_c = preconsolidation stress and

σ'_z = effective vertical stress from depth at which the sample was taken

If you prefer to plot the stress-strain curve as void ratio versus log-effective stress, you can use the value of e_f , computed in Step 37, to convert the strain readings into void ratio.

10.6.3: Computation of the Coefficient of Consolidation

To determine the coefficient of consolidation, c_v , you must have taken readings throughout an entire 24-hour loading period. The coefficient of consolidation is computed from the plot of vertical deflection versus time. Since you have deflection versus time data for each load increment, it is possible to compute a value of c_v for each load increment. You will find that the computed c_v for each load increment is different. In particular, there may be significant differences between the values of c_v computed during recompression compared to the values computed during compression on the virgin curve.

We generally compute c_v at 50% consolidation since this is halfway through the consolidation process and represents a good average for c_v during a given load increment. To compute c_v , we must know the time required for the sample to reach 50% consolidation and the length of the drainage path. We can then compute c_v using Equation 10.8 and Figure 10.4.

There are two methods for computing c_v , the log-time method and the square root of time method. The simplest is the log-time method.

Log-time Method

In the log-time method, we first find the deformation at 0% consolidation, δ_0 , and 100% consolidation, δ_{100} . According to Terzaghi's one-dimensional consolidation theory, the deformation of the sample will be proportional to the degree of consolidation, U . Therefore, the deformation at 50% consolidation, δ_{50} , will be halfway between δ_0 and δ_{100} . The detailed procedure follows.

43. For a given load increment, plot the dial gauge readings as a function of log-time and connect with a smooth curve, as shown in Figure 10.7. You can plot the raw dial gauge readings. There is no need to convert them to strain.
44. To estimate the point of 100% primary consolidation, extend the linear tail of the curve back toward the y-axis (line 1 in Figure 10.7). Then draw a line tangent to the point of inflection in the central portion of the curve (line 2 in Figure 10.7). The point where these two lines intersect is taken to be the point at which primary consolidation ends, or the point where the degree of consolidation is 100%. Determine δ_{100} from the point where these lines intersect.

We cannot extend the plot of deformation versus log-time back to a time of zero since zero time does not exist on the semi-log plot. Instead, we will take advantage of the fact that the early part of the curve has a parabolic shape. Using this assumption, the procedure is as follows:

45. Select a time, t_A , in the early portion of the curve, as shown in Figure 10.7. Then compute the time, t_B , using

$$t_B = 4t_A \quad (10.14)$$

46. Compute $\Delta\delta$ the difference between the dial gauge readings at t_A and t_B .
47. Compute the deformation at time zero, δ_0 , as

$$\delta_0 = \delta_A - \Delta\delta \quad (10.15)$$

as shown in Figure 10.7.

48. Compute the thickness of the sample at t_{50} using the original thickness and δ_{50} . The drainage path, H_{dr} , will be one-half of the thickness, since the sample is doubly drained.
49. Insert t_{50} and H_{dr} into Equation 10.8 and compute c_v as

$$c_v = \frac{T_{50} H_{dr}^2}{t_{50}} \quad (10.16)$$

where

T_{50} = time factor at 50% consolidation

$t_{50} = 0.196$, from Figure 10.4

Square Root of Time Method

One shortcoming of the log-time method is that the test must be run well past 100% consolidation in order to back-calculate t_{50} . This can take considerable time. Fortunately, there is a faster method, called the *square root of time* method. The theory of this method is beyond the scope of this manual, but the procedure follows.

50. Plot the deformation-time data as deformation versus the square root of time, as shown in Figure 10.8.
51. Ignoring the first few data points, draw a line through the linear portion of the deflection versus square root of time curve (line 1 in Figure 10.8). The point where the line intersects the y-axis is the deflection reading at zero deformation, δ_0 .
52. At any convenient place on line 1, pick a point, A , and measure the distance, X_A , from the y-axis to point A . Along the same horizontal line, measure the distance $X_B = 1.15 X_A$, and plot the point B (see Figure 10.8).
53. Draw a straight line from δ_0 through B (line 2 in Figure 10.8). The point at which line 2 intersects the curve represents 90% consolidation with the coordinates $\sqrt{t_{90}}$ and δ_{90} , as shown in Figure 10.8.
54. Compute δ_{50} using

$$\delta_{50} = \delta_0 + \frac{5}{9}(\delta_{90} - \delta_0) \quad (10.17)$$

and graphically determine $\sqrt{t_{50}}$ as shown in Figure 10.8.

55. Using the value of t_{50} determined above, and the drainage path of the sample, H_{dr} , compute c_v using Equation 10.16.

10.7: PRESENTATION OF RESULTS

Reports of consolidation tests generally present the computed stress-strain curve. The typical format of the stress-strain curve varies among geographic regions. In the Western U.S., it is generally plotted as strain versus log-effective stress. In the Midwest and Eastern U.S., it is generally plotted as void ratio versus log-effective stress. Either method is acceptable. The preconsolidation stress, σ'_c , is generally shown on the stress-strain curve.

The compressibility on the recompression and virgin curves is always reported. If the stress-strain curve is presented as strain versus log-effective stress, then the compressibilities are presented as $\frac{C_r}{1+e_0}$ and $\frac{C_c}{1+e_0}$. If the stress-strain curve is presented as void ratio versus log-effective

stress, then the compressibilities are presented as C_r and C_c .

The value of the coefficient of consolidation, c_v , can vary significantly throughout the test. In research reports, c_v may be presented as a function of the applied effective stress, σ'_z . In reports for construction projects, a design value of c_v is generally reported. This design value is generally less than the average c_v calculated, but greater than the minimum c_v calculated, during testing.

10.8: TYPICAL RESULTS

All of the parameters measured in the consolidation test can vary dramatically depending on the type of soil being tested. Table 10.3 presents a general comparison of compressibility with soil classification.

The coefficient of consolidation, c_v , can vary even more greatly than the compressibility. The values of c_v can vary from 0.007 ft²/day for Chicago glacial clays to 0.33 ft²/day for Boston Blue clay (Holts and Kovacs, 1981). Regionally, the value of c_v for a specific soil deposit may be well established. You should check your data against reported values for similar soils.

10.9: COMMON ERROR SOURCES

The quality of the field sample will have an overwhelming effect on the quality of the data from a consolidation test. High-quality Shelby tube-type samples will be significantly less disturbed than samples from a thick-walled California-type sampler. If you are testing medium to soft clays, it is critical that you acquire high-quality field samples.

During data acquisition, by far the most common error is associated with incorrect dial gauge readings. It is very easy to misread an analogue dial gauge. To counteract this problem, you should 1) practice reading the dial gauge before starting the test, and 2) plot your stress-strain data during the test to spot potential problems.

Table 10.1: Recommended load sequence.

Applied Stress	Notes
100 lb/ft ²	Seating Load
200 lb/ft ²	Loading Sequence
400 lb/ft ²	
800 lb/ft ²	
1,600 lb/ft ²	
2,400 lb/ft ²	
4,800 lb/ft ²	
6,400 lb/ft ²	
12,800 lb/ft ²	
3,200 lb/ft ²	Unloading Sequence
800 lb/ft ²	
200 lb/ft ²	

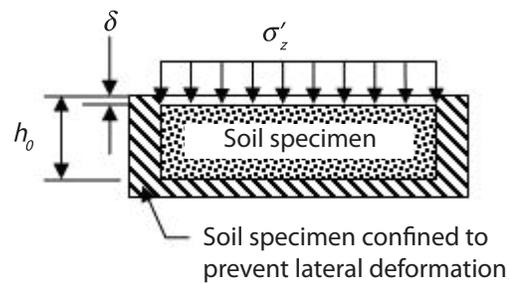
Table 10.2: Recommended time sequence for reading when computing the coefficient of consolidation, c_v .

Elapsed Time of Reading	
6 sec (0.1 min)	20 min
12 sec (0.2 min)	30 min
30 sec (0.5 min)	1 hr
1 min	2 hr
2 min	4 hr
4 min	8 hr
8 min	12 hr
10 min	24 hr

Table 10.3: Relative compressibility and soil classification.

$\frac{C_r}{1+e_0}$ or $\frac{C_c}{1+e_0}$	Relative Compressibility	Soil Classification		
0 – 0.05	Very Slightly Compressible	Clayey/Silty Sand & Stiff Clay		
0.05 – 0.10	Slightly Compressible		Medium Clay	
0.10 – 0.20	Moderately Compressible			
0.20 – 0.35	Highly Compressible			Soft Clay
> 0.35	Very Highly Compressible			

Compressibility definitions from Coduto, 1999.

**Figure 10.1:** Loading conditions for the one-dimensional consolidation test.

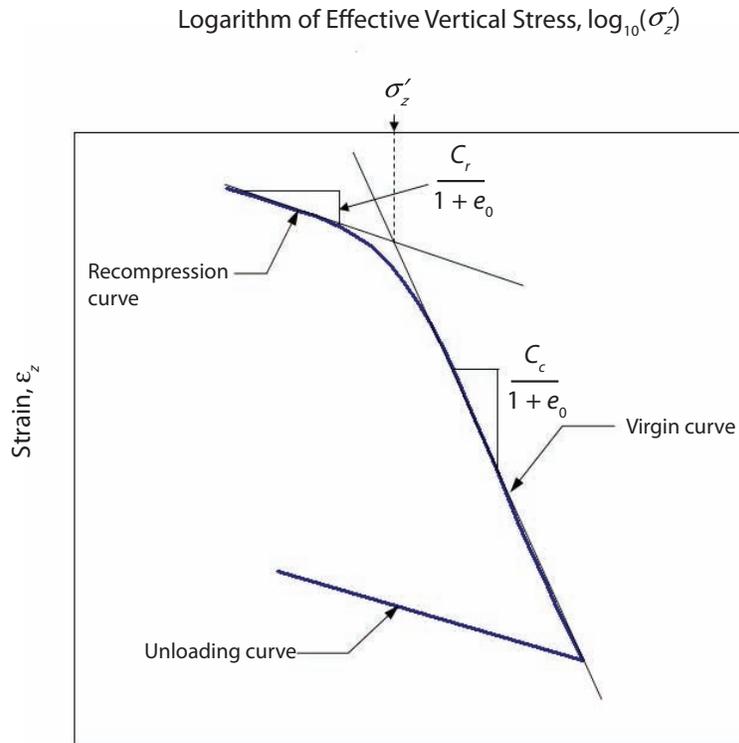


Figure 10.2: Stress-deformation curve for a soil plotted as vertical strain versus log-effective stress.

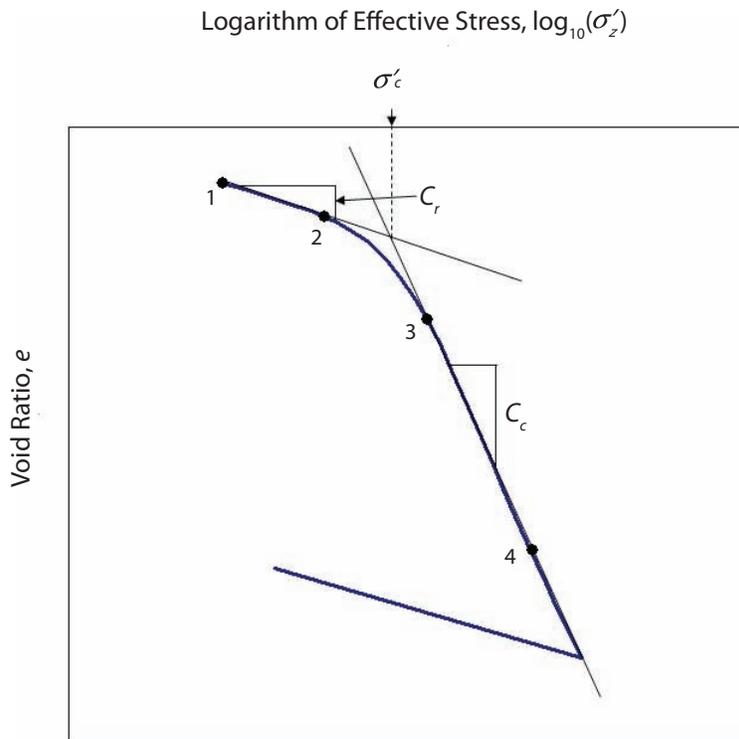


Figure 10.3: Stress-deformation curve for a soil plotted as void ratio versus log-effective stress.

