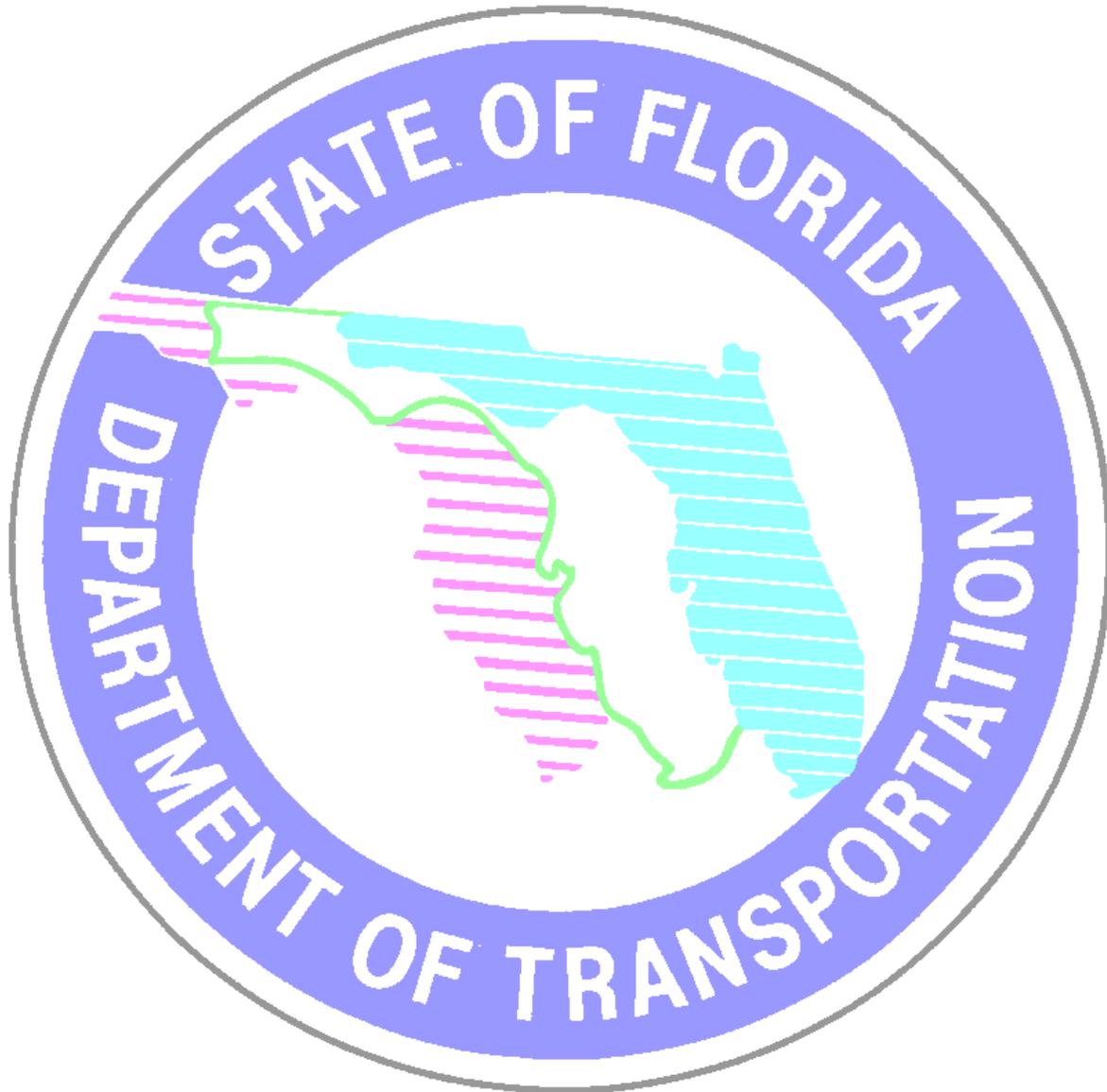


Soils and Foundation Handbook

2000



State Materials Office
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Chapter 1

1 Introduction

The purpose of this handbook is to provide Geotechnical Engineers with a guide to the proper procedures in the performance of geotechnical activities for the Florida Department of Transportation. Specifically, this handbook is intended to define the tasks involved in performing a subsurface investigation and the geotechnical aspects of the design and construction of roadways and roadway structures. General guidelines are presented covering the geotechnical phases of a typical project.

As each project presents unique considerations and requires engineering judgement based on a thorough knowledge of the individual situation, this handbook is not intended to serve as the geotechnical scope of services on individual projects. Instead, the scope of services for each project should be formulated using this handbook as a reference. The scope of services dictates the specific practices, which are to be used on a particular project. Additionally, the scope defines the required interaction between the Department's Geotechnical Engineer and those performing the geotechnical work.

The design and construction of a roadway and related structures is a complex operation involving the participation of many department units and outside agencies. The key to the successful completion of the project is communication. It is essential that good communication, coordination and interaction exist between the Geotechnical Engineer and these other units and agencies. This interaction should continue throughout all project phases to ensure a reliable and cost-effective design and minimize construction problems.

This handbook is designed to present information in the same sequence, as it would occur during project development. A general outline of the tasks, which should be performed by a Geotechnical Engineer during a project, is shown in Sections **1.1.1** through **1.1.4**. The details of these tasks are discussed and amplified in subsequent chapters.

Finally, it should be noted that this is intended neither as an all-encompassing and comprehensive procedural handbook, nor as a design handbook. Methods of subsurface investigation and of analyzing data and solving problems are not discussed in detail. The lists of references at the end of each chapter are but a few of the many sources of information that will provide the engineer with greater insight into investigation procedures and analysis and problem solving techniques. Further assistance is available from the District Geotechnical Engineer, the State Geotechnical Materials Engineer in Gainesville, and the State Geotechnical Engineer and State Construction Geotechnical Engineer in Tallahassee.

1.1 Geotechnical Tasks in Highway Projects

1.1.1 Planning, Development, and Engineering Phase

- Prepare geotechnical scope of services for consultant projects.
- Assist in corridor and route selection.
- Review existing information.
- Perform field reconnaissance of site and existing structures.
- Plan and supervise field investigation program, field and laboratory testing.
- Analyze all data available.
- Prepare preliminary geotechnical report summarizing available data and providing recommendations
- Identify potential construction requirements and problems (predrilling requirements, vibration and sound impacts).

1.1.2 Project Design Phase

- Perform additional field investigations and provide additional or revised recommendations if called for in geotechnical report or if project has substantially changed since earlier investigations.
- Assist structural engineer in interpreting and applying geotechnical recommendations to design and special provisions and/or supplemental specifications.
- Design and if applicable perform load test programs or special instrumentation monitoring as deemed necessary.
- Review plans, special provisions and/or supplemental specifications.
- Identify construction activities and techniques to minimize potential construction requirements and problems (predrilling requirements, vibration and sound impacts).

1.1.3 Construction Phase

- Establish construction criteria for geotechnical portions of project.
- Inspect construction procedures to assure compliance with design and specifications.
- Assist in design, installation, performance, monitoring, and evaluation of load test programs and/or instrumentation systems.
- Assist in solution of unforeseen foundation and/or roadway soils problems.

1.1.4 Post-Construction Phase

- Assist in assessment of and provide solutions to roadway and structure maintenance problems, which are related to the geotechnical characteristics of the site.
- Summarize construction procedures and/or problems and any changes in design made during construction.
- Provide information to State Geotechnical files for reference during the design of future projects.

Chapter 2

2 Subsurface Investigation Procedures

Because of the varying complexity of projects and soil conditions, it is impossible to establish a rigid format to be followed in conducting subsurface investigations; however, there are basic steps that should be considered for any project. By outlining and describing these steps, it will be possible to standardize procedures and considerably reduce time and expense often required to go back and obtain information not supplied by the initial investigation.

The basic steps are summarized in this and subsequent chapters. In this chapter, review of existing data is discussed, as well as commonly used methods for performing field explorations. Guidelines for minimum investigations for various types of projects are presented in **Chapter 3**; field and laboratory test methods are discussed in **Chapters 4 & 5**, respectively. Refer also to ASTM D 420 and D 5434.

2.1 Review of Project Requirements

The first step in performing a subsurface investigation is a thorough review of the project requirements. It is necessary that the information available to the Geotechnical Engineer include the project location, alignment, structure locations, structure loads, approximate bridge span lengths and pier locations, and cut and fill area locations. The Geotechnical Engineer should have access to typical section, plan and profile sheets, and cross sections with a template for the proposed roadway showing cuts and fills. This information aids the Geotechnical Engineer in planning the investigation and minimizes expensive and time-consuming backtracking.

2.2 Office Review of Available Data

After gaining a thorough understanding of the project requirements, the Geotechnical Engineer should collect all relevant available information on the project site. Review of this information can aid the engineer in understanding the geology, geography and topography of the area and assist him in laying out the field explorations and locating potential problems. Contact the District Geotechnical Engineer for assistance in obtaining sources of this available data. Existing data may be available from the following sources:

2.2.1 Topographic Maps

These maps are prepared by the U.S. Geological Survey (USGS) and the U.S. Coast and Geodetic Survey (USCGS) and are readily available. They are sometimes also prepared on a larger scale by the Department during early planning phases of a project. These maps portray physical features, configuration and elevation of the ground surface, and surface water features. This data is valuable in determining accessibility for field equipment and possible problem areas.

2.2.2 Aerial Photographs

These photographs are available from the Department and other sources. They are valuable in that they can provide the basis for reconnaissance and, depending on the age of the photographs, show manmade structures, excavations, or fills that affect accessibility and the planned depth of exploration. Historical photographs can also help determine the reasons and/or potential of general scour and sinkhole activity.

2.2.3 Geological Maps and Reports

Considerable information on the geological conditions of an area can often be obtained from geological maps and reports. These reports and maps often show the location and relative position of the different geological strata and present information on the characteristics of the different strata. This data can be used directly to evaluate the rock conditions to be expected and indirectly to estimate possible soil conditions since the parent material is one of the factors controlling soil types. Geological maps and reports can be obtained from the USGS, Florida Geological Survey, university libraries, and other sources.

2.2.4 Soils Conservation Service Surveys

These surveys are compiled by the U.S. Department of Agriculture usually in the form of county soils maps. These surveys can provide valuable data on surface soils including mineralogical composition, grain size distribution, depth to rock, water table information, drainage characteristics, geologic origin, and presence of organic deposits.

2.2.5 Potentiometric Surface Map

The potentiometric surface elevation shown on the map (see **Figure 1**) can supplement and be correlated with what was found in the field by the drillers. The Potentiometric Surface map can be obtained from the local Water Management District office.

2.2.6 Adjacent Projects

Data may be available on nearby projects from the Department, or county or city governments. The Department may have soils data on file from state projects and as-built drawings and pile driving records for the final structure. This data is extremely useful in setting preliminary boring locations and depths and in predicting problem areas. Maintenance records for existing nearby roadways and structures may provide additional insight into the subsurface conditions. For example, indications of differential settlement or slope stability problems may provide the engineer with valuable information on the long-term characteristics of the site.

2.3 Field Reconnaissance

Following review of the existing data, the Geotechnical Engineer should visit the project site. This will enable the engineer to gain first-hand knowledge of field conditions and correlate this information with previous data. The form included as **Figure 2** indicates the type of information the engineer should look for. In particular, the following should be noted during the field reconnaissance:

1. Nearby structures should be inspected to ascertain their foundation performance and potential to damage from vibration or settlement from foundation installation. Also, the structure-s usages must be looked at to check the impact the foundation installation may have (i.e. a surgical unit, printing company, etc.).
2. On water crossings, banks should be inspected for scour and the streambed inspected for evidence of soil deposits not previously indicated.
3. Note any feature that may affect the boring program, such as accessibility, structures, overhead utilities, signs of buried utilities, or property restrictions.
4. Note any feature that may assist in the engineering analysis, such as the angle of any existing slopes and the stability of any open excavations or trenches.
5. Any drainage features, including signs of seasonal water tables.
6. Any features that may need additional borings or probing such as muck pockets.

2.4 Field Exploration Methods

Assuming access and utility clearances have been obtained and a survey base line has been established in the field, field explorations are begun based on the information gained during the previous steps. Many methods of field exploration exist; some of the more common are described below. These methods are often augmented by in-situ testing (see **Chapter 4**).

2.4.1 Test Pits and Trenches

These are the simplest methods of inspecting subsurface soils. They consist of excavations performed by hand, backhoe, or dozer. Hand excavations are often performed with posthole diggers or hand augers. They offer the advantages of speed and ready access for sampling. They are severely hampered by limitations of depth and by the fact they cannot be used in soft or loose soils or below the water table. In Florida their use is generally limited to borrow pits.

2.4.2 Boreholes

Borings are probably the most common method of exploration. They can be advanced using a number of methods, as described below. Upon completion, all borings should be backfilled in accordance with applicable Department of

Environmental Protection and Water Management District regulations. In many cases this will require grouting.

2.4.2.1 Auger Borings

Rotating an auger while simultaneously advancing it into the ground either hydraulically or mechanically advances auger borings. The auger is advanced to the desired depth and then withdrawn. Samples of cuttings can be removed from the auger; however, the depth of the sample can only be approximated. These samples are disturbed and should be used only for material identification. This method is used to establish soil strata and water table elevations, or to advance to the desired stratum before Standard Penetration Testing (SPT) or undisturbed sampling is performed. However, it cannot be used effectively in soft or loose soils below the water table without casing or drilling mud to hold the hole open. See ASTM D 1452 (AASHTO T 203).

2.4.2.2 Hollow-Stem Auger Borings

A hollow-stem auger consists of a continuous flight auger surrounding a hollow drill stem. The hollow-stem auger is advanced similar to other augers; however, removal of the hollow stem auger is not necessary for sampling. SPT and undisturbed samples are obtained through the hollow drill stem, which acts like a casing to hold the hole open. This increases usage of hollow-stem augers in soft and loose soils. See ASTM D 6151 (AASHTO T 251).

2.4.2.3 Wash Borings

In this method, the boring is advanced by a combination of the chopping action of a light bit and the jetting action of water flowing through the bit. This method of advancing the borehole is used only when precise soil information is not required between sample intervals.

2.4.2.4 Percussion Drilling

In this method, the drill bit advances by power chopping with a limited amount of water in the borehole. Slurry must be periodically removed. The method is not recommended for general exploration because of the difficulty in determining stratum changes and in obtaining undisturbed samples. However, it is useful in penetrating materials not easily penetrated by other methods, such as those containing boulders.

2.4.2.5 Rotary Drilling

A downward pressure applied during rapid rotation advances hollow drill rods with a cutting bit attached to the bottom. The drill bit cuts the material and drilling fluid washes the cuttings from the borehole. This is, in most cases, the fastest method of advancing the borehole and can be used in

any type of soil except those containing considerable amounts of large gravel or boulders. Drilling mud or casing can be used to keep the borehole open in soft or loose soils, although the former makes identifying strata change by examining the cuttings difficult.

2.4.2.6 Coring

A core barrel is advanced through rock by the application of downward pressure during rotation. Circulating water removes ground-up material from the hole while also cooling the bit. The rate of advance is controlled so as to obtain the maximum possible core recovery. The minimum core barrel to be used shall be HW (2.4 inch {61 mm} diameter), but it is preferable to use a 4-inch (101.6 mm) diameter core barrel. Refer to ASTM D 2113 (AASHTO T 225).

2.4.3 Soundings

A sounding is a method of exploration in which either static or dynamic force is used to cause a rod tipped with a testing device to penetrate soils. Samples are not usually obtained. The depth to rock can easily be deduced from the resistance to penetration. The resistance to penetration can be measured and correlated to various soil properties. See **Chapter 4** for details of the cone penetrometer.

2.4.4 Geophysical Methods

These are nondestructive exploratory methods in which no samples can be taken. Geophysical methods can provide information on the general subsurface profile, the depth to bedrock, depth to groundwater, and the location of granular borrow areas, peat deposits, or subsurface anomalies. Results can be significantly affected by many factors however, including the presence of groundwater, non-homogeneity of soil stratum thickness, and the range of wave velocities within a particular stratum. For this reason, geophysical explorations should always be accompanied by conventional borings and an experienced professional must interpret results. (See ASTM **PS 78** soon to be released as **D 6429** and US Army Corps of Engineers Engineering Manual **EM-1110-1-1802**) Geophysical methods commonly used for engineering purposes include:

2.4.4.1 Seismic Refraction and Reflection

These methods rely on the fact that shock waves travel through different materials at different velocities. The times required for an induced shock wave to travel to set detectors after being refracted or reflected by the various subsurface materials are measured. This data is then used to interpret material types and thickness. Seismic refraction is limited to material stratifications in which velocities increase with depth. For the seismic refraction method, refer to ASTM D 5777. Seismic investigations can be performed from the surface or from various depths within borings. For cross-hole seismic techniques, see ASTM D 4428.

2.4.4.2 Resistivity

This method is based on the differences in electrical conductivity between subsurface strata. An electric current is passed through the ground between electrodes and the resistivity of the subsurface materials is measured and correlated to material types. Several electrode arrangements have been developed, with the Wenner (4 equally spaced electrodes) being the most commonly used in the United States. Refer to ASTM G 57.

2.4.4.3 Ground Penetrating Radar (GPR)

The velocity of electromagnetic radiation is dependent upon the material through which it is traveling. GPR uses this principle to analyze the reflections of radar signals transmitted into the ground by a low frequency antenna. Signals are continuously transmitted and received as the antenna is towed across the area of interest, thus providing a profile of the subsurface material interfaces.

2.4.5 Soil Sampling

Common methods of sampling during field explorations include those listed below. All samples should be properly preserved and carefully transported to the laboratory such that sample integrity is maintained. See ASTM D 4220.

2.4.5.1 Bag Bulk Samples

These are disturbed samples obtained from auger cuttings or test pits. The quantity of the sample depends on the type of testing to be performed, but can range up to 50 lb (25 kg) or more. Testing performed on these samples includes classification, moisture-density, Limerock Bearing Ratio (LBR), and corrosivity tests. A portion of each sample should be placed in a sealed container for moisture content determination.

2.4.5.2 Split-Barrel

Also known as a split-spoon sample, this method is used in conjunction with the Standard Penetration Test (see **Chapter 4**). The sampler is a 2-inch (50.8 mm) (O.D.) split barrel which is driven into the soil with a 140-pound (63.5 kg) hammer dropped 30 inches (760 mm). After it has been driven 18 inches (450 mm), it is withdrawn and the sample removed. The sample should be immediately examined, logged and placed in sample jar for storage. These are disturbed samples and are not suitable for strength or consolidation testing. They are adequate for moisture content, gradation, and Atterberg Limits tests, and valuable for visual identification. See ASTM D 1586.

2.4.5.3 Shelby Tube

This is thin-walled steel tube, usually 3 inches (76.2 mm) (O.D.) by 30 inches (910 mm) in length. It is pushed into the soil with a relatively rapid, smooth stroke and then retracted. This produces a relatively undisturbed

sample provided the Shelby tube ends are sealed immediately upon withdrawal. Refer to ASTM D 1587 (AASHTO T 207).

This sample is suitable for strength and consolidation tests. This sampling method is unsuitable for hard materials. Good samples must have sufficient cohesion to remain in the tube during withdrawal. Refer to ASTM D 1587 (AASHTO T 207).

2.4.5.4 Piston Samplers

2.4.5.4.1 Stationary

This sampler has the same standard dimensions as the Shelby Tube, above. A piston is positioned at the bottom of the thin-wall tube while the sampler is lowered to the bottom of the hole, thus preventing disturbed materials from entering the tube. The piston is locked in place on top of the soil to be sampled. A sample is obtained by pressing the tube into the soil with a continuous, steady thrust. The stationary piston is held fixed on top of the soil while the sampling tube is advanced. This creates suction while the sampling tube is retrieved thus aiding in retention of the sample. This sampler is suitable for soft to firm clays and silts. Samples are generally less disturbed and have a better recovery ratio than those from the Shelby Tube method.

2.4.5.4.2 Floating

This sampler is similar to the stationary method above, except that the piston is not fixed in position but is free to ride on the top of the sample. The soils being sampled must have adequate strength to cause the piston to remain at a fixed depth as the sampling tube is pushed downward. If the soil is too weak, the piston will tend to move downward with the tube and a sample will not be obtained. This method should therefore be limited to stiff or hard cohesive materials.

2.4.5.4.3 Retractable

This sampler is similar to the stationary sampler, however, after lowering the sampler into position the piston is retracted and locked in place at the top of the sampling tube. A sample is then obtained by pushing the entire assembly downward. This sampler is used for loose or soft soils.

2.4.5.4.4 Hydraulic (Osterberg)

In this sampler, a movable piston is attached to the top of a thin-wall tube. Sampling is accomplished as hydraulic pressure pushes the movable piston downward until it contacts a stationary piston positioned at the top of the soil sample. The distance over which the sampler is pushed is fixed; it cannot be over-pushed. This sampler is used for very soft to firm cohesive soils.

2.4.5.5 Rock Core Sampling

Rock cores are obtained using core barrels equipped with diamond or tungsten-carbide tipped bits. There are three basic types of core barrels: Single tube, double tube, and triple tube. Single tube core barrels generally provide poor recovery rates in Florida limestone and their use is not allowed. Double tube and triple tube are required and are described below. (Note: face discharge bits generally provide better return in Florida limestone). See also ASTM D 2113 (AASHTO T 225). Refer to ASTM D 5079 for practices of preserving and transporting rock core samples.

2.4.5.5.1 Double Tube Core Barrel

This core barrel consists of inner and outer tubes equipped with a diamond or tungsten-carbide drill bit. As coring progresses, fluid is introduced downward between the inner and outer tubes to cool the bit and to wash ground-up material to the surface. The inner tube protects the core from the highly erosive action of the drilling fluid. In a rigid type core barrel, both the inner and outer tubes rotate. In a swivel type, the inner tube remains stationary while the outer tube rotates. Several series of swivel type core barrels are available. Barrel sizes vary from EWG or EWM (0.845 inch (21.5 mm) to 6 inch (152.4 mm) I.D.). The larger diameter barrels are used in highly erodible materials, such as Florida limestone, to generally obtain better core recovery. The minimum core barrel to be used shall be HW (2.4 inch (61 mm) I.D.), and it is recommended using 4 inch (101.6 mm) diameter core barrels to better evaluate the Florida limestone properties.

2.4.5.5.2 Triple Tube Core Barrel

Similar to the double tube, above, but has an additional inner liner, consisting of either a clear plastic solid tube or a thin metal split tube, in which the core is retained. This barrel best preserves fractured and poor quality rock cores.

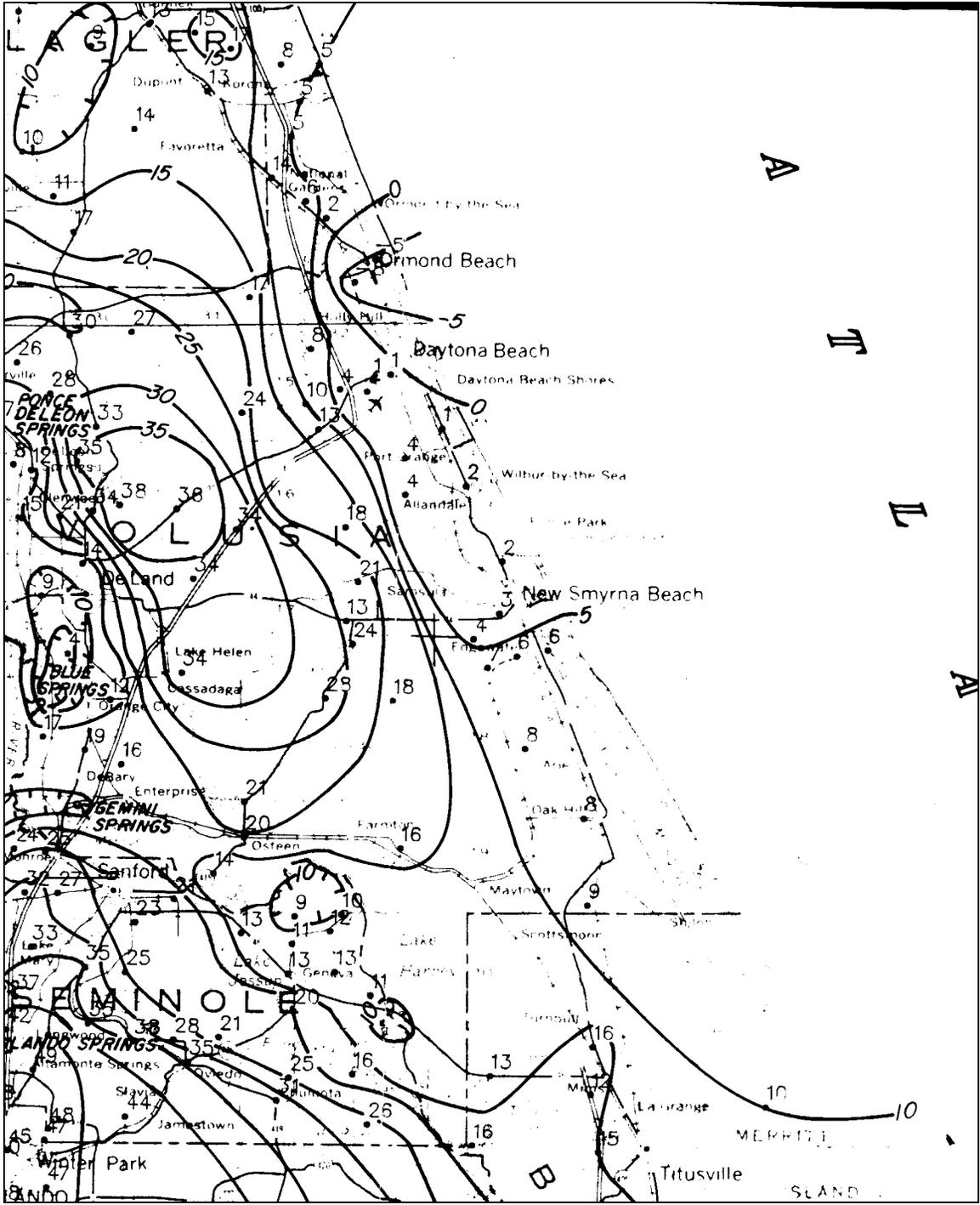


Figure 1, Excerpt from the Potentiometric Surface of the St. Johns River Water Management District and Vicinity, Florida, September 1993 map

STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION
FIELD RECONNAISSANCE REPORT

FORM 675-020-14
 MATERIALS
 08/94

PROJECT NO.: _____ COUNTY _____ STA NO. _____

REPORTED BY: _____ DATE _____

<p>1. STAKING OF LINE <input type="checkbox"/> WELL STAKED <input type="checkbox"/> POORLY STAKED (WE CAN WORK) <input type="checkbox"/> POORLY STAKED (WE MUST REPLACE) <input type="checkbox"/> REQUEST DIVISION TO RESTAKE</p>	<p>8. BRIDGE SITE - CONTINUED CUT SECTION - METERS _____ FILL SECTION - METERS _____ IF STREAM CROSSING: WILL BARGE BE NECESSARY: _____ EASILY PLACED IN WATER _____ CAN CABLE BE STRETCHED ACROSS STREAM <input type="checkbox"/> YES <input type="checkbox"/> NO HOW LONG? _____ CURRENT: <input type="checkbox"/> SWIFT <input type="checkbox"/> MODERATE <input type="checkbox"/> SLOW IF PRESENT BRIDGE NEARBY: TYPE OF FOUNDATION _____ ANY PROBLEMS EVIDENT IN OLD BRIDGE (DESCRIBE ON BACK) IS WATER NEARBY FOR WET DRILLING - METERS _____</p>
<p>2. BENCH MARKS IN PLACE: <input type="checkbox"/> YES <input type="checkbox"/> NO DISTANCE FROM BRIDGE - METERS _____</p>	<p>9. GROUND WATER TABLE CLOSE TO SURFACE - METERS _____ NEARBY WELLS - DEPTH - METERS _____ INTERMEDIATE DEPTH - METERS _____</p>
<p>3. PROPERTY OWNERS GRANTED PERMISSION: <input type="checkbox"/> YES <input type="checkbox"/> NO REMARKS ON BACK</p>	<p>10. ROCK BOULDERS OVER AREA? <input type="checkbox"/> YES <input type="checkbox"/> NO DEFINITE OUTCROP? <input type="checkbox"/> YES <input type="checkbox"/> NO (SHOW SKETCH ON BACK) WHAT KIND? _____</p>
<p>4. UTILITIES WILL DRILLERS ENCOUNTER UNDERGROUND UTILITIES? <input type="checkbox"/> YES <input type="checkbox"/> NO AT WHICH HOLES? _____ WHAT TYPE? _____ WHO TO SEE FOR DEFINITE LOCATION _____</p>	<p>11. SPECIAL EQUIPMENT NECESSARY</p>
<p>5. GEOLOGIC FORMATION</p>	<p>12. REMARKS ON ACCESS DESCRIBE ANY PROBLEMS ON ACCESS</p>
<p>6. SURFACE SOILS <input type="checkbox"/> SAND <input type="checkbox"/> CLAY <input type="checkbox"/> SANDY CLAY <input type="checkbox"/> SILT <input type="checkbox"/> MUCK <input type="checkbox"/> OTHER</p>	<p>13. DEBRIS AND SANITARY DUMPS STATIONS _____ REMARKS _____</p>
<p>7. GENERAL SITE DESCRIPTION <input type="checkbox"/> LEVEL <input type="checkbox"/> ROLLING <input type="checkbox"/> HILLSIDE <input type="checkbox"/> VALLEY <input type="checkbox"/> SWAMP <input type="checkbox"/> GULLIED GROUND COVER <input type="checkbox"/> CLEARER <input type="checkbox"/> FARMED <input type="checkbox"/> BUILDINGS <input type="checkbox"/> HEAVY WOODS <input type="checkbox"/> LIGHT WOODS <input type="checkbox"/> OTHER REMARKS ON BACK</p>	<p>8. BRIDGE SITE REPLACING _____ WIDENING _____ RELOCATION _____ RIG TYPE _____ <input type="checkbox"/> TRUCK MOUNTED SKID RIG <input type="checkbox"/> SKID RIG <input type="checkbox"/> ROCK CORING RIG <input type="checkbox"/> WASH BORING EQUIPMENT <input type="checkbox"/> WATER WAGON <input type="checkbox"/> PUMP <input type="checkbox"/> HOSE - METERS _____</p>

Figure 2, Field Reconnaissance Report

2.5 References

1. Cheney, Richard S. & Chassie, Ronald G., Soils and Foundations Workshop Manual – Second Edition, FHWA HI-88-009, 1993.
2. NAVFAC DM-7.1 - Soil Mechanics, Department of the Navy, Naval Facilities Engineering Command, 1986.
3. Hannigan, P.J., Goble, G.G., Thendean, G., Likins, G.E., and Rausche, F., Manual on Design and Construction of Driven Pile Foundations, FHWA-HI-97-013 and 014, 1996.
4. Fang, Hsai-Yang, Foundation Engineering Handbook Second Edition, Van Nostrand Reinhold Company, New York, 1990.
5. AASHTO, Manual on Subsurface Investigations, Washington DC, 1988.
6. George Munfakh, Ara Arman, Naresh Samtani and Raymond Castelli, Subsurface Investigations, FHWA-HI-97-021, 1997 (Pending)
7. Recommended Guidelines for Sealing Geotechnical Exploratory Holes, National Cooperative Highway Research Program, NCHRP Report 378
8. Engineering Manual 1110-1-1802, Geophysical Exploration for Engineering and Environmental Investigations, Department of Army, U.S. Army Corps of Engineers, 1995

2.6 Specifications and Standards

<u>Subject</u>	<u>ASTM</u>	<u>AASHTO</u>	<u>FM</u>
Guide to Site Characterization for Engineering, Design, and Construction Purposes	D 420	T 86	-
Standard Practice for Soil Investigation and Sampling by Auger Borings	D 1452	T 203	-
Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils	D 1586	T 206	-
Standard Practice for Thin-Walled Tube Geotechnical Sampling of Soils	D 1587	T 207	1-T 207
Standard Practice for Diamond Core Drilling for Site Investigation	D 2113	T 225	-
Standard Practices for Preserving and Transporting Soil Samples	D 4220	-	-
Standard Test Methods for Crosshole Seismic Testing	D 4428	-	-
Standard Test Method for Determining Subsurface Liquid Levels in a Borehole or Monitoring Well (Observation Well)	D 4750	-	-

<u>Subject</u>	<u>ASTM</u>	<u>AASHTO</u>	<u>FM</u>
Standard Practices for Preserving and Transporting Rock Core Samples	D 5079	-	-
Standard Guide for Field Logging of Subsurface Explorations of Soil and Rock	D 5434	-	-
Standard Guide for Using the Seismic Refraction Method for Subsurface Investigation	D 5777	-	-
Standard Practice for Using Hollow-Stem Augers for Geotechnical Exploration and Soil Sampling	D 6151	T 251	-
Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method	G 57	T 288	-
Provisional Guide for Selecting Surface Geophysical Methods	PS 78	-	-

Chapter 3

3 Subsurface Investigation Guidelines for Highways and Related Structures

A subsurface investigation should be performed at the site of all new structure, roadway construction, widenings, extensions, and rehabilitation locations as directed by the District Geotechnical Engineer or project scope.

This chapter presents guidelines to plan a subsurface investigation program. As the requirements will vary with the project conditions, engineering judgment is essential in tailoring the investigation to the specific project.

The amounts and types of data obtained during a subsurface investigation are often constrained by limitations of time, manpower, equipment, access, or funds. However, as a minimum, the investigation should provide sufficient data for the Geotechnical Engineer to recommend the most efficient design. Without sufficient data, the engineer must rely on conservative designs, with high factors of safety, the use of which may cost considerably more than an extended exploration program.

A comprehensive subsurface investigation program might include both conventional borings and other specialized field investigatory or testing methods. While existing data can provide some preliminary indication of the necessary extent of exploration, more often it will be impossible to finalize the investigation plan until some field data is available. Therefore, close communication between the engineer and driller is essential. The results of preliminary borings should be reviewed as soon as possible so that additional borings and in-situ testing, if necessary, can be performed without remobilization and with a minimum loss of time.

3.1 General Requirements

The extent of the exploration will vary considerably with the nature of the project. However, the following general standards apply to all investigation programs or as appropriate for the specific project and agreed upon by the District Geotechnical Engineer:

1. Preliminary exploration depths should be estimated from data obtained during field reconnaissance, existing data, and local experience. The borings should penetrate unsuitable founding materials (organic soils, soft clays, loose sands, etc.) and terminate in competent material. Competent materials are those suitable for support of the foundations being considered.
2. All borings shall be extended below the estimated scour depths.
3. Each boring, sounding, and test pit should be given a unique identification number for easy reference.
4. The ground surface elevation and actual location should be noted for each boring, sounding, and test pit. Offshore borings should be referenced to mean sea level with the aid of a tide gauge. (Note: There are two vertical

datum. They are the 1927 datum and the 1988 datum; ensure that the proper one is being referenced.)

5. A sufficient number of samples, suitable for the types of testing intended, should be obtained within each layer of material.
6. Water table observation within each boring or test pit should be recorded when first encountered, at the end of each day and after sufficient time has elapsed for the water table to stabilize. Refer to ASTM D 4750. Other groundwater observations (artesian pressure, etc.) should also be recorded.
7. Unless serving as an observation well, each borehole, sounding, and test pit should be backfilled or grouted according to applicable environmental guidelines. Refer to Reference 6.

3.2 Guidelines for Minimum Explorations

Following is a description of the recommended minimum explorations for various types of projects. It is stressed that these guidelines represent the minimum extent of exploration and testing anticipated for most projects and must be adapted to the specific requirements of each individual project. The District Geotechnical Engineer should be consulted for assistance in determining the requirements of a specific project. Additionally, the Engineer should verify that the Federal Highway Administration (FHWA) minimum criteria are met. Refer to Reference 3.

It is noted that the guidelines below consider the use of conventional borings only. While this is the most common type of exploration, the Engineer may deem it appropriate on individual projects to include soundings, test pits, geophysical methods, or in-situ testing as supplementary explorations or as substitutes for some, but not all, of the conventional borings noted in the following sections.

3.2.1 Roadway Soil Surveys

Soil survey explorations are made along the proposed roadway alignment for the purpose of defining subsurface materials. This information is used in the design of the pavement section, as well as in defining the limits of unsuitable materials and any remedial measures to be taken. Soil survey information is also used in predicting the probable stability of cut or fill slopes.

Minimum criteria for soil surveys vary substantially, depending on the location of the proposed roadway, the anticipated subsurface materials, and the type of roadway. The following are basic guidelines covering general conditions. It is important that the engineer visit the site to ensure that all features are covered. In general, if a structure boring is located in close proximity to a planned soil survey boring, the soil survey boring may be omitted.

- a. At least one boring shall be placed at each 100-foot (30 m) interval. Generally, borings are to be staggered left and right of the centerline to cover the entire roadway corridor. Borings may be spaced further apart if pre-existing information indicates the presence of uniform subsurface conditions. Additional borings shall be located as necessary to define the

limits of any undesirable materials or to better define soils stratification.

- b. In areas of highly variable soil conditions, additional borings shall be located at each interval considering the following criteria.
 - 1) For interstate highways, three borings are to be placed at each interval, one within the median and one within each roadway.
 - 2) For four lane roadways, two borings are to be placed at each interval, one within each roadway.
- c. For roadway widenings that provide an additional lane, one boring shall be placed within the additional lane at each interval.
- d. In areas of cut or fill, where stability analysis is anticipated, a minimum of two additional borings shall be placed at each interval near the outer reaches of the sloped areas.
- e. In all cases, at least three samples per mile (two samples per kilometer) or 3 per project whichever is greater shall be obtained for each stratum encountered. Each of the samples representing a particular stratum shall be obtained from a different location, with sampling locations spread out over each mile (kilometer). Samples should be of adequate size to permit classification and moisture content testing.
- f. Additional samples shall be obtained to permit LBR and corrosion testing. As a minimum, three LBR samples per mile (two samples per kilometer) or 3 per project whichever is greater per stratum of all materials, which can be used in accordance with Standard Indexes 500, or 505 shall be obtained and tested. LBR samples shall also be obtained of all strata located in excavation areas (i.e., water retention areas, ditches, cuts, etc.), which can be used in accordance with Standard Indexes 500 or 505. Corrosion series samples shall be obtained (unless no structures are to be installed) on a frequency of at least one sample per stratum per 1,500 feet (450 m) of alignment. When a rigid pavement is being considered for design, obtain sufficient samples to perform laboratory permeability tests based upon the requirements given in the Rigid Pavement Design Manual.
- g. Borings in areas of little or no grade change shall extend a minimum of 5 feet (1.5 m) below grade, drainage pipe or culvert invert level whichever is deeper. Every 500 feet (150 m), one boring shall be extended to a nominal depth of 20 feet (6 m) below grade. The 20 feet (6 m) borings apply to projects with proposed buried storm sewer systems; project specifics may dictate adjustments. Borings may or may not include Standard Penetration Tests (SPT), depending on the specific project and its location.
- h. In areas of cut, borings shall extend a minimum of 10 feet (3 m) below the proposed grade. If poor soil conditions are encountered at this depth, borings shall be extended to firm materials or to a depth below grade equal to the depth of cut, whichever occurs first. Bag, SPT, undisturbed and core samples shall be obtained as appropriate for analyses.

- i. In areas of fill, borings shall extend to firm material or to a depth of twice the embankment height. Bag, SPT, and undisturbed samples shall be obtained as appropriate.
- j. Areas of muck must be probed to delineate both the vertical and the horizontal extents.

3.2.2 Structures

The purpose of structure borings is to provide sufficient information about the subsurface materials to permit design of the structure foundations and related geotechnical construction. The following general criteria should satisfy this purpose on most projects; however, it is the engineer's responsibility to assure that appropriate explorations are carried out for each specific project.

All structure borings shall include Standard Penetration Testing (SPT) at regular intervals unless other sampling methods and/or in-situ testing (as defined in **Chapter 4**) are being performed.

3.2.2.1 Bridges

- 1) Perform at least one 2.5-inch (63.5 mm) minimum diameter borehole at each pier or abutment location. The hole pattern should be staggered so that borings occur at the opposite ends of adjacent piers. Pier foundations or abutments over 100 feet (30 m) in plan length may require at least two borings, preferably at the extremities of the proposed substructure. For structure widenings, the total number of borings may be reduced depending on the information available for the existing structure.
- 2) If pier locations are unknown, their probable approximate locations may be deduced based on experience and a preliminary design concept for the structure. If this is not possible, place borings at no more than 100-foot (30 m) intervals along the alignment. Additionally, for projects which include a water crossing that includes a pier in the water, at least one boring should be located in the water when practical depending on the width of the crossing.
- 3) Borings shall be continued until all unsuitable foundation materials have been penetrated and the predicted stress from the foundation loading is less than 10% of the original overburden pressure (see **Figure 3** and **Figure 4**), or until a minimum of 10 feet (3 m) of competent rock has been penetrated. If no data is available for predicting the foundation stress, extend the boring until at least 20 feet (6 m) of bedrock or other competent bearing material (N-values of 50 or greater) is encountered. (Scour and lateral requirements must be taken into account.)
- 4) When using the Standard Penetration Test, split-spoon samples shall be obtained at a maximum interval of 2.5 to 3.0 feet (one meter) and at

the top of each stratum. Continuous SPT sampling in accordance with ASTM D 1586 is recommended in the top 15 to 20 feet (5 to 6 m) unless the material is obviously unacceptable as a founding material.

- 5) When cohesive soils are encountered, undisturbed samples shall be obtained at 5-foot (1.5 m) intervals in at least one boring. Undisturbed samples shall be obtained from more than one boring where possible.
- 6) When rock is encountered, successive core runs shall be made with the objective of obtaining the best possible core recovery. **SPT's shall be performed between core runs, typically at 5-foot (1.5 m) intervals.**
- 7) In-situ vane, pressuremeter, or dilatometer tests (See **Chapter 4**) are recommended where soft clays are encountered.
- 8) Corrosion tests (see **Chapter 4**) are required on all new bridge projects. As a minimum one on the soil and one on the water shall be done.
- 9) In the case of a water crossing, samples of streambed materials and each underlying stratum shall be obtained for determination of the median particle diameter, D_{50} , needed for scour analysis.
- 10) For projects with large ship impacts the pressuremeter test is recommended to be performed within seven (7) foundation element diameters below the deepest scour elevation at the pier location.

3.2.2.2 Approach Embankments

- 1) At least one boring shall be taken at the point of highest fill; usually the borings taken for the bridge abutment will satisfy this purpose.

If settlement or stability problems are anticipated, as may occur due to the height of the proposed embankment and/or the presence of poor foundation soils, additional borings shall be taken along the alignment. The first of these borings shall be no more than 15 feet (5 m) from the abutment. The remaining borings shall be placed at 100-foot (30 m) intervals until the height of the fill is less than 5 feet (1.5 m). Borings shall be taken at the toes of the proposed embankment slopes as well as the embankment centerline.
- 2) Borings shall be continued until the superimposed stress is less than 10% of the original overburden pressure (see **Figure 5**) and unsuitable founding materials have been penetrated.
- 3) Sampling and in-situ testing criteria are the same as for bridges, above.

3.2.2.3 Retaining Walls

- 1) At retaining wall locations borings shall be taken at a maximum interval of one per 150 feet (50 m) of the wall, as close to the wall alignment as possible. Borings shall be extended below the bottom of the wall a minimum of twice the wall height or at least 10 feet (3 m)

into competent material. This applies to all walls, proprietary systems as well as precast and cast-in-place.

- 2) Sampling and in-situ testing criteria are the same as for bridges.

3.2.2.4 Buildings

In general, one boring should be taken at each corner and one in the center. This may be reduced for small buildings. For extremely large buildings or highly variable site conditions, one boring should be taken at each support location. Other criteria are the same as for bridges.

3.2.2.5 Drainage Structures

- 1) Borings shall be taken at proposed locations of box culverts. Trenches or hand auger borings may suffice for smaller structures.
- 2) For box culverts, borings shall extend a minimum of 15 feet (5 m) below the bottom of the culvert or until firm material is encountered, whichever is deeper.
- 3) For smaller structures, borings or trenches shall extend at least 5 feet (1.5 m) below the bottom of the structure or until firm material is encountered, whichever is deeper.
- 4) Corrosion testing must be performed for each site. Material from each stratum above the invert elevation and any standing water shall be tested. For drainage systems parallel to roadway alignments, tests shall be performed at 1,500-foot (500 m) intervals along the alignment.

3.2.2.6 High Mast Lighting, Strain Poles and Sign Structures

- 1) One boring shall be taken at each designated location.
- 2) Borings shall be 50 feet (15 m) into suitable soil or 5 feet (1.5 m) into competent rock. Deeper borings may be required for cases with higher torsional loads.
- 3) Other criteria are the same as for bridges.

3.2.2.7 Mast Arms Assemblies

- 1) One boring (Auger, SPT or CPT) shall be taken in the area of each designated location (for uniform sites a boring can cover several foundation locations).
- 2) For Standard Mast Arm Assemblies verify that the soil strength properties meets or exceeds the soil strength properties shown on the Mast Arm Assemblies Table in the Plans Preparation Manual. A site-specific design only must be done for those that have poorer strength properties.
- 3) For mast arm assemblies not covered in the standards or at locations

with poorer soil strength properties an analysis and design must be done.

3.2.2.8 Tunnels

Due to the greatly varying conditions under which tunnels are constructed, investigation criteria for tunnels shall be established for each project on an individual basis.

3.2.2.9 Other Structures

Contact the District Geotechnical Engineer for instructions concerning other structures not covered in this section.

3.2.3 Borrow Areas

Test pits, trenches, and various types of borings can be used for exploration of potential borrow areas. Samples should be obtained to permit classification, moisture, compaction, permeability test, LBR, and/or corrosion testing of each material type, as applicable. The extent of the exploration will depend on the size of the borrow area and the amount and type of borrow needed.

3.2.4 Retention Ponds

A minimum of 2 borings shall be taken per 40,000 feet² (4,000 m²) of pond, with a minimum depth of 5 feet (1.5 m) below the deepest elevation of the pond, or until a confining layer is encountered or local Water Management District criteria are satisfied. A minimum of 2 field permeability tests per pond shall be performed, with this number increasing for larger ponds.

Sufficient testing must be accomplished to verify whether the excavated material can be used for embankment fill. Also, if rock is to be excavated from the pond sufficient borings and soundings must be accomplished to estimate the volume of rock to be removed and the hardness of the rock.

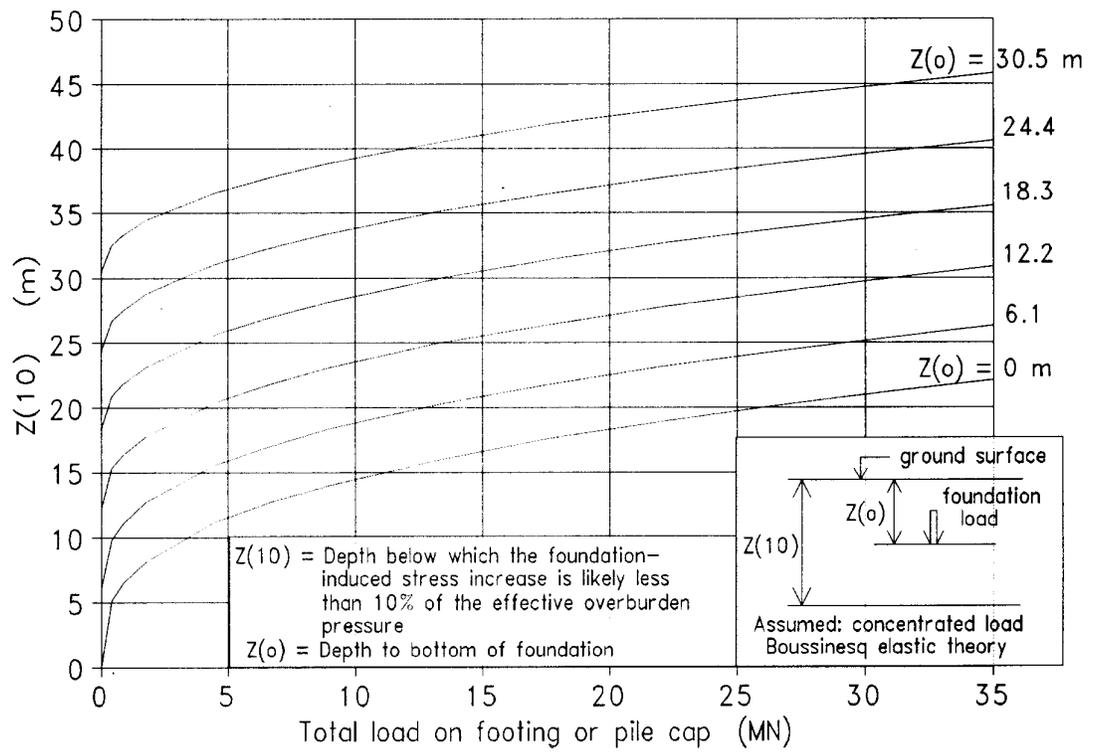


Figure 3, Depth below which the Foundation-Induced Vertical Normal Stress Increase is likely less than 10% of the Effective Overburden Pressure (Metric)(Adapted from Schmertmann, 1967)

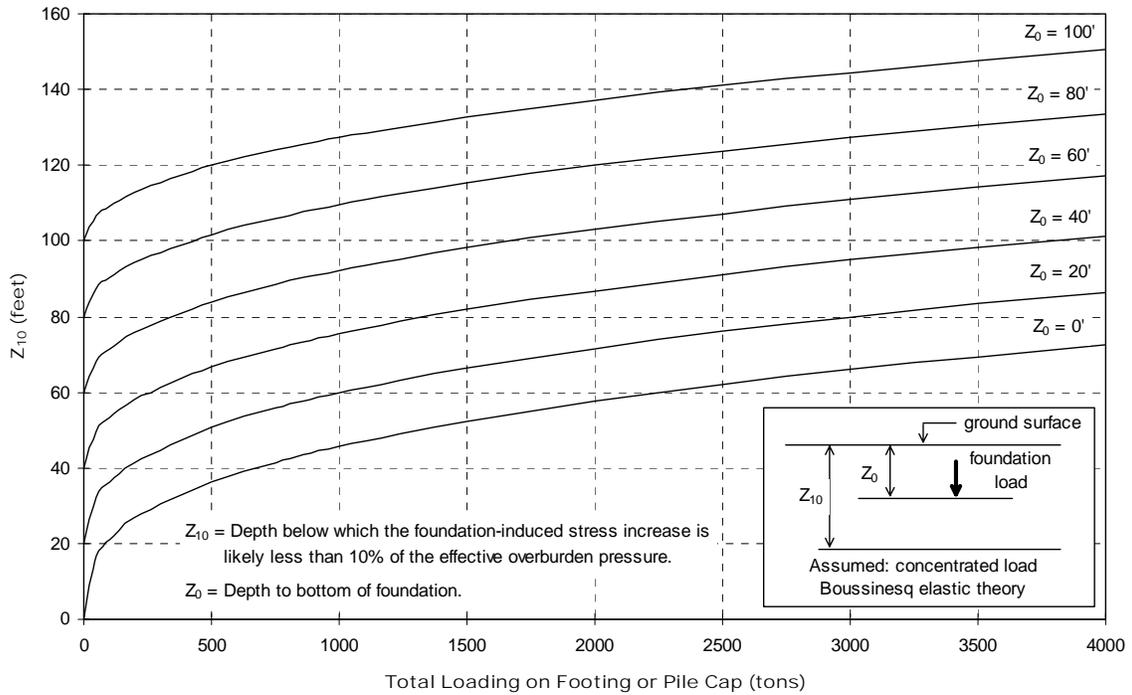


Figure 4, Depth below which the Foundation-Induced Vertical Normal Stress Increase is likely less than 10% of the Effective Overburden Pressure (English)(Adapted from Schmertmann, 1967)

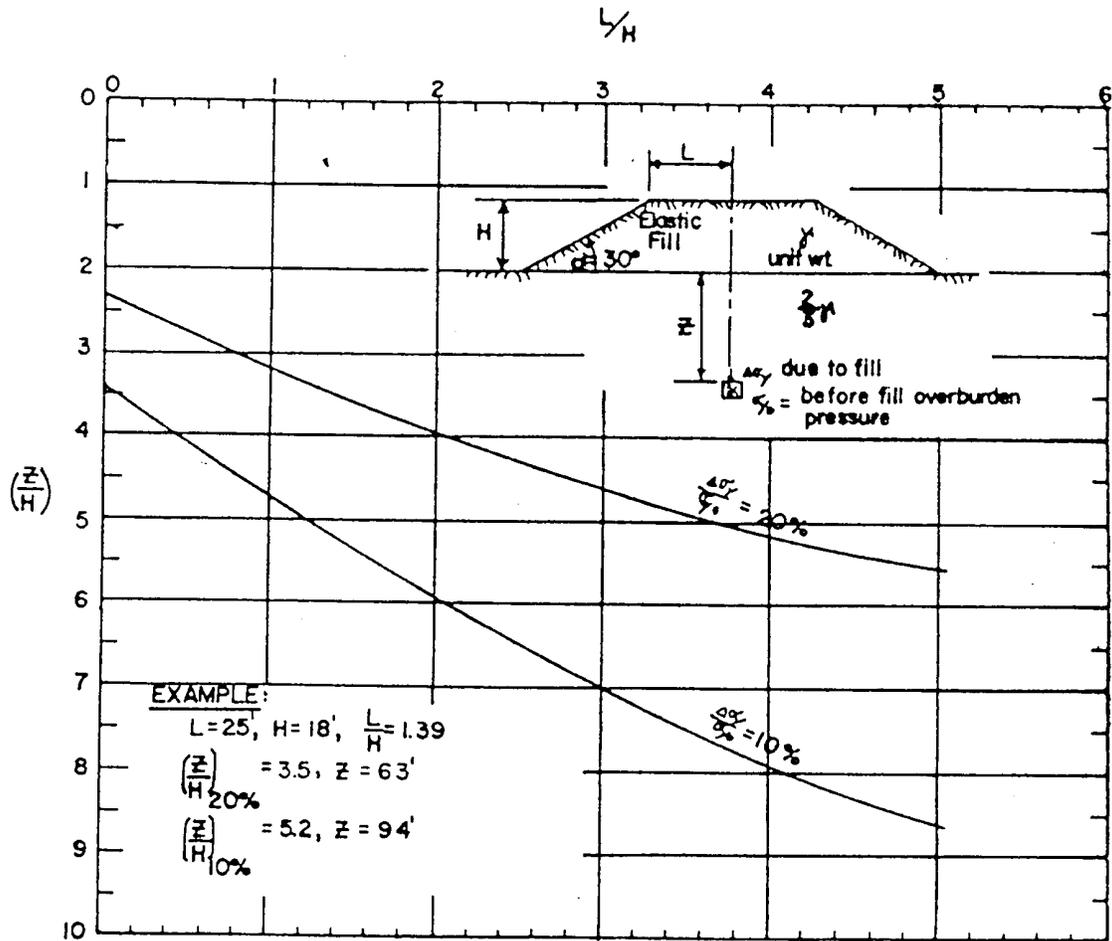


Figure 5, Chart for Determining the Maximum Depth of Significant Increase in Vertical Stress in the Foundation Soils Resulting from an Infinitely Long Trapezoidal Fill (both fill and foundation assumed homogeneous, isotropic and elastic). (After Schmertmann, 1967)

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7. Rigid Pavement Design Manual, FDOT, (Current version)

3.4 Specifications and Standards

<u>Subject</u>	<u>ASTM</u>	<u>AASHTO</u>	<u>FM</u>
Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils	D 1586	T 206	-
Standard Test Method for Determining Subsurface Liquid Levels in a Borehole or Monitoring Well (Observation Well)	D 4750	-	-

Chapter 4

4 In-situ Testing

The testing described in this chapter provides the Geotechnical Engineer with soil and rock parameters determined in-situ. This is important on all projects, especially those involving soft clays, loose sands and/or sands below the water table, due to the difficulty of obtaining representative samples suitable for laboratory testing. For each test included, a brief description of the equipment, the test method, and the use of the data is presented.

4.1 Standard Penetration Test (SPT)

This test is probably the most widely used field test in the United States. It has the advantages of simplicity, the availability of a wide variety of correlations for its data, and the fact that a sample is obtainable with each test.

A standard split barrel sampler is advanced into the soil by dropping a 140-pound (63.5-kilogram) safety or automatic hammer on the drill rod from a height of 30 inches (760 mm). (Note: Use of a donut hammer is not permitted). The sampler is advanced a total of 18 inches (450 mm). The number of blows required to advance the sampler for each of three 6-inch (150 mm) increments is recorded. The sum of the number of blows for the second and third increments is called the Standard Penetration Value, or more commonly, N-value (blows per foot {300 mm}). Tests shall be performed in accordance with ASTM D 1586.

During design, the N-values may need to be corrected for overburden pressure. A great many correlations exist relating the corrected N-values to relative density, angle of internal friction, shear strength, and other parameters. Design methods are available for using N-values in the design of driven piles, embankments, spread footings and drilled shafts.

The SPT values should not be used indiscriminately. They are sensitive to the fluctuations in individual drilling practices and equipment. Studies have also indicated that the results are more reliable in sands than clays. Although extensive use of this test in subsurface exploration is recommended, it should always be augmented by other field and laboratory tests, particularly when dealing with clays. The type of hammer (safety or automatic) shall be noted on the boring logs, since this will affect the actual input driving energy.

A method to measure the energy during the SPT has been developed (ASTM D 4633). Since there is a wide variability of performance in SPT hammers, this method is useful to evaluate an individual hammer's performance. The SPT installation procedure is similar to pile driving because it is governed by stress wave propagation. As a result, if force and velocity measurements are obtained during a test, the energy transmitted can be determined. Once this is known, the N-values from that SPT can be modified to a standard N_{60} using the following equation:

$$N_{60} E_{60} = N_{\text{field}} E_{\text{measured}}$$

Where:

E_{60} = 60% of the theoretical potential energy (210 ft-pounds {285 N-m})

N_{field} = field observed N-value

E_{measured} = measured energy.

4.2 Cone Penetrometer Test (CPT)

The Cone Penetrometer Test is a quasi-static penetration test in which a cylindrical rod with a conical point is advanced through the soil at a constant rate and the resistance to penetration is measured. A series of tests performed at varying depths at one location is commonly called a sounding.

Several types of penetrometers are in use, including mechanical (mantle) cone, mechanical friction-cone, electric cone, electric friction-cone, and piezocone penetrometers. Cone penetrometers measure the resistance to penetration at the tip of the penetrometer, or the end-bearing component of resistance. Friction-cone penetrometers are equipped with a friction sleeve, which provides the added capability of measuring the side friction component of resistance. Mechanical penetrometers have telescoping tips allowing measurements to be taken incrementally, generally at intervals of 8 inches (200 mm) or less. Electric (or electronic) penetrometers use electric force transducers to obtain continuous measurements with depth. Piezocone penetrometers are electric penetrometers, which are also capable of measuring pore water pressures during penetration.

For all types of penetrometers, cone dimensions of a 60-degree tip angle and a 1.55 in^2 (10 cm²) projected end area are standard. Friction sleeve outside diameter is the same as the base of the cone. Penetration rates should be between 0.4 to 0.8 in/sec (10 to 20 mm/sec). Tests shall be performed in accordance with ASTM D 3441 (which includes mechanical cones) and ASTM D 5778 (which includes piezocones).

The penetrometer data is plotted showing the end-bearing resistance, the friction resistance and the friction ratio (friction resistance divided by end bearing resistance) as functions of depth. Pore pressures, if measured, can also be plotted with depth. The results should also be presented in tabular form indicating the interpreted results of the raw data. See **Figure 6**, **Figure 7**, and **Figure 8** (Note: the log for a standard cone penetration test would only include the first three plots: tip resistance, local friction, and friction ratio; shown in **Figure 7**).

The friction ratio plot can be analyzed to determine soil type. Many correlations of the cone test results to other soil parameters have been made, and design methods are available for spread footings and piles. The penetrometer can be used in sands or clays, but not in rock or other extremely dense soils. Generally, soil samples are not obtained with soundings, so penetrometer exploration should always

be augmented by SPT borings or other borings with soil samples taken.

The piezocone penetrometer can also be used to measure the dissipation rate of the excessive pore water pressure. This type of test is useful for subsoils, such as fibrous peat or muck that are very sensitive to sampling techniques. The cone should be equipped with a pressure transducer that is capable of measuring the induced water pressure. To perform this test, the cone will be advanced into the subsoil at a standard rate of 0.8 inch/sec (20 mm/sec). Pore water pressures will be measured immediately and at several time intervals thereafter. Use the recorded data to plot a pore pressure versus log-time graph. Using this graph one can directly calculate the pore water pressure dissipation rate or rate of settlement of the soil.

4.3 Dynamic Cone Penetrometer Test

This test is similar to the cone penetrometer test except, instead of being pushed at a constant rate, the cone is driven into the soil. The number of blows required to advance the cone in 6-inch (150 mm) increments is recorded. A single test generally consists of two increments. Tests can be performed continuously to the depth desired with an expendable cone, which is left in the ground upon drill rod withdrawal, or they can be performed at specified intervals by using a retractable cone and advancing the hole by auger or other means between tests. Samples are not obtained.

Blow counts can generally be used to identify material type and relative density. In granular soils, blow counts from the second 6-inch (150 mm) increment tend to be larger than for the first increment. In cohesive soils, the blow counts from the two increments tend to be about the same. While correlations between blow counts and engineering properties of the soil exist, they are not as widely accepted as those for the SPT.

4.4 Dilatometer Test (DMT)

The dilatometer is a 3.75-inch (95 mm) wide and 0.55-inch (14 mm) thick stainless steel blade with a thin 2.4-inch (60 mm) diameter expandable metal membrane on one side. While the membrane is flush with the blade surface, the blade is either pushed or driven into the soil using a penetrometer or drilling rig. Rods carry pneumatic and electrical lines from the membrane to the surface. At depth intervals of 8 inch (200 mm), the pressurized gas expands the membrane and both the pressure required to begin membrane movement and that required to expand the membrane into the soil 0.04 inches (1.1 mm) are measured. Additionally, upon venting the pressure corresponding to the return of the membrane to its original position may be recorded (see **Figure 9**, **Figure 10**, and **Figure 11**). Refer to References 5, 6, and 7.

Through developed correlations, information can be deduced concerning material type, pore water pressure, in-situ horizontal and vertical stresses, void ratio or relative density, modulus, shear strength parameters, and consolidation parameters. Compared to the pressuremeter, the flat dilatometer has the advantage of reduced soil disturbance during penetration.

4.5 Pressuremeter Test (PMT)

This test is performed with a cylindrical probe placed at the desired depth in a borehole. The Menard type pressuremeter requires pre-drilling of the borehole; the self-boring type pressuremeter advances the hole itself, thus reducing soil disturbance. The Menard probe contains three flexible rubber membranes (see **Figure 12**). The middle membrane provides measurements, while the outer two are “guard cells” to reduce the influence of end effects on the measurements. When in place, the guard cell membranes are inflated by pressurized gas while the middle membrane is inflated with water by means of pressurized gas. The pressure in all the cells is incremented and decremented by the same amount. The measured volume change of the middle membrane is plotted against applied pressure. Tests shall be performed in accordance with ASTM D 4719.

Studies have shown that the “guard cells” can be eliminated without sacrificing the accuracy of the test data provided the probe is sufficiently long. Furthermore, pumped air can be substituted for the pressurized gas used to inflate the membrane with water. The TAXAM pressuremeter is an example of this type.

Results are interpreted based on semi-empirical correlations from past tests and observation. In-situ horizontal stresses, shear strength, bearing capacities, and settlement can be estimated using these correlations. The pressuremeter test is very sensitive to borehole disturbance and the data may be difficult to interpret for some soils.

4.6 Field Vane Test

This test consists of advancing a four-bladed vane into cohesive soil to the desired depth and applying a measured torque at a constant rate until the soil fails in shear along a cylindrical surface. (See **Figure 13**) The torque measured at failure provides the undrained shear strength of the soil. A second test run immediately after remolding at the same depth provides the remolded strength of the soil and thus information on soil sensitivity. Tests shall be performed in accordance with ASTM D 2573.

This method is commonly used for measuring shear strength in soft clays and organic deposits. It should not be used in stiff and hard clays. Results can be affected by the presence of gravel, shells, roots, or sand layers. Shear strength may be overestimated in highly plastic clays and a correction factor should be applied.

4.7 Percolation Test

The percolation test is used to ascertain the vertical percolation rate of unsaturated soil, i.e., the rate at which the water moves through near surface soils. The most common tests consist of digging a 4 to 12 inch (100 to 300 mm) diameter hole to the stratum for which information is required, cleaning and backfilling the bottom with coarse sand or gravel, filling the hole with water and providing a soaking period of sufficient length to achieve saturation. During the soaking period, water is added as necessary to prevent loss of all water. The percolation rate is then obtained

by filling the hole to a prescribed water level and measuring the drop in water level over a set time. The times required for soaking and for measuring the percolation rate vary with the soil type; local practice should be consulted for specific requirements. See also References 8 and 9.