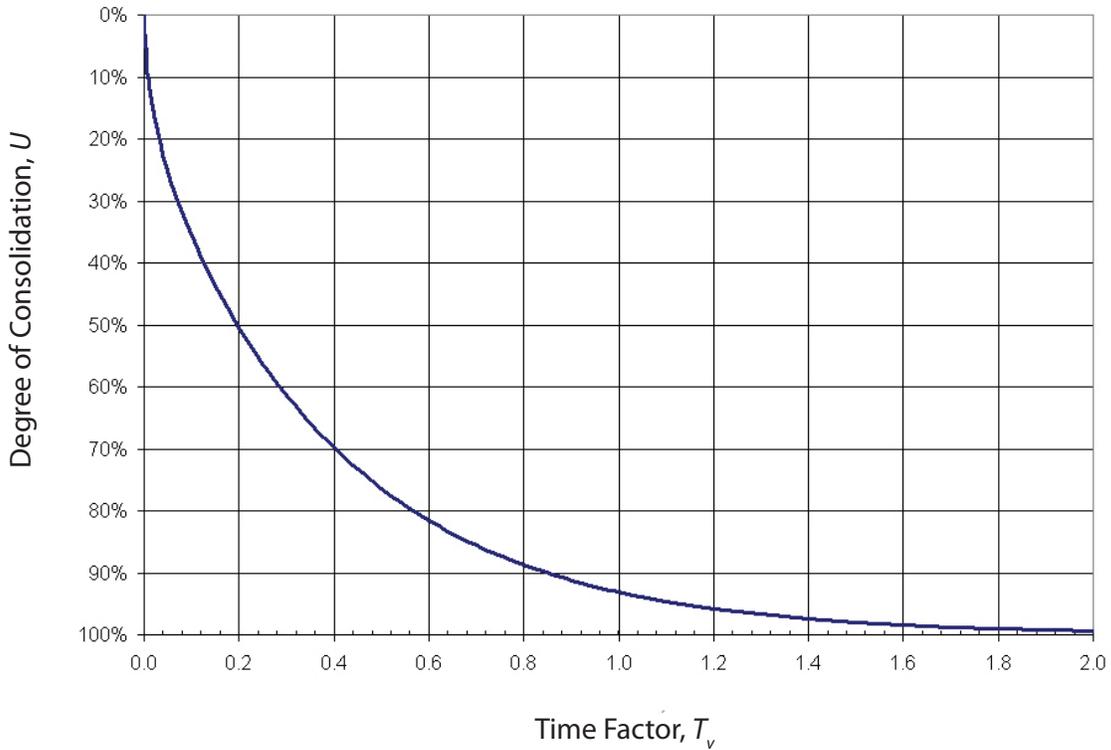


Figure 10.4: Relationship between the Degree of Consolidation and the Time Factor for Terzaghi's one-dimensional theory of consolidation.

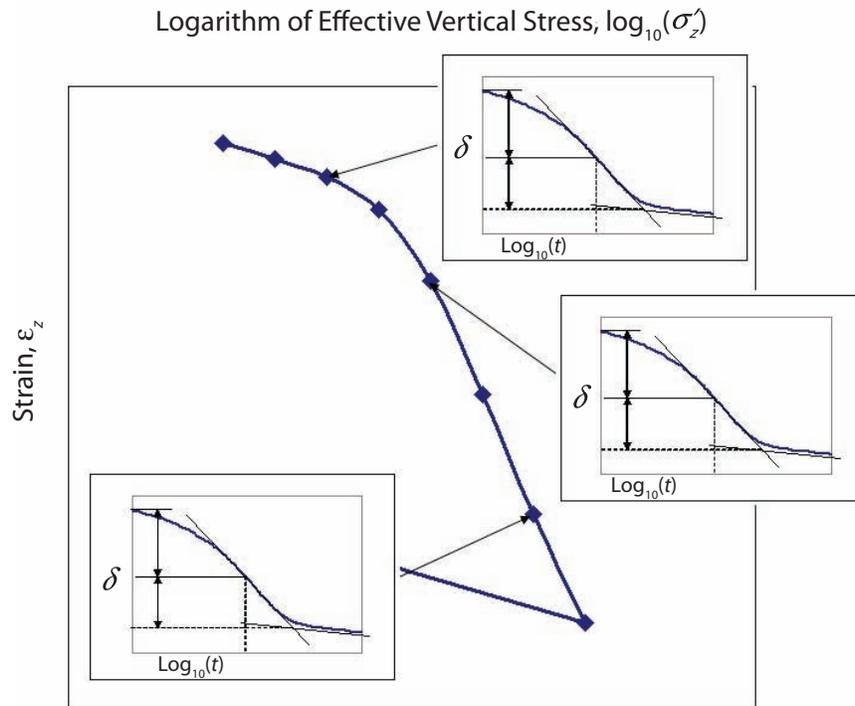


Figure 10.5: Illustration of how each point on the soil stress-strain curve comes from an individual load increment of deformation versus time.

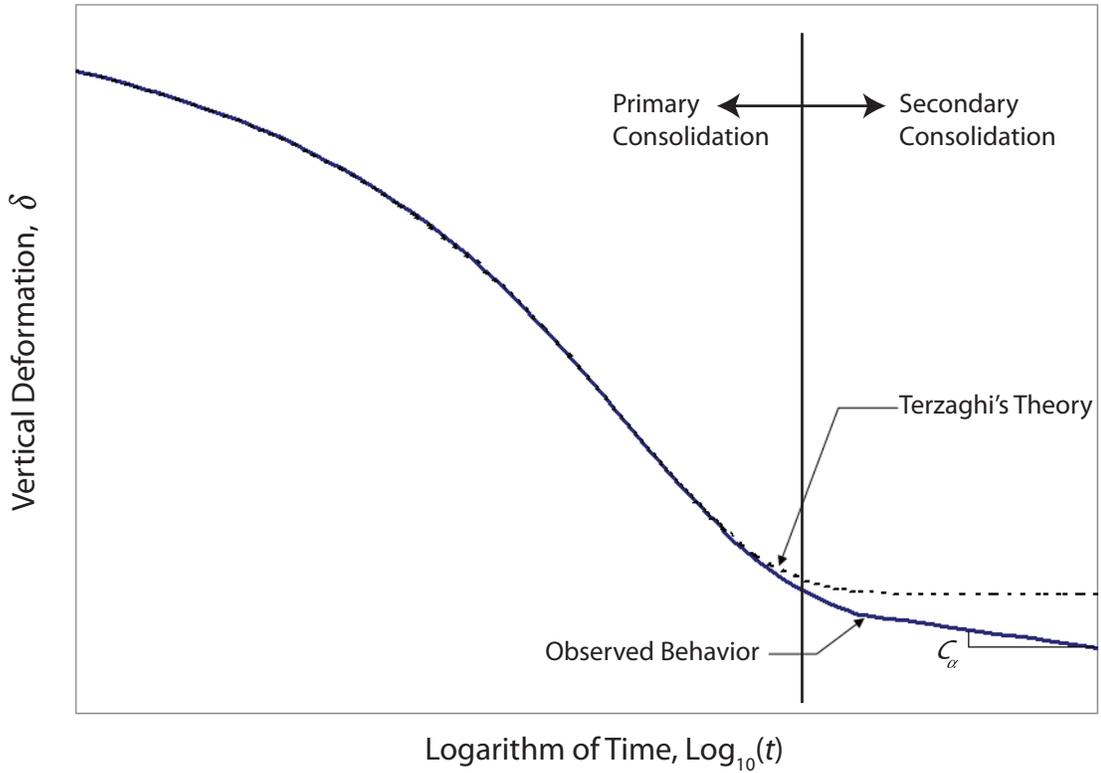


Figure 10.6: Typical plot of vertical deflection versus log-time, showing difference between Terzaghi's theory and observed behavior and the effects of secondary consolidation.

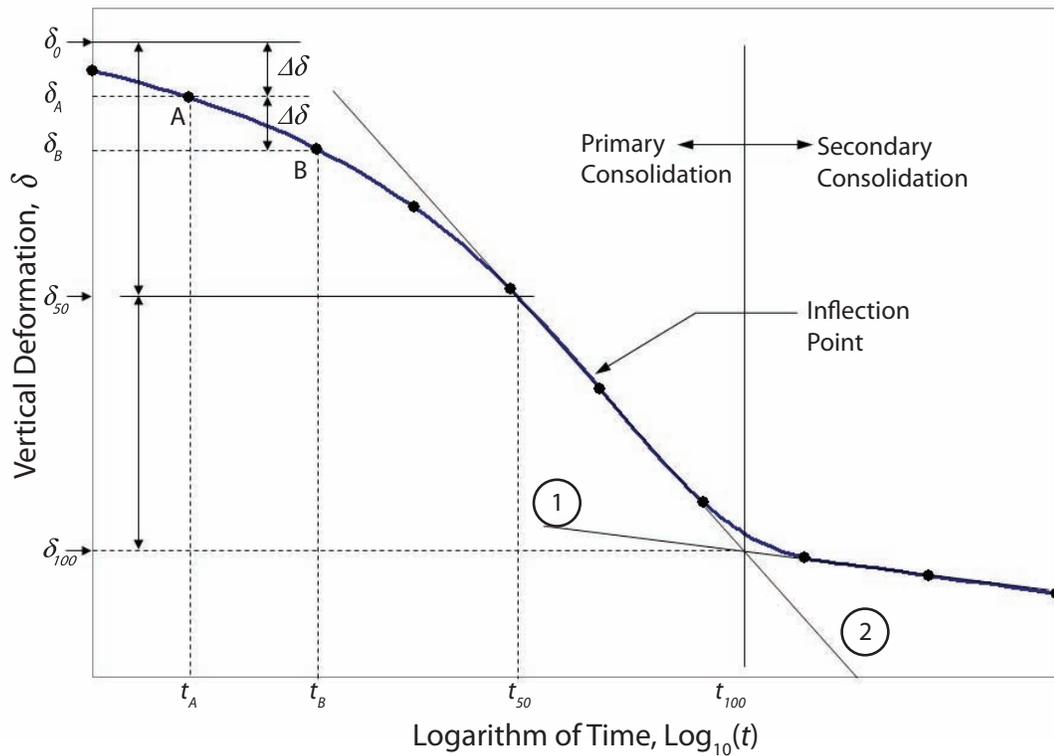


Figure 10.7: Typical plot of vertical deflection versus log-time for a consolidation test, showing procedure for determining, δ_{100} , δ_{50} , and t_{50} .

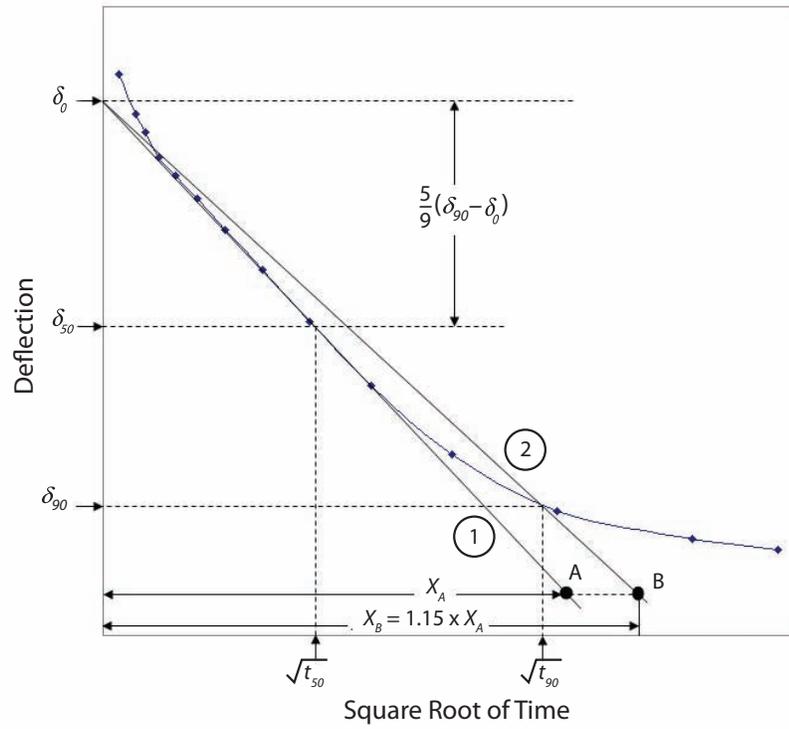


Figure 10.8: Typical plot of vertical deflection versus square root of time.