Results of this test are generally used in evaluating site suitability for septic system drainage fields.

4.8 Infiltration Test

The infiltration rate of a soil is the maximum rate at which water can enter the soil from the surface under specified conditions. The most common test in Florida uses a double-ring infiltrometer. Two open cylinders, approximately 20 inch (500 mm) high and 12 to 24 inch (300 to 600 mm) in diameter, are driven concentrically into the ground. The outer ring is driven to a depth of about 6 inch (150 mm), the inner ring to a depth of 2 to 4 inch (50 and 100 mm). Both are partially filled with water. As the water filtrates into the soil, measured volumes are added to keep the water levels constant. The volumes of water added to the inner ring and to the annular space during a specific time interval, equivalent to the amounts, which have infiltrated the soil. These are converted into infiltration rates, expressed in units of length per unit time, usually inches (millimeters) per hour. The infiltration rate is taken as the maximum infiltration velocity occurring over a period of several hours. In the case of differing velocities for the inner ring and the annular space, the maximum velocity from the inner ring should be used. The time required to run the test is dependent upon soil type. Tests shall be performed in accordance with ASTM D 3385.

Drainage engineers in evaluating runoff, ditch or swale infiltration use information from this test.

4.9 Permeability Test

Permeability, also known as hydraulic conductivity, is the measure of the rate of flow of water through soils, usually measured when the soil is saturated. Permeability differs from infiltration or percolation rates in that permeability values are corrected for the hydraulic boundary conditions, including the hydraulic gradient, and thus is representative of a specific soil property. Some types of field permeability tests are discussed below.

4.9.1 Seepage Test

These tests can be constant head, falling head, or rising head tests. The constant head test is the most generally applicable and, in areas of unknown permeability, should be performed first. The falling head and rising head methods are used in areas where the permeability is low enough to permit accurate measurement of the change in water level. Results are used in the design of exfiltration systems. The more commonly performed tests include:

4.9.1.1 Open-End Borehole Test

This test can be conducted as either a constant head or a variable head test. An open-end pipe or casing is installed to the desired depth within a uniform soil. The pipe/casing is then cleaned out flush with the bottom of the pipe/casing while the hole is kept filled with water. Clear water is added through a metering system to maintain gravity flow at a constant head until measurements indicate a steady-state flow is achieved. The permeability is calculated from the rate of steady-state flow, height of head and radius of pipe (see **Figure 14**). For in-situ variable head tests, see Reference 2.

4.9.1.2 Exfiltration Test

This test is performed as a constant head test. A 7-inch (175 mm) diameter (or larger) hole is augered to a standard depth of 10 feet (3 meters). Approximately 0.125 ft³ (0.0035 m³) of 0.5-inch (13 mm) diameter gravel is poured to the bottom of the hole to prevent scour. A 6-inch (150 mm) diameter (or larger), 9-foot (2.75 meter) long casing which is perforated with 0.5 inch (12.7 mm) holes on 2-inch (51 mm) centers over the bottom 6.0 feet (1.8 m) is then lowered into the hole. Water is added and the amount required to maintain a constant water level over specified time intervals is recorded. See Reference 10.

4.9.2 Pumping Test

Pumping tests are used in large-scale investigations to more accurately measure the permeability of an area. The results are used in the design of dewatering systems and other situations where the effects of a change in the water table are to be analyzed.

Pumping tests require a test hole and at least one observation well, although several observation wells at varying distances from the test hole are preferable. As water is pumped from the test hole, water level changes within each observation well and corresponding times is recorded. Pumping is continued at a constant rate until the water level within each observation well remains constant. Permeability calculations are made based on the rate of pumping, the measured draw down, and the configuration of the test hole and observation wells. Refer to ASTM D 4050.

4.10 Environmental Corrosion Tests

These tests are carried out on soil and water at structure locations, on structural backfill materials and on subsurface materials along drainage alignments to determine the corrosion classification to be considered during design. For structures, materials are classified as slightly, moderately, or extremely aggressive, depending on their pH, resistivity, chloride content, and sulfate content. (Refer to the latest Structures Design Guidelines, for the criteria, which defines each class). For roadway drainage systems, test results for each stratum are presented for use in determining alternate culvert materials. Testing shall be performed in the field and/or the

laboratory according to the standard procedures listed below.

4.10.1 pH of Soils

- a) ASTM G 51
- b) FM 5-550

4.10.2 pH of Water

- a) ASTM G 51
- b) FM 5-550

4.10.3 Chloride Ion in Water

- a) ASTM D 512
- b) FM 5-552

4.10.4 Chloride Ion in Soil

- a) ASTM D 512 (using supernatant from soils)
- b) FM 5-552

4.10.5 Sulfate Ion in Brackish Water

- a) ASTM D 4130 (using supernatant from soils)
- b) FM 5-553

4.10.6 Sulfates in Soil

- a) ASTM D 4130 (using supernatant from soils)
- b) FM 5-553

4.10.7 Electrical Resistance of Water

- a) ASTM D 1125
- b) FM 5-551

4.10.8 Electrical Resistance of Soil

- a) ASTM G 57
- b) FM 5-551

4.11 Grout Plug Pull-out Test

This test is performed when the design of drilled shafts in rock is anticipated. However, the values obtained from this test should be used carefully. Research has indicated that the results are overly conservative.

A 4-inch (100 mm) diameter (minimum) by 30-inch (760 mm) long core hole is made to the desired depth in rock. A high strength steel bar with a bottom plate and a reinforcing cage over the length to be grouted is lowered to the bottom of the hole. Sufficient grout is poured into the hole to form a grout plug approximately 2 feet (600 mm) long. After curing, a center hole jack is used to incrementally apply a tension load to the plug with the intent of inducing a shear failure at the grout -

limestone interface. The plug is extracted, the failure surface examined, and the actual plug dimensions measured.

The ultimate shear strength of the grout-limestone interface is determined by dividing the failure load by the plug perimeter area. This value can be used to estimate the skin friction of the rock-socketed portion of the drilled shaft.

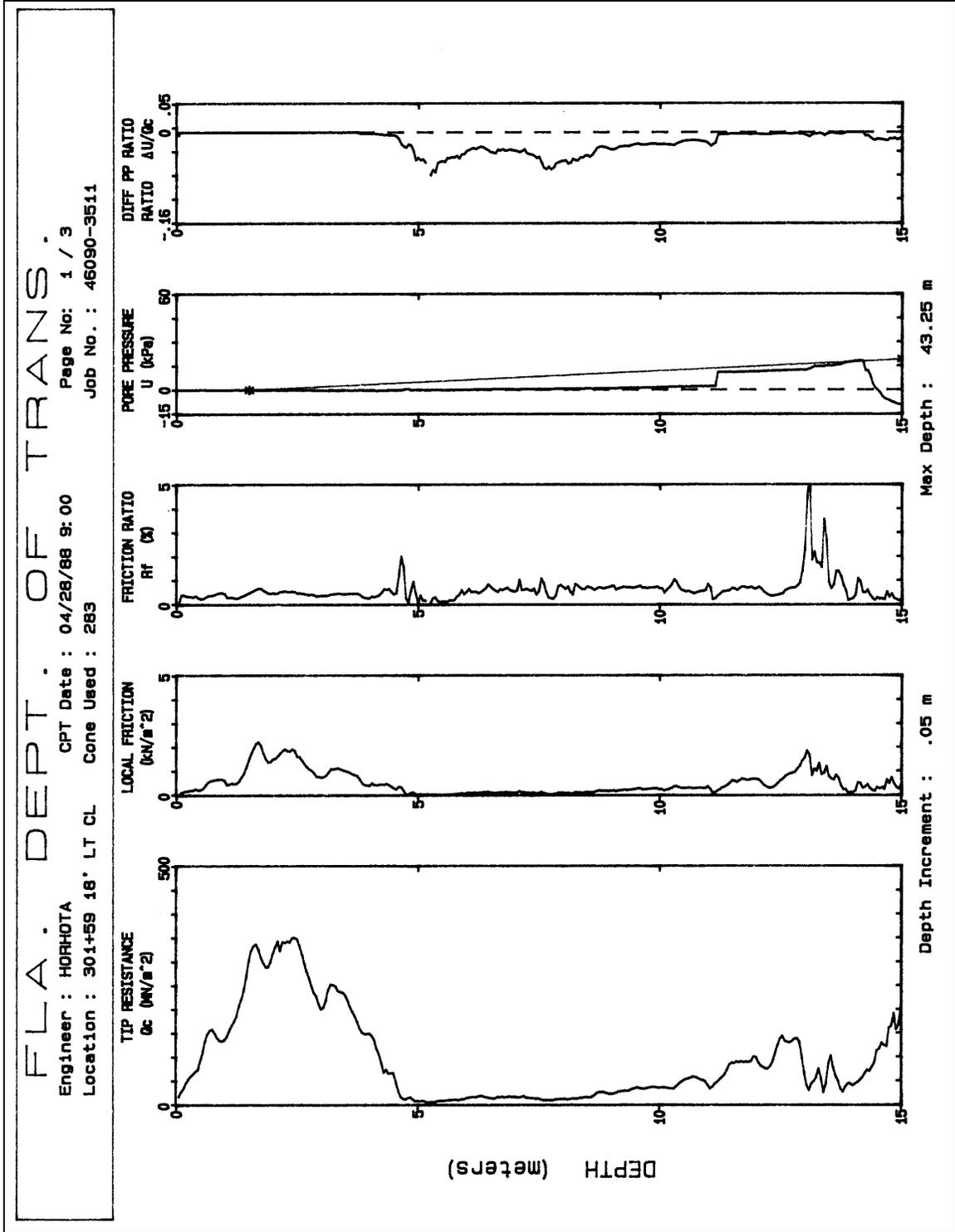


Figure 7, Typical Log from Electric Piezocone

FLORIDA D.O.T MATERIALS OFFICE

CPT DATE :04/28/88 9:00 ENGINEER :BLANTON
 LOCATION :301+59 6 m LT CL Cone Used :283
 Job No. :46090-3511 Water table (meters) : 1.5
 Tot. Unit Wt. (avg) : 510 N/m³

DEPTH (meters)	Qc (avg) (MN/m ²)	Fs (avg) (kN/m ²)	Rf (avg) (%)	SIGV' (MPa)	SOIL BEHAVIOUR TYPE	Eq - Dr (%)	PHI deg.	SPT N	Su (MPa)
0.50	53.24	0.16	0.31	0.04	sand to silty sand	>90	>48	13	UNDEFINED
1.00	139.74	0.56	0.40	0.13	sand	>90	>48	27	UNDEFINED
1.50	205.64	0.71	0.35	0.22	sand	>90	>48	39	UNDEFINED
2.00	312.59	1.77	0.57	0.28	gravelly sand to sand	>90	>48	50	UNDEFINED
2.50	341.26	1.77	0.52	0.31	gravelly sand to sand	>90	>48	>50	UNDEFINED
3.00	262.08	1.08	0.41	0.35	gravelly sand to sand	>90	>48	42	UNDEFINED
3.50	236.04	1.00	0.42	0.38	sand	>90	>48	45	UNDEFINED
4.00	173.89	0.67	0.39	0.42	sand	>90	46-48	33	UNDEFINED
4.50	92.91	0.41	0.44	0.45	sand to silty sand	70-80	44-46	22	UNDEFINED
5.00	17.01	0.13	0.79	0.49	sandy silt to clayey silt	UNDFND	UNDFD	7	1.07
5.50	6.64	0.01	0.21	0.52	sensitive fine grained	UNDFND	UNDFD	3	.38
6.00	10.38	0.03	0.32	0.55	sandy silt to clayey silt	UNDFND	UNDFD	4	.62
6.50	16.33	0.10	0.60	0.59	sandy silt to clayey silt	UNDFND	UNDFD	6	1.01
7.00	15.86	0.10	0.61	0.62	sandy silt to clayey silt	UNDFND	UNDFD	6	.97
7.50	14.86	0.09	0.63	0.66	sandy silt to clayey silt	UNDFND	UNDFD	6	.90
8.00	10.37	0.06	0.61	0.69	sandy silt to clayey silt	UNDFND	UNDFD	4	.60
8.50	13.54	0.09	0.67	0.73	sandy silt to clayey silt	UNDFND	UNDFD	5	.80
9.00	22.86	0.16	0.70	0.76	silty sand to sandy silt	<40	34-36	7	UNDEFINED
9.50	29.15	0.21	0.71	0.80	silty sand to sandy silt	<40	36-38	9	UNDEFINED
10.00	35.88	0.26	0.72	0.83	silty sand to sandy silt	<40	36-38	11	UNDEFINED
10.50	39.31	0.28	0.71	0.87	silty sand to sandy silt	40-50	36-38	13	UNDEFINED
11.00	53.59	0.30	0.56	0.90	sand to silty sand	50-60	38-40	13	UNDEFINED
11.50	58.47	0.30	0.52	0.94	sand to silty sand	50-60	38-40	14	UNDEFINED
12.00	92.18	0.64	0.69	0.97	sand to silty sand	60-70	40-42	22	UNDEFINED
12.50	94.25	0.44	0.47	1.01	sand to silty sand	60-70	40-42	23	UNDEFINED
13.00	125.46	1.04	0.83	1.04	sand to silty sand	70-80	40-42	30	UNDEFINED
13.50	50.89	1.15	2.26	1.08	sandy silt to clayey silt	UNDFND	UNDFD	19	3.24
14.00	51.81	0.39	0.76	1.11	silty sand to sandy silt	40-50	36-38	17	UNDEFINED
14.50	68.95	0.32	0.46	1.15	sand to silty sand	50-60	38-40	17	UNDEFINED
15.00	154.16	0.41	0.27	1.18	sand	70-80	42-44	30	UNDEFINED
15.50	214.47	0.46	0.22	1.22	gravelly sand to sand	80-90	42-44	34	UNDEFINED
16.00	239.03	0.43	0.18	1.25	gravelly sand to sand	80-90	42-44	38	UNDEFINED
16.50	168.43	0.21	0.13	1.29	sand	70-80	42-44	32	UNDEFINED
17.00	102.13	0.13	0.13	1.32	sand	60-70	38-40	20	UNDEFINED
17.50	101.49	0.22	0.22	1.36	sand	60-70	38-40	19	UNDEFINED
18.00	171.24	0.28	0.16	1.39	sand	70-80	40-42	33	UNDEFINED
18.50	174.32	0.23	0.13	1.43	sand	70-80	40-42	33	UNDEFINED
19.00	191.14	0.25	0.13	1.46	sand	70-80	42-44	37	UNDEFINED

Dr - All sands (Jamiolkowski et al. 1985) PHI - Robertson and Campanella 1983 Su: Nk= 15

**** Note: For interpretation purposes the PLOTTED CPT PROFILE should be used with the TABULATED OUTPUT from CPTINTR1 (v 3.04) ****

Figure 8, Typical Interpreted Output from Electric Cone Penetrometer

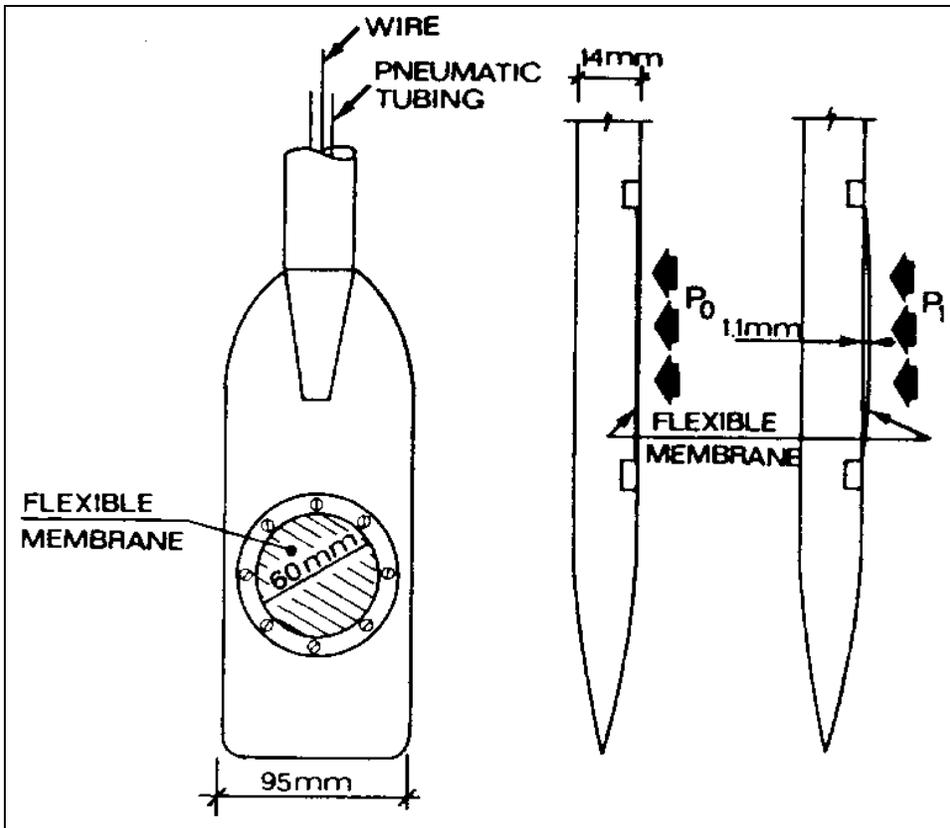


Figure 9, Schematic of the Marchetti Flat Dilatometer (After Baldi, et al., 1986)

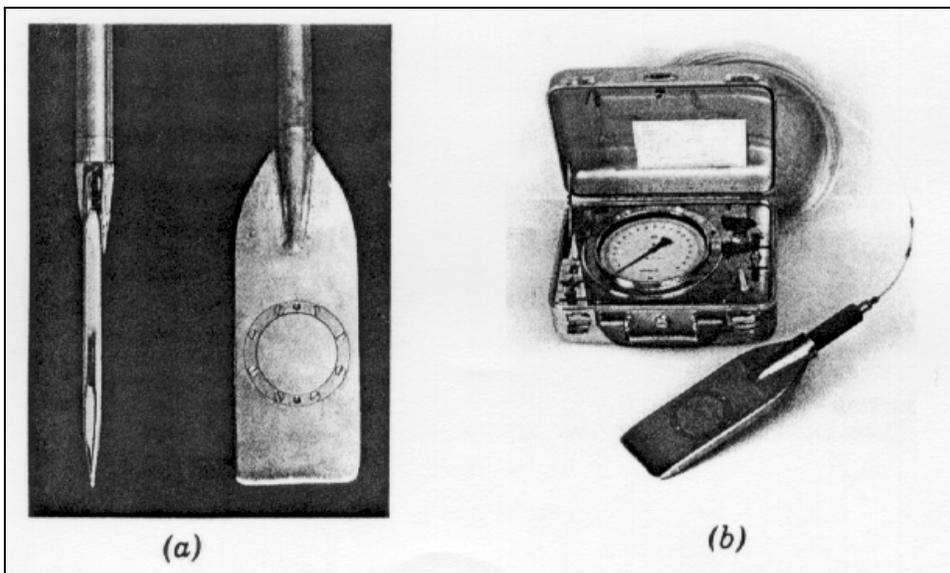


Figure 10, Dilatometer (After Marchetti 1980)

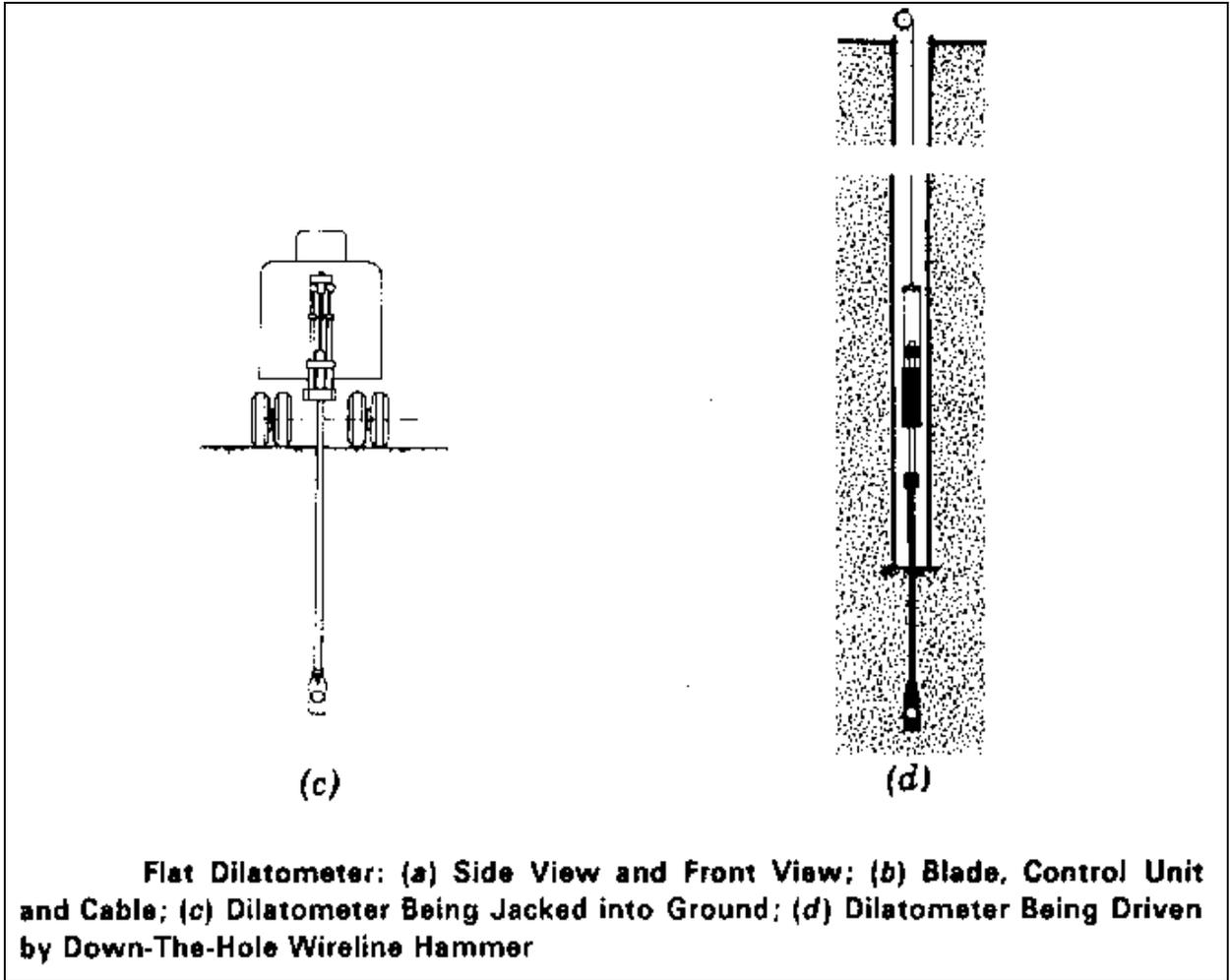


Figure 11, Dilatometer (Continued)

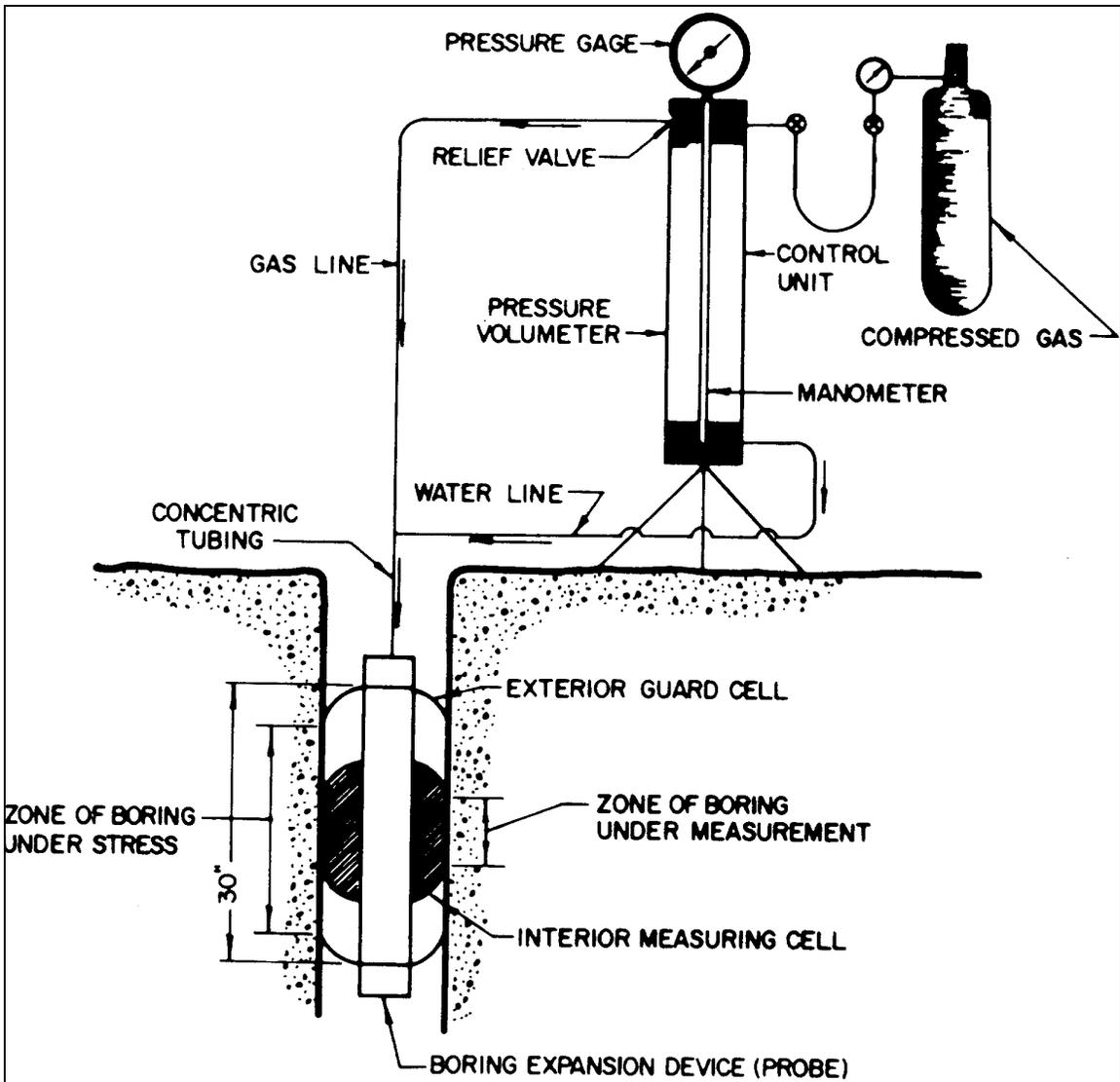


Figure 12, Menard Pressuremeter Equipment (After NAVFAC, 1986)

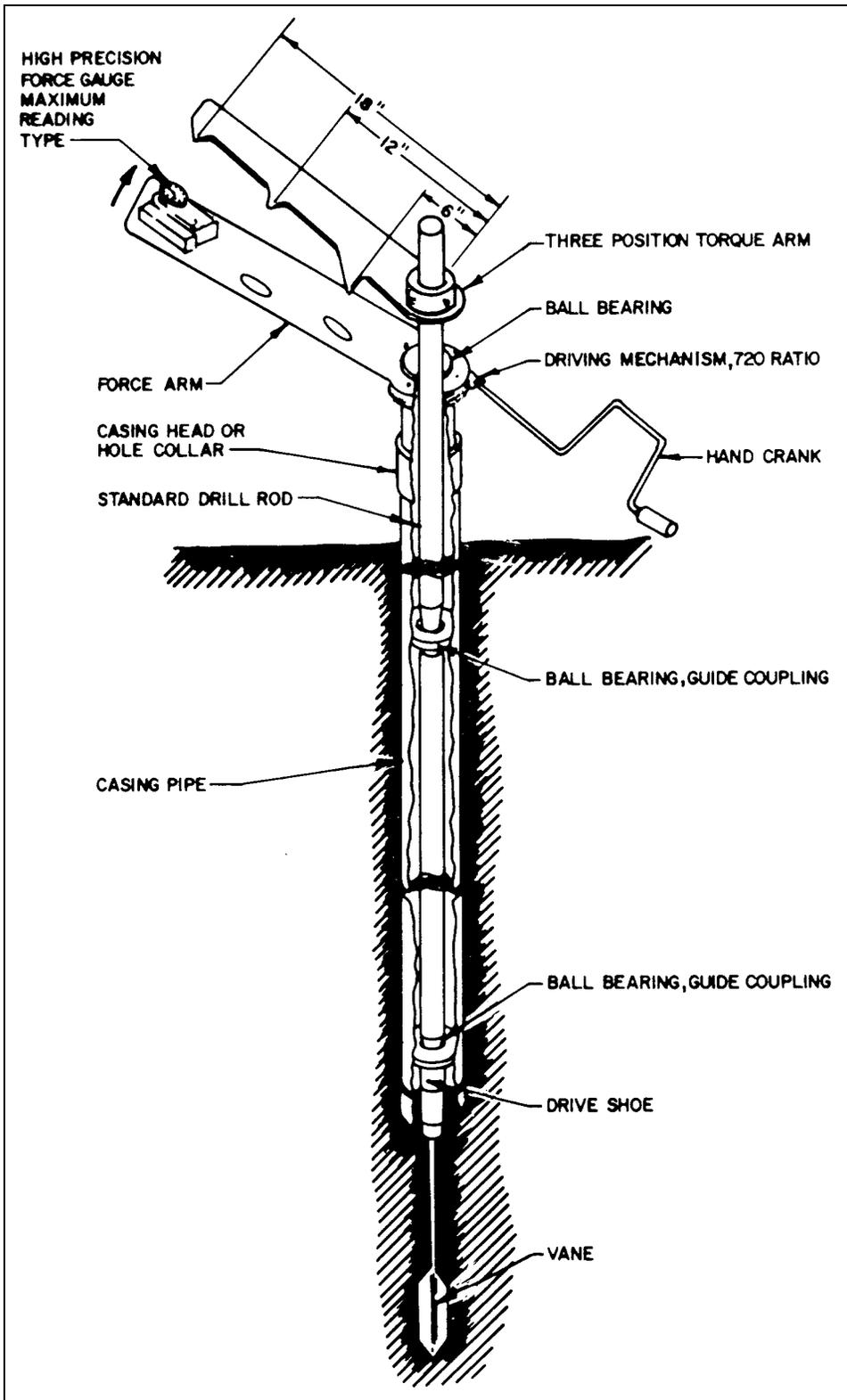


Figure 13, Vane Shear Test Equipment (After NAVFAC, 1986)

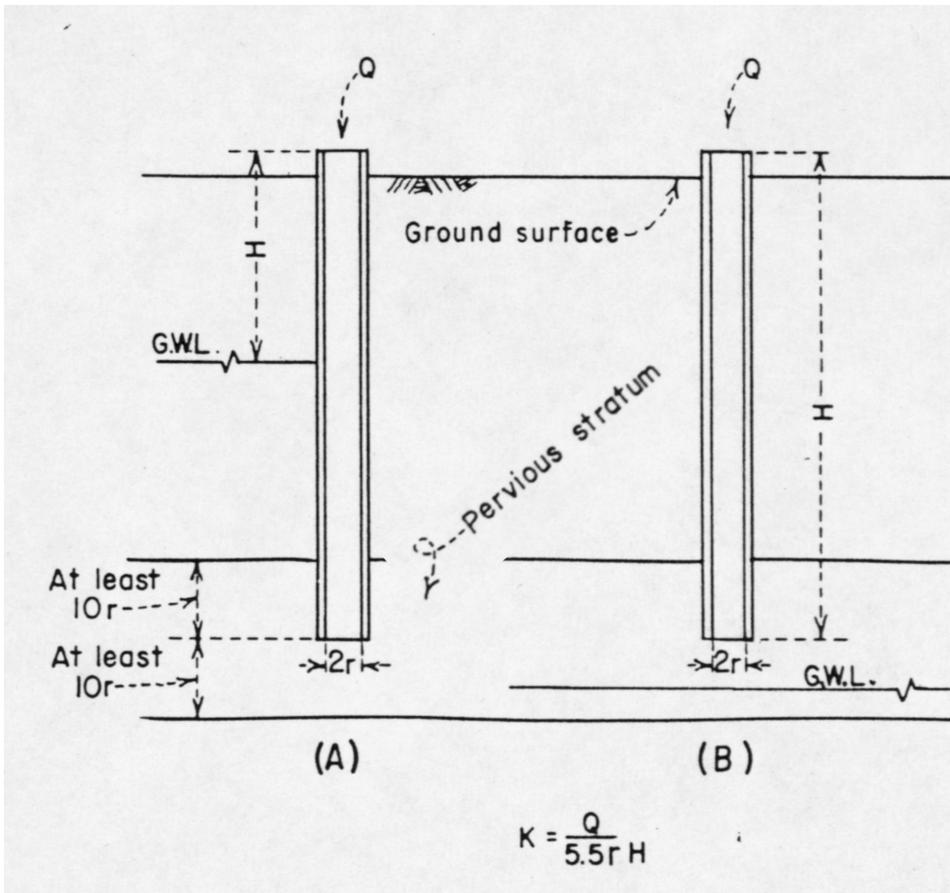


Figure 14, Open-end Borehole Test (After Bureau Reclamation, 1961)

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15. Jean-Louis Briaud and Jerome Miran; Manual on the Dilatometer Test, FHWA-SA-91-044, 1992
16. Structures Design Guidelines, Florida Department of Transportation, (Current version)

4.13 Specifications and Standards

<u>Subject</u>	<u>ASTM</u>	<u>AASHTO</u>	<u>FM</u>
Dilatometer	-	-	-
Chloride Content - Soil (Retaining wall backfill)	-	-	5-556
Standard Test Methods for Chloride Ion In Water	D 512	-	5-552
Standard Test Methods for Electrical Conductivity and Resistivity of Water	D 1125	-	5-551
Standard Test Methods for pH of Water	D 1293	-	5-550
Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils	D 1586	T 206	-
Standard Test Method for Field Vane Shear Test in Cohesive Soil	D 2573	T 223	-
Standard Test Method for Deep, Quasi-Static, Cone and Friction-Cone Penetration Tests of Soil	D 3441	-	-
Standard Test Method for Infiltration Rate of Soils in Field Using Double-Ring Infiltrometer	D 3385	-	-
Standard Test Method (Field Procedure) for Withdrawal and Injection Well Tests for Determining Hydraulic Properties of Aquifer Systems	D 4050	-	-
Standard Test Method for Sulfate Ion in Brackish Water, Seawater, and Brines	D 4130	-	5-553
Standard Test Method for Pressuremeter Testing in Soils	D 4719	-	-
Standard Test Method for Determining Subsurface Liquid Levels in a Borehole or Monitoring Well (Observation Well)	D 4750	-	-
Standard Practices for Preserving and Transporting Rock Core Samples	D 5079	-	-
Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils	D 5778	-	-
Standard Test Method for Measuring pH of Soil for Use in Corrosion Testing	G 51	T 289	5-550
Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method	G 57	T 288	5-551

Chapter 5

5 Laboratory Tests

As with other phases of a subsurface investigation program, the laboratory testing must be intelligently planned in advance but flexible enough to be modified based on test results. The ideal laboratory program will provide the engineer with sufficient data to complete an economical design, yet not tie up laboratory personnel and equipment with superfluous testing. The cost for laboratory testing is insignificant compared to the cost of an over-conservative design.

As noted in Chapter 1, this handbook is not intended as a procedural or a design handbook. Detailed instructions on test procedures will be found in the References and Specifications and Standards listed at the end of the chapter. This chapter is limited to a brief description of the tests, their purpose and the uses of the resulting data.

Not every test outlined below is applicable to every project. Engineering judgement must be exercised in setting up a testing program that will produce the information required on each specific project.

5.1 Soils

5.1.1 Grain-Size Analysis

This test is performed in two stages: sieve analysis for coarse-grained soils (sands, gravels) and hydrometer analysis for fine-grained soils (clays, silts). Soils containing both types are tested in sequence, with the material passing the No. 200 sieve (0.075 mm or smaller) analyzed by hydrometer.

5.1.1.1 Sieve Analysis

This test provides a direct measurement of the particle size distribution of a soil by causing the sample to pass through a series of wire screens with progressively smaller openings of known size. The amount of material retained on each sieve is weighed. See ASTM C 136.

5.1.1.2 Hydrometer

This test is based on Stokes Law. The diameter of a soil particle is defined as the diameter of a sphere which has the same unit mass and which falls at the same velocity as the particle. Thus, a particle size distribution is obtained by using a hydrometer to measure the change in specific gravity of a soil-water suspension as soil particles settle out over time.

Results are reported on a combined grain size distribution plot as the percentage of sample smaller than, by weight, versus the log of the particle diameter. These data are necessary for a complete classification of the soil. The curve also provides other parameters, such as effective diameter (D_{10}) and

coefficient of uniformity (C_u). Tests shall be performed in accordance with ASTM D 422 (AASHTO T 88).

5.1.2 Moisture Content

The moisture content, w , is defined as the ratio of the weight of water in a sample to the weight of solids. The wet sample is weighed, and then oven-dried to a constant weight at a temperature of about 230° F (110° C). The weight after drying is the weight of solids. The change in weight, which has occurred during drying, is equivalent to the weight of water. For organic soils, a reduced drying temperature of approximately 140° F (60° C) is sometimes recommended. Tests shall be performed in accordance with ASTM D 2216 (AASHTO T 265).

The moisture content is valuable in determining the properties of soils and can be correlated with other parameters. A good technique is to plot the moisture content from SPT samples as a function of depth.

5.1.3 Atterberg Limits

The liquid limit, plastic limit and shrinkage limit are all Atterberg Limits. However, for classification purposes, the term Atterberg Limits generally refers to the liquid and plastic limits only. The tests for these two are described here; the shrinkage limit test is described in Section 5.1.8 of this chapter.

The liquid limit (LL) is the moisture content of a soil at the boundary between the liquid and plastic states. The plastic limit (PL) is the moisture content at the boundary between the plastic and semi-solid states. The plasticity index (PI) is the difference between the LL and PL. The results are generally reported as LL/PI values and can be plotted on the same graph as the moisture content above. These values are useful in soil classification and have been correlated with other parameters.

5.1.3.1 Liquid Limit

The liquid limit is determined by ascertaining the moisture content at which two halves of a soil cake will flow together for a distance of 0.5 inch (13 mm) along the bottom of the groove separating the halves, when the bowl they are in is dropped 25 times for a distance of 0.4 inches (10 mm) at the rate of 2 drops/second. Tests shall be performed in accordance with ASTM D 4318 (AASHTO T 89).

5.1.3.2 Plastic Limit

The plastic limit is determined by ascertaining the lowest moisture content at which the material can be rolled into threads 0.125 inches (3.2 mm) in diameter without crumbling. Tests shall be performed in accordance with ASTM D 4318 (AASHTO T 90).

5.1.4 Specific Gravity of Soils

The specific gravity of soil, G_s , is defined as the ratio of the mass in air of a given volume of soil particles to the mass in air of an equal volume of gas free distilled water at a stated temperature (typically 68° F {20° C}). The specific gravity is determined by means of a calibrated pycnometer, by which the mass and temperature of a deaired soil/distilled water sample is measured. Tests shall be performed in accordance with ASTM D 854 (AASHTO T 100). This method is used for soil samples composed of particles less than the No. 4 sieve (4.75 mm). For particles larger than this sieve, use the procedures for Specific Gravity and Absorption of Coarse Aggregate (ASTM C 127 or AASHTO T 85).

The specific gravity of soils is needed to relate a weight of soil to its volume, and it is used in the computations of other laboratory tests.

5.1.5 Strength Tests

The shear strength of a soil is the maximum shearing stress the soil structure can resist before failure. Soils generally derive their strength from friction between particles (expressed as the angle of internal friction, ϕ), or cohesion between particles (expressed as the cohesion, c in units of force/unit area), or both. These parameters are expressed in the form of total stress (c, ϕ) or effective stress (c, ϕ) The total stress on any subsurface element is produced by the overburden pressure plus any applied loads. The effective stress equals the total stress minus the pore water pressure.

The common methods of ascertaining these parameters in the laboratory are discussed below. All of these tests should be performed only on undisturbed samples.

5.1.5.1 Unconfined Compression Tests

While under no confining pressure, a cylindrical sample is subjected to an axial load until failure. This test is only performed on cohesive soils. Total stress parameters are obtained. The cohesion is taken as one-half the unconfined compressive strength, q_u . This test is a fast and economical means of approximating the shear strength at shallow depths, but the reliability is poor with increasing depth. Tests shall be performed in accordance with ASTM D 2166 (AASHTO T 208).

5.1.5.2 Triaxial Compression Tests

In this test a cylindrical sample is subjected to an axial load until failure while also being subjected to confining pressure approximating the in-situ stress conditions. Various types of tests are possible with the triaxial apparatus as summarized below.

5.1.5.2.1 Unconsolidated-Undrained (UU), or Q Test

In this test the specimen is not permitted to change its initial water content before or during shear. The results are total stress parameters. This test is used primarily in the calculation of immediate embankment stability during quick-loading conditions. Refer to ASTM D 2850 (AASHTO T 296).

5.1.5.2.2 Consolidated-Undrained (CU), or R Test

In this test the specimen is allowed to consolidate under the confining pressure prior to shear, but no drainage is permitted during shear. A minimum of three tests at different confining pressures is required to derive the total stress parameters. If pore pressure measurements are taken during testing, the effective stress parameters can also be derived. Refer to ASTM D 4767 (AASHTO T 297).

5.1.5.2.3 Consolidated-Drained (CD), or S Test

This test is similar to the CU test (above) except that drainage is permitted during shear and the rate of shear is very slow. Thus, the buildup of excess pore pressure is prevented. As with the CU test, a minimum of three tests is required. Effective stress parameters are obtained. This test is used to determine parameters for calculating long-term stability of embankments.

5.1.5.3 Direct Shear

In this test a thin soil sample is placed in a shear box consisting of two parallel blocks and a normal force is applied. One block remains fixed while the other block is moved parallel to it in a horizontal direction. The soil fails by shearing along a plane that is forced to be horizontal. A series of at least three tests with varying normal forces is required to define the shear strength parameters for a particular soil. This test is typically run as a consolidated-drained test on cohesionless materials. Tests shall be performed in accordance with ASTM D 3080 (AASHTO T 236).

5.1.5.4 Miniature Vane Shear (Torvane) and Pocket Penetrometer

These tests are used only as an index of the undrained shear strength (S_u) of clay samples and should not be used in place of a laboratory test program. Both tests consist of hand-held devices that are pushed into the sample and either a torque resistance (torvane) or a tip resistance (pocket penetrometer) is measured. They can be performed in the lab or in the field, typically on the ends of undisturbed thin-walled tube samples, as well as along the sides of test pits. Miniature vane shear tests shall be performed in accordance with ASTM D 4648.

5.1.6 Consolidation Test

When large loads such as embankments are applied to the surface, cohesive subsoils will consolidate, i.e., settle over time, through a combination of the rearrangement of the individual particles and the squeezing out of water. The amount and rate of settlement is of great importance in construction. For example, an embankment may settle until a gap exists between an approach and a bridge abutment. The calculation of settlement involves many factors, including the magnitude of the load, the effect of the load at the depth at which compressible soils exist, the water table, and characteristics of the soil itself. Consolidation testing is performed to ascertain the nature of these characteristics.

5.1.6.1 One-Dimensional Test

The most often used method of consolidation testing is the one-dimensional test. In this test, a specimen is placed in a consolidometer (oedometer) between two porous stones, which permit drainage. Specimen size can vary depending on the equipment used. Various loading procedures can be used during a one-dimensional test with incremental loading being the most common. With this procedure the specimen is subjected to increasing loads, usually beginning at approximately 1/16 tsf (5 kPa) and doubling each increment up to 16 tsf (1600 kPa). After each load application the change in sample height is monitored incrementally for, generally, 24 hours. To evaluate the recompression parameters of the sample, an unload/reload cycle can be performed during the loading schedule. To better evaluate the recompression parameters for over consolidated clays, the unload/reload cycle may be performed after the preconsolidation pressure has been defined. After the maximum loading has been reached, the loading is removed in decrements. Tests shall be performed in accordance with ASTM D 2435 (AASHTO T 216).

The data from a consolidation test is usually presented on an e -log p curve, which plots void ratio (e) as a function of the log of pressure (p), or an ϵ -log p curve where ϵ equals % strain. The parameters necessary for settlement calculation can be derived from these curves: compression index (C_c), recompression index (C_r), preconsolidation pressure (p_o or P_c) and initial void ratio (e_o). A separate plot is prepared of change in sample height versus log time for each load increment; from this, the coefficient of consolidation (c_v) and coefficient of secondary compression (C_α) can be derived. These parameters are used to predict the rate of primary settlement and amount of secondary compression.

5.1.6.2 Constant Rate of Strain Test

Other loading methods include the Constant Rate of Strain Test (ASTM D 4186) in which the sample is subjected to a constantly changing load while maintaining a constant rate of strain; and the single-increment test, sometimes used for organic soils, in which the sample is subjected only to the

load expected in the field. A direct analogy is drawn between laboratory consolidation and field settlement amounts and rates.

5.1.7 Organic Content

Organic soils demonstrate very poor engineering characteristics, most notably low strength and high compressibility. In the field these soils can usually be identified by their dark color, musty odor and low unit weight. The most used laboratory test for design purposes is the Ignition Loss test, which measures how much of a sample's mass burns off when placed in a muffle furnace. The results are presented as a percentage of the total sample mass. Tests shall be performed in accordance with ASTM D 2974 (AASHTO T 267).

5.1.8 Shrinkage and Swell

5.1.8.1 Shrinkage

These tests are performed to determine the limits of a soil's tendency to lose volume during decreases in moisture content. The shrinkage limit (SL) is defined as the maximum water content at which a reduction in water content will not cause a decrease in volume of the soil mass. Tests shall be performed in accordance with ASTM D 427 (AASHTO T 92).

5.1.8.2 Swell

Some soils, particularly those containing montmorillonite clay, tend to increase their volume when their moisture content increases. These soils are unsuitable for roadway construction. The swell potential can be estimated from the test methods shown in ASTM D 4546 (AASHTO T 258).

5.1.9 Permeability

The laboratory determination of soil permeability can be performed by one of the following test methods. Permeability can also be determined either directly or indirectly from a consolidation test.

5.1.9.1 Constant-Head Test

This test uses a permeameter into which the sample is placed and compacted to the desired relative density. Water (preferably de-aired) is introduced via an inlet valve until the sample is saturated. Water is then allowed to flow through the sample while a constant head is maintained. The permeability is measured by the quantity of flow of discharge over a specified time. This method is generally used only with coarse-grained soils. Tests shall be performed in accordance with ASTM D 2434 (AASHTO T 215).

5.1.9.2 Falling-Head Test

This test uses an apparatus and procedure similar to the constant-head test (above), but the head is not kept constant. The permeability is measured by the decrease in head over a specified time. This method is generally used for fine-grained soils. Tests shall be performed in accordance with FM 5-513.

5.1.9.3 Flexible Wall Permeability

For fine-grained soils, tests performed using a triaxial cell are generally preferred. In-situ conditions can be modeled by application of an appropriate confining pressure. The sample can be saturated using back pressuring techniques. Water is then allowed to flow through the sample and measurements are taken until steady-state conditions occur. Tests shall be performed in accordance with ASTM D 5084.

5.1.10 Environmental Corrosion Tests

These tests are performed to determine the corrosion classification of soil and water. A series of tests includes pH, resistivity, chloride content, and sulfate content testing. The testing can be done either in the laboratory or in the field. See **Section 4.3** for a list of test procedures.

5.1.11 Compaction Tests

These tests are used to determine the optimum water content and maximum dry density, which can be achieved for a particular soil using a designated compactive effort. Results are used to determine appropriate methods of field compaction and to provide a standard by which to judge the acceptability of field compaction.

Compacting a sample in a test mold of known volume using a specified compactive effort performs the test. The water content and the weight of the sample required to fill the mold are determined. Results are plotted as density versus water content. By varying the water content of the sample, several points on the moisture-density curve shall be obtained in accordance with the standard procedures specified.

The compactive effort used is dependent upon the proposed purpose of the site and the loading to which it will be subjected. The most commonly used laboratory test compactive efforts are described below.

5.1.11.1 Standard Proctor

This test method uses a 5.5-pound (2.5 kg) rammer dropped from a height of 12 inches (305 mm). The sample is compacted in three layers. Tests shall be performed in accordance with ASTM D 698 (AASHTO T 99).

5.1.11.2 Modified Proctor

This test method uses a 10-pound (4.54 kg) rammer dropped from a height of 18 inches (457 mm). The sample is compacted in five layers. Tests shall be performed in accordance with ASTM D 1557 (AASHTO T 180).

5.1.12 Relative Density Tests

Proctor tests often do not produce a well-defined moisture-density curve for cohesionless, free-draining soils. Additionally, maximum densities from Proctor tests may be less than those obtained in the field or by vibratory methods. For these soils, it may be preferable to perform tests, which determine standard maximum and minimum densities of the soil. The density of the in-situ soil can then be compared with these maximum and minimum densities and its relative density and/or percent compaction can be calculated.

5.1.12.1 Maximum Index Density

This test requires that either oven-dried or wet soil be placed in a mold of known volume, and that a 2-psi (14 kPa) surcharge load is applied. The mold is then vertically vibrated at a specified frequency for a specified time. The weight and volume of the sample after vibrating are used to calculate the maximum index density. Tests shall be performed in accordance with ASTM D 4253.

5.1.12.2 Minimum Index Density

This test is performed to establish the loosest condition, which can be attained by standard laboratory procedures. Several methods can be used, but the preferred method is to carefully pour a steady stream of oven-dried soil into a mold of known volume through a funnel. Funnel height should be adjusted continuously to maintain a free fall of the soil of approximately 0.5 inches (13 mm). Tests shall be performed in accordance with ASTM D 4254.

5.1.13 Limerock Bearing Ratio (LBR)

This test is used to determine the bearing value of limerock and other soils, which are used as base, stabilized subgrade, or embankment materials in Florida. This value is then used in the design of pavements.

A minimum of four, and preferably five, samples is compacted at varying moisture contents to establish a moisture-density curve for the material. Compaction procedures are similar to those of the modified Proctor test. There are two options, the soaked and the unsoaked methods. For the soaked method, the samples are soaked for a period of 48 hours under a surcharge mass of at least 2.5 lb (1.13 kg). For the unsoaked method, the samples are tested without any soak period. For both methods a penetration test is then performed on each sample by causing a 1.95-inch (49.5 mm) diameter piston to penetrate the soil at a constant rate and to a depth of 0.5 inches (12.7 mm). A load-penetration curve is plotted for each sample and the LBR corresponding to 0.1-inch (2.5 mm) penetration is

calculated. The maximum LBR for a material is determined from a plot of LBR versus moisture content. Tests shall be performed in accordance with FM 5-515.

5.1.14 Florida Bearing Value (FBV)

Although no longer commonly performed by the Department, this test provides valuable information for friable A-3 soils, specifically, uniformly graded dune sands.

A 1.3-pound (600 gm) of oven-dried soil sample is mixed with 10.4 ml of water and compacted into a cylindrical cup approximately 3 inches (76.2 mm) in diameter and 3 inches (76.2 mm) high by a 1,200-pound (544.3 kg) evenly distributed load. A circular bearing plate 1 in² (645 mm²) in area is placed on the surface of the sand. The cup is then placed under the lever arm of the Florida Bearing Value machine and the arm is leveled. A constantly increasing load is applied to the sample by allowing steel shot to run into a bucket on the end of the lever arm. The load is increased until the bearing plate is deflected at a rate of 0.01 inch (0.25 mm) in five seconds or until failure abruptly occurs. Tests are performed in accordance with FM 5-517.

5.1.15 Resilient Modulus Test (Dynamic)

This test is used to determine the dynamic elastic modulus of a base or subgrade soil under conditions that represent a reasonable simulation of the physical conditions and stress states of such materials under flexible pavements subjected to wheel loads. A prepared cylindrical sample is placed in a triaxial chamber and conditioned under static or dynamic stresses. A repeated axial stress is then applied at a fixed magnitude, duration, and frequency. The resilient modulus, M_r , is calculated by dividing the deviator stress by the resilient axial strain. This value is used in the design and evaluation of pavement systems. Tests shall be performed in accordance with AASHTO T 294.

5.2 Rock Cores

Laboratory tests on rock are performed on small samples of intact cores. However, the properties of in-situ rock are often determined by the presence of joints, bedding planes, etc. It is also important that the rock cores come from the zone that the foundations are founded in. Laboratory test results must therefore be considered in conjunction with knowledge of the in-situ characteristics of the rock mass. Some of the more common laboratory tests are:

5.2.1 Unconfined Compression Test

This test is performed on intact rock core specimens, which preferably have a length of at least two times the diameter. The specimen is placed in the testing machine and loaded axially at an approximately constant rate such that failure occurs within 2 to 15 minutes. **Note: the testing machine must be of the proper size for the samples being tested.** Tests shall be performed in accordance with ASTM D 2938.

5.2.2 Absorption and Bulk Specific Gravity

Absorption is a measure of the amount of water, which an initially dry specimen can absorb during a 48-hour soaking period. It is indicative of the porosity of the sample. Bulk specific gravity is used to calculate the unit weight of the material. Tests shall be performed in accordance with ASTM C 97.

5.2.3 Splitting Tensile Strength Test

This test is an indirect tensile strength test similar to the point load test; however, the compressive loads are line loads applied parallel to the core's axis by steel bearing plates between which the specimen is placed horizontally. Loading is applied continuously such that failure occurs within one to ten minutes. The splitting tensile strength of the specimen is calculated from the results. Tests shall be performed in accordance with ASTM D 3967 **except the minimum t/D (length-to-diameter) ratio shall be 1.0 when testing.**

5.2.4 Triaxial Compression Strength

This test is performed to provide shearing strengths and elastic properties of rock under a confining pressure. It is commonly used to simulate the stress conditions under which the rock exists in the field. Tests shall be performed in accordance with ASTM D 2664.

5.2.5 Unit Weight of Sample

This is a direct determination of either the moist or total weight of the rock core sample divided by the total cylindrical volume of the intact sample (for the total/moist unit weight), or the oven-dried weight divided by the total volume (for the dry unit weight). This measurement includes any voids or pore spaces in the sample, and therefore can be a relative indicator of the strength of the core sample. Samples should be tested at the moisture content representative of field conditions, and samples should be preserved until time of testing. Moisture contents shall be performed in accordance with ASTM D 2216.

5.3 References

1. Lambe, T. William, Soil Testing for Engineers, John Wiley & Sons, Inc. New York, NY, 1951.
2. NAVFAC DM-7.1 - Soil Mechanics, Department of the Navy, Naval Facilities Engineering Command, 1986.
3. George Munfakh, Ara Arman, Naresh Samtani and Raymond Castelli, Subsurface Investigations, FHWA-HI-97-021, 1997 (Pending)

5.4 Specifications and Standards

<u>Subject</u>	<u>ASTM</u>	<u>AASHTO</u>	<u>FM</u>
Permeability - Falling Head	-	-	5-513
Limerock Bearing Ratio	-	-	5-515
Florida Bearing Value	-	-	5-517
Resilient Modulus – Soil	-	T 294	-
Absorption and Bulk Specific Gravity of Dimension Stone	C 97	-	-
Standard Test Method for Specific Gravity and Absorption of Coarse Aggregate	C 127	T 85	1-T 85
Standard Test Method for Particle-Size Analysis of Soils	D 422	T 88	1-T 088
Test Method for Shrinkage Factors of Soils by the Mercury Method	D 427	T 92	1-T 092
Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft ³ (600 kN-m/m ³))	D 698	T 99	5-525
Standard Test Method for Specific Gravity of Soils	D 854	T 100	1-T 100
Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft ³ (2,700 kN-m/m ³))	D 1557	T 180	5-521
Standard Test Method for Unconfined Compressive Strength of Cohesive Soil	D 2166	T 208	1-T 208
Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock	D 2216	T 265	1-T 265
Standard Test Method for Permeability of Granular Soils (Constant Head)	D 2434	T 215	1-T 215
Standard Test Method for One-Dimensional Consolidation Properties of Soils	D 2435	T 216	1-T 216
Standard Test Method for Triaxial Compressive Strength of Undrained Rock Core Specimens Without Pore Pressure Measurements	D 2664	-	-
Standard Test Method for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression	D 2850	T 296	1-T 296
Standard Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens	D 2938	-	-

<u>Subject</u>	<u>ASTM</u>	<u>AASHTO</u>	<u>FM</u>
Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils	D 2974	T 267	1-T 267
Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions	D 3080	T 236	1-T 236
Standard Test Method for Splitting Tensile Strength of Intact Rock Core Specimens	D 3967	-	-
Standard Test Method for One-Dimensional Consolidation Properties of Soils Using Controlled-Strain Loading	D 4186	-	-
Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table	D 4253	-	-
Standard Test Method for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density	D 4254	-	-
Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils	D 4318	T 89	1-T 089 & 1-T-90
Standard Test Methods for One-Dimensional Swell or Settlement Potential of Cohesive Soils	D 4546	T 258	-
Standard Test Method for Laboratory Miniature Vane Shear Test for Saturated Fine-Grained Clayey Soil	D 4648	-	-
Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils	D 4767	T 297	1-T 297
Standard Practices for Preserving and Transporting Rock Core Samples	D 5079	-	-
Standard Test Method for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter	D 5084	-	-

Chapter 6

6 Materials Description, Classification, and Logging

During field exploration a log must be kept of the materials encountered. A field engineer, a geologist, or the driller usually keeps the field log. Details of the subsurface conditions encountered, including basic material descriptions, and details of the drilling and sampling methods should be recorded. Upon delivery of the samples to the laboratory, an experienced technician will generally verify or modify material descriptions and classifications based on the results of laboratory testing and/or detailed visual-manual inspection of samples. See ASTM D 5434

Material descriptions, classifications, and other information obtained during the subsurface explorations are heavily relied upon throughout the remainder of the investigation program and during the design and construction phases of a project. It is therefore necessary that the method of reporting this data is standardized. Records of subsurface explorations should follow as closely as possible the standardized format presented in this chapter.

6.1 Materials Description and Classification

A detailed description for each material stratum encountered should be included on the log. The extent of detail will be somewhat dependent upon the material itself and on the purpose of the project. However, the descriptions should be sufficiently detailed to provide the engineer with an understanding of the material present at the site. Since it is rarely possible to test all of the samples obtained during an exploration program, the descriptions should be sufficiently detailed to permit grouping of similar materials and choice of representative samples for testing.

6.1.1 Soils

Soils should be described in general accordance with the Description and Identification of Soils (Visual - Manual Procedure) of ASTM D 2488. This procedure employs visual examination and simple manual tests to identify soil characteristics, which are then included in the material description. For example, estimates of grain-size distribution by visual examination indicate whether the soil is fine-grained or coarse-grained. Manual tests for dry strength, dilatancy, toughness, and plasticity indicate the type of fine-grained soil. Organics are identified by color and odor. A detailed soil description should comply with the following format:

- Color
- Constituents
- Grading
- Relative Density or Consistency
- Moisture Content
- Particle Angularity and Shape

Additional Descriptive Terms Classification

6.1.1.1 Color

The color description is restricted to two colors. If more than two colors exist, the soil should be described as multi-colored or mottled and the two predominant colors given.

6.1.1.2 Constituents

Constituents are identified considering grain size distribution and the results of the manual tests. In addition to the principal constituent, other constituents which may affect the engineering properties of the soil should be identified. Secondary constituents are generally indicated as modifiers to the principal constituent (i.e., sandy clay or silty gravel). Other constituents can be included in the description through the use of terms such as with, some and trace.

6.1.1.3 Grading

6.1.1.3.1 Coarse-Grained Soils

Coarse-grained soils are defined as either:

6.1.1.3.1.1 Well-Graded

Soil contains a good representation of all particle sizes from largest to smallest.

6.1.1.3.1.2 Poorly-Graded

Soil contains particles about the same size. A soil of this type is sometimes described as being uniform.

6.1.1.3.1.3 Gap-Graded

Soil does not contain one or more intermediate particles sizes. A soil consisting of gravel and fine sand would be gap graded because of the absence of medium and coarse sand sizes.

6.1.1.3.2 Fine-Grained Soil

Descriptions of fine-grained soils should not include a grading.

6.1.1.4 Relative Density or Consistency

Relative density refers to the degree of compactness of a coarse-grained soil. Consistency refers to the stiffness of a fine-grained soil.

Standard Penetration Test N-values (blows per foot {300 mm}) are usually used to define the relative density and consistency as follows:

Table 1, Relative Density or Consistency

<i>Granular Materials</i>		
Relative Density	Safety Hammer SPT N-Value (Blow/Foot {300 mm})	Automatic Hammer SPT N-Value (Blow/Foot {300 mm})
Very Loose	Less than 4	Less than 3
Loose	4 – 10	3 – 7
Medium Dense	10 – 30	7 – 21
Dense	30 – 50	21 – 35
Very Dense	Greater than 50	Greater than 35
<i>Silts and Clays</i>		
Consistency	Safety Hammer SPT N-Value (Blow/Foot {300 mm})	Automatic Hammer SPT N-Value (Blow/Foot {300 mm})
Very Soft	Less than 2	Less than 1
Soft	2 – 4	1 – 3
Firm	4 – 8	3 – 6
Stiff	8 – 15	6 – 11
Very Stiff	15 – 30	11 – 21
Hard	Greater than 30	Greater than 21

If SPT data is not available, consistency can be estimated based on visual-manual examination of the material. Refer to ASTM D 2488 for consistency criteria.

6.1.1.5 Moisture Content

The in-situ moisture content of a soil should be described as dry, moist, or wet.

6.1.1.6 Particle Angularity and Shape

Coarse-grained soils are described as angular, sub-angular, sub-rounded, or rounded. Gravel, cobbles, and boulders can be described as flat, elongated, or flat and elongated. Descriptions of fine-grained soils will not include a particle angularity or shape.

6.1.1.7 Additional Descriptive Terms

Any additional descriptive terms considered to be helpful in identifying the soil should be included. Examples of such terms include calcareous, cemented, and gritty. Material origins or local names should be included in parentheses (i.e., fill, ironrock)

6.1.1.8 Classification

A soil classification should permit the engineer to easily relate the soil description to its behavior characteristics. All soils should be classified according to one of the following two systems.

6.1.1.8.1 Unified Soil Classification System (USCS)

This system is used primarily for engineering purposes and is particularly useful to the Geotechnical Engineer. Therefore, they should be used for all structural-related projects; such as bridges, retaining walls, buildings, etc. Precise classification requires that a grain size analysis and Atterberg Limits tests be performed on the sample. The method is discussed in detail in ASTM D 2487 and a summary is reprinted in **Figure 15** and **Figure 16** for convenience.

6.1.1.8.2 AASHTO Classification System

This system is used generally to classify soils for highway construction purposes and therefore will most often be used in conjunction with roadway soil surveys. Like the Unified System, this system requires grain size analysis and Atterberg Limit tests for precise classification. The system is discussed in detail in ASTM 3282 or AASHTO M 145, and a summary is reprinted in **Figure 17** and **Figure 18** for convenience.

6.1.2 Rocks

In Florida, only sedimentary rocks are encountered within the practical depths for structure foundations. Descriptions of sedimentary rocks are based on visual observations and simple tests. Descriptions should comply with the following format:

Color
Constituents
Weathering
Grain Size
Hardness
Cementation
Additional Descriptive Terms

6.1.2.1 Color

As with soils, the description should be limited to two predominant colors.

6.1.2.2 Constituents

The principal constituent is the rock type constituting the major portion of the stratum being investigated. Since the formations encountered in Florida normally consist of only one rock type, the use of modifying

constituents will generally not be applicable; however, when more than one rock type is present in any given formation, both should be included in the description.

6.1.2.3 Weathering

The degree of weathering should be described. Classical classification systems do not apply to Florida rock.

6.1.2.4 Hardness

Classical classification systems do not apply to Florida rock.

6.1.2.5 Cementation

The degree of cementation should be identified as well cemented to poorly cemented.

6.1.2.6 Additional Description Terms

Use any additional terms that will aid in describing the type and condition of the rock being described. Terms such as fossiliferous, friable, indurated, and micaceous are to be used where applicable. Formation names should be included in parentheses.

6.2 Logging

The standard boring log included as **Figure 19** and **Figure 20**, or its equivalent as approved by the District Geotechnical Engineer, shall be used for all borings and test pits. A sample completed log is included as **Figure 21** and **Figure 22**. The majority of information to be included on this form is self-explanatory. Information that should be presented in the remarks column includes:

6.2.1 Comments on Drilling Procedures and/or Problems

Any occurrences, which may indicate characteristics of the in-situ material, should be reported. Such occurrences include obstructions; difficulties in drilling such as caving, surging sands, or caverns; loss of drilling fluid; change in drilling method; and termination of boring above planned depth.