Strength Determination: Direct Shear Test

11.1: PURPOSE

Many geotechnical engineering projects require knowledge of the shear strength of soils. Examples include bearing capacity, slope stability, and lateral earth pressure. The direct shear test is one of the simplest tests used to determine soil strength. The test can be performed on both undisturbed and laboratory-prepared soil samples. The goal of the direct shear test is to determine the Mohr-Coulomb shear strength parameters of the soil: the friction angle, ϕ , and the cohesion, c .

11.2: THEORY AND DEFINITIONS

In the direct shear test, we place a soil sample in a device called a *shear box*. There are two basic shapes of shear boxes: circular and square. The shape description refers to the horizontal cross-sectional shape of the shear box. The shear box is similar to a consolidometer cell in that it allows for application of a vertical stress with vertical strain, but prevents the sample from deforming in the horizontal direction. The shear box differs from the consolidometer cell in that it is split at mid-height on a horizontal plane and thereby allows us to shear the soil sample along this horizontal plane. Figure 11.1 shows the loading conditions for the direct shear tests.

11.2.1: Determining Strength Parameters

In the direct shear test, we apply a known vertical load to a soil sample and allow the soil to consolidate. We then apply a horizontal stress and force the soil to fail on a horizontal plane. Since the soil is forced to fail on a horizontal plane, both the shape and the area of the failure plane are known. The applied normal stress, σ , and applied shear stress, τ , are both defined as

$$\sigma = \frac{F_v}{A} \quad (11.1)$$

$$\tau = \frac{F_h}{A} \quad (11.2)$$

where

F_v = the applied vertical force

F_h = the applied horizontal force

A = cross-sectional area of the sample on a horizontal plane

During the test, we will measure the applied horizontal force as a function of the horizontal displacement, as shown in Figure 11.2. The applied shear force will increase to a peak value, as

shown in Figure 11.2. If the soil is a loose sand or normally consolidated clay, the applied shear stress will level off at the peak values, as shown in Curve (a) of Figure 11.2. If the soil is a dense sand or an overconsolidated clay, the shear stress will reach a peak value, after which it will decrease and become asymptotic to a lower value, as shown in Curve (b) of Figure 11.2. In this second case, the lower asymptote represents the *residual shear* force.

Using Equation 11.2, we can convert these forces to the peak shear stress, τ_p , and the residual shear stress, τ_r . The peak strength represents the ultimate strength the soil can reach, and the residual strength represents the strength the soil will have after a large amount of shear deformation. The residual strength is useful for analyzing the stability of landslides that have previously failed and undergone significant shear deformation. For intact soils, we generally base analyses on the peak strength, using an appropriate factor of safety. Some soils, particularly loose sands and normally consolidated clays, show no significant loss in strength after reaching a peak. In this case, the peak and residual strengths are equal.

From a single direct shear test, we get a single point on the peak shear strength envelope (σ , τ_p), and a single point on the residual shear strength envelope (σ , τ_r). However, to determine the strength parameters ϕ and c , we must have more than one point on the failure envelope. To draw a straight-line Mohr-Coulomb failure envelope, we need at least two points on the failure envelope. In general, we require at least three points to reduce the uncertainties in testing. Therefore, a complete direct shear test actually consists of three tests of the same soil, each performed at a different normal stress.

Figure 10.3 is a plot of shear stress versus normal stress, showing how each data point comes from a single direct shear test. Each test is performed at a different normal stress, thus generating three different points on the plot. The Mohr-Coulomb failure envelope is plotted as the best-fit line to these three points, using the least squares method. Using this failure envelope, we can determine both the friction angle, ϕ' , and the cohesion, c' , of the soil.

11.2.2: Vertical Deformation During Shear

In addition to measuring the horizontal force and deformation, we generally measure the vertical deformation during the direct shear test. This measurement gives us insight to the soil behavior during shear and helps us to distinguish loose sands from dense sands, as well as normally consolidated clays from overconsolidated clays.

Figure 11.4 is a plot of vertical deformation versus horizontal deformation during direct shear testing. Positive values of vertical deformation indicate expansion or dilation of the soil during shear. Negative values of vertical deformation indicate compression or contraction of the soil during shear. All soils show some initial contraction during shear, as shown in Figure 11.4. Loose sands or normally consolidated clays continue to contract during shear because the soil particles in these soils are loosely packed and the shearing process allows the particles to rearrange themselves in a tighter configuration, thus causing the soil to compress or contract, as shown in Curve (a) of Figure 11.4. In contrast, dense sand and overconsolidated clays dilate during shear after the initial contraction, as shown in Curve (b) of Figure 11.4. The dilation occurs because the soil particles are packed tightly together. In order for the soil to shear, the particles must slide over one another, thus causing the soil to expand or dilate.

11.2.3: Pore Pressures, Effective and Total Stresses

One of the shortcomings of the direct shear test is that there is neither control over the drainage of the sample nor any way to measure the pore pressures during testing. In general, we want to

know the effective stress strength parameters, ϕ' and c' . We know the total stresses applied on the failure plane from the applied vertical and horizontal stresses. However, during testing, the soil tends to generate shear-induced excess pore pressure. If the sample is contractive, it tends to generate positive excess pore pressure. If it is dilative, it tends to generate negative excess pore pressure. Since we cannot control the drainage condition or measure the pore pressure, the only way we can assure that we can know the effective stress state on the failure surface is to ensure the excess pore pressure is zero. To do this, we must ensure that the soil is tested in a fully drained state.

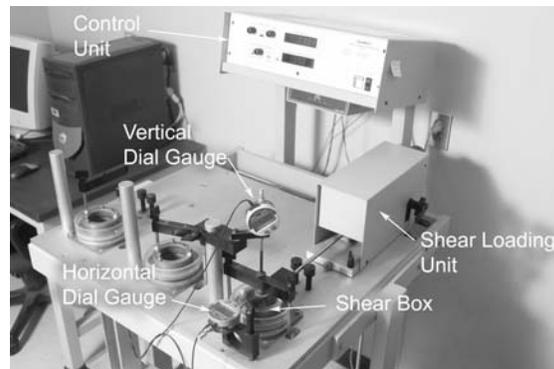
There are two approaches to ensuring that the soil is fully drained. If you are testing a clean sand, then it is possible to test the sand in a dry state, where there is no pore fluid and therefore no pore pressure. For soils with any significant amount of clay or silt, it is necessary to inundate the sample with water, allow the sample to fully consolidate under the applied vertical stress, and shear the sample slowly enough that any excess pore pressure generated has time to dissipate.

11.3: APPLICABLE STANDARDS

- ASTM D3080: Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions

11.4: EQUIPMENT AND MATERIALS

- Geomatic® direct shear machine
 - Shear boxes (3)
 - Vertical loading systems (3)
 - Calibration/display unit
 - Shear loading mechanism
 - Calibrated weights
 - Vertical dial gauge
 - Horizontal dial gauge
 - Computer
 - Geomatic software Version 6.0
- Plastic sample extruding rod
- Thin porous stone
- Thick porous stone
- Loading cap
- Steel ball
- Filter paper
- Allen wrenches: $\frac{3}{16}$ " and $\frac{5}{32}$ "
- Plastic sheeting
- USB storage device (student-provided)



11.5: PROCEDURE

11.5.1: Calibrating Geomatic® Direct Shear Apparatus

1. Turn on the machine and allow it to warm up for at least 15 minute before use.
2. Ensure that the shear loading arm is not connected to anything; then zero the shear load by using the zero dial on the calibration/display unit.
3. Ask your instructor for the R-cal calibration value. Press the R-cal button and wait until the number stops changing. While pressing the R-cal button, adjust the span dial on the calibration/display unit until the display reads the correct calibration value.
4. Now you must set the deflection rate to the default value of 0.05 in/min. To do so, ensure the forward/reverse switch is in the forward position. Next, ensure the speed switch is set to in/min. Now turn the rate adjustment knob until the display reads 50.0. Note that the display is in 0.001 inches, so a display of 50 is $50 \times 0.001 = 0.050$ in/min.



Once you have calibrated the apparatus, do not change either of the shear load calibration knobs (zero or span). When you have to adjust the shear load to zero from now on, you will do so by adjusting the manual crank on the side of the shear load mechanism, as described later.

11.5.2: Sample Preparation

5. You will need three undisturbed samples of the same soil to perform the test. Select a sample tube with a full six rings of soil. Inspect the top and bottom ring to ensure the sample is from a single soil layer.
6. Before trimming any samples, perform a unit weight test on the entire tube sample, following the procedures in Chapter 4.
7. Trim three individual ring samples from the tube sample. Ensure that each sample is flush with the sample ring on both the top and the bottom. When you have finished trimming the samples, store them on the counter with plastic on the top and the bottom so they will not dry out.
8. Make sure the lower shear box is clean. To do so, remove the machine screws holding it to the shear table, remove the shear box from the table, clean the shear box, and bolt the shear box back onto the table.



9. Before attaching the upper shear box, back out the three leveling machine screws so they do not extend below the bottom of the upper shear box.



10. Attach the upper shear box to the lower shear box using the nylon screws. Finger-tighten the screws. Make sure that the bolt hole for the shear arm is at the back of the shear box.



11. Place the thinner porous stone in the bottom of the shear box and center a piece of filter paper on top of the stone.



12. There is a lip cut in the upper shear box that exactly fits a sample ring. Place the sample (still in its ring) on top of the upper shear box, ensuring that the ring is properly seated in the lip.



13. Center a piece of filter paper on top of the sample; place the thick porous stone on top of the filter paper.



14. Using the plastic extruding rod, push the sample down until the sample is seated completely inside the shear box. Try to use one smooth motion when pushing the sample into the shear box. Remove the extruding rod and sample ring.



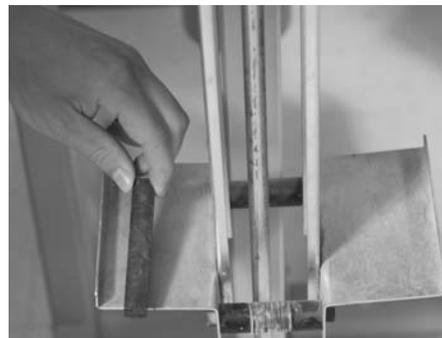
15. Place the black loading cap on top of the porous stone and put the steel ball on top of the loading cap. Ensure the cap is centered on the stone.



16. Swing the cross bar into position over the steel ball and tighten the two thumbscrews.



17. Place a 100 lb/ft² weight into the weight hanger to seat the loading arm. Examine the sample and loading arm to ensure it is properly assembled.
18. Set up samples in the other two shear boxes using the same procedures.



11.5.3: Sample Consolidation

Before proceeding, you must determine the range of normal stress that is appropriate during testing. The range of normal stress that you use during testing should span the range of normal stress that is applicable in the project for which your strength data will be used. If your project will have normal stress in the range of 200 to 1,000 lb/ft², then you don't want to perform testing at 4,000 to 10,000 lb/ft². Conversely, if your project will have normal stress in the range of 2,000 to 6,000 lb/ft², then you don't want to perform testing at 100 to 1,000 lb/ft². It is usually sufficient to compute the vertical effective stress at the depths of interest in your project and use these values to determine the range of normal stress to use during testing.

19. Estimate the range of effective stress of interest to your project and select three normal loads that span this range. In general, your loads should be between 1,000 and 6,000 lb/ft². Before proceeding, discuss with your instructor the load you plan to use.

If we were to follow the correct ASTM testing procedures, we would now inundate the samples with water, load the samples with the appropriate normal stress, and perform a 24-hour consolidation test on the samples. We would use the data from the consolidation test to ensure each sample was fully consolidated and to determine a shear loading rate that would allow us to test the sample slowly enough that no excess pore pressure would be generated. However, we will forgo the 24-hour consolidation test and test the samples at their natural moisture content without adding water. This procedure is sufficient for teaching purposes and will yield reasonable strength parameters for unsaturated sands, silty sands, and slightly clayey sands. While the strength parameters you measure are sufficient for educational purposes, this procedure does not provide the correct strength parameters for your soil, and it should not be used in practice.

20. For each test sample, place the appropriate weights on the weight hanger and remove the 100 lb/ft² seating load.

11.5.4: Attaching the Shear Loading Unit to the Shear Box

21. Move the shear loading unit to align with the sample being tested. Ensure that the unit is secured to the shear table with the thumbscrew.