6.2.2 Test Results

Results of tests performed on samples in the field, such as pocket penetrometer or torvane tests should be noted. Results of tests on in-situ materials, such as field vane tests, should also be recorded.

6.2.3 Rock Quality Designation (RQD)

In addition to the percent recovery, the RQD should be recorded for each core run. RQD is a modified core recovery, which is best used on NX size core or

larger (HW is FDOT minimum size allowed). It describes the quality of rock based on the degree and amount of natural fracturing. Determined the RQD by summing the lengths of all core pieces equal to or longer than 4 inches (100 mm) (ignoring fresh irregular breaks caused by drilling) and dividing that sum by the total length of the core run.

Expressing the RQD as a percentage, the rock quality is described as follows:

<u>RQD (%)</u>	<u>Description of Rock Quality</u>
0 - 25	Very poor
25 - 50	Poor
50 - 75	Fair
75 - 90	Good
90 - 100	Excellent

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests ^A			Soil Classification		
	Group Symbol	Group Name ^B			
COARSE-GRAINED SOILS More than 50 % retained on No. 200 sieve	Gravels More than 50 % of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5 % fines ^C	Cu ≥ 4 and 1 ≤ Cc ≤ 3 ^E	GW	Well-graded gravel ^F
		Less than 5 % fines ^C	Cu < 4 and/or 1 > Cc > 3 ^E	GP	Poorly graded gravel ^F
		Gravels with Fines More than 12 % fines ^C	Fines classify as ML or MH	GM	Silty gravel ^{F,G,H}
		More than 12 % fines ^C	Fines classify as CL or CH	GC	Clayey gravel ^{F,G,H}
FINE-GRAINED SOILS 50 % or more passes the No. 200 sieve	Sands 50 % or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5 % fines ^D	Cu ≥ 6 and 1 ≤ Cc ≤ 3 ^E	SW	Well-graded sand ^I
		Less than 5 % fines ^D	Cu < 6 and/or 1 > Cc > 3 ^E	SP	Poorly graded sand ^I
		Sands with Fines More than 12 % fines ^D	Fines classify as ML or MH	SM	Silty sand ^{G,H,I}
		More than 12 % fines ^D	Fines classify as CL or CH	SC	Clayey sand ^{G,H,I}
HIGHLY ORGANIC SOILS	Silts and Clays Liquid limit less than 50	inorganic	PI > 7 and plots on or above "A" line ^J	CL	Lean clay ^{K,L,M}
		organic	PI < 4 or plots below "A" line ^J	ML	Silt ^{K,L,M}
		inorganic	Liquid limit – oven dried < 0.75 Liquid limit – not dried < 0.75	OL	Organic clay ^{K,L,M,N} Organic silt ^{K,L,M,O}
		organic	PI plots on or above "A" line PI plots below "A" line	CH	Fat clay ^{K,L,M}
HIGHLY ORGANIC SOILS	Silts and Clays Liquid limit 50 or more	inorganic	Liquid limit – oven dried < 0.75 Liquid limit – not dried < 0.75	MH	Elastic silt ^{K,L,M}
		organic	PI plots on or above "A" line PI plots below "A" line	OH	Organic clay ^{K,L,M,P} Organic silt ^{K,L,M,Q}
		Primarily organic matter, dark in color, and organic odor		PT	Peat

Figure 15, Unified Soil Classification System (After ASTM, 1993)

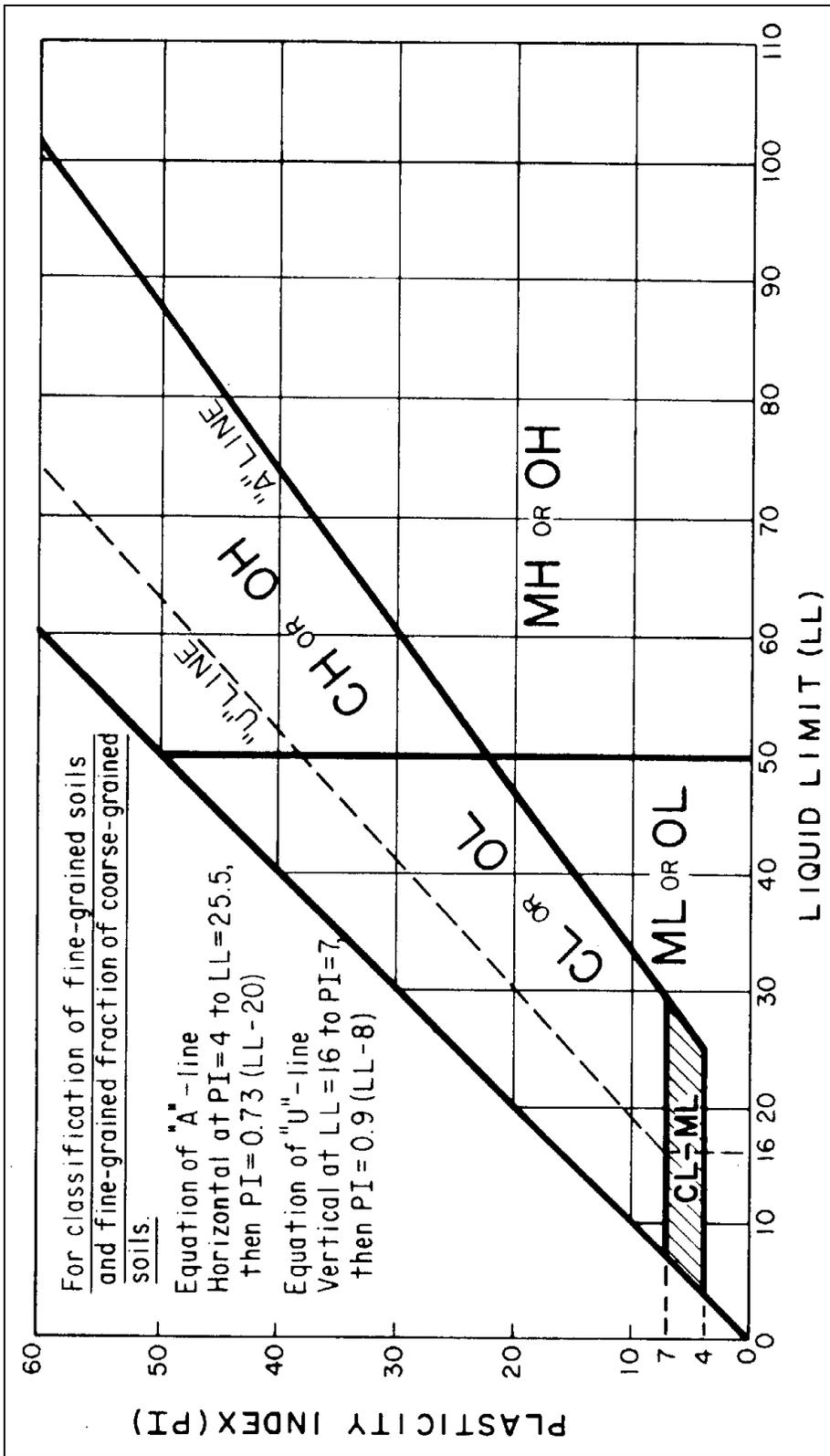


Figure 16, Unified Soil Classification System (After ASTM, 1993)(Cont.)

Classification of Soils and Soil-Aggregate Mixtures						
General Classification	Granular Materials (35 % or less passing No. 200)		Silt-Clay Materials (More than 35 % passing No. 200)			
Group Classification	A-1	A-3 ^A	A-2	A-4	A-5	A-6
Sieve analysis, % passing:						
No. 10 (2.00 mm)
No. 40 (425 μm)	50 max	51 min
No. 200 (75 μm)	25 max	10 max	35 max	36 min	36 min	36 min
Characteristics of fraction passing No. 40 (425 μm):						
Liquid limit	^B	40 max	41 min	40 max
Plasticity index	6 max	N.P.	^B	10 max	10 max	11 min
General rating as subgrade	Excellent to Good			Fair to Poor		
^A The placing of A-3 before A-2 is necessary in the "left to right elimination process" and does not indicate superiority of A-3 over A-2. ^B See Table 2 for values. Reprinted with permission of American Association of State Highway and Transportation Officials						

Classification of Soils and Soil-Aggregate Mixtures						
General Classification	Granular Materials (35 % or less passing No. 200)			Silt-Clay Materials (More than 35 % passing No. 200)		
Group classification	A-1		A-2	A-2		A-7
	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6
	A-3		A-2-7	A-4	A-5	A-6
	A-7		A-7-5,	A-7-6		A-7-6
Sieve analysis, % passing:						
No. 10 (2.00 mm)	50 max
No. 40 (425 μm)	30 max	50 max	51 min
No. 200 (75 μm)	15 max	25 max	10 max	35 max	35 max	35 max
Characteristics of fraction passing No. 40 (425 μm):						
Liquid limit	40 max	41 min	40 max
Plasticity index	6 max	N.P.	...	10 max	10 max	11 min
Usual types of significant constituent materials	Stone Fragments, Gravel and Sand		Fine Sand	Silty or Clayey Gravel and Sand		Silty Soils
General rating as subgrade	Excellent to Good			Fair to Poor		
^A Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30 (see Fig. 1). Reprinted with permission of American Association of State Highway and Transportation Officials.						

Figure 17, AASHTO Soil Classification System (After ASTM, 1993)

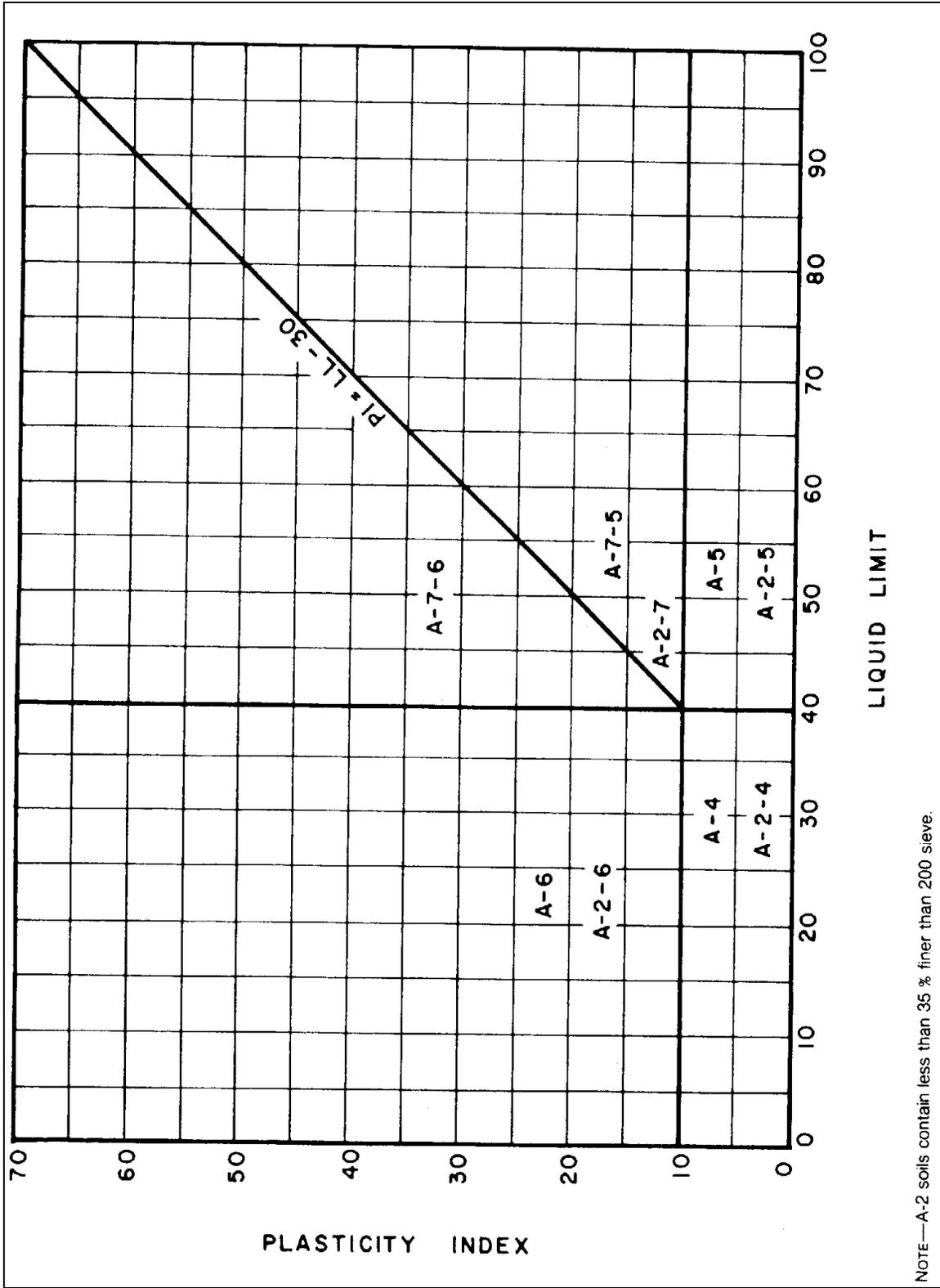


Figure 18, AASHTO Soil Classification System (After ASTM, 1993) (Cont.)

STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION
FIELD BORING LOG

FORM 675-020-13
 MATERIALS - 05/94

SHEET 1 OF 5

PROJECT NO. 79100-1523 NAME S.R. 40 / Tomoka River COUNTY Volusia DISTRICT 5
 LOCATION STA 14+80, 27.5 m RT CL Survey TOWNSHIP 14S RANGE 31E SECTION 25
 ROAD NUMBER State Road # 40 SURFACE ELEVATION +0.68 m, NGVD
 EQUIPMENT TYPE CME 45 RIG NO. 7476 BORING NO. 4
 DATE STARTED 8/27/90 COMPLETED 8/28/90 DRILLED BY Jenkins
 LOGGED BY Dawson BORING TYPE: AUGER, WASHED, PERCUSSION, ROTARY, Rotary
 WATER TABLE: 0 HR. 0.46 m 24 HRS. 0.46 m HRS. _____ CASED, UNCASD, DRILLING MUD, Cased/Uncased

SAMPLE CONDITIONS: DISTURBED GOOD LOST CORE SAMPLE

SAMPLE TYPES: A: AUGER SB: SPLIT BARREL S: SHELBY TUBE RC: ROCK CORE _____ SIZE

TESTS: W.C.: WATER CONTENT (%) T: TORVANE (kPa) V: IN-SITU VANE TEST (kPa)

ELEV. (M)	DEPTH (M)	S.P.T. BLOWS	MATERIAL DESCRIPTION	SAMPLES			TESTS	REMARKS
				CON.	NO. TYPE	REC. (%)		
		2						
		4	Dark brown fine SAND, trace peat (SP)	G	S-1	100		
		4						
		2	Light grey to dark brown fine SAND (SP)	G	S-2	20		
		3						
		2						
		2		G	S-3	50	WC = 20 -200 = 3	
		2						
-0.82	1.5	3		G	S-4	100		
		3						
		2						
		3		G	S-5	100		
		4						
		3						
		4		G	S-6	100	WC = 20 -200 = 3	
		3						
		5						
-2.32	3.0	6		G	S-7	100		
		3						
		9						
		13		G	S-8	100		
		8						
		7	Greenish-grey silty fine SAND, few shell fragments (SM)	G	S-9	100	WC = 29 -200 = 18	
		5						
		6						
		9						
-3.82	4.5	4		G	S-10	100		
			Light to dark grey fine SAND with silt, trace to few shell (SP-SM)					
		7						
		8						
		11		G	S-11	75		
-5.32	6.0	7						
		2						

RECYCLED PAPER 

Figure 22, Metric Typical Boring Log

6.3 References

1. Cheney, Richard S. & Chassie, Ronald G., Soils and Foundations Workshop Manual – Second Edition, FHWA HI-88-009, 1993.
2. NAVFAC DM-7.1-Soil Mechanics, Department of the Navy, Naval Facilities Engineering Command, 1986.
3. George Munfakh, Ara Arman, Naresh Samtani and Raymond Castelli, Subsurface Investigations, FHWA-HI-97-021, 1997 (Pending)

6.4 Specifications and Standards

<u>Subject</u>	<u>ASTM</u>	<u>AASHTO</u>	<u>FM</u>
Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System)	D 2487	M 145	-
Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)	D 2488	-	-
Standard Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes	D 3282	M 145	-
Standard Guide for Field Logging of Subsurface Explorations of Soil and Rock	D 5434	-	-

Chapter 7

7 Field Instrumentation

7.1 Instrumentation

Field instrumentation can be used on major projects during the analysis and design phase to assist the engineer in refinement of the design. An instrumented test embankment constructed during the preliminary stages of a project to assist in settlement prediction is an example.

On projects where analysis has indicated potential problems with embankment or structure settlement or stability, construction must be monitored through the use of field instrumentation. The location of such instrumentation should be included in the foundation design. This instrumentation allows the engineer to assess the settlement rate and evaluate stability as construction proceeds. The installation of this instrumentation and the interpretation of the ensuing data should be made by the Geotechnical Engineer in consultation with the construction engineer. Also included in the design package should be special provisions and the hold points, time or limitations of construction (for example, fill shall halt until settlement is less than 1 inch (25 mm) per 24 hours, etc.) needs to be indicated for the contractor. Many of the special provisions are available from the District or State Geotechnical Engineers.

Additionally, field instrumentation can be installed to provide data on existing structures or embankments. For example, slope indicators placed within an unstable area of an existing slope can provide the engineer with information, which is valuable in assessing the cause of the problem and in designing the necessary remedial measures.

Many of the instruments described in this chapter involve equipment such as inclinometer casing, settlement platform risers, or junction boxes, which protrude above ground in the construction area. These protuberances are particularly susceptible to damage from construction equipment. The Geotechnical Engineer must work with the construction engineer to ensure that the contractor understands the importance of these instruments and the need to protect them. The special provisions should carry penalties attached to them for the negligent damage to these instruments occurring during construction.

The most commonly used types of instrumentation are discussed below (Reference 2 and 4 is recommended for more detail):

7.1.1 Inclinometers (Slope Indicators)

These instruments are used to monitor embankment or cut slope stability. An inclinometer casing consists of a grooved metal or plastic tube that is installed in a borehole. The bottom of the tube must be in rock or dense material, which will not experience any movement, thereby achieving a stable point of fixity. A sensing probe is lowered down the tube and deflection of the tube is measured.

Successive readings can be plotted to provide the engineer with information about the rate of subsurface movement with depth (see **Figure 23**). Refer to ASTM D 4622 (AASHTO T 254).

Care must be taken when installing the casing so that spiraling of the casing does not occur because of poor installation techniques. This will result in the orientation of the grooves at depth being different than at the surface. This can be checked with a spiral-checking sensor, and the data adjusted with most new computerized data reduction routines. Also, the space between the borehole wall and the casing should be backfilled with a firm grout, sand, or gravel. For installation in highly compressible soils, use of telescoping couplings should be used to prevent damage of the casing.

To monitor embankment construction, inclinometers should be placed at or near the toes of slopes of high-fill embankments where slope stability or lateral squeeze is considered a potential problem. The casing should penetrate the strata in which problems are anticipated. Readings should be taken often during embankment construction. Fill operations should be halted if any sudden increase in movement rate is detected. The special provision 144 Digital Inclinometer Casing and Pneumatic Pore-Pressure Transducers Assembly should be modified for site conditions, other pore-pressure transducer types and included in the contract package.

7.1.2 Settlement Indicators

Settlement instruments simply record the amount and rate of the settlement under a load; they are most commonly used on projects with high fill embankments where significant settlement is predicted. The simplest form is the settlement platform or plate, which consists of a square wooden platform or steel plate placed on the existing ground surface prior to embankment construction. A reference rod and protecting pipe are attached to the platform. As fill operations progress, additional rods and pipes are added. (See **Figure 24** or **Standard Index 540**). Settlement is evaluated by periodically measuring the elevation of the top of the reference rod. Benchmarks used for reference datum shall be known to be stable and remote from all possible vertical movement. It is recommended to use multiple benchmarks and to survey between them at regular intervals.

Settlement platforms should be placed at those points under the embankment where maximum settlement is predicted. On large jobs two or more per embankment are common. The platform elevation must be recorded before embankment construction begins. This is imperative, as all future readings will be compared with the initial reading. Readings thereafter should be taken periodically until the embankment and surcharge (if any) are completed, then at a reduced frequency. The settlement data should be plotted as a function of time. The Geotechnical Engineer should analyze this data to determine when the rate of settlement has slowed sufficiently for construction to continue. The special provision 141 Settlement Plates should be modified for site conditions and included in the contract package.

A disadvantage to the use of settlement platforms is the potential for damage to the reference rod by construction equipment.

An alternative to settlement plates is probe extensometers in which a probe lowered down a compressible pipe can identify points along the pipe either mechanically or electrically, and thereby, the distance between these points can be determined. Surveying at the top of the pipe needs to be performed to get absolute elevations if the pipe is not seated into an incompressible soil layer. This method allows a settlement profile within the compressible soil layer to be obtained. Care must be taken during installation and grouting the pipe in the borehole so that it is allowed to settle in the same fashion as the surrounding soil.

7.1.3 Piezometers

Piezometers are used to measure the amount of water pressure within the saturated pores of a specific zone of soil. The critical levels to which the excess pore pressure will increase prior to failure can be estimated during design. During construction, the piezometers are used to monitor the pore water pressure buildup. After construction, the dissipation of the excess pore water pressure over time is used as a guide to consolidation rate. Thus, piezometers can be used to control the rate of fill placement during embankment construction over soft soils.

The simplest type of piezometer is an open standpipe extending through the fill, but its use may be limited by the response time lag inherent in all open standpipe piezometers. More useful and common in Florida is the pneumatic piezometer, which consists of a sensor body with a flexible diaphragm attached. This sensor is installed in the ground and attached to a junction box with twin tubes. The junction box outlet can be connected to a readout unit. Pressurized gas is applied to the inlet tube. As the applied gas pressure equals and then exceeds the pore water pressure, the diaphragm deflects allowing gas to vent through the outlet tube. The gas supply is then turned off and the diaphragm returns to its original position when the pressure in the inlet tube equals the pore water pressure. This pressure is recorded (see **Figure 25**). Refer to AASHTO T 252. Also available are vibrating wire and electrical resistance piezometers, but use of the electrical resistance piezometers is generally limited to applications where dynamic responses are to be measured.

Piezometers should be placed prior to construction in the strata in which problems are most likely to develop. If the problem stratum is more than 10 feet (3 m) thick, more than one piezometer should be placed, at varying depths. The junction box should be located at a convenient location but outside the construction area if possible.

The pore water pressure should be checked often during embankment construction. After the fill is in place, it can be monitored at a decreasing frequency. The data should be plotted (as pressure or feet (meters) of head) as a function of time. A good practice is to plot pore water pressure, settlement, and embankment elevation on the same time-scale plot for comparison. The special provision 144 Digital Inclinometer Casing and Pneumatic Pore-Pressure

Transducers Assembly should be modified for site conditions and included in the contract package.

7.1.4 Tiltmeters

Tiltmeters measure the inclination of discreet parts of structures from the norm. They are most commonly used to monitor tilting of bridge abutments and decks or retaining walls, and can also be used to monitor rotational failure surfaces in landslides. Types range from a simple plumb line to more sophisticated equipment.

7.1.5 Monitoring Wells

A monitoring or observation well is used to monitor groundwater levels or to provide ready access for sampling to detect groundwater contamination. It consists of a perforated section of pipe or well point attached to a riser pipe, installed in a sand-filled borehole.

Monitoring wells should also be installed in conjunction with piezometers to provide a base reference necessary for calculating changes in pore pressure. The monitoring well should be placed in an unimpacted area of construction to reflect the true static water table elevation.

7.1.6 Vibration Monitoring

It is sometimes desirable to monitor the ground vibrations induced by blasting, pile driving, construction equipment, or traffic. This is especially critical when construction is in close proximity to sensitive structures or equipment, which may become damaged if subjected to excessive vibration.

A vibration-monitoring unit typically consists of a recording control unit, one or more geophones, and connecting cables. Sound sensors to detect noise levels are also available. Geophones and/or sound sensors are placed at locations where data on vibration levels is desired. Peak particle velocities, principle frequencies, peak sound pressure levels, and actual waveforms can be recorded. Results are compared with pre-established vibration-limiting criteria, which are based on structure conditions, equipment sensitivity, or human tolerance.

7.1.7 Special Instrumentation

Earth pressure cells and strain gauges fall into this category of special instruments. They are not normally used in monitoring construction projects but only in research and special projects. These instruments require experienced personnel to install and interpret the data. Consult the State Materials Office for assistance.

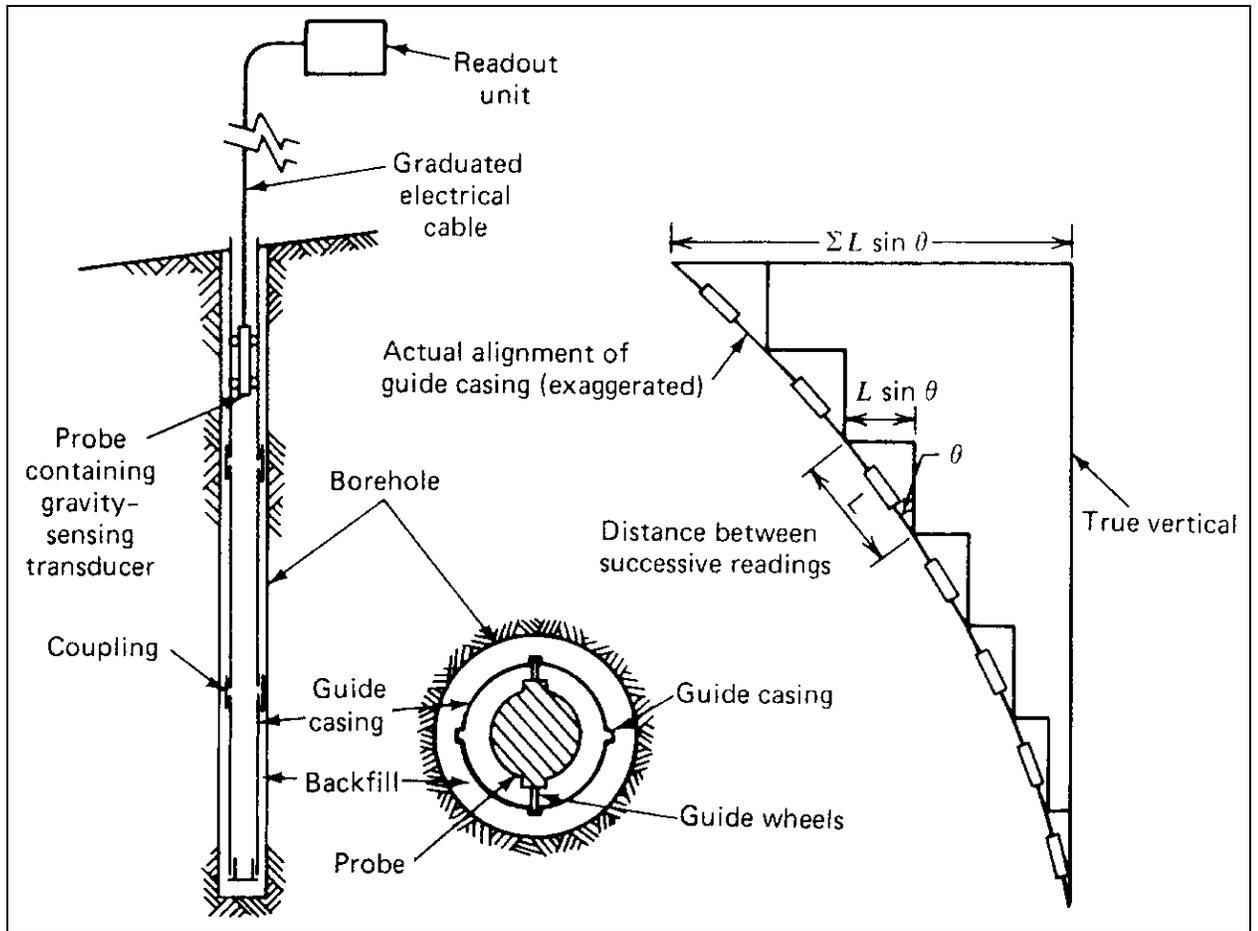


Figure 23, Principle of Inclinometer Operation (After Dunncliff, 1988)

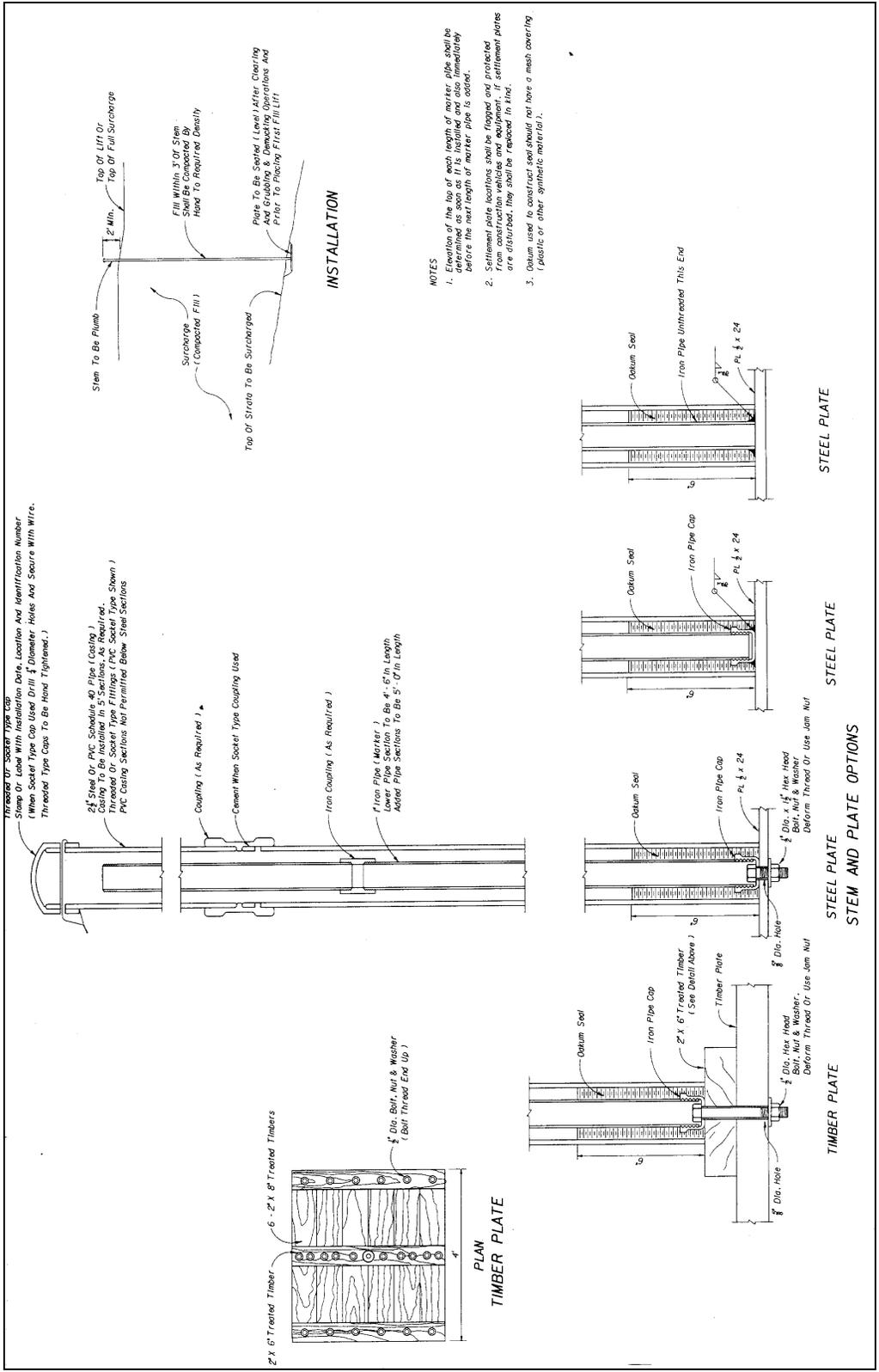


Figure 24, Typical Settlement Platform Design (After FDOT, 1992)

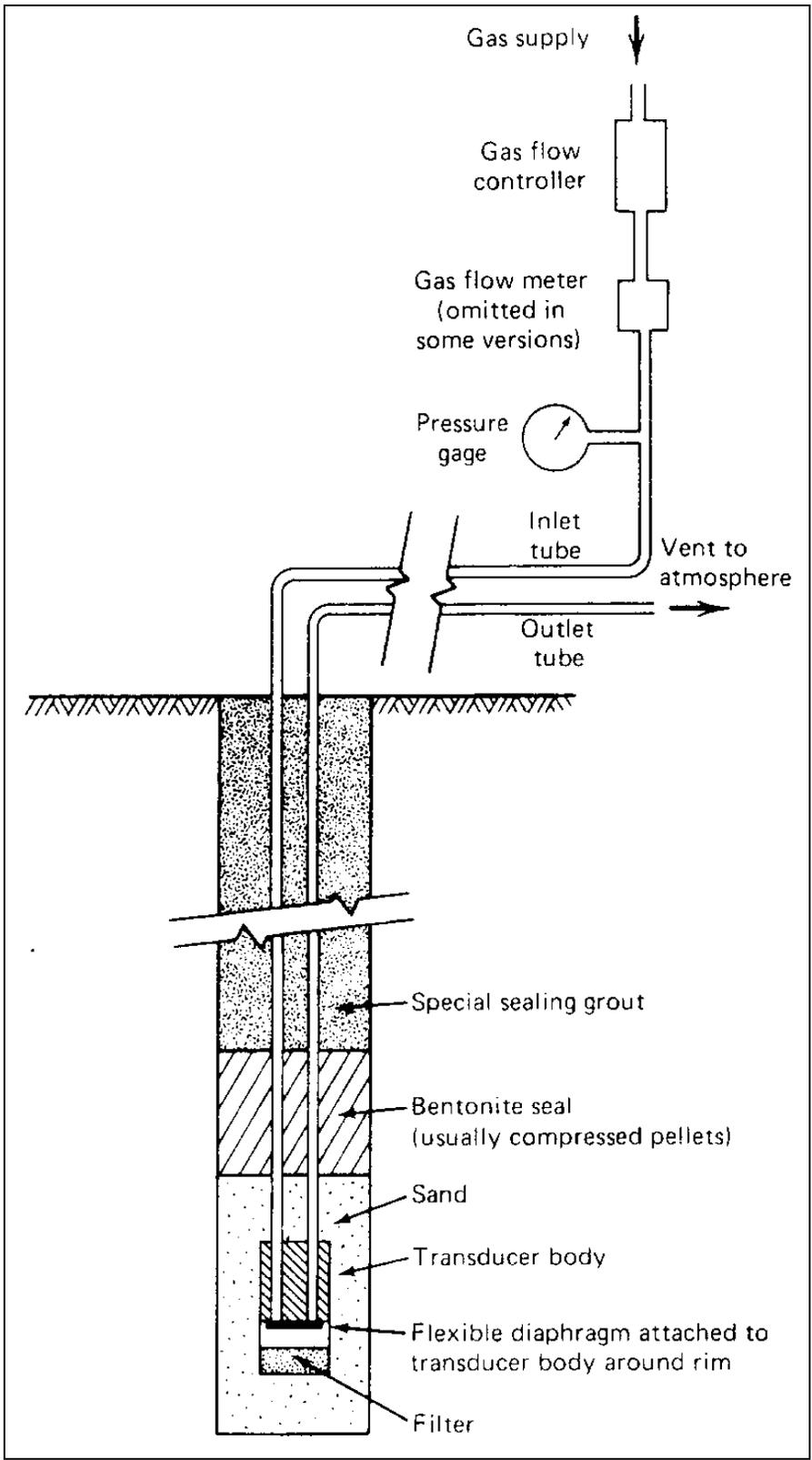


Figure 25, Typical Pneumatic Piezometer (After Dunnicliff, 1988)

7.2 References

1. Cheney, Richard S. & Chassie, Ronald G., Soils and Foundations Workshop Manual – Second Edition, FHWA HI-88-009, 1993.
2. Dunicliff, John, Geotechnical Instrumentation for Monitoring Field Performance, Wiley-Interscience, New York, 1993.
3. Roadway and Traffic Design Standards, Florida Department of Transportation, (Current version).
4. Geotechnical Instrumentation, FHWA-HI-98-034, 1998 (Pending)

7.3 Specifications and Standards

<u>Subject</u>	<u>ASTM</u>	<u>AASHTO</u>	<u>FM</u>	<u>STD. INDEX</u>
Settlement Platform	-	-	-	540
Pore Pressure	-	T 252	-	-
Standard Test Method for Rock Mass Monitoring Using Inclinometers	D 4622	T 254	-	-

Chapter 8

8 Analysis and Design

Once all exploration and testing have been completed, the Geotechnical Engineer must organize and analyze all existing data and provide design recommendations. The scope of the analysis will of course depend upon the scope of the project and the soils involved.