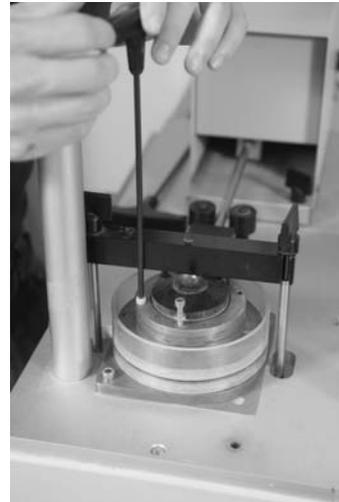
22. Loosen the clutch on the shear loading unit (the outer knob on the side of the unit), and adjust the loading arm using the hand crank until the hole in the loading arm aligns exactly with the hole in the upper shear box.



23. Bolt the loading arm to the upper shear box with the stainless steel machine screw, using the Allen wrench to tighten the machine screw.
24. Zero the shear load by cranking the load arm backwards or forwards until the shear load reads zero. Do **not** use the zero dial to zero the shear load. Tighten the clutch knob, ensuring the shear load remains zeroed.



25. Remove the nylon screws holding the shear box together.



26. You now must raise the upper shear box off the lower shear box to reduce friction between the two halves of the shear box. To do this, lower the three leveling screws, using only your fingers, until you feel the screws just touch the lower shear box. Using the Allen wrench, lower each leveling screw an additional $\frac{3}{4}$ of a turn. (Some of the screws have a shoulder or cap that allows them to be lowered to exactly the right depth.)
27. Once all three leveling screws have been lowered, raise each of them back up 2 full turns so they no longer contact the lower shear box.



28. Put the vertical dial gauge in place. The foot of the dial gauge should rest on the rounded pin on top of the cross arm above the sample. The sample top will travel $\frac{1}{4}$ " back toward the shear loading unit. Make sure the pin is located near the front of the dial gauge foot so the dial gauge will not fall off the pin during loading. The dial gauge should be set in the middle of its travel range so it can measure both positive and negative vertical displacement.

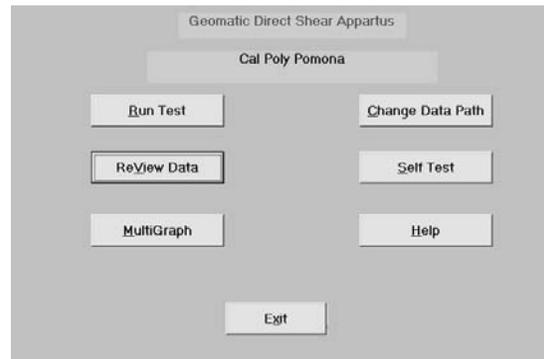


29. Place the horizontal dial gauge so its foot rests against the tall stainless steel machine screw on the front of the upper shear box. Ensure the dial is pointed directly in the direction of travel.
30. Push the reset button on each dial gauge to zero the gauges.

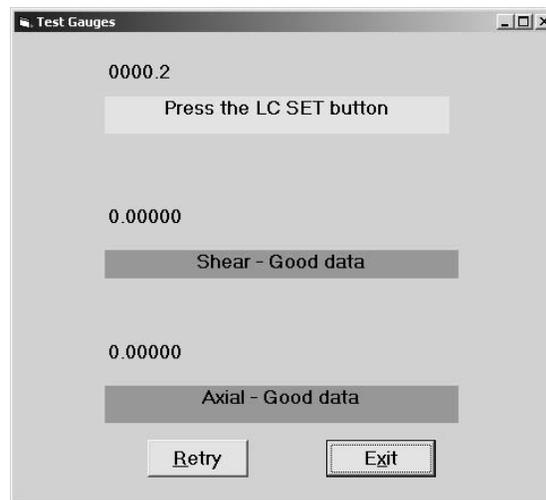


11.5.5: Setting up the Computer

31. Log on to the computer and insert your USB storage device.
32. Go to Start—> Programs—> Geomatic—> Shear
You should see the following window.
33. Click the Change-Data-Path button to redirect the data into your storage device.
34. Click the Run-Test button.



35. You should now see the Test-Gauges window. All three boxes should read zero and be green. If any of the boxes is yellow, it means that particular gauge is not set to zero. Reset the gauges to zero as needed and click Retry. When all boxes are green, click Exit.



36. The Run-Test window should appear. Fill in all the data boxes from Client down to, and including, “Confining Pressure (psf).” The rest of the data should be correct, but check with your instructor to be sure.
37. Click on the box labeled “Unknown.dat.” This will allow you to provide a name for your data file. You should pick a file name that will help you identify the particular test you are running. A suggested format is shown below:

BxxSyy-nnnn.dat

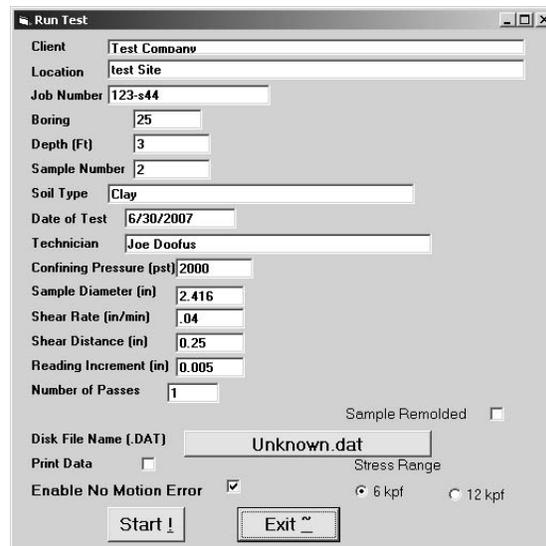
where

xx is the boring number

yy is the sample number

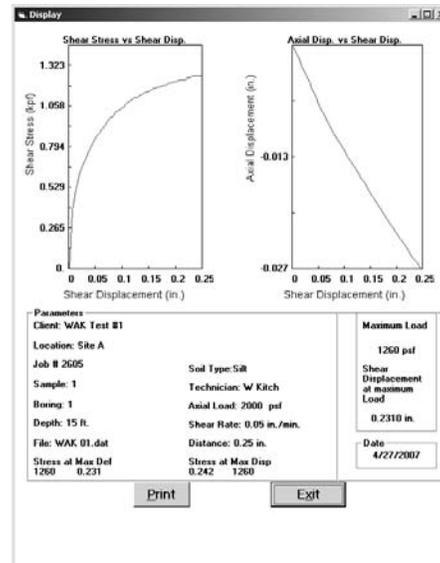
nnnn is the normal stress in lb/ft²

Be sure to use a different file name for each test, or you will overwrite your previous data file.



38. After entering the file name, click Start. The following screen should appear. Click Start again, and the shear apparatus should start running.
39. When the test is complete, record the peak load data from the screen so you can double check your data when you plot them in your report.
40. Click Exit, and your data file will be closed.

The file saved on your USB drive is a simple text file. You can open it with any text editor or import it into Excel for plotting. The format of the data file is shown in Figure 11.7.



11.5.6: After Running the Test

41. Carefully remove the digital dial gauges.
42. Remove the weights from the hanger arm.
43. Loosen the thumbscrew on the cross bar, and swing the cross bar out of the way.
44. Reach under the table and push up the plunger under the sample. This will push your sample up through the shear box. Remove the sample from the shear box.
45. Carefully open the sample along the shear plane. Examine the shear plane. The shear plane should be relatively flat and smooth. If you see large chunks of soil or large pieces of gravel in the shear plane, make a note. Any large particles in the shear plane may result in inaccurate measurements of the shear strength.
46. Unbolt the shear box (upper and lower sections) from the table. Clean the box and bolt it back to the table.

11.6: DATA REDUCTION

The first steps in data reduction are to compute the applied normal and shear stresses from the measured loads. Data reduction is simplified when using the Geomatic® direct shear machine. The data collection program automatically computes the applied shear stress. Each weight has a number stamped on its side. This number considers both the sample area and load arm ratio and is equal to the applied normal stress in lb/ft². Therefore, you can skip Steps 48 and 49 when using this machine.

47. Compute the applied normal stress using Equation 11.1.
48. For each data point, compute the applied shear stress using Equation 11.2.
49. Plot shear stress, τ , versus horizontal displacement, δ_h , for each value of normal stress, σ .
50. From the plot of τ versus δ_h , determine the peak shear stress, τ_p , and the residual shear stress τ_r , for each value of σ .
51. Create a Mohr-Coulomb failure envelope by plotting (τ_p , σ) from each test on a plot of shear stress, τ , versus normal stress, σ , as shown in Figure 11.3.
52. Draw a best-fit line through the data points on the Mohr-Coulomb plot.
53. Compute the cohesion, c , as the intercept of the best-fit line and the internal angle of friction, ϕ , as the angle the line makes with the horizontal. Be careful when determining ϕ . If your plotting program does not plot the x- and y-axes to the same scale, you will have to determine ϕ by computing the inverse tangent of the slope of the line rather than measuring it with a protractor.

11.7: PRESENTATION OF RESULTS

The direct shear data are generally presented as shown in Figures 11.5. The shear stress data from all three tests are plotted on a single graph as a function of horizontal displacement. Similarly, the vertical displacement data from all three tests are plotted on a single graph as a function of horizontal displacement, as shown in Figure 11.6. The peak strength values from Figure 11.5 are then plotted as a function of normal stress, as shown in Figure 11.3 and the parameters c and ϕ are determined from this plot. If residual strength parameters are needed, a similar graph can be generated using residual strengths from Figure 11.5.

The final values for c and ϕ are generally summarized both on boring logs from where the soil samples were taken and in a summary table in the report.

11.8: TYPICAL RESULTS

Clean sands will have a cohesion that is essentially zero and an effective friction angle, ϕ' , generally ranging between 28° and 45° . Loose soils will be at the lower end of this range, and dense soils at the upper end.

Saturated clay soils can have an effective friction angle, ϕ' , ranging from 10° to 35° . Kaolinite will be at the high end of this range. Illite will be in the middle of this range. Montmorillonite and Bentonite will be at the lower end of this range. The effective cohesion intercept, c' , can range from less than 100 lb/ft^2 for normally consolidated clays to over 2000 lb/ft^2 for highly overconsolidated clays.

11.9: COMMON ERROR SOURCES

Three of the most common sources of errors in the direct shear test are disturbance of the sample, failure to allow complete saturation and consolidation of the sample, and shearing the soil too rapidly.

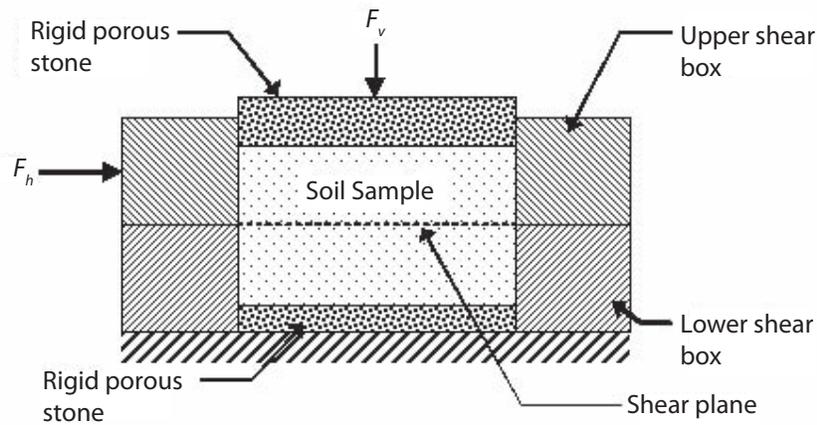
The strength of a soil is highly dependant on its density, particle orientation and structure, and cementation. Sample disturbance nearly always has the effect of reducing soil density, breaking up soil structure, and reducing soil cementation. The net effect is to reduce the strength of the sample being tested. Therefore, it is important to acquire high-quality undisturbed samples when performing direct shear tests and then to handle those samples carefully during testing.

As discussed above, the direct shear test has no way to control sample drainage or to measure pore pressures during testing. It is essential, therefore, to ensure that the sample is under zero pore pressure during testing. This can only be accomplished by saturating the soil to eliminate negative pore pressure present in partially saturated soils and ensuring that consolidation is complete and no excess pore pressures exist. Testing partially saturated clays and clayey sands will generally lead to an overestimate of the cohesion due to negative pore pressures present in the unsaturated soil. Testing soils that are not fully consolidated will generally lead to underestimating the shear strength, due to positive pore pressures present in the sample.

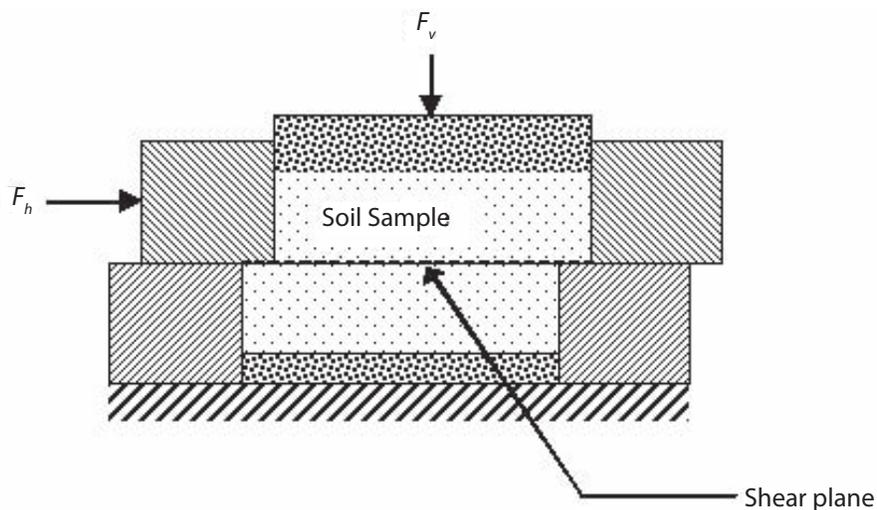
Shearing samples too quickly will have an effect similar to testing unsaturated samples or samples that are not fully consolidated. If a soil is sheared too quickly, shear-induced excess pore pressures

will not be able to dissipate. These shear-induced pore pressures will change the effective stress in the sample and affect the measured shear strength.

Shearing a normally consolidated soil too quickly will generally cause an underestimate of the shear strength because these soils will generate positive shear-induced pore pressures, leading to lower effective stresses and lower shear strength. Conversely, shearing an overconsolidated soil too quickly will lead to an overestimate of the shear strength because these soils will generate negative shear-induced pore pressures, leading to higher effective stresses and higher shear strength. You must be very careful to shear highly overconsolidated soils slowly enough, since errors generated by rapid shearing will produce unconservative strength values.



a) Loading conditions during consolidation before shear displacement.



b) Loading conditions during shear displacement.

Figure 11.1: Loading conditions during the direct shear test.

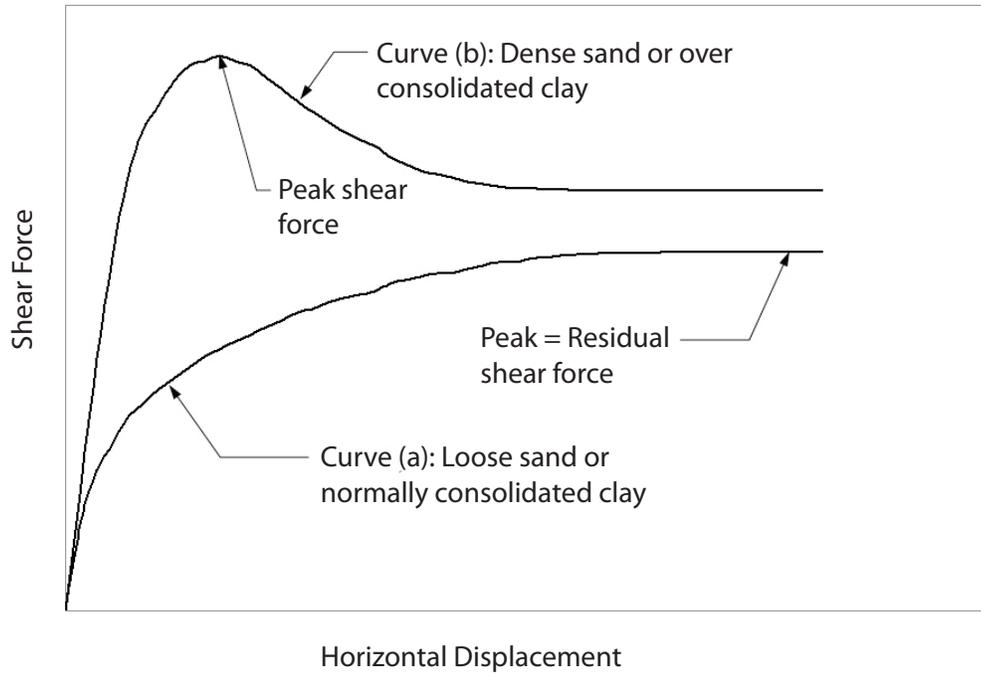


Figure 11.2: Typical curves of shear force versus horizontal displacement for direct shear tests.

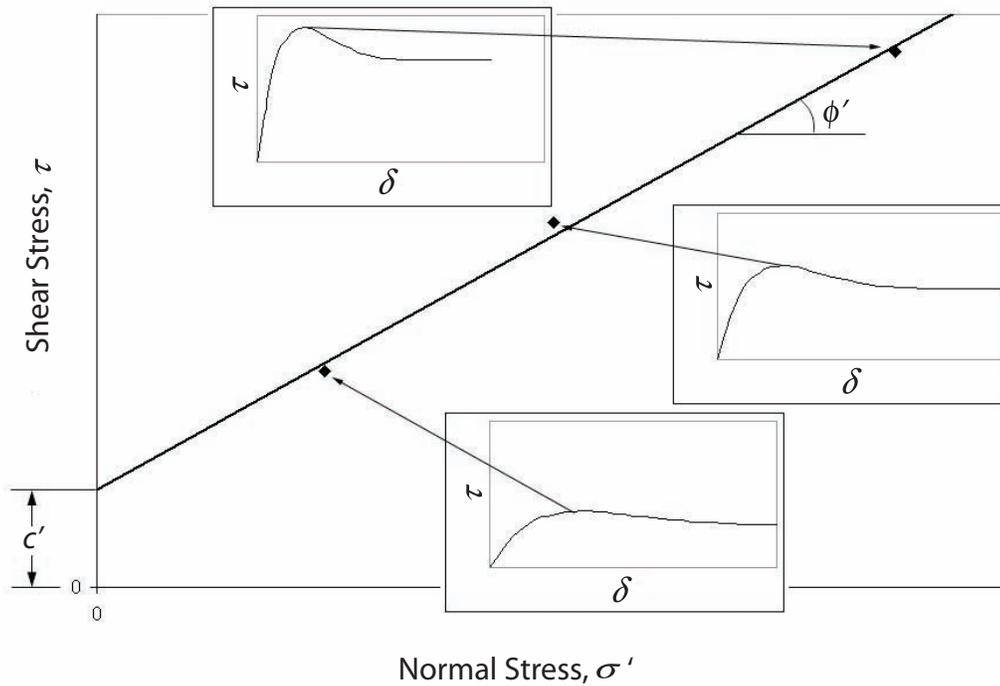


Figure 11.3: Typical Mohr-Coulomb failure envelope developed from a series of three direct shear tests showing the effective cohesion intercept, c' , and effective friction angle, ϕ' .

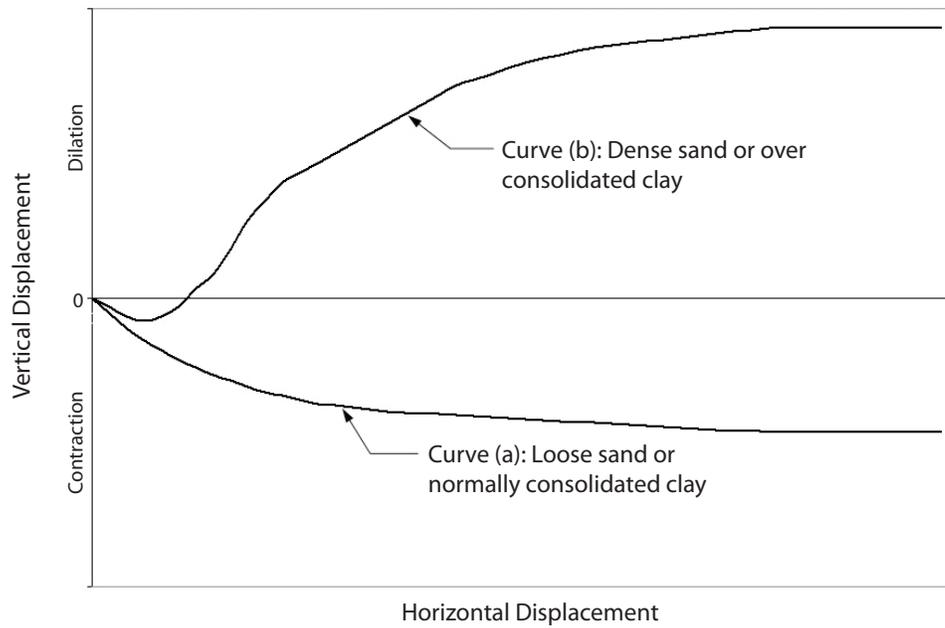


Figure 11.4: Typical Mohr-Coulomb failure envelope developed from a series of three direct shear tests showing the effective cohesion intercept, c' , and effective friction angle, ϕ' .

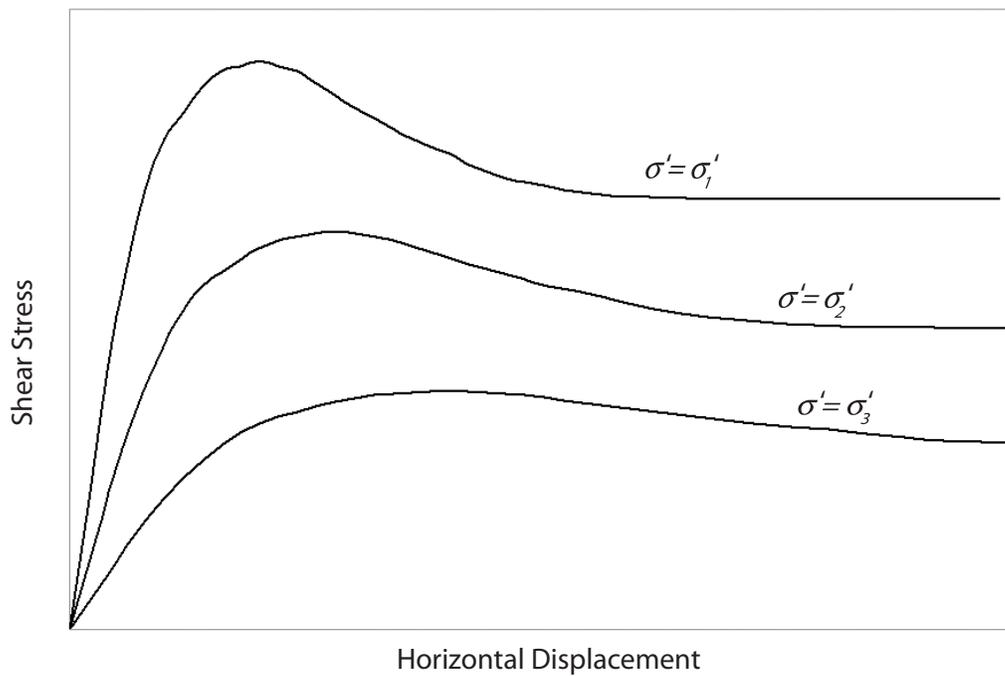


Figure 11.5: Typical plot of shear stress versus horizontal displacement showing data from three tests on a single graph.