This chapter will discuss the major factors, which must be considered during the analysis and design phase and possible methods of solving potential problems. **Table 2** and **Table 3** provide guidelines as to the types of analyses, which should be performed. The references cited in the text provide suggested methods of analysis and design. A list of computer programs, which are used by the Department to aid analysis, is given by **Table 4** through **Table 12**.

In using these references and computer programs, the engineer should remember that engineering technology progresses rapidly and those methods are being improved or new methods introduced frequently. The engineer should keep abreast of the state-of-the-art in order to produce the most efficient and economical designs; although, the engineer needs to consult with the District Geotechnical Engineer when new techniques are to be utilized. The suggested references, programs, and solutions represent only a few possibilities and should by no means be considered exhaustive.

8.1 Roadway Embankment Materials

The suitability of in-situ materials for use as roadway embankment is determined by analysis of the results of soil survey explorations. Embankment materials must comply with Standard Indexes 500 and 505.

The subsurface materials identified during soil survey explorations should be classified, usually according to the AASHTO classification system, and stratified. Soils must be stratified such that similar soils are contained within the same stratum. Stratifications shall be based upon the material utilization requirements of Standard Indexes 500 and 505. If testing identifies dissimilar types within the same stratum, additional sampling and testing may be required to better define the in-situ materials. Restratification may be required. On occasion, dissimilar soil types may be grouped for such reasons as borderline test results or insufficient quantities of in-situ material to economically justify separation during construction. These cases should be the exception, not the norm. Some engineering judgment must undoubtedly be used in stratifying soil types. All conclusions should be clearly explained and justified in the geotechnical report. In all cases, the soil stratifications must meet the approval of the District Geotechnical Engineer.

Once stratified, each stratum must be analyzed to define characteristics that may affect the design. Such characteristics include:

8.1.1 Limits of Unsuitable Materials

The limits of all in-situ materials considered unsuitable for pavement embankments should be defined and the effect of each material on roadway performance should be assessed. Refer to Standard Indexes 500 and 505 for requirements on excavation and replacement of these materials. In areas where complete excavation is not required but the potential for problems exists, possible solutions to be considered include stabilization with lime, cement, or flyash, placement of geotextile, surcharging, and combinations of these and other methods.

8.1.2 Limerock Bearing Ratio (LBR)

A design LBR value should be chosen based on test results and the stratification of subsurface materials. The design value should be representative of actual field conditions. One of the two following methods is generally applied to the LBR test data to account for variabilities in materials, moisture contents, and field versus laboratory conditions.

8.1.2.1 $\pm 2\%$ of Optimum Method

The LBR values corresponding to moisture contents of +2% optimum and -2% optimum of the maximum LBR value are found (Refer to **Table 13**). The average of these values is used as the design LBR. It may be substantially lower than the average of the maximum LBRs.

8.1.2.2 90% Method

Maximum LBR values are sorted into ascending or descending order. For each value, the percentage of values, which are equal to or greater than that value, is calculated. These percentages are plotted versus the maximum LBR values. The LBR value corresponding to 90% is used as the design value (Refer to **Figure 26**). Thus, 90% of the tests result in a maximum LBR equal to or greater than the design value.

8.1.3 Corrosivity

Results of field and/or laboratory tests should be reviewed and the potential for corrosion of the various structure foundation and drainage system components should be assessed.

8.1.4 Drainage

The permeability and infiltration rate of the embankment materials should be estimated based on test results or knowledge of the material characteristics. This data, along with data on the depth to groundwater, can then be used in assessing the need for and in designing drainage systems, including pavement underdrains and retention, detention, and infiltration ponds.

8.1.5 Earthwork Factor

Earthwork factors used in estimating cut and fill quantities should be estimated based on local experience.

8.1.6 Other Considerations

Other characteristics which can be detected from soil survey explorations and which can affect the roadway design include expansive soils, springs, sinkholes, potential grading problems due to the presence of rock, etc. The effect of these characteristics on roadway performance should be assessed.

8.2 Foundation Types

As an absolute minimum, spread footings, driven piles and drilled shafts should be considered as potential foundation types for each structure. For sound barrier walls auger-cast piles may be the preferred foundation. On some projects, one or more of these alternatives will be obviously not feasible for the subsurface conditions present. Analysis of design capacity should be based on SPT and/or cone penetrometer results, laboratory and/or in-situ strength tests, consolidation tests, and the results of instrumentation programs, if available.

8.2.1 Spread Footings

The use of spread footings is generally controlled by the depth to material of adequate bearing capacity and the potential for settlement of footings placed at this depth.

8.2.1.1 Design Procedure

References 3, 5, 6 and 24 offer good methods. Reference 6 was developed specifically for the Florida Department of Transportation.

8.2.1.2 Considerations

Varying depths of footings should be considered to achieve maximum economy of design. For water crossings, depth of scour will be a controlling factor, which may preclude consideration of spread footings. Settlement possibilities, including the amount of total settlement, rate of settlement, and the potential for differential settlement, should be addressed. Ground improvement methods which permit the use of spread footings in otherwise marginal cases (grouting, vibratory compaction, etc.) should be considered where their use might be more economical than deep foundations.

8.2.2 Driven Piles

Driven piles must be designed for axial and lateral loading conditions as applicable.

8.2.2.1 Design Procedure

References 3, 6, 7 and 8 are all recommended. Reference 7 in particular gives an excellent overview of design procedures. Static analysis computer programs are available for assessment of axial design capacity.

8.2.2.2 Considerations

Various pile types and sizes should be analyzed to achieve an optimum design. For water crossings, depth of scour must be considered for both axial and lateral load analyses. Pile group effects, settlement and downdrag should be addressed as applicable. Test pile locations should be recommended and the need for static and/or dynamic testing addressed. The driveability of the piles should be considered. See the Structures Design Guidelines for load limits of different pile sizes.

8.2.3 Drilled Shafts

As with driven piles, drilled shafts must be designed considering both axial and lateral loads.

8.2.3.1 Design Procedure

Reference 9 is a comprehensive study.

8.2.3.2 Considerations

Various drilled shaft sizes should be analyzed to achieve an optimum design. For water crossings, depth of scour must be considered. Allowable settlement and any anticipated construction problems should be considered. The method of construction (dry, slurry, or casing) should be addressed, as this will affect the side friction and end bearing values assumed during design. Both the unit side friction and mobilized end bearing values should be analyzed and presented. Load tests on test shafts should be specified when necessary to verify capacity and/or constructability. Test shafts (test holes) are always required for bridges, and their locations should be specified in the plans. Refer to the Structures Design Guidelines for additional considerations.

8.2.4 Auger-Cast-Piles

As with driven piles and drilled shafts, auger-cast-piles must be designed considering both axial and lateral loads however lateral loads typically governs. Auger-cast-piles may not be used for bridge foundations.

8.2.4.1 Design Procedure

Reference 27 is a comprehensive study.

8.2.5 Micro Piles

In special cases micro piles may be the preferred foundation system. This would typically be in cases of limited access at foundations that are to be strengthened.

8.2.5.1 Design Procedure

Reference 28 is a comprehensive study.

8.3 Foundation Analysis

Along with an axial analysis (as outlined in the previous section) for deep foundations, the following factors must also be addressed.

8.3.1 Lateral Loads

Lateral load analyses for deep foundations shall be performed on all retaining structures and almost all bridges permitting navigation. The Structural Engineer using soil parameters provided by the Geotechnical Engineer shall perform the analyses for bridges. The Geotechnical Engineer shall check the final lateral load analysis for correct soil property application. The associated minimum tip elevations requirement (elevation where structure stability is achieved plus 10 feet {3 meters}) must be reviewed.

Designs may need to be changed if lateral deflection is excessive. Reference 10 is recommended.

8.3.2 Scour

For structures over water, scour susceptibility may control the design. All new structures shall be designed in accordance with FHWA Technical Advisory T5140.20.

Design for scour requires coordination of efforts between the Hydraulics Engineer, Geotechnical Engineer, and the Structures Engineer. This multi-discipline effort, which is needed for the proper iterative procedure used for scour design, is described in the FDOT Structures Design Guidelines.

8.3.3 Downdrag

For piles driven through a compressible soil layer(s), which is subject to consolidation, a load transfer (negative skin friction) occurs due to the compressible soil settling around the pile. The downward forces created by this process are known as downdrag. The results of these downdrag forces can be either excessive settlements or overstressing the pile if it is an end bearing pile.

Driving additional piles to carry these additional downdrag loads is expensive. To minimize the downdrag forces, bitumen coatings may be used to reduce the load transferred by the compressing soil(s), but a means for protecting this coating during driving must be used. Other means to reduce downdrag is to use a polyethylene wrap around the pile within the embankment fill after driving,

or to place the embankment fill and allow the compressible soil(s) to consolidate prior to driving. The Geotechnical Engineer shall provide the downdrag values along with recommended methods to reduce the effect of downdrag.

8.3.4 Construction Requirements

This would identify any project specific requirements that may be required for constructability. This would include items like preaugaring, jetting, vibration monitoring artesian water, etc. It would also identify any nearby structures and occupants usages that would be impacted from the installation of the foundations and special techniques required to minimize these impacts.

8.4 Embankment Settlement/Stability

These factors should be addressed concurrently, as various options to solve settlement problems will also impact stability.

8.4.1 Settlement

Settlement calculations should be based on the results of consolidation tests performed on high-quality samples. For embankments over soft soils requiring reinforcement, see Roadway and Traffic Design Standards Index 501 for standard details.

8.4.1.1 Design Procedure

References 3 and 11 are recommended.

8.4.1.2 Considerations

The results of consolidation calculations should be plotted on a time-settlement curve. If excessive settlement over too lengthy a time period is predicted (the criteria can vary) the engineer must propose a method of dealing with the problem. Not every possible solution is applicable to every project because of constraints of construction time, stability, etc. The Geotechnical Engineer may also need to design and monitor a field instrumentation program.

8.4.1.3 Possible Solutions

1. Reduce fill height. This is seldom practical except in planning phase.
2. Provide waiting period to allow for the majority of consolidation to occur.
3. Increase surcharge height.
4. Use a lightweight fill.
5. Install wick drains within the compressible material.
6. Excavate soft compressible material and backfill with granular soil.

7. Ground modification such as stone columns, dynamic compaction, etc.
8. Combinations of some of the above.

8.4.2 Stability

Stability analyses are performed based on the results of in-situ strength tests and/or laboratory strength tests on high quality samples. A range of possible material strengths is often considered, thus providing the engineer with a range of soil resistance from which to judge the stability of the slope. Any construction or utility placement that will require trenching or excavation will need a stability analysis.

8.4.2.1 Design Procedure

References 3 and 20 are recommended. Various computer programs are available to assist in the analysis.

8.4.2.2 Considerations

Soil resistance should be calculated for all possible slope conditions (i.e., surcharge loading, varying fill heights and/or slopes, varying water tables, etc.). The engineer must design a method of dealing with potential stability problems and may need to design and monitor a field instrumentation program.

8.4.2.3 Possible Solutions

1. Realign highway.
2. Reduce fill height.

Note: These first two solutions are seldom practical unless the problem is identified early in the planning phase.
3. Flatten slope (Right of way requirements?).
4. Staged construction, to allow soft soil to gain strength through consolidation.
5. Excavate and replace soft soils.
6. Place berms at toe.
7. Use lightweight fills.
8. Ground modification such as stone columns, dynamic compaction, etc.
9. Include geotextile or geogrid within the embankment.
10. Combinations of some of the above.

8.5 Retaining Wall Design

All retaining walls; including gravity walls, cantilever walls, crib walls, and mechanically stabilized earth (MSE) walls and soil nail walls; must be designed with adequate soil resistance against bearing, sliding, overturning, and overall stability. A design analysis is still required when standard index walls are used on a project.

8.5.1 Design Procedure

Reference 5 is recommended. References 12, 13, 14, 15, 16, 17 and 19 are recommended for MSE walls. Reference 25 is recommended for soil nail walls.

8.5.2 Consideration

The use of proprietary MSE wall systems is growing more common as right-of-ways become limited and congestion grows. FDOT maintains a list of wall systems pre-approved for use as permanent and critical temporary walls.

For all proprietary systems, the Geotechnical Engineer is responsible for external stability and assuring that the design is compatible with the actual subsurface conditions. The system proprietor is responsible for internal stability. Control drawings will be provided to the proprietary wall companies, which indicate the minimum lengths of reinforcement required for external stability. Drawings produced by the proprietor will show the actual reinforcement lengths required. These lengths will be the longer of those required for external stability, as given by the Geotechnical Engineer, and those required for internal stability, as calculated by the proprietor. Refer to the FDOT Structures Design Guidelines and the FDOT Plans Preparation Manual for procedures on design of proprietary walls.

8.6 Steepened Slopes

All steepened slopes must be designed for external stability including all failure possibilities such as sliding, deep-seated overall instability, local bearing capacity failure at the toe (lateral squeeze), and excessive settlement from both short- and long-term conditions. Reinforcement requirements must be designed to adequately account for the internal stability of the slope. See Roadway and Traffic Design Standards Index 501 for standard details.

8.6.1 Design Procedure

References 13 and 17 are recommended for MSE walls.

Table 2, Geotechnical Engineering Analysis Required for Embankments, Cut Slopes, Structure Foundations and Retaining Walls (After FHWA, 1985)

GEOTECHNICAL ENGINEERING ANALYSIS REQUIRED FOR EMBANKMENTS, CUT SLOPES, STRUCTURE FOUNDATIONS, AND RETAINING WALLS					
Soil Classification	Embankment and Cut Slopes	Structure Foundations (Bridges & Retaining Walls)	Retaining Walls (Conventional, Gravity & Reinforced Soil)	Subsidence Earth Pressure	Stability Analysis
Unified AASHTO ¹ Soil Type	Slope Stability ² Analysis	Bearing Capacity Analysis	Settlement Analysis	Subsidence Earth Pressure	Stability Analysis
GM A-0-a CBWL well graded	Stability analysis generally not required if cut or fill slope is 1-1/2 to 1 to 1 or flatter, and water table in cut slope is drawn down by underdrains. Erosion of slopes may be a problem for SM or SP soils.	Required for spread footings, pile, or drilled shaft foundations. Spread footings generally adequate except possibly for SC soils.	Analysis generally not needed except for SC soils or for large, heavy structures. Empirical correlations with SPT values usually used to estimate settlement.	GM, SP, SM, & SC soils generally suitable for backfill behind or in retaining or overbearing walls. Minimum P.S. = 1.5 applied.	All walls should be designed to provide atleast F.S.=2 against overbearing & minimum P.S. = 1.5 applied.
GP A-1-a CBWL poorly graded				GM, SC, SM, & SC soils generally suitable or have less than 10% fines.	External slope stability considerations same as generally given for cut slopes & embankments.
GM A-1-b CBWL silty				External earth pressure analysis required using soil angle at internal friction.	
GC A-2-6 A-2-7 clayey					
SM A-1-b well graded					
SP A-3 SAND poorly graded					
SM A-2-8 A-2-5 silty					
SC A-2-6 A-2-7 clayey					
ML A-4 SILT In- organic Silt Sandy	Stability analysis required unless non-plastic. Erosion of slopes may be a problem.	Analysis required. Spread footings generally adequate.	Analysis required. Can use SPT values if non-plastic.	These soils are not recommended for use directly behind or in retaining or reinforced soil walls.	
OL A-6 CLAY In- organic Lean Clay	Required	Analysis required. Deep foundations generally required unless soil has been preloaded.	Analysis required. Lab consolidation test data needed to estimate settlement amount & time.		
OL A-4 SILT organic	Required	Required	Required		

¹ Approximate correlation to Unified Soil Classification System is preferred for geotechnical engineering usage - AASHTO system was developed for rating pavement subgrades.
² These are general guidelines - detailed slope stability analysis may not be required where past experience in area in similar soils or rock gives required slope angles.

Table 3, Geotechnical Engineering Analysis (Continued)

Soil Classification		Embankment and Cut Slopes		Structure Foundations (Bridges & Retaining Walls)		Retaining Walls (Conventional, Crib, & Reinforced Soil)	
Unified	AASHTO ¹	Slope Stability ²	Embankment Foundation Settlement Analysis	Bearing Capacity Analysis	Settlement Analysis	Lateral Earth Pressure	Stability Analysis
HI	A-5	SILT in-organic Stability analysis required. Erosion of slopes may be a problem.	Required	Analysis required. Deep foundations generally required unless soil has been pre-loaded.	Analysis required. Lab consolidation test data needed to estimate settlement amount and time.	These soils are not recommended for use directly behind or in retaining walls.	All walls should be designed to provide minimum F.S.=2 against overturning & minimum F.S.=1.5 against sliding along base. External slope stability considerations same as previously given for cut slopes & Embankments.
CH	A-7	CLAY inorganic "RC clays"	Required	Deep foundation required unless peat excavated and replaced.	Highly compressible not suitable for foundation support.		
OH	A-7	CLAY organic	Required	Analysis required for spread footings or drilled shafts - usually empirical related to RQD.3	Analysis only required where rock is badly fractured (low RQD value). May require special testing such as pressuremeter.		
PT	--	PEAT muck	Required	Deep foundation required unless peat excavated and replaced.	Highly compressible not suitable for foundation support.		
Rock		Fills- Analysis not required for slopes 1-1/2 H. to 1 W. or flatter Cuts- Analysis required but depends on spacing, orientation, and strength of discontinuities, and durability of the rock.	Required Long-term settlement can be significant.	Analysis required for spread footings or drilled shafts - usually empirical related to RQD.3	Analysis only required where rock is badly fractured (low RQD value). May require special testing such as pressuremeter.	Lateral earth pressure analysis required using rock backfill angle of internal friction.	

REMARKS:

Soils - Temporary groundwater control may be needed for foundation excavations in GM through SM soils.

Backfill specifications for reinforced soil walls using metal reinforcement should meet the following requirements to insure use of non-corrosive backfill:

1. pH 5-10
2. Resistance 3,000 ohm-cm
3. Chlorides 200 ppm
4. Sulfates 1,000 ppm

Rock - Durability of shales (silt-stones, clay-stones, mud-stones, etc.) to be used in fills, should be checked. Non-durable shales should be embanked as soils, i.e., placed in maximum 12" loose lifts & compacted with heavy sheepfoot or grid rollers.

¹Approximate correlation to Unified (Unified Soil Classification system is preferred for geotechnical engineering usage - AASHTO system was developed for rating pavement subgrades).

²These are general guidelines - detailed slope stability analysis may not be required where past experience in area in similar soils or rock gives required slope angles.

³RQD = Rock Quality Designation = Sum of pieces of rock core 4" or greater in length divided by total length of core run.

8.7 Computer Programs used in FDOT

Table 4, Driven Piles

SPT94/ SPT 97	Lai, P., et al., <u>Static Pile Bearing Analysis Program for Concrete & Steel Piles - SPT94, 1994/1997.</u> http://www.dot.state.fl.us/structures/index.htm	Computes static pile capacities based on SPT data. Used for precast concrete, or steel H- or pipe piles. PC-version of modified Bulletin RB-121-C.
CONEPILE	Malerk, T.O., <u>User's Manual - CONEPILE</u> , FDOT, 1980.	Computes static pile capacities based on cone penetrometer data. Developed for mechanical cone penetrometer data.
PL-AID	University of Florida, McTrans, Transportation Research Center, 1989.	Computes static pile capacities from CPT data, and predicts settlement based on SPT and CPT data. Used for precast concrete or steel pipe piles.
WEAP	Gobel, G.G. & Rausche, Frank, <u>WEAP 87, Wave Equation Analysis of Pile Foundations</u> , Volumes I-V, FHWA, 1987.	Dynamic analysis of pile capacity and drivability.
FLPier	University of Florida http://www.dot.state.fl.us/structures/index.htm	The Lateral Pile Group Structural Analysis Program is a 3-D nonlinear substructure analysis program.
PILE LOAD TEST DATA BASE	University of Florida, FDOT	Database consisting of results from in-situ tests and load tests. The program Access is used to review the data.

Table 5, Drilled Shafts

SHAFT98	University of Florida http://www.dot.state.fl.us/structures/index.htm	Computes drilled shaft capacities using FHWA method with modifications for shafts tipped in rock.
SHAFT	University of Florida, McTrans, Transportation Research Center, 1989	Lotus template for data reduction from drilled shaft load tests.
FLPier	University of Florida http://www.dot.state.fl.us/structures/index.htm	The Lateral Pile Group Structural Analysis Program is a 3-D nonlinear substructure analysis program.
Drilled Shaft Axial Load Test Database	University of Florida, FDOT	Data Consisting of results from in-situ tests and load tests. Requires Access database program.

Table 6, Lateral Loads

FLPier	University of Florida http://www.dot.state.fl.us/structures/index.htm	The Lateral Pile Group Structural Analysis Program is a 3-D nonlinear substructure analysis program.
COM624P	<u>COM624P - Laterally Loaded Pile Analysis Program for the Microcomputer, Version 2.0</u> , FHWA-SA-91-048, 1993. http://www.fhwa.dot.gov/bridge/software.HTM	Computes deflections and stresses for laterally loaded piles and drilled shafts.
Lateral Load Test Database	University of Florida	Database of lateral load tests. Database uses Excel.

Table 7, Spread Footings

CBEAR	<u>CBEAR Users Manual</u> , FHWA-SA-94-034, 1996. http://www.fhwa.dot.gov/bridge/software.HTM	Computes ultimate bearing capacity of spread or continuous footings on layered soil profiles.
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Table 8, Sheet Piling

CWALSHT	Dawkins, William P., <u>Users Guide: Computer Program For Design and Analysis of Sheet Pile Walls by Classical Methods</u> , Waterways Experiment Station, 1991.	Design and analysis either anchored cantilevered sheet pile retaining walls. Moments, shear, and deflection are shown graphically.
Shoring	Civil Tech, <u>CT-SHORING WINDOWS 3.X, 95, NT VERSION Users Manual</u>	Excavation supporting system design and analysis.

Table 9, Slope Stability (Programs are for ASD)

PCSTABL	<u>PC-STABL5M Users Manual</u> , FHWA, 1990. <u>PC-STABL6 Users Manual</u> , FHWA, 1990.	Calculates factor of safety against rotational, irregular, or sliding wedge failure by simplified Bishop or Janbu, or Spencer method of slices. Version 6 is used for embankments w/reinforcement by simplified Bishop method.
RSS	<u>RSS Reinforced Slope Stability A Mircocomputer Program User-s Manual</u> , FHWA-SA-96-039, 1997 http://www.fhwa.dot.gov/bridge/software.HTM	A computer program for the design and analysis of reinforced soil slopes (RSS Reinforced Slope Stability). This program analyzes and designs soil slopes strengthened with horizontal reinforcement, as well as analyzing unreinforced soil slopes. The analysis is performed using a two-dimensional limit equilibrium method.
XSTABL	Interactive Software Designs, Inc., <u>XSTABL An Integrated Slope Stability Analysis Program for Personal Computers Reference Manual</u> .	Program performs a two dimensional limit equilibrium analysis to compute the factor of safety for a layered slope using the modified Bishop or Janbu methods.

Table 10, Embankment Settlement

EMBANK	<u>EMBANK Users Manual, FHWA-SA-92-045, 1993.</u>	Calculates compression settlement due embankment loads.
DILLY	University of Florida, McTrans Transportation Research Center, 1989.	Reduces data from dilatometer tests and calculates settlements of footings and embankments.

Table 11, Soil Nailing

GoldNail	Golder Associates, <u>GoldNail A Stability Analysis Computer Program for Soil Nail Wall Design Reference Manual Version 3.11</u>	The program is a slip-surface, limiting-equilibrium, slope-stability model based on satisfying overall limiting equilibrium (translational and rotational) of individual free bodies defined by circular slip surfaces. GoldNail can analyze slopes with and without soil nail reinforcement or structural facing.
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Table 12, MSE Walls and Steepened Slopes

MSEW 1.0	ADAMA Engineering, Inc., <u>Mechanically Stabilized Earth Walls Software Version 1.0</u>	The program can be applied to walls reinforced with geogrids, geotextiles, wire mesh, or metal strips. It allows for reduction factors associated with polymeric reinforcement or for corrosion of metallic reinforcement.
RSS	Reinforced Steepened Slopes	A computer program for the design and analysis of reinforced soil slopes (RSS Reinforced Slope Stability). This program analyzes and designs soil slopes strengthened with horizontal reinforcement, as well as analyzing unreinforced soil slopes. The analysis is performed using a two dimensional limit equilibrium method.

NOTE:

- 1) The programs included in this list are generally available from public sources. Many additional programs, which perform similar tasks, can be obtained from the private sector.
- 2) Many of the programs listed are continually updated or revised. It is the user's responsibility to become familiarize with the latest versions.
- 3) FDOT's programs are available on the FDOT's Structures Internet site. The address is: <http://www.dot.state.fl.us/structures/>

Table 13, $\pm 2\%$ of Optimum Method

TEST NO.	MAXIMUM LBR	LBR AT MOISTURE CONTENTS: (OF OPTIMUM LBR)	
		- 2%	+ 2%
1	165	30	18
2	35	25	25
3	64	60	55
4	35	12	8
5	85	20	45
6	55	45	20
7	33	7	10
MEAN LBR VALUE:	67.42	28.42	24.42
AVERAGE = 26.42 (26) => DESIGN LBR = 26			

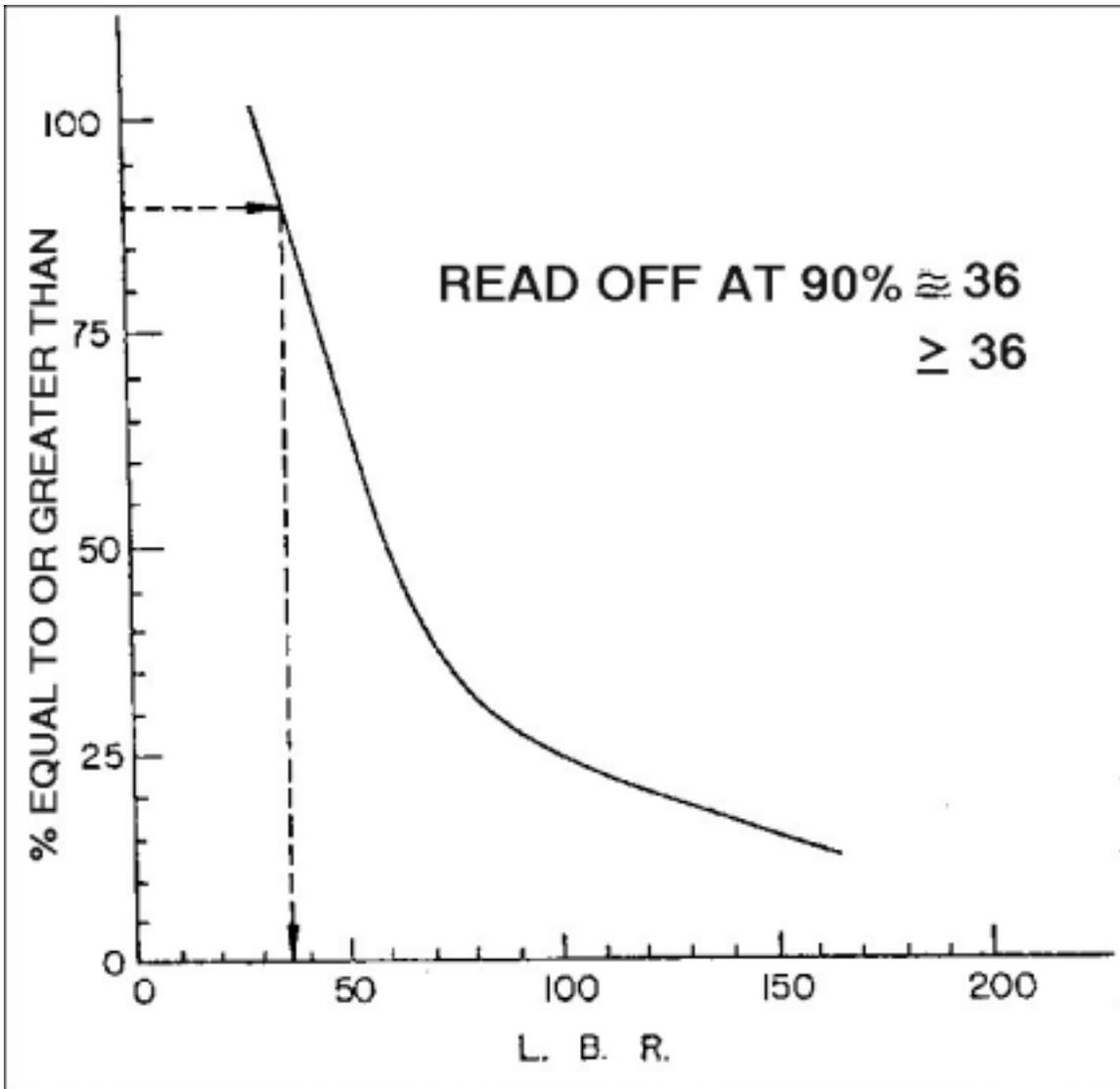


Figure 26, Design Example 1 (LBR Design Methods) 90% Method

8.8 References

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Chapter 9

9 Presentation of Geotechnical Information

Upon completion of the subsurface investigation and analysis, the information, which has been obtained, must be compiled in a format, which will present to others the results of the work, which has been performed. This compilation will serve as the permanent record of all geotechnical data known to be pertinent to the project and will be referred to throughout the design, construction, and service life of the project. It is perhaps the most critical function of the geotechnical process.