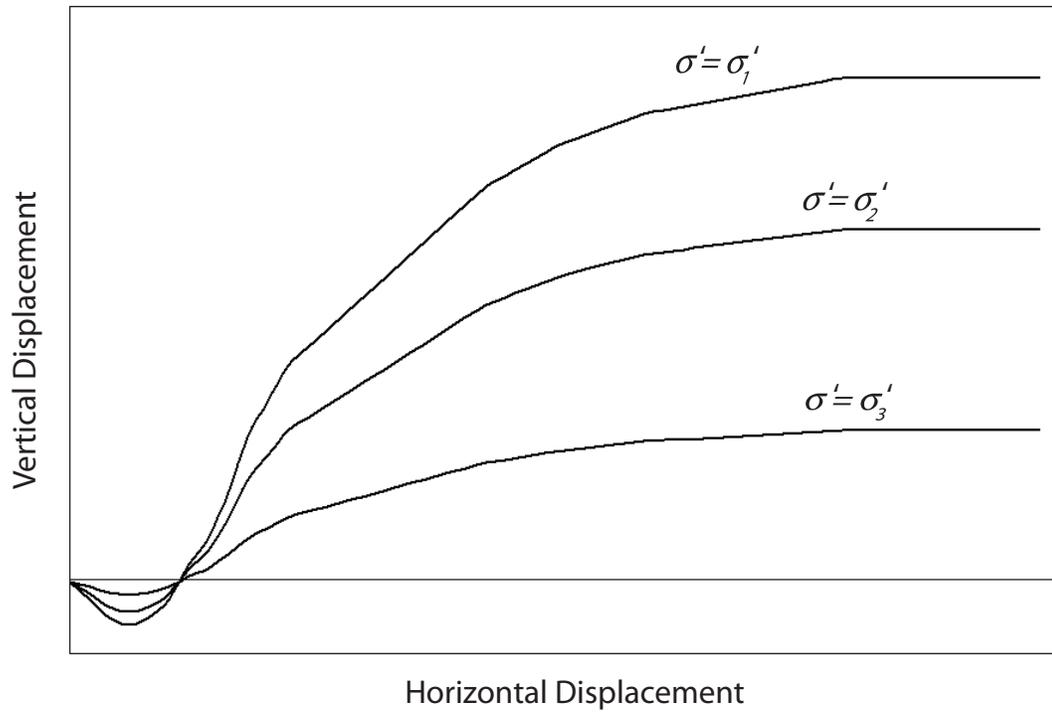


Figure 11.6: Typical plot of vertical displacement versus horizontal displacement showing data from three tests on a single graph.

Acme Distributors	←	Client		
2007-15-1654	←	Job #		
4/27/2007	←	Date of test		
G Perez	←	Technician		
Pomona Ave	←	Location		
2.416	←	Sample diameter		
2000	←	Axial (normal) stress (psf)		
1	←	# of passes of shear apparatus		
0.25	←	Maximum shear displacement (in)		
ML Olive Brown Silt	←	Soil type		
1	←	Boring #		
3	←	Sample #		
15	←	Sample Depth (ft)		
0.05	←	Shear rate (in/min)		
0.006 384 -0.0008	←	Shear data		
0.012 504 -0.0015		Column #1	Column #2	Column #3
0.016 576 -0.0023		Horz disp (in)	Shear stress (psf)	Vert disp (in)
0.021 636 -0.0029				+ up, - down
0.026 684 -0.0036				
0.042 804 -0.0058				
0.057 888 -0.0079				
0.061 912 -0.0084				
0.067 936 -0.0092				
0.071 960 -0.0097				
0.082 996 -0.0109				
0.086 1020 -0.0114				
0.092 1032 -0.0119				
0.106 1080 -0.0134				
0.117 1104 -0.0145				
0.122 1116 -0.0150				
0.131 1128 -0.0160				
0.136 1140 -0.0164				
0.146 1152 -0.0175				
0.157 1176 -0.0185				
0.167 1188 -0.0195				
0.171 1188 -0.0199				
0.181 1200 -0.0209				
0.192 1212 -0.0219				
0.196 1224 -0.0223				
0.202 1224 -0.0229				
0.207 1236 -0.0233				
0.211 1236 -0.0236				
0.217 1236 -0.0242				
0.221 1248 -0.0247				
0.227 1248 -0.0253				
0.237 1260 -0.0261				
0.242 1260 -0.0265				
0.246 1260 -0.0269				
999	←	Signals end of shear data		
1260	←	Maximum shear stress (psf)		
.2310	←	Horizontal disp at point of maximum shear stress (in)		
Geomatic Shear V.632	←	Software version		

Figure 11.7: Annotated sample data file from Geomatic shear test.

Reporting Results: The Geotechnical Report

12.1: BACKGROUND

The objective of the report is to gather all of the data from your field and lab work and present it in the form of a professional engineering report. This is not a research report or an academic report about what you learned from a particular test. It is a professional engineering report providing the results of your investigation along with your recommendations to the client. The report must be bound with a comb, spiral, or similar binding. An outline for the report follows. You may modify the outline as you see fit.

12.2: REPORT OUTLINE

12.2.1: Cover Sheet and Letter of Transmittal

The first page of the report is a cover sheet that contains the project name, project number, client name, date of report, and name of your company. (You should make up a company name for your group.)

Just behind the cover sheet is a cover letter. The cover letter is a short (normally one page) letter that basically says, “Here’s the work you asked us to do. Thanks for the opportunity. Please call us again.” This is where the official signatures for the report go. This is often called a *transmittal letter*.

Following the cover/transmittal letter is a table of contents for the report. It should include all major headings and all appendices.

12.2.2: Introduction

This is a short introduction outlining the purpose of your report.

12.2.3: Project Description

This section presents a short description of the project for which this investigation is being done. You will include in this section any design criteria that has been given to you. You may wish to add subsections here.

12.2.4: Scope

This section includes a description of the scope of work for your part of the project. State simply and clearly what it is you have been contracted to do. Do not take on more responsibility than you are getting paid for.

12.2.5: Site Exploration Procedures

Field Exploration

This section describes the basic procedures used for the subsurface exploration. This is not a “how to” guide or a blow-by-blow procedure. It’s an overview describing the important aspects of the process.

Laboratory Testing

This section briefly describes each of the laboratory tests performed and their purpose. The descriptions should focus on what is determined from each test rather than on specific procedures. Again, this is not a “how to” section, but an overview of the tests, why they were performed, and what was measured in each test. Be sure to reference the exact location in the appendices where the data can be found (e.g., Figure A-3, or Table B-6). It’s important to present tables and figures that summarize the results in this section. Don’t expect the reader to search through your lab data to find the important results. Summarize them clearly for the reader, either in tables or figures.

12.2.6: Site Conditions

This section contains a short description of the actual site at the time of your field exploration. Note any significant surficial geologic or hydrologic conditions and the general topography. You should also note the current state of construction (i.e., whether the site is vacant, whether there are existing structures or pavement on site, whether grading has started, etc.)

12.2.7: Geologic Conditions

Regional Geology

This discussion should come from your desk study. You don’t need a book. Just enough of the regional geology so the client knows the general soil and rock conditions to be expected. This section will cover an area of tens to hundreds of miles, depending on the geologic setting.

Local Geology

This section covers the geology at and within a mile or so of the immediate site. Again, the information comes from your desk study.

Geologic Hazards

In California, nearly every geotechnical report will have a section covering geologic hazards of the area. This generally includes nearest faults and ground motion parameters, landslide potential, and liquefaction potential. For the purposes of our report, we will keep this section short. Use data from the background studies provided to give a general discussion of geologic hazards.

12.2.8: Subsurface Conditions

In this section, you will provide a short summary of the soil profile you found at the site. Briefly describe the soil types and thickness of soil layers found at the site. Be sure to refer to the appendix that contains your subsurface data.

12.2.9: Groundwater Conditions

Here, describe groundwater conditions encountered on the site. You may also wish to refer to other sources of groundwater data, such as regional maps of historic groundwater levels. Again, refer to the appendix with your data.

12.2.10: Engineering Recommendations

This is where your engineering judgment comes in. The recommendations you make depend on the scope of the project and what you found in your investigation. In this project, as a minimum, you should provide guidance about earthwork and foundations. Be clear in your guidance. Don’t be wishy-washy. Make clear, understandable recommendations.

Earthwork

This is where you provide your guidance on removal and or improvement of soils unsuitable for the project. Specifically, you need to make clear recommendations about the suitability of the surficial soils to support a slab on grade and/or pavements. At a minimum, you should provide guidance on

- removal and recompaction depth, lateral extent, and compaction criteria (if applicable)
- surface drainage
- general compaction requirements
- compaction requirements for special uses, such as soils below structural slabs and pavements
- required inspection during compaction operations

Be sure to reference appropriate appendices.

Foundations

This is where you provide specific recommendations for the foundation design. At a minimum, you should provide recommendations on

- allowable bearing stress
- minimum footing depth and width

You should indicate what is driving the allowable bearing stress (settlement or bearing capacity) and clearly state the criteria (e.g., less than 1" total settlement, or a factor of safety against bearing capacity failure of 3). Be sure to reference appropriate appendices.

12.2.11: General Comments

This is the place for whatever else you think needs to be said. In particular, you should reiterate the scope of your services and describe any limitations to your recommendations.

12.2.12: Limitations

Geotechnical reports, in particular, should always have a section that describes the limitations of the data you have taken and the recommendations you have made. Briefly discuss any limitations of your interpretations and designs. This should include a description of the standard of care you provided, the data upon which your work was done, and the limitations of your data and analyses.

12.2.13: Figures and Tables

Do not put figures (or plates, as they are sometimes called) in the body of the report. First of all, it makes pagination difficult. Secondly, in technical reports, people often need just the figures or tables. It's more convenient to find and copy them if they're all in one location. Be sure every figure is numbered and has a title—every figure—each and every one! When you want to refer to a figure in the body of the report, be sure to refer to it by its proper number. Do not expect your readers to search through your figures to determine which one you want them to review!

12.2.14: Appendix A: Field Exploration

This appendix contains all of your finalized boring logs. Be sure they are properly numbered and labeled. The numbering should start with the appendix letter (e.g., Figure A-1: Log of Boring B06).

12.2.15: Appendix B: Laboratory Testing

This appendix contains the final results of all of your lab tests. For each test, you must clearly identify the boring and depth from which the test sample came, as well as describe the soil classification. Again, clearly number and label every table and figure. Every table or figure will have a "prepared by" and "checked by" signature.

12.2.16: Appendix C: Laboratory Data Sheets

Geotechnical reports do not always include all of the raw data. It depends upon the nature of the project. For our project, you are to include all your raw data for each test in this appendix. Each section should also include a short discussion of your test results. In particular, note any problems or limitations of your data. In professional practice, one would not present suspect data; you would have to repeat the test. However, in our learning environment, we realize you will not always get it right the first time and often won't realize something is wrong until you begin preparing your report. This appendix is where you can discuss such limitations or problems. Every data sheet will have a "prepared by" and "checked by" signature.

12.2.17: Appendix D: Foundation Calculations

This appendix presents all of your computations and calculations for the foundation analysis. Hand calculations are fine, as long as they are neat, logical, and legible. If you are using spreadsheets or programs for calculations, you must include at least one hand calculation for each different spreadsheet or program you use. The purpose of the hand calculations is to verify the automated computations. All calculations will have a "prepared by" and "checked by" signature.

Appendix A

Field Identification of Soils

Complete classification of soils using the Unified Soil Classification System (USCS) requires lab testing that can take several days. However, you will often need to rapidly classify soils in the field. Fortunately, with a little bit of experience, it is possible to quickly classify most soils without the use of any specialized equipment. Two excellent guides to visual classification of soils are ASTM D2488: *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)* and the U.S. Army Field Manual 5-410, *Military Soils Engineering*, (pgs 5-17 through 5-28). This appendix draws heavily from these two references.

A.1: EQUIPMENT

These tests can be performed using only a water bottle. The following items are helpful in performing visual classification, but are not necessary:

- field logging guide from Appendix A
- pie pan or pizza pan
- #4 and #200 sieve (small 5" diameter sieves are available and are ideal for this purpose)
- pocket penetrometer or Torvane® device

A.2: PROCEDURE

A.2.1: Color and Smell

1. Describe the color of the soil. If you have any form of standardized color chart, such as that shown in Appendix A, use it. The color should be described when the soil is moist. If the soil is dry, add a bit of water before determining the color.
2. Describe the wetness of the soil using the moisture classification provided in Appendix A.
3. If the sample is intact, estimate the soil consistency using a pocket penetrometer, if available, or by attempting to push your thumb into the sample. Use the consistency descriptions in Appendix A.
4. Smell the soil. Classify the smell as none, earthy, organic, or chemical, using the descriptions in Appendix A.
5. If the soil has an organic smell, is spongy, and contains large amounts of undecayed plant matter, the soil is peat (PT).

A.2.2: Fine- versus Coarse-Grain Determination

The objective of this step is to determine whether the soil is coarse-grained (gravel and sand) or fine-grained (silt and clay). The dividing line between coarse- and fine-grain particles is the #200 sieve, which has an opening size of 0.75mm. This is also approximately the smallest size that can be seen by the naked eye.

6. Spread a portion of the sample out in a pan or on a plastic sheet. Visually estimate the percent of soil that would pass the #200 sieve (percent that is finer than 0.75 mm). If you have a #200 sieve, you may use this to separate the fine and coarse fractions. If you don't have a sieve, assume that any individual particles that you can see with your naked eye are coarse-grained.

7. If 50% or more of the soil is coarse-grained, then the soil is sand and/or gravel. Proceed to Step B.2.3.
8. If more than 50% is fine-grained, then the soil is clay and/or silt. Proceed to Step B.2.4.

A.2.3: Coarse-Grained Soil Classification

9. Visually determine what percentage of the coarse-grained fraction of the soil would be retained on the #4 sieve (percent that is larger than 4.75 mm). If you have a #4 sieve, you may use it. If not, estimate the percentage using the particle size chart in Appendix A. If 50% or more of the coarse fraction is retained on the #4 sieve, then the soil is gravel. If less than 50% of the coarse fraction is retained on the #4 sieve, then the soil is sand.
10. Determine if the soil is clean or not. Visually determine if less than 10% of the sample passes the #200 sieve. If so, then the soil is a clean sand (SP or SW) or a clean gravel (GP or GW). If 10% or more of the soil passes the number #200 sieve, then the soil is clayey or silty sand or gravel (SC, SM, GC, or GM). You will have to test the fine-grain portion of the sample using procedures in Section B.2.4 to complete the classification of the soil.
11. Determine gradation by observing the relative diversity of soil particle sizes. If the soil contains a very large range of particle sizes in a relatively uniform distribution, then the soil is well-graded and is either SW or GW. If not, it is poorly graded and is either SP or GP. Most soils are poorly graded. When in doubt, assume poorly graded.
12. Examine a number of individual particles for their shape. Describe the particle shape using the titles and visual aid found in Appendix A.
13. If your soil is a clean sand or gravel, classification is complete; there is no need to perform the tests in Section B.2.4. If not, proceed.

A.2.4: Fine-Grained Soil Classification: Distinguishing Silts from Clays

Silts and clays are distinguished primarily based on their relative plasticity and dry strength. Clays have a greater ability to absorb water and remain in a plastic state than do silts. Clays also tend to have a higher strength when dried than do silts.

14. Separate out a small handful of soil finer than the #40 sieve. Use the particle size chart in Appendix A to estimate #40 sieve size.

Dry Strength Test

15. Mold a sample of the soil into a 1" diameter ball, adding water if necessary. Flatten the ball into a disk about ½" thick.
16. Set the disk aside to dry. You may put the disk on a warm surface, such as the hood of a car, to speed drying.
17. When the disk is completely dry, test its drying strength by breaking the disk between your fingers. Use the dry strength description in Table B.1 to describe its strength.

Dilatancy or Wet Shaking Test

18. Select enough soil to form a ball approximately ½" in diameter. Form the soil into a ball, adding enough water that the soil is soft but does not have free water on the surface.
19. Flatten the ball of soil out to form a pat in the palm of your right hand. With your right palm facing up, slap your right hand down into your left hand several times. Slap vigorously.
20. Observe the surface of the soil pat to see if water appears. The water will be seen as a sheen on the surface of the pat. Note how quickly the sheen forms, if at all.
21. If a sheen does form, flex the hand holding the soil pat open and close it slightly. Observe if the sheen goes away and, if so, how quickly.
22. Classify the reaction to the dilatancy test as none, slow, or rapid, based on the descriptions in Table B.2.

Ribbon Test

23. Select enough soil to make a cylinder of soil $\frac{1}{2}$ " to $\frac{3}{4}$ " in diameter and 3" to 4" long. Mold the soil with enough water that it is plastic but not sticky.
24. Carefully squeeze the soil between your thumb and forefinger to form a ribbon $\frac{1}{8}$ " to $\frac{1}{4}$ " thick. Let the ribbon dangle below your hand as you form it. Try to make the ribbon as long as possible without it breaking. Note the length of the ribbon at the time it breaks. Describe the plasticity of the soil as nonplastic, low, medium, or high, based on the descriptions in Table B.3.

Determination of Fine-Grained Soil Type

25. Using the results of the dry strength, dilatancy, and ribbon tests, determine the soil classification using Table B.4.

A.2.5: Completing Classification

Follow the flowsheet in Figure B.1 to determine the USCS group symbol for the soil.

To verify the classification, you must perform a complete lab analysis and classify the soil according to ASTM D2487: *Classification of Soils for Engineering Purposes (Unified Soil Classification System)*.

Table A.1: Dry Strength Test Classification (based on ASTM 2488 and FM 5-410).

Strength Classification	Behavior of Dry Specimen
None	Crumbles into powder with slightest finger pressure
Low	Crumbles into powder with moderate finger pressure
Medium	Breaks into pieces with considerable finger pressure
High	Cannot be broken into pieces with finger pressure, but can be broken using thumb and forefinger against a hard surface
Very High	Cannot be broken using thumb and forefinger against a hard surface

Table A.2: Dilatancy Test Classification (based on ASTM 2488 and FM 5-410).

Strength Classification	Behavior of Specimen During Test
None	No sheen forms on surface of pat during shaking
Slow	Sheen appears slowly on pat during shaking and does not disappear or disappears slowly when flexing hand after shaking
Fast	Sheen appears quickly on pat during shaking and disappears quickly when flexing hand after shaking

Table A.3: Plasticity Classification Using Ribbon Test (based on FM 5-410).

Strength Classification	Behavior of Specimen During Test
Nonplastic	No ribbon can be formed
Low	Ribbon can be formed with difficulty but will hold together lengths of only 3" or less
Medium	Ribbon can be formed and will hold together for lengths of 3" to 6"
High	Ribbon can be formed and will hold together for a length of 6" or more

Table A.4: Classification of Fine Soil Based on Dry Strength, Dilatancy, and Plasticity (based on ASTM 2488 and FM 5-410).

USCS Group Symbol	Dry Strength	Dilatancy	Plasticity
ML	None to Low	Slow to Rapid	Nonplastic to Low
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

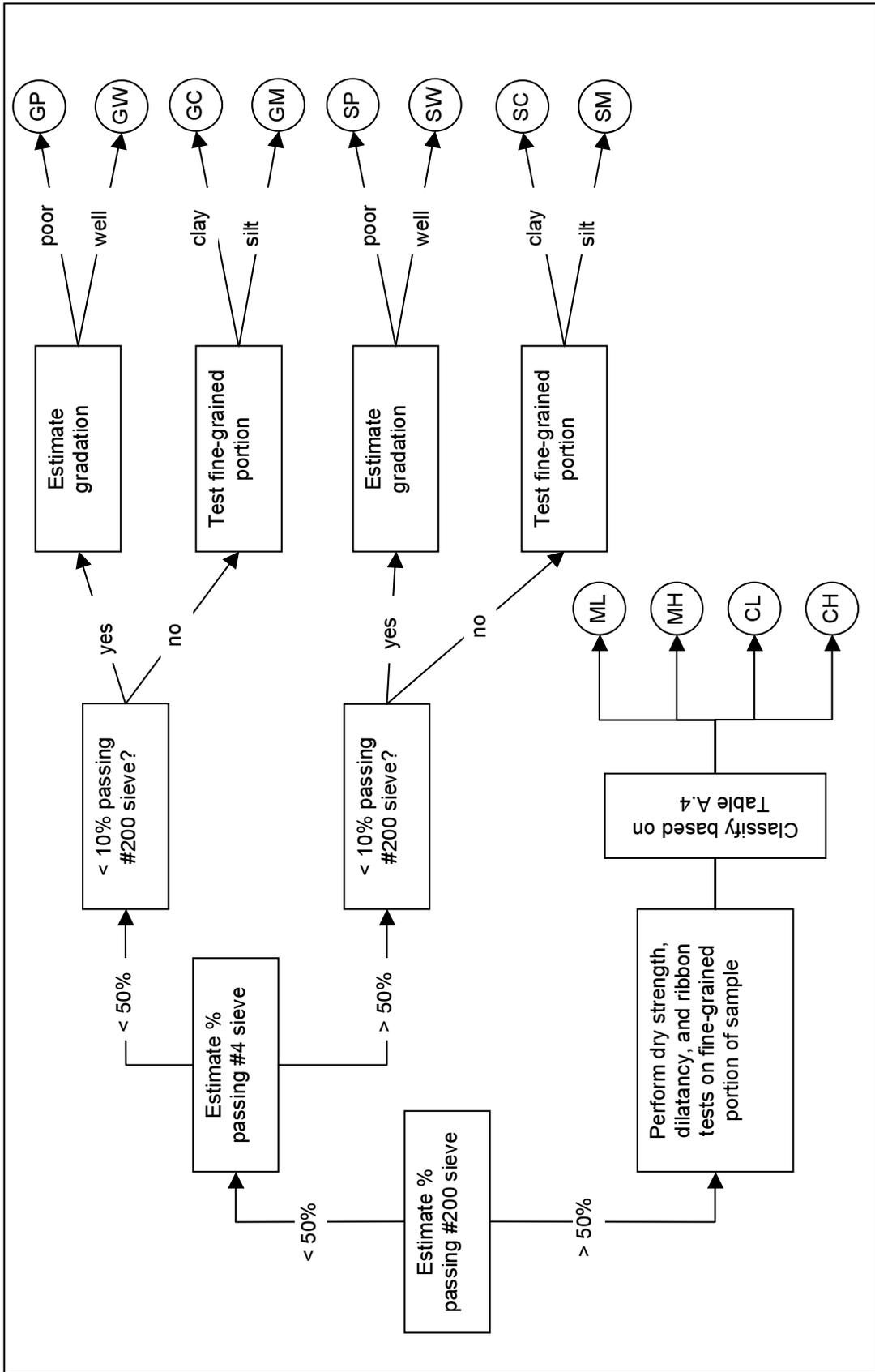


Figure A.1: Field classification flowsheet.

Appendix B

Safe Practices Plans

B.1: GEOTECHNICAL LAB

B.1.1: Applicability

- Faculty and staff in Civil Engineering Department working in the Geotechnical Engineering Lab
- Students enrolled in geotechnical engineering courses

B.1.2: Hazards

Potential hazards in this lab include the following:

- **Heavy Equipment and Supplies:** There are numerous heavy objects used in the lab, including buckets and bags of soil, compaction hammers, and weights for testing machines. If improperly handled, these could injure persons either by falling on persons or causing back injuries through improper lifting techniques.
- **Hot Surfaces:** There are several ovens in the labs, including bench ovens and microwave ovens. Interior surfaces of these ovens and materials being removed from the ovens can burn unprotected skin.
- **Slippery Floors and Tripping Hazards:** There are a number of potential tripping and slipping hazards, including low drawers and cabinet doors, wet floors, and loose soil on floors.
- **Cutting Devices:** Cutting and scraping devices, such as hacksaws, utility knives, wire saws, and soil scrapers, are used in the lab. Improper use can cause cuts and scrapes.
- **Chemical Usage:** Chemical usage is limited in the lab, but some potentially hazardous chemicals are used, including sodium hexametaphosphate, isopropyl alcohol, and lubricants.
- **Ladder:** Some work in the lab requires use of a ladder. Improper usage could lead to falls.
- **Pinching Hazards:** Many test machines and tools have potential pinch points, including vice, sample ejector, and proctor molds. Unsafe usage of these machines or tools can cause cuts, bruises, or blisters on hands and fingers.

B.1.3: Safe Lab Practice

The following practices will be followed to mitigate potential hazards in the lab:

- No food or drink is allowed in the lab at any time.
- No open-toed shoes, sandals, or flip-flops may be worn in the lab at any time.
- Students will work in groups of two or more in the lab at all times. (Exception: A single student is permitted in the lab for the purposes of weighing samples or reading gauges, as long as no equipment is operated.)
- All injuries, equipment damage, and hazardous conditions will be reported immediately to the instructor or lab technician.
- The lab will be kept clean and orderly at all times. Spilled soils, materials, or liquids will be cleaned up immediately. At the end of each session, the lab will be tidied up and the floors swept.
- Machines must be turned off and locked unless specifically approved and directed by the instructor.