The data is typically compiled in a geotechnical report. The purpose of the geotechnical report is to present the data collected in a clear manner, to draw conclusions from the data and to make recommendations for the geotechnical related portions of the project. The format and contents of the geotechnical report are somewhat dependent on the type of project. Most projects will generally require either a roadway soil survey or a structure related foundation investigation, or both. For reports prepared by consultants the consultant's recommendations shall be documented and retained. The department's final decision shall be documented separately (i.e. in letter form to the structures engineer in charge of the project).

This chapter describes the format for presentation of geotechnical data for each type of project. General outlines of the topics to be discussed in the geotechnical report are presented. Not every project will follow these formats exactly, however; for any given project, certain items may be unnecessary while other items will need to be added. Also included in this chapter are discussions on the finalization and distribution of the geotechnical report and on the incorporation of its recommendations into the design.

9.1 Roadway Soil Survey

The geotechnical report for a roadway soil survey present conclusions and recommendations concerning the suitability of in-situ materials for use as embankment materials. Special problems affecting roadway design, such as slope stability or excessive settlement may also be discussed if applicable. The following is a general outline of the topics, which should be included.

9.1.1 General Information

- a. List of information provided to the geotechnical consultant (alignment, foundation layout, 30% plans, scour estimate, etc.).
- b. Description of the project, including location, type, and any design assumptions.
- c. Description of significant geologic and topographic features of the site.
- d. Description of width, composition, and condition of existing roadway.
- e. Description of methods used during the subsurface explorations, in-situ testing, and laboratory testing.

- f. Soil conservation (SCS/USDA) and USGS maps.

9.1.2 Conclusion and Recommendations

- a. Explanation of stratification of in-situ materials including ground water table.
- b. Evaluation of strength and extent of unsuitable soils within the proposed alignment including their probable effect on roadway performance. The extent of removal of the unsuitable material should be stated. Recommendations for special construction considerations, which minimize anticipated problems should be included.
- c. Recommended design LBR based on the most conservative value from either the 90% Method or the $\pm 2\%$ of Optimum LBR Method.
- d. Estimated soil drainage characteristics and permeability or infiltration rates. In the case of rigid pavement design, include average laboratory permeability values for each stratum based on the requirements given in the Rigid Pavement Design Manual.
- e. Recommendations for cut or fill sections when seepage, stability or settlements are significant.
- f. Recommendations for any cast-in-place or MSE walls.
- g. Any storm water retention pond considerations.
- h. Effect of roadway construction (vibratory rollers, utility excavations, etc.) on surrounding structures and effect on the usage of the structures during roadway construction.

9.1.3 Roadway Soils Survey Sheet

This sheet presents a material description and results of classification and corrosivity tests for each stratum. Recommendations for material utilization in accordance with Standard Indexes 500 and 505 are provided. Visual classification of muck is not sufficient; organic content test results should be included in the material description. The number of lab tests runs for each stratum shall be included for corrosion tests results as well as classification tests. Include the range of values of all tests performed for each stratum. The Report of Test Results sheet is included in the construction plans. **Figure 27** is an example of a typical test results sheet.

9.1.4 Roadway Cross Sections

Simplified boring logs are plotted on the cross section sheets included in the construction plans. Each material stratum is numbered corresponding to the strata on the test results sheet. **Figure 28** is an example of a typical cross sections sheet. If cross sections sheets are to be prepared by others, the appropriate subsurface information should be provided. The Geotechnical Engineer should

then verify that the data has been correctly incorporated. Removal of unsuitable materials should be indicated on the cross sections.

9.2 Structures Investigation

9.2.1 Introduction

The geotechnical report for a structure presents the conclusions and recommendations for the most suitable foundation types and information required for incorporating such foundations into the design of the structure. Recommendations for related work, such as approach embankments and retaining walls, are also included. Special construction considerations are noted. Items stated in the FDOT Specification 455 shall not be repeated and copied into the report. Only the site-specific items should be recommended for the special provisions. The following is a general guide to the contents of a typical structure foundation report.

9.2.2 Scope of Investigation

- a. Description of type of project, location of project, and any assumptions related to the project.
- b. Vicinity map, including potentiometric map, USGS and soil survey maps (SCS/USDA), depicting project location.
- c. Summary of general content of report.

9.2.3 Interpretation of Subsurface Conditions

- a. Description of the methods used in the field investigation, including the types and frequencies of all in-situ tests.
- b. Description of the laboratory-testing phase, including any special test methods employed.
- c. Boring location plan and plots of boring logs and cone soundings. Note the size of rock core sampled, and the minimum acceptable rock core diameter to be used shall be 2.4 inch (61 mm) (although 4 inch {101.6 mm} diameter rock cores are preferable). See **Figure 29** and **Figure 30** for examples of Report of Core Borings and Report of Cone Soundings sheets. These sheets are included in the final plans. Standard soil type symbols used in plotting the borings are shown in **Figure 31**.
- d. Estimated depths of scour used (usually determined by the Hydraulics Engineer), if applicable.
- e. Environmental class for both substructure and superstructure, based on results of corrosivity tests. This information is also reported on the Report of Core Borings sheet. For extremely aggressive classification note what parameter placed it in that category.

- f. Summary table of soil parameters determined from field and laboratory testing.
- g. Table of soil parameters to use with computer modeling (such as the FLPIER program). These parameters can be broken up into zones across the bridge length.
- h. MSE or cast-in-place wall recommendations.

9.2.4 Existing Structures Survey and Evaluation

Structures in close proximity to construction activities should be evaluated for potential damages caused by these activities. The usage of the structures should also be included in this evaluation. This needs to happen early in the design process. Vibration, settlement, noise and any other damaging results of these construction activities should be considered in the evaluation. When warranted, the recommendations should include possible means of reducing the damaging effects of the construction activity, such as time restraints on certain operations, underpinning, monitoring, or even purchasing of the property. **Table 14** shows what is needed in a report. **Table 15** and the notes that follow are examples of what may be shown on the plan sheets.

Where there is a potential impact on existing structures in the surrounding area, the report should include the structures address, type of construction, the estimated vibration level that may cause damage, the usage (storage building, hospital, etc.), what the potential problem may be and what actions should be taken to minimize the impact.

Table 14, Existing Structures Evaluation Table for Geotechnical Report

Address	Structure Type	Potential Vibration Damage Level	Structure Usage	Potential Problem	Recommendation
230 Walnut Street	Concrete	4 in/sec	Storage Units	Damage from vibration	Vibration monitoring during installation of piers 3 – 7.
235 Walnut Street	Brick	1.5 in/sec	House	Damage from vibration	Vibration monitoring during installation of piers 13 – 14.
238 Spruce Ave.	Concrete	2 in/sec	Hotel	Noise	Limit pile drive from 9 am to 7 pm
245 Spruce Ave.	Stucco	0.75 in/sec	House	Vibration causing cracking of stucco	Pre & Post survey, repair any new cracks.

Table 15, Plan Note and Table for Existing Structures

Address	Structure Type	Structure Usage	
230 Walnut Street	Concrete	Storage Units	Perform vibration and settlement monitoring during the installation of piers 3-7
235 Walnut Street	Brick	House	Perform vibration and settlement monitoring during the installation of piers 13-14

Typical Notes:

Noise Restrictions: The contractor shall strictly adhere to all local noise ordinances. All pile driving operations shall be limited to the hours of 7:00 am to 6 pm. Methods of maintaining construction noise levels may include but not be limited to temporary noise barriers, enclosures for equipment, mufflers, etc. There will be no separate payment for any of these measures.

Vibration: The contractor shall provide surveys and settlement/vibration monitoring of the existing structures listed, as per FDOT Standard Specifications. The cost of all vibration monitoring as required here and specified in Section 455 shall be paid for under Pay Item No. 455-18, Protection of Existing Structures.

9.2.5 Structure Foundation Analysis and Recommendations

Alternate foundation recommendations should be provided for all structures including recommendations for spread footings, driven piles, and drilled shafts. An explanation should be included for any of these alternates judged not to be feasible. The types of analyses performed should be summarized.

9.2.5.1 Spread Footings

1. Reason(s) for selections and exclusions.
2. Elevation of bottom of footing or depth to competent bearing material.
3. Allowable soil pressure based on settlement and bearing capacity.
4. Settlement potential.
5. Recommendation for special provisions for footing construction, including compaction requirements and the need for particular construction methods such as dewatering or proof rolling.
6. Sinkhole potential.
7. Soil improvement method(s).

9.2.5.2 Driven Piles

1. Suitable pile types and reasons for design selections and exclusions.
2. Plots of soil resistance for selected pile size alternates. Plots should be developed indicating both Davisson's curve and ultimate soil resistance versus elevation and should show end bearing and skin friction as well as total resistance. The Davisson capacity is equivalent to the LRFD's nominal ultimate resistance (R_n). To use this curve for design, the engineer should enter the factor design load divided by the resistance Φ factor. The ultimate bearing capacity curve is used to examine the Required Driving Resistance to see if it is exceeding the ultimate resistance defined in the Structures Design Guideline for the pile size(s) used. Various plots as per the FDOT Structures Design Guidelines shall also be included. Depth of scour should be accounted for on each plot.

Unless otherwise specified, separate pile analyses for recommended pile sizes are to be performed for each SPT boring and/or CPT sounding. A corresponding pile capacity curve for each analysis must also be provided. When more than one boring is taken at a pile group or when it is appropriate to otherwise generalize the soil strata, the corresponding pile capacity curves are to be shown on the same plot and a recommended relationship established for that particular structure(s). Refer to the FDOT Structures Design Guidelines.

3. Recommendations for minimum pile length or bearing elevation (non-lateral).
4. Minimum pile spacing shall be at least three times the diameter/width of the pile size used.
5. Estimated pile settlement or pile group settlement, if significant.
6. Effects of scour, downdrag, and lateral squeeze, if applicable.
7. Maximum driving resistance to be encountered in reaching the estimated bearing elevation. Capacity analyses may have included the estimated scour of substantial amounts of materials. The presence of this material during driving will contribute added resistance to driving.
8. Recommended locations of test piles and pile installation criteria for dynamic monitoring.
9. Selection of load test types, locations and depths where applicable. For static, Statnamic or Osterberg load testing the ultimate load the test should be taken to must be shown in the plans (this should be a minimum of 3 times the design load for ASD design or 2 times the factored design load for LFD or LRFD designs.).
10. Recommendations for special provisions for pile installation (special needs or restrictions). Special construction techniques may be

needed to minimize the effects of foundation installation discussed in **Section 9.2.4.**

11. Present recommendations for information to be placed in the pile installation table when applicable as recommended in FDOT Structures Design Guidelines.
12. Present soil parameters to be used for lateral analysis accounting for installation techniques and scour. The Geotechnical Engineer shall check the final lateral load analysis for correct soil property application.

9.2.5.3 Drilled Shafts

1. Include plots of soil resistance versus elevation for selected alternate shaft sizes. Plots should be developed for both factored ($\phi \square Q_{ult}$) and ultimate soil resistance and should show end bearing, skin friction and total resistance (end bearing should not be discounted). Depths of scour analyzed should be included.

Unless otherwise specified, separate shaft analyses for the recommended shaft sizes are to be performed for each SPT boring and/or CPT sounding. A soil resistance versus elevation curve for each analysis must also be provided. When more than one boring is taken at a shaft group or when it is appropriate to otherwise generalize the soil strata, the corresponding soil resistances versus elevation curves are to be shown on the same plot and a recommended relationship established for that particular structure(s). The unit skin friction and end bearing values used for the analyses should be presented.

2. Recommendations for minimum shaft length or bearing elevation, for shaft diameter, and design soil resistance. The minimum socket length should be indicated, if applicable (non-lateral).
3. Minimum shaft spacing or influence of group effects on capacity.
4. Effects of scour, downdrag, and lateral squeeze, if any.
5. Estimated drilled shaft settlement or shaft group settlement.
6. Recommended locations of test. Selection of load test types, locations and depths. For static, Statnamic or Osterberg load testing the ultimate load the test should be taken to must be shown in the plans (this should be a minimum of 3 times the design load for ASD design or 2 times the factored design load for LFD or LRFD designs (provided there is equipment capable of performing the test to that load)).
7. Recommendations for special provisions for shaft installation (special needs or restrictions). Special construction techniques may

be needed to minimize the effects of foundation installation discussed in **Section 9.2.4**.

8. As recommended in the FDOT Structures Design Guidelines, when applicable, present recommendations for information to be placed in the drilled shaft installation table.
9. Include the potentiometric Surface Map information.
10. Present soil parameters to be used for lateral analysis accounting for installation techniques and scour. The Geotechnical Engineer shall check the final lateral load analysis for correct soil property application.

9.2.6 Approach Embankments Considerations

9.2.6.1 Settlement

1. Estimated magnitude and rate of settlement.
2. Evaluation of possible alternatives if magnitude or time required for settlement is excessive and recommended treatment based on economic analysis, time and environmental constraints.

9.2.6.2 Stability

1. Estimated factor of safety.
2. Evaluation of possible treatment alternatives if factor of safety is too low. Recommended treatment based on economic analysis, time and environmental constraints.

9.2.6.3 Construction Considerations

1. Special fill requirements and drainage at abutment walls.
2. Construction monitoring program.
3. Recommendations for special provisions for embankment construction.

9.2.7 Retaining Walls and Seawalls

- a. Recommended wall type.
- b. Recommended lateral earth pressure parameters.
- c. Factored soil resistance or alternate foundation recommendations.
- d. Settlement potential.
- e. Factored soil resistance and loads with respect to sliding and overturning (including standard index wall designs).
- f. Overall stability of walls.

- g. Recommendations for special provisions for fill material (except MSE walls), drainage.
- h. Special considerations for tiebacks, geotextiles, reinforcing materials, etc., if applicable.
- i. MSE reinforcement lengths required for external stability, if applicable. See the FDOT Structures Design Guidelines and the FDOT Plans Preparation Manual for details.

9.2.8 Steepened Slopes

- a. Estimated factor of safety for internal and external stability.
- b. Spacings and lengths of reinforcement to provide a stable slope.
- c. Design parameters for reinforcement (allowable strength, durability criteria, and soil-reinforcement interaction). (See Roadway and Traffic Design Standards Index 501)
- d. Fill material properties.
- e. Special drainage considerations (subsurface and surface water runoff control).

9.2.9 Technical Special Provisions

The department has available a number of Technical Special Provisions for various items of work. These Technical Special Provisions can be obtained from the District Geotechnical Engineer and include:

- a. 119 Dynamic Compaction
- b. 120 Surcharge Embankment
- c. 141 Settlement Plate Assemblies
- d. 144 Digital Inclinometer Casing And Pore-Pressure Transducers Assembly
- e. 442 Vertical Plastic Drainage Wicks
- f. 455 Crosshole Sonic Logging
- g. 455 Osterberg Load Test
- h. 455 Statnamic Load Test

9.2.10 Appendix

All structure investigation reports should include an appendix, which contains the following information.

- a. Report of Core Boring Sheets. (See **Figure 29**) (Note the FDOT Geotechnical CADD Standard menu is available for Microstation.)
- b. Report of Cone Sounding Sheet. (See **Figure 30**)

- c. Data logs or reports from specialized field tests.
- d. Laboratory test data sheets. The following are examples of what should be provided.
 - 1. Rock Cores: Location, elevation, Maximum Load, Core Length, Core Diameter, Moist Density, Dry Density, Split Tensile Strength, Unconfined Compressive Strength and Strain at Failure.
 - 2. Gradations: Location, elevation, test results.
 - 3. Corrosion Tests: Location, elevation, test results.
- e. Engineering analyses and notes.
- f. FHWA checklist.
- g. Any other pertinent information.

9.3 Final or Supplementary Report

To obtain the optimum benefit from the geotechnical investigation, it is imperative that the Geotechnical Engineer and the project design and construction engineers interact throughout the duration of the project. The input from the Geotechnical Engineer should be incorporated into the project as it develops. Often, the geotechnical report, which is initially prepared, is considered preliminary. As the design of the project progresses, the geotechnical recommendations may have to be modified. When the project approaches the final design stage, the Geotechnical Engineer should prepare a final or supplementary report to revise his assumptions and recommendations if necessary in accordance with the final design plans. The following topics should be included in this report.

- 1. Final recommended foundation type and alternates.
- 2. Size and bearing elevation of footing or size, length, and number of piles or drilled shafts at each structural foundation unit.
- 3. Final factored design loads.
- 4. Requirements for construction control for foundation installation.
- 5. Possible construction problems, such as adjacent structures, and recommended solutions.

If revisions to the preliminary report are not necessary, a letter should be submitted stating that the initial report is final.

9.4 Signing and Sealing

Geotechnical documents shall be signed and sealed by the Professional Engineer in responsible charge in accordance with Florida Statutes and the Rules of the State Board of Professional Engineers. The following documents are included:

Table 16, Signing and Sealing Placement

Placement of Signature/Seal Geotechnical Report	First page of official copy
Supplemental Specifications and Special Provisions	First page of official copy
Roadway Soils Survey Sheet	Title Block
Report of Core Borings Sheet	Title Block
Report of Cone Soundings Sheet	Title Block

For supplemental specifications and special provisions, which cover other topics in addition to Geotechnical Engineering, the engineer in responsible charge of the geotechnical portions should indicate the applicable pages.

Originals of the sheets for plans shall be signed and dated by the responsible engineer within the space designated "Approved By". One record set of full size prints shall be signed, sealed, and dated.

9.5 Distribution

The following offices should be provided copies of geotechnical reports, as applicable.

1. Project Manager.
2. District Geotechnical Engineer.
3. District Design Engineer.
4. District Structural Design Section.
5. Roadway Design Section.
6. State Geotechnical Engineer (for Category II structures).

9.6 Plan and Specification Review

In addition to writing a report, the Geotechnical Engineer should review all phases of the plans and specifications to ensure that the geotechnical recommendations have been correctly incorporated.

FDOT Standard and Supplemental Specifications should not be changed except in rare cases, then only with the approval of the District Geotechnical Engineer.

9.7 Electronic File

The consultant shall submit an electronic copy of the final approved geotechnical report in either WordPerfect or Word format. It shall include the boring log sheets in DGN format, and it shall also include the input files used in the analysis programs (SPT97, Shaft98, FLPier, etc.).

9.8 Units

For Metric projects all test results units presented in geotechnical reports shall be in accordance with ASTM SI-10, Standard Practice for Use of the SI International System of Units (SI): The Modernized Metric System and ASTM E 621 Standard Practice for the Use of Metric (SI) Units in Building Design and Construction.

9.9 Unwanted

Some of the things we do not wish to see in the report are:

1. Do not include FDOT specifications in the report. The specification can be referenced by number or section number but do not put the specification in the report we can look it up if need be.
2. Changes to the specification without valid justification. For example do not change the MSE wall backfill gradation, base your design on our specified material.
3. Long verbal descriptions when a simple table will do it better.

STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

DATE OF SURVEY: 9/30-10/27/87
B.Z.S.-2/94
 SURVEY MADE BY: STUBILING, Kirkland
JOHN HUBBELLING
 SUBMITTED BY: J.N. Puckett, P.E.

DISTRICT: One
 ROAD NO.: 35 & 64 E.B.W.
 COUNTY: Hardee

PROJECT NO. : 06010-3501 CONST.

CROSS SECTION SOIL SURVEY FOR THE DESIGN OF ROADS

SURVEY BEGINS STA. : 494+00 SURVEY ENDS STA. : 554+00 SR 35
 SURVEY BEGINS STA. : 48+00 SURVEY ENDS STA. : 54+00 SR 64 West
 SURVEY BEGINS STA. : 10+00 SURVEY ENDS STA. : 17+00 SR 64 East

STATION NO.	ORGANIC CONTENT				SIEVE ANALYSIS RESULTS				ATTERBERG LIMITS (L.S.)		DESCRIPTION	CORROSION TEST RESULTS			
	NO. OF TESTS	% MOISTURE	% ORGANIC CONTENT	NO. OF TESTS	NO. 10	NO. 20	NO. 40	NO. 60	NO. 100	NO. 200		LIQUID LIMIT	PLASTIC INDEX	NO. OF TESTS	RESISTIVITY (Ohm cm)
1	1	100	0.0	1	100	100	100	100	100	100	100	0	1	10000	0.0
2	1	100	0.0	1	100	100	100	100	100	100	100	0	1	10000	0.0
3	1	100	0.0	1	100	100	100	100	100	100	100	0	1	10000	0.0
4	1	100	0.0	1	100	100	100	100	100	100	100	0	1	10000	0.0
5	1	100	0.0	1	100	100	100	100	100	100	100	0	1	10000	0.0
6	1	100	0.0	1	100	100	100	100	100	100	100	0	1	10000	0.0
7	1	100	0.0	1	100	100	100	100	100	100	100	0	1	10000	0.0
8	1	100	0.0	1	100	100	100	100	100	100	100	0	1	10000	0.0

NOTES:

- (1) Strata boundaries are approximate. Make final check after grading.
- (2) \checkmark Water table encountered.
- (3) GNE - Groundwater not encountered.

EMBANKMENT AND SUBGRADE MATERIAL

- The material from Stratum Number 1 is Asphaltic Concrete over Rock Bases.
- The material from Strata Number 2 and 3 appears satisfactory for use in the embankment when utilized in accordance with Index 505.
- The material from Strata Number 4 and 5 appears satisfactory for use in the embankment when utilized in accordance with Index 505. However, this material contains excessive moisture and is difficult to dry and compact. It should be used in the embankment only if the water content is within the limits of compaction. The material from Stratum 5 may not be used in the subgrade portion of the roadbed due to its organic content.
- The material from Stratum Number 6 is MCK/A-8 material and shall remain in place.
- The material from Stratum Number 7 is high plastic A-7 material and shall be removed in accordance with Index 505. It may be used within the project limits as indicated in Index 505 only when accepted within the project limits and is not to be used when obtained from outside the project limits.
- The material from Stratum Number 8 is plastic A-2-A-2-7 and A-3 material and shall be removed in accordance with Index 505. It may be placed above the existing water level (at the time of construction) to within 1.2 meters of the proposed base. It should be placed uniformly in the lower portion of the embankment for some distance along the project rather than full depth for shorter distances.

Figure 27, Typical Report of Test Results Sheet

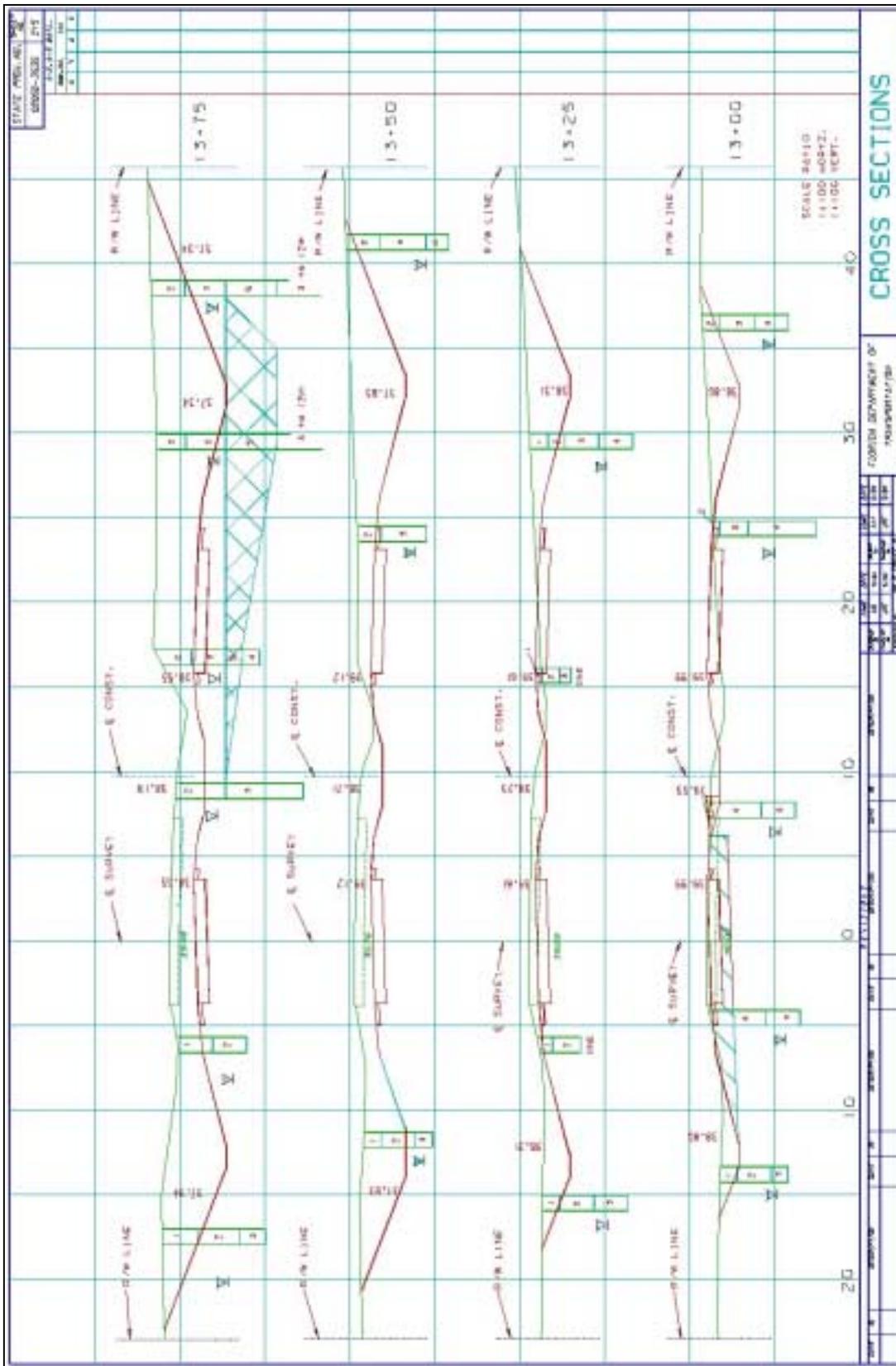


Figure 28, Typical Roadway Cross-Section Sheet

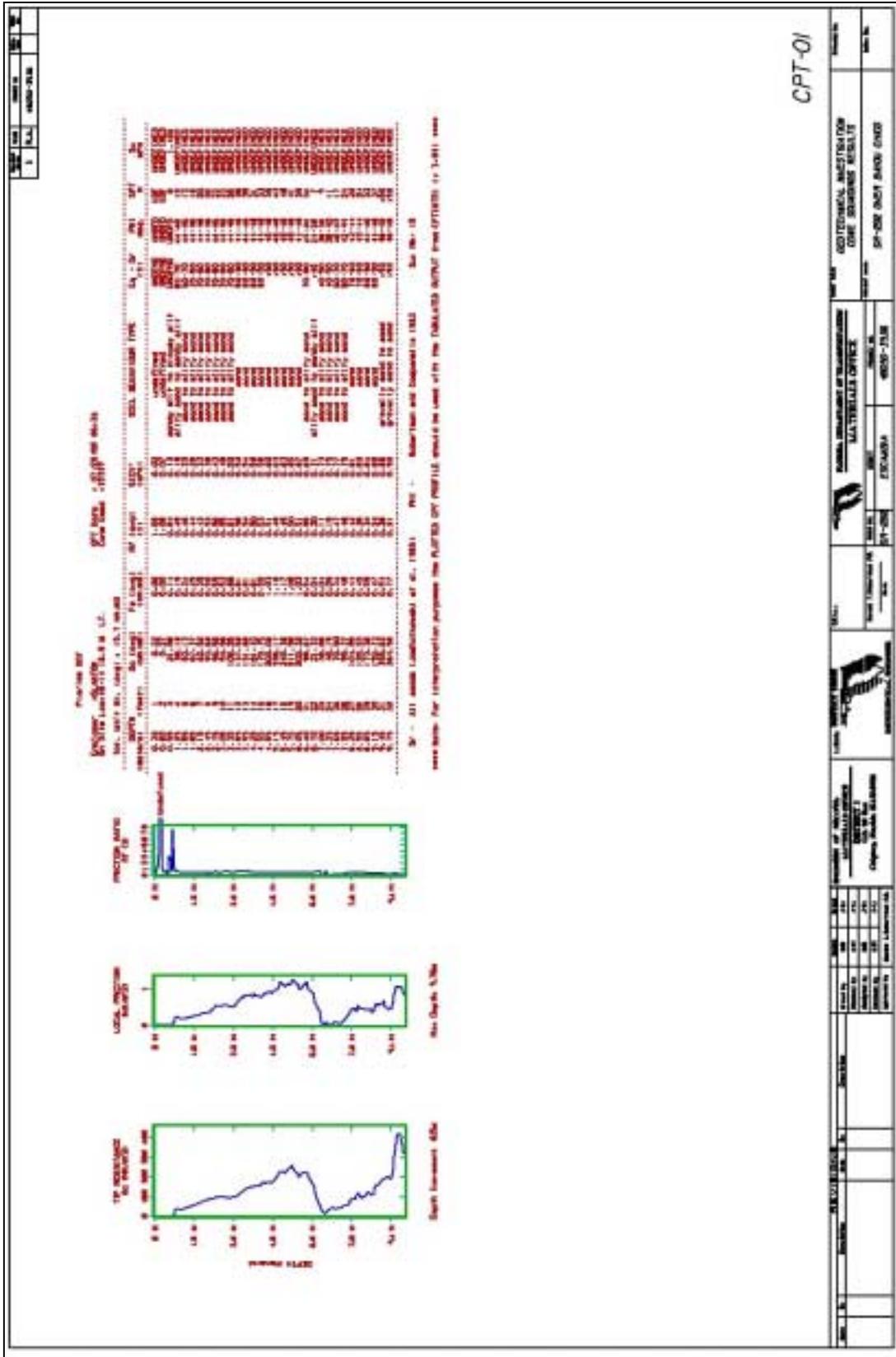


Figure 30, Typical Report of Cone Soundings Sheet

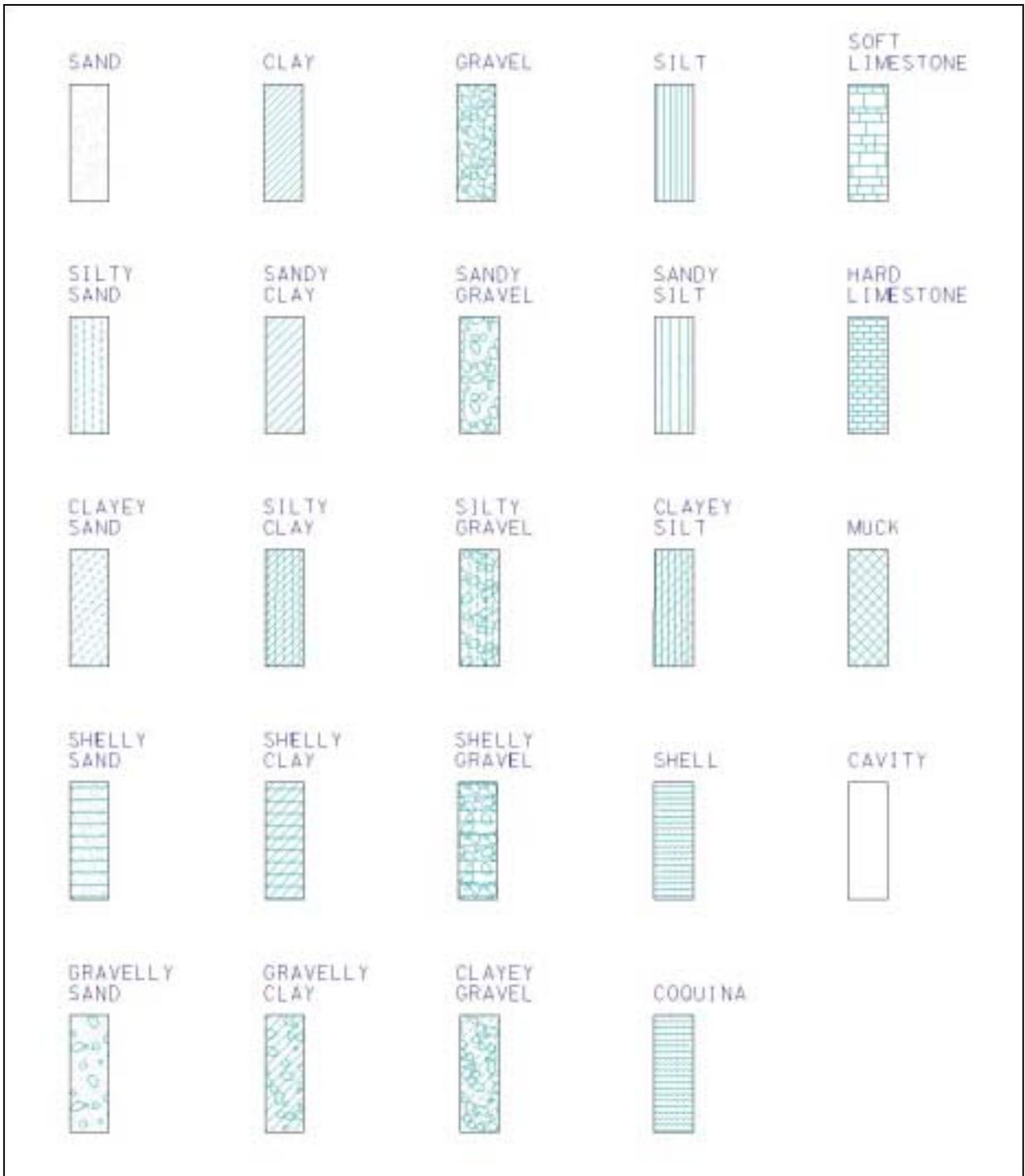


Figure 31, Standard Soil Type Symbols

9.10 Specifications and Standards

<u>Subject</u>	<u>ASTM</u>	<u>AASHTO</u>	<u>FM</u>
Standard Practice for the Use of Metric (SI) Units in Building Design and Construction	E 621	-	-
Standard for Use of the International System of Units (SI): The Modern Metric System	SI-10	-	-

Chapter 10

10 Construction and Post-Construction

A Geotechnical Engineer's involvement does not end with the completion of the final report; he may also be involved in the preconstruction, construction and maintenance phases of a project.