- Before operating equipment, check your surroundings for any debris and/or individual that may harm you or others around you.
- Never attempt to repair machines yourself; shut off the machine and call the lab instructor or technician.
- Approved ladders will be used to reach elevated locations; there will be no climbing or standing on stools, chairs, or furniture. The top two steps of the stepladder will not be used as steps.
- Tools and equipment will be in kept in good condition and ready for use.
- Report all lab-related accidents and injuries to the professor.
- Aisles shall be kept clear at all times; Pick up any debris on the ground and close any open cabinet doors and drawers.
- Proper lifting techniques will be used when handling heavy items; keep the back straight and lift with the legs.

B.1.4: Personal Protective Equipment (PPE)

The following PPE is required at all times while in the lab:

- **Closed-toed Shoes:** Proper footwear must be worn at all times; sandals and flip-flops are not allowed.

The following PPE will be maintained in the lab in good working order and used when required:

- **Oven Mitts:** Protective hand mitts must be kept near bench ovens and used when handling hot items in ovens.
- **Safety Glasses:** Protective eyewear must be worn where there is a danger of flying particles.
- **Chemical Splash Goggles:** Protective eyewear must be worn when handling chemicals.
- **Protective Gloves:** Protective gloves must be worn to guard against cuts and burns.
- **Hearing Protection:** Ear protection may be worn when operating the sieve machine.
- **Dust Masks:** Protective nose and mouth gear may be worn when working with dry soils that create dust.

B.1.5: Special Equipment Safety

- **Always follow posted operating procedures.**
- **Microwave:** Do not leave an operating microwave unattended. The user will remain at the microwave at all times while in use. A timer may not be set for more than 2 minutes. If longer heating is required, an additional 2-minute period may be used. Do not place any metallic objects in the microwave or operate the microwave when empty.
- **Sieve Shakers:** Do not operate sieve shakers with the doors open. The operator must stay in the room when the shaker is being operated. Immediately shut down the shaker if noise indicates the sieves have become loose.
- **Soil Pulverizer:** Students are not allowed to operate the pulverizer. Eye protection is required while operating the pulverizer. Unplug the motor before removing the safety shield.

B.2: DRILLING AND SAMPLING OPERATIONS

B.2.1: Applicability

- Faculty and staff in Civil Engineering Department using drilling and sampling equipment in the field or present during drilling and sampling
- Students enrolled in geotechnical engineering lab course

B.2.2: Hazards

- **Overhead Equipment:** Heavy drill rods will extend above heads of people and could cause injury if they fell on someone.
- **Sharp Metal Edges:** The drilling augers and samplers have sharp points. Drill rods can have sharp burrs where pipe wrenches have been used.
- **Heavy Equipment:** Much of the equipment used is heavy and could strain or injure muscles or cause back injuries if proper lifting techniques are not used.
- **Hot Sunny Weather:** Extreme heat and sun exposure can cause sunburn, heat exhaustion, or heat stroke.

B.2.3: Safe Practices

- Before operating equipment, check your surroundings for any debris or individuals that may interfere with operations or could potentially be harmed by operations.
- Use a spotter when lifting drill rods overhead.
- Use two people when lifting drill rods into or out of a hole.
- Use proper lifting techniques for heavy objects; keep the back straight and lift with the legs.
- Report all unsafe conditions to the instructor.
- Report all work-related accidents and injuries to the instructor.
- Apply sunscreen before coming to the field.
- Bring and drink plenty of water.

B.2.4: Required PPE

- **Hard Hat:** Protective head gear must be worn at all times.
- **Safety Glasses:** Protective eyewear must be worn at all times.
- **Closed-toed Leather Shoes:** Proper footwear must be worn at all times; canvas shoes, sandals, or flip-flops are not allowed.
- **Work Gloves:** Protective gloves must be worn when handling heavy drill rods.

References

- Coduto, Donald P. (1999), *Geotechnical Engineering: Principles and Practices*, 2nd ed., Prentice Hall, New Jersey
- US Army (2001), *Engineering Manual (EM) 110-1-1804: Engineering and Design, Geotechnical Investigations*
- US Army (1997), *Field Manual (FM) 5-410: Military Soils Engineering*



SOIL DESCRIPTION

Color

GROUP NAME (Symbol): color, grain size, moisture, density/consistency, other descriptors, etc.

i.e.: Poorly Graded sand (SP): Brown, fine to medium grained, slightly micaceous, moist, dense, slight hydrocarbon odor

UNIFIED SOIL CLASSIFICATION SYSTEM¹

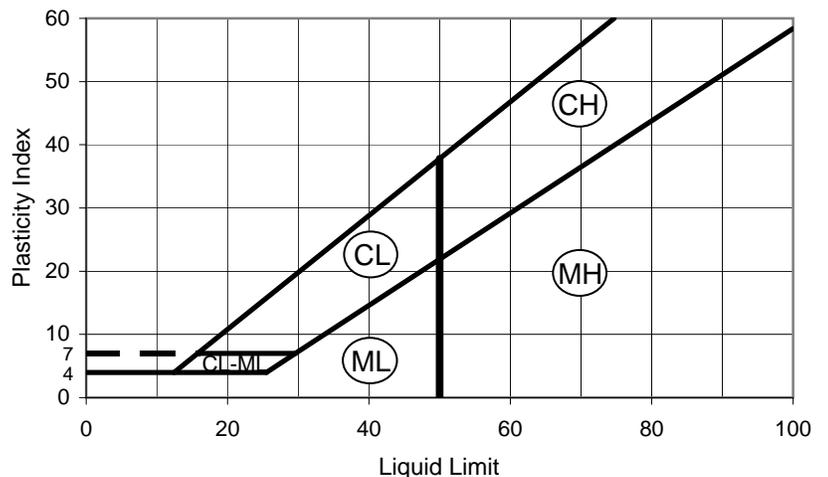
Dark Gray

COARSE-GRAINED SOILS < 50% fines passes #200 sieve	GRAVELS < 50% coarse fraction passes #4	< 5% fines (Passes #200 sieve)	GW	Well-graded Gravels, gravel-sand mixtures, little or no fines	Light Gray	
		5 to 12% fines use dual classification e.g. GP-GM	GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines	Olive Brown	
			GM	Silty Gravels, poorly graded gravel-sand-silt mixtures	Gray Brown	
		> 12% fines	GC	Clayey gravels, poorly graded gravel-sand-clay mixture	Gray Brown	
	SANDS > 50% coarse fraction passes #4	< 5% fines (Passes #200 sieve)	SW	Well-graded sands, gravelly sands, little or no fines	Olive	
		5 to 12% fines use dual classification e.g. SP-SM	SP	Poorly-graded sands, gravelly sands, little or no fines	Dark Brown	
			SM	Silty sands, poorly graded sand-gravel-silt mixtures	Dark Brown	
		> 12% fines	SC	Clayey sands, poorly graded sand-gravel-silt mixtures	Dark Brown	
	FINE-GRAINED SOILS > 50% fines passes #200 sieve	SILTS AND LEAN CLAYS Liquid Limit < 50		ML	Inorganic silts, silty fine sands and clayey silts, silty clays	Red Gray
				CL	Inorganic lean clay, gravelly clays, sandy clays and silty clays, low to medium plasticity	Red
OL				Organic silts and clays, low plasticity	Yellow Brown	
ELASTIC SILTS AND FAT CLAYS Liquid Limit > 50		MH	inorganic elastic silts diatomaceous or micaceous silt, medium to high plasticity	Yellow Brown		
		CH	Inorganic fat clay, high plasticity	Brown		
		OH	Organic silts and clays, medium to high plasticity	Yellow		
HIGHLY ORGANIC SOILS			PT	Peat, humus, swamp soils with high organic content	Yellow	

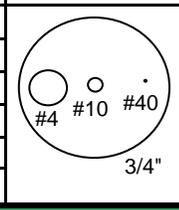
LABORATORY CLASSIFICATION CRITERIA¹

GW	C_u greater than 4; C_c between 1 and 3	
GP	Not meeting all gradation requirements for GW	
SW	C_u greater than 6; C_c between 1 and 3	
SP	Not meeting all gradation requirements for SW	
CH	Liquid Limit (LL) > 50 and Plasticity Index (PI) above A-line	
MH	Liquid Limit (LL) > 50 and Plasticity Index (PI) below A-line	
CL	LL < 50 and PI above 'A' line	Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols
ML	LL < 50 and PI below 'A' line	

PLASTICITY CHART



Red Yellow

GRAIN SHAPE							
	ANGULAR	SUBANGULAR	SUBROUNDED	ROUNDED	WELL ROUNDED		
RELATIVE DENSITY - COARSE GRAINED SOIL ³			GRAIN SIZE ¹				
Relative Density	Sampler		Description	Grain Size		N-value conversion $N_{60} = 0.15N_{crr}$	
	SPT (blow/ft)	CALIFORNIA (blow/ft)		(Inches)	Sieve		
Very Loose	<4	<5	Boulders	> 12	> 12 in		
Loose	4 to 10	5 to 12	Cobbles	3 to 12	3 in to 12 in		
Medium Dense	11 to 30	13 to 35	Gravel	Course	3/4 to 3		3 in to 3/4 in
				Fine	0.19 to 0.75		3/4 to #4
Dense	31 to 50	36 to 60	Sand	Course	0.079 to 0.19		#4 to #10
				Medium	0.017 to 0.079		#10 to #40
Very Dense	>50	>60	Fine	0.0029 to 0.017	#40 to #200		
Use N_1 values corrected for overburden.			Fines	< 0.0029	< #200		
CEMENTATION ²			ODOR ³		OTHER DESCRIPTORS		
Classification	Description		Description		Examples:		
Weak	Crumbles or breaks with handling or little finger pressure		None	no odor	1. Mineral content (e.g. slightly micaceous, carbonate stringers, caliche, etc.)		
Moderate	Crumbles or breaks with considerable finger pressure			Earthy		Musty or moldy smell	
Strong	Will not crumble or break with finger pressure		Chemical		methane, hydrocarbon, or other chemicals	2. Staining (e.g. Iron-oxide staining)	
MOISTURE CLASSIFICATION ²				Organic	Manure or decay	3. Debris (e.g. fragments of brick present)	
Classification	Description						
Dry	Dusty; dry to the touch						
Slightly Moist	Some moisture, but still has a dry appearance						
Moist	Damp, But no visible water		Organic	Manure or decay	4. Interbedded soil (e.g. 1/2-in inerbeds of silt)		
Very Moist	Enough moisture to wet the hands						
Wet	Saturated; visible free water						
CONSISTENCY CLASSIFICATION FOR FINE-GRAINED SOILS ⁴							
Classification	Description	SPT N_{60} value	Undrained Shear Strength, S_u				
			(kPa)	(lb/ft ²)			
Very Soft	Thumb penetrate easily; extrudes between fingers when squeezed	< 2	< 12	<250			
Soft	Thumb will penetrate soil about 25 mm; molds with light finger pressure	2-4	12-25	250-500			
Medium	Thumb will penetrate about 6 mm with moderate effort; molds with strong finger pressure	4-8	25-50	500-1000			
Stiff	Thumb indents easily, and will penetrate 12 mm with great effort	8-15	50-100	1000-2000			
Hard	Thumb will not indent soil, but thumbnail readily indents it	15-30	100-200	2000-4000			
Very Hard	Thumbnail will not indent soil or will indent it only with difficulty	> 30	>200	>4000			
LOG KEY SYMBOLS		UTILITY COLOR CODE		GRADATION PARAMETERS			
	Standard Penetration Slit Spoon Sampler (SPT)		RED Electric	$C_u = \frac{D_{60}}{D_{10}}$ $C_c = \frac{D_{30}^2}{D_{10}D_{60}}$			
	California Sampler		ORANGE Telephone - CATV				
	Shelby Tube		YELLOW Gas - Oil	REFERENCES 1 - ASTM Test Methods D 2487 and D2488 2 - Coduto (1999) <i>Geotechnical Engineering principles and Practices</i> 3 - Laney, M., <i>Terracon Field Logging Guide</i> 4 - Terzaghi & Peck (1967) <i>Soil Mechanics in Engineering Practice</i>			
	CPP Sampler		GREEN Sewer				
	Bulk/ Bag Sample		BLUE Water				
	Stabilized Ground water		PURPLE Reclaimed water lines				
	Groundwater At time of Drilling		PINK Survey	Call Before Digging USA Dig Alert 1-800-227-2600 CPP Facilities 909-869-3030			
			WHITE Proposed excavation			BACK	
			Ver 1.3 CVS & WAK 20080813				