During construction, in-situ materials and construction methods for geotechnical elements must be inspected to assure compliance with the design assumptions and the project specifications. Such inspection tasks include subgrade and/or embankment compaction control, assurance of proper backfilling techniques around structural elements, and routine footing, drilled shaft, and piling installation inspection. While the Geotechnical Engineer may not regularly be involved in these inspections, he must assure that sufficient geotechnical information is provided to a qualified inspector. He must also be prepared to review the procedures and the inspection records if needed.

Where there are existing structures, which may be sensitive to vibrations or movement, pre-construction and post-construction surveys of the structures should be performed. Mitigating action shall be taken to reduce the impact. It may also be desirable to monitor construction-induced vibrations, groundwater level changes, and/or settlement or heave of the structures. A Geotechnical Engineer should be involved in the placement of these monitoring devices as well as the interpretation of the resulting data.

On major projects especially, several other aspects of the construction phase may require significant input from the Geotechnical Engineer. Involvement of the Geotechnical Engineer is often required post-construction as well. Tasks, which in all cases require the direct involvement of a Geotechnical Engineer, include those discussed below.

10.1 Dynamic Pile Driving Analysis

Wave equation analysis is preferred over the use of traditional dynamic formulas for predicting the capacity of a driven pile. The wave equation uses a mass-spring-dashpot system to dynamically model the behavior of a pile subjected to impact driving. The latest version of the WEAP computer program is recommended. Based on pile driving equipment data supplied by the contractor, the Geotechnical Engineer can use the wave equation program to determine the relationship between ultimate pile capacity and the penetration resistance (the number of blows per foot {meter}). The program also determines the relationship between stresses induced in the pile during driving and the penetration resistance. These relationships are then used to determine the suitability of the proposed driving system and to determine in the field if adequate pile capacity can be obtained.

10.2 Dynamic Monitoring of Pile Driving

Measurements of the dynamic pile response can be obtained during driving by the Pile Driving Analyzer (PDA). (See **Figure 32** and **Figure 33**). These measurements are used to determine:

1. Pile capacity

2. Driving stresses and probable damage to the pile
3. Energy transfer to the pile and therefore the efficiency and suitability of the pile driving system.
4. The soil parameters used in wave equation analysis
5. Possible reasons for pile installation problems.

On major projects, dynamic monitoring of pile driving can be used together with static load tests to confirm design-bearing capacities. Quite often, the use of dynamic measurements decreases the number of static load tests required. This will result in time, as well as, cost savings to a load test program. On smaller projects, dynamic measurements alone may serve as a load test. The advancement in the design of the PDA system in recent years has made this equipment an indispensable tool for the field-testing and inspection of driven piles. Refer to ASTM D 4945.

10.3 Load Tests

Many major projects involving driven piles or drilled shafts will require the use of load tests. These tests are conducted to verify that actual pile or shaft response to loading is as assumed by the designer, and to ensure that the actual ultimate capacities are not less than the computed ultimate loads used during design. The project Geotechnical Engineer should be involved in the load testing itself, and the interpretation of the resultant data. He should be prepared to modify designs where necessary based on load test data.

10.3.1 Static Load Tests

Three types are commonly used based on type of loading: axial compression (refer to ASTM D 1143) (see **Figure 35**), axial tension (refer to ASTM D 3689), or lateral load (refer to ASTM D 3966). In each case, the test typically consists of a jack/load cell system to apply a loading based on the desired application against a reaction system and measuring the resulting displacement. Use of the state-owned load test equipment needs to be scheduled as early as possible of the anticipated time of the load test, and needs to be arranged through the State Materials Office, which maintains this equipment.

10.3.2 Statnamic Load Tests

Statnamic applies axial loads up to 3,400 tons (30 MN) (see **Figure 36**) or lateral loads up to 3,400 tons (30 MN)(see **Figure 37**). The load application is between a static load and a dynamic load. The associated dynamic effects are accounted for giving you the static load curve. Foundations tested include high capacity drilled shafts, steel piles, auger cast piles, timber piles, batter piles in clay, rock, silt, and sand. It can be used to test bridge foundations, pile groups, spread footings, and piles in water. It can also be used to evaluate the lateral capacity of foundations. No reaction piles are required. The duration of loading is on the order of 10 Hz. Statnamic's built-in load cell and laser sensor provide direct measurements of load-displacement behavior. Statnamic produces load versus displacement results immediately on site. Currently there is no ASTM

standard on this type of testing. Use of the state-owned 30 MN reaction weights needs to be scheduled as early as possible of the anticipated time of the load test, and needs to be arranged through the State Materials Office, which maintains this equipment. 10.3.3 Osterberg Load Tests

The Osterberg Load Cell is placed on the bottom of a pile or anywhere in a drilled shaft (see **Figure 38**). The cell expands to jack against the foundation's end bearing capacity so no reaction system is required. The cell can be placed above the bottom of a drilled shaft to equal out the loading. Or multiple cells can be used to isolate various zones. Currently there is no ASTM standard on this type of testing.

10.4 Pile/Drilled Shaft Damage Assessment

Various test methods are available to assess the quality of the in-place deep foundation unit. These quality assurance tests need to be performed by qualified personnel and the results need to be analyzed and interpreted by experienced engineers in order to provide meaningful results.

10.4.1 Pile Integrity Testing

The use of low strain impact non-destructive testing has become common to determine cracks or breaks in driven piles caused by high stresses, necking or voids which might have occurred during the construction of drilled shafts, or the actual length of piles for existing structures (one such product, the P.I.T., is shown in **Figure 34**). The Geotechnical Engineer should evaluate results of these tests. Refer to ASTM D 5882.

10.4.2 Crosshole Sonic Logging

This test is used to determine the integrity of drilled shafts and slurry walls. The test involves lowering probes to the bottoms of water-filled access tubes, and recording the compression waves emitted from a source probe in one tube by a receiver in another tube at the same elevation. The probes are pulled back to the surface and this procedure is repeated at various test depths in order to obtain a profile of the entire depth of the shaft or wall. Potential defects are indicated by delays in the signal arrival time and lower energies at a given test depth. Since access tubes are needed for this test, the design of the reinforcement cage must take the total number and location of these tubes into account.

10.5 Drilled Shaft Construction

Using the wet method during construction of a drilled shaft, mineral slurry is used to maintain a positive head inside the open shaft in order to keep the hole open prior to placement of concrete. In order to ensure the mineral slurry shall meet the requirements to perform properly, the following control tests shall be performed: density, viscosity, sand content, and pH of the slurry. Refer to FM 8-R13B-1, 8-R13B-2, 8-R13B-3, and 8-R13B-4, respectively.

In order to evaluate the quality of the rock directly below the shaft excavation, rock cores may be taken to a minimum depth of 5 feet (1.5 m) and up to 20 feet (6 m)

below the bottom of the drilled shaft excavation. The core barrel must be approved prior to usage, and shall be designed to provide core samples from 4 to 6 inches (100 to 150 mm) in diameter and allow the cored material to be removed in an undisturbed state. Refer to ASTM D 2113 and ASTM D 5079.

10.6 Shaft Inspection Device (SID)

A piece of equipment that is used to inspect the bottom cleanliness of drilled shafts prior to placement of concrete through the use of an inspection bell which houses a high resolution video camera (See **Figure 39**). The inspection bell is lowered from a service platform to the bottom of the shaft, and the operator can view the condition of the bottom via the camera. The bell is fitted with a depth gage to indicate the thickness of debris on the shaft bottom. The SID also has the capability to sample the sidewalls of shafts in soil in order to evaluate the buildup of slurry along the sidewalls. Use of the state-owned shaft inspection devices need to be scheduled as early as possible of the anticipated use, and need to be arranged through the State Materials Office, which maintains this equipment.

10.7 Field Instrumentation Monitoring

Field instrumentation is often used during construction and afterward to assure that actual field conditions are in agreement with the assumptions made during design or to monitor changes in conditions, which may occur during construction. Refer to **Chapter 7** for descriptions of some of the more common types of field instrumentation.

All field instrumentation should be installed, and have readings taken, by qualified personnel under the supervision of a Geotechnical Engineer. A Geotechnical Engineer should interpret all data and recommend any necessary action. For example, in projects where surcharging or precompression is required to improve the foundation soils, waiting periods are required. It is essential that the Geotechnical Engineer communicate with the construction engineer when required waiting periods determined from actual measurements differ from predicted periods so that the project schedule can be properly adapted.

10.8 Troubleshooting

No matter how carefully a project was investigated and designed, the possibility exists that unforeseen problems will arise during construction or afterward. The Geotechnical Engineer should be prepared to investigate when such problems occur. He should then recommend changes in design or construction method if necessary to minimize construction down time. If it is determined that maintenance problems have a geotechnical basis, he should recommend remedial actions that will eliminate, or at least reduce, the problems.

10.9 Records

Invaluable geotechnical information is obtained during all construction projects. This data is often helpful during the design of other projects under similar conditions. Problems, which occurred during construction of one project, can

possibly be avoided on future projects if the design engineer has access to information about the problems.

Complete records of the geotechnical aspects of the construction and maintenance phases of a project should be kept. Any specialized construction procedures or design changes should be noted. Construction and maintenance problems and their solutions should be described in detail. This information should then be provided to the District Geotechnical Engineer and the State Geotechnical Engineer in Tallahassee.

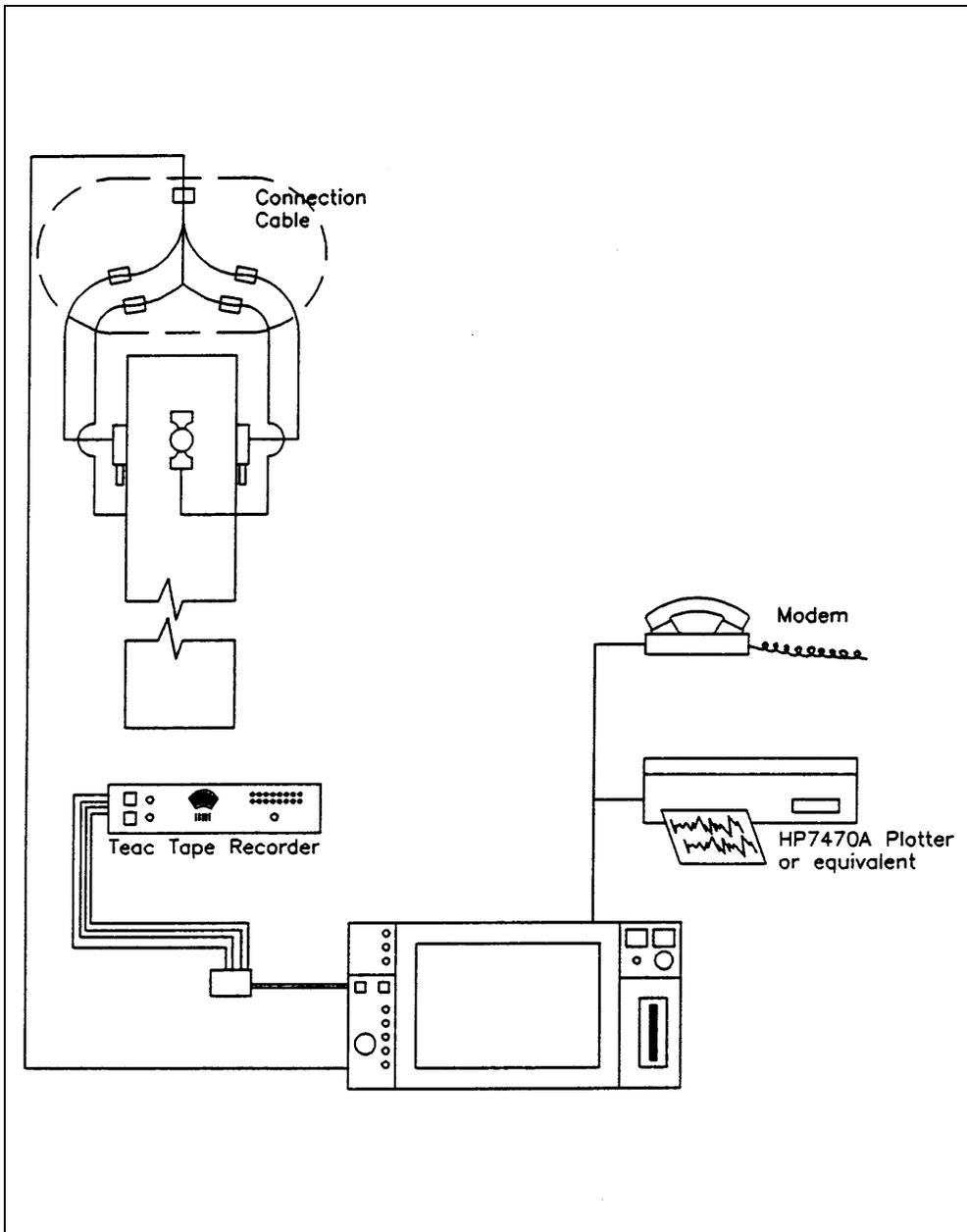


Figure 32, Schematic of Pile Driving Analyzer and Data Recording System (After PDI, 1993)



Figure 33, Pile Driving Analyzer, Model PAK (After PDI, 1993)



Figure 34, Pile Integrity Tester (After PDI, 1993)



Figure 35, Static Load Test



Figure 36, Statnamic Axial Load Test



Figure 37, Lateral Static Load Test

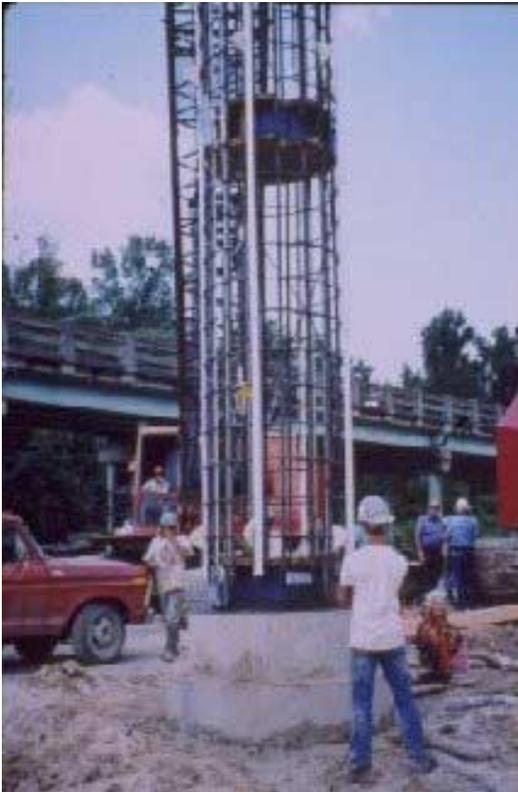


Figure 38, Osterberg Load Cells



Figure 39, Shaft Inspection Device

10.10 References

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10.11 Specifications and Standards

<u>Subject</u>	<u>ASTM</u>	<u>AASHTO</u>	<u>FM</u>
Statnamic Load Test	-	-	-
Osterberg Load Test	-	-	-
Viscosity of Slurry	-	-	8-RP13B-2
PH of Slurry	-	-	8-RP13B-4
Standard Test Method for Piles Under Static Axial Compressive Load	D 1143	-	-
Standard Test Method for Individual Piles Under Static Axial Tensile Load	D 3689	-	-
Standard Test Method for Piles Under Lateral Loads	D 3966	-	-
Standard Test Method for Density of Bentonitic Slurries	D 4380	-	8-RP13B-1
Standard Test Method for Sand Content by Volume of Bentonitic Slurries	D 4381	-	8-RP13B-3

<u>Subject</u>	<u>ASTM</u>	<u>AASHTO</u>	<u>FM</u>
Standard Test Method for High-Strain Dynamic Testing of Piles	D 4945	T 298	-
Standard Practices for Preserving and Transporting Rock Core Samples	D 5079	-	-
Standard Test Method for Low Strain Integrity Testing of Piles	D 5882	-	-

Appendix A

Determination of Design Skin Friction for Drilled Shafts Socketed in the Florida Limestone

By: Peter Lai

Introduction

The highly variable strength properties of the Florida limestone formation always prompted the question of what design skin friction should be used for a drilled shaft socketed in it. Some engineers even decide that doing any tests on rock cores obtained from the project site is senseless because of the uncertainties associated with a spatial variability of the limestone. This presentation provides a method that may be helpful for determining a reasonable design skin friction value from a number of laboratory unconfined compression and split tensile tests.

Design Method

On the basis of the study done by the University of Florida, the following method proposed by Prof. McVay seems to be the most appropriate for the Florida limestone. The ultimate skin friction for the portion socketed in the rock is expressed as

$$f_{su} = \frac{1}{2} \sqrt{q_u} \sqrt{q_t}$$

where : f_{su} is the ultimate side friction,
 q_u is the unconfined compression strength of rock core, and
 q_t is the split tensile strength (McVay, 1992).

$$(f_{su})_{DESIGN} = REC f_{su}$$

To consider the spatial variations of the rock qualities, the average **REC** (% recovery in decimal) is applied to the ultimate unit side friction, f_{su} , and the product is used as the design ultimate side friction.

The Department engineers have used this method for several years now and it has provided fairly good design skin friction as compared with load test data. However, there are some uncertainties of how to obtain the q_u , q_t , and **REC**.

Rock Sampling and Laboratory Testing

The main thing that makes the design method work is the quality of the rock cores. The rock core sample quality is hinged on the sampling techniques as well as the size and type of the core barrel used. The porous nature of the Florida limestone makes the larger diameter sampler more favorable than the smaller diameter sampler. Therefore, in the FDOT's *Soils and Foundation Handbook*, a minimum core barrel size of 61 mm (2.4") I.D. is required and a 101.6 mm (6") I.D. core barrel is recommended for better evaluation of the Florida limestone properties. Furthermore, the handbook also recommends using a double barrel as a minimum to have better percentage recovery as well as RQD. After obtaining the better quality core samples, the engineer can select more representative specimens for laboratory unconfined compression and split tensile tests. Thus better shear strength test data can be obtained for more an accurate design skin friction.

Data Reduction Method

The data reduction method presented here is intent to provide a means to obtain a more reliable q_u , q_t , and REC values that can provide realistic design skin friction for the

rock formation yet be conservative. This method involves the following steps of analyses.

1. Find the mean values and standard deviations of both the q_u , and q_t strength tests.
2. Establish the upper and lower bounds of each type of strength tests by using the mean values, +/- the standard deviations.
3. Discount all the data that are larger or smaller than the established upper and lower bounds, respectively.
4. Recalculate the mean values of each strength test using the data set that fall within the boundaries.
5. Establish the upper and lower bounds of q_u , and q_t .
6. Use the q_u , and q_t obtained from steps 4 and 5 to calculate the ultimate skin friction, f_{su} .
7. Multiply the ultimate skin friction f_{su} by the mean REC (in decimal) to account for the spatial variability.
8. The allowable or design skin friction can then be obtained by applying an appropriated factor of safety or load factor.

An example data set is provided for demonstration (see **Table A-1**).

Table A- 1

Boring No.	Core Sample Elevations		% REC	q _u , ksf	q _t , ksf
	Top	Bottom			
B-1	-62.24	-65.42	30		64.4
B-1	-72.42	-75.42	67	194.3	54.7
B-1	-82.42	-87.42	13		228.4
B-2	-36.58	-41.58	18	338.2	
B-9	-74.42	-82.42	5		53
B-9	-89.42	-94.4	43		49.3
B-9	-89.4	-94.4	43		65.8
S-12	-30	-35	60	422.4	136.7
S-12	-35	-40	48	234	38.7
S-12	-50	-55	48		39.2
B-7	-44.4	-52.4	18		87
B-7	-92.9	-97.4	98		52.6
B-7	-97.4	-102.4	66		235
B-7	-134.4	-142.4	35	281.2	129.3
B-11	-34.2	-39.2	38		288
B-11	-34.2	-39.2	38	758.9	378.1
B-11	-34.2	-39.2	38		225.2
B-11	-76.4	-81.4	33		52.6
B-11	-90.4	-95.4	60		137.4
N-14	-40	-43	63	778.7	
B-10	-33.4	-41.4	46	566.9	297.6
B-10	-33.4	-41.4	46		105.3
B-10	-46.4	-51.4	69	888.8	99.7
B-10	-46.4	-54.4	69	425.8	121
B-10	-46.4	-51.4	69		131.5
B-8	-48.9	-57.9	48	317.4	110.9
B-8	-48.9	-57.9	48	545.5	108
B-8	-48.9	-57.9	48		153.6
B-8	-59.9	-67.9	50	570.2	80.8
B-8	-99.9	-107.9	17		28.1
N-17	-58.1	-63	33	864.0	90.5
S-15	-48.5	-53.5	55	102.8	
S-15	-48.5	-53.5	55		34.9
S-15	-65	-70	61	76.7	15.3
B-6	-64.1	-72.1	51	116.4	24.8
B-6	-74	-82	57	730.7	202.8
B-6	-114	-122	45		41.9
N-25	-58.8	-63.3	85	53.1	
N-25	-68.8	-73.3	80	562.5	
N-25	-73.3	-78.3	47	662.9	
SUM			1941	9491.4	3962.1
MEAN			48.5	451.9	116.5
STANDARD DEVIATION				268.5	88.5
UPPER BOUND				720.4	205.1
LOWER BOUND				183.4	27.9

Table A- 2

Boring No.	Core Sample Elevations		% REC	q _u , ksf	q _t , ksf
	Top	Bottom			
B-1	-62.24	-65.42	30		64.4
B-1	-72.42	-75.42	67	194.3	54.7
B-1	-82.42	-87.42	13		228.4
B-2	-36.58	-41.58	18	338.2	
B-9	-74.42	-82.42	5		53
B-9	-89.42	-94.4	43		49.3
B-9	-89.4	-94.4	43		65.8
S-12	-30	-35	60	422.4	136.7
S-12	-35	-40	48	234	38.7
S-12	-50	-55	48		39.2
B-7	-44.4	-52.4	18		87
B-7	-92.9	-97.4	98		52.6
B-7	-97.4	-102.4	66		235
B-7	-134.4	-142.4	35	281.2	129.3
B-11	-34.2	-39.2	38		288
B-11	-34.2	-39.2	38	758.9	378.1
B-11	-34.2	-39.2	38		225.2
B-11	-76.4	-81.4	33		52.6
B-11	-90.4	-95.4	60		137.4
N-14	-40	-43	63	778.7	
B-10	-33.4	-41.4	46	566.9	297.6
B-10	-33.4	-41.4	46		105.3
B-10	-46.4	-51.4	69	888.8	99.7
B-10	-46.4	-54.4	69	425.8	121
B-10	-46.4	-51.4	69		131.5
B-8	-48.9	-57.9	48	317.4	110.9
B-8	-48.9	-57.9	48	545.5	108
B-8	-48.9	-57.9	48		153.6
B-8	-59.9	-67.9	50	570.2	80.8
B-8	-99.9	-107.9	17		28.1
N-17	-58.1	-63	33	864.0	90.5
S-15	-48.5	-53.5	55	102.8	
S-15	-48.5	-53.5	55		34.9
S-15	-65	-70	61	76.7	15.3
B-6	-64.1	-72.1	51	116.4	24.8
B-6	-74	-82	57	730.7	202.8
B-6	-114	-122	45		41.9
N-25	-58.8	-63.3	85	53.1	
N-25	-68.8	-73.3	80	562.5	
N-25	-73.3	-78.3	47	662.9	
SUM			1941	5121.3	1800.9
MEAN			48.5	426.7	78.3
STANDARD DEVIATION				147.3	35.9
UPPER BOUND				574.1	114.2
LOWER BOUND				279.3	42.3

Use the upper and lower bounds of q_u and q_t as guides to reduce the data set so that no data are higher than the upper bound value and no data are lower than the lower bound value. The modified data set is presented in the **Table A-2**.

By using the above q_u and q_t values the following f_{su} values can be calculated;

Upper bound

$$f_{su} = \frac{1}{2} * \sqrt{574} * \sqrt{114} = 128 \text{ ksf}$$

Lower bound

$$f_{su} = \frac{1}{2} * \sqrt{279} * \sqrt{42.3} = 54 \text{ ksf}$$

Mean value

$$f_{su} = \frac{1}{2} * \sqrt{426.8} * \sqrt{78.3} = 91.4 \text{ ksf}$$

The design ultimate skin friction can also be obtained by applying the mean %REC to the above high and low values respectively and obtain;

Upper Design Boundary

$$(f_{su})_{DESIGN} = .485 * 128 = 62 \text{ ksf}$$

Lower Design Boundary

$$(f_{su})_{DESIGN} = .485 * 54 = 26.3 \text{ ksf}$$

Mean Design Value

$$(f_{su})_{DESIGN} = .485 * 91.4 = 44.3 \text{ ksf}$$

A safety factor or load factor should be applied to these skin friction values depend on the construction methods used. The following table may be used as a guide to obtain an appropriate safety factor for the service load design (SLD) or a load factor for the load factor design (LFD). However, it should be noted that all these will be changed when Load and Resistance Factor Design (LRFD) method becomes effective.

Service Load Design

Drilled shaft construction	Factor of Safety	Performance Factor
With load test	2.0	0.7
Without load test	2.5	0.6

The mobilized ultimate end bearing capacity is a function of shaft tip movement as well as the load-shedding mechanism along the shaft. To obtain an accurate estimate of the mobilized end bearing capacity, the engineer should first calculate the shaft tip

movement, which includes both the elastic shortening of the shaft and the yielding of the bearing soils. This will involve a trial-and-true process called Q-Z method by first assuming a tip movement and calculate the load-shedding along the shaft so that the resistance and the applied load will be the same. However, based on the load test database the percentage of the ultimate end bearing mobilized for various shaft sizes can be roughly estimated by using the following;

Drilled shaft diameter, mm	Nominal mobilized ult. end bearing*
< 1 200	0.10*S _u
1 200 - 1 850	0.15*S _u
> 1 850	0.25*S _u

- The ultimate unit end bearing is equal to 0.5*S_u, where S_u is the unconfined compression strength of the bearing rock.

It should be noted that the mobilized end bearing presented are for your reference only. Engineers shall perform their analysis by using appropriate method(s) and test data to verify these estimated results.

Appendix B

Specifications and Standards

ASTM

<u>Subject</u>	<u>ASTM</u>
Absorption and Bulk Specific Gravity of Dimension Stone	C 97
Standard Test Method for Specific Gravity and Absorption of Coarse Aggregate	C 127
Guide to Site Characterization for Engineering, Design, and Construction Purposes	D 420
Standard Test Method for Particle-Size Analysis of Soils	D 422
Test Method for Shrinkage Factors of Soils by the Mercury Method	D 427
Standard Test Methods for Chloride Ion In Water	D 512
Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft ³ (600 kN-m/m ³))	D 698
Standard Test Method for Specific Gravity of Soils	D 854
Standard Test Methods for Electrical Conductivity and Resistivity of Water	D 1125
Standard Test Method for Piles Under Static Axial Compressive Load	D 1143
Standard Test Methods for pH of Water	D 1293
Standard Practice for Soil Investigation and Sampling by Auger Borings	D 1452
Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft ³ (2,700 kN-m/m ³))	D 1557
Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils	D 1586
Standard Practice for Thin-Walled Tube Geotechnical Sampling of Soils	D 1587
Standard Practice for Diamond Core Drilling for Site Investigation	D 2113
Standard Test Method for Unconfined Compressive Strength of Cohesive Soil	D 2166
Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock	D 2216
Standard Test Method for Permeability of Granular Soils (Constant Head)	D 2434
Standard Test Method for One-Dimensional Consolidation Properties of Soils	D 2435
Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System)	D 2487

<u>Subject</u>	<u>ASTM</u>
Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)	D 2488
Standard Test Method for Field Vane Shear Test in Cohesive Soil	D 2573
Standard Test Method for Triaxial Compressive Strength of Undrained Rock Core Specimens Without Pore Pressure Measurements	D 2664
Standard Test Method for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression	D 2850
Standard Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens	D 2938
Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils	D 2974
Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions	D 3080
Standard Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes	D 3282
Standard Test Method for Infiltration Rate of Soils in Field Using Double-Ring Infiltrometer	D 3385
Standard Test Method for Deep, Quasi-Static, Cone and Friction-Cone Penetration Tests of Soil	D 3441
Standard Test Method for Individual Piles Under Static Axial Tensile Load	D 3689
Standard Test Method for Piles Under Lateral Loads	D 3966
Standard Test Method for Splitting Tensile Strength of Intact Rock Core Specimens	D 3967
Standard Test Method (Field Procedure) for Withdrawal and Injection Well Tests for Determining Hydraulic Properties of Aquifer Systems	D 4050
Standard Test Method for Sulfate Ion in Brackish Water, Seawater, and Brines	D 4130
Standard Test Method for One-Dimensional Consolidation Properties of Soils Using Controlled-Strain Loading	D 4186
Standard Practices for Preserving and Transporting Soil Samples	D 4220
Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table	D 4253
Standard Test Method for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density	D 4254
Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils	D 4318
Standard Test Method for Density of Bentonitic Slurries	D 4380

<u>Subject</u>	<u>ASTM</u>
Standard Test Method for Sand Content by Volume of Bentonitic Slurries	D 4381
Standard Test Methods for Crosshole Seismic Testing	D 4428
Standard Test Methods for One-Dimensional Swell or Settlement Potential of Cohesive Soils	D 4546
Standard Test Method for Rock Mass Monitoring Using Inclinedometers	D 4622
Standard Test Method for Laboratory Miniature Vane Shear Test for Saturated Fine-Grained Clayey Soil	D 4648
Standard Test Method for Pressuremeter Testing in Soils	D 4719
Standard Test Method for Determining Subsurface Liquid Levels in a Borehole or Monitoring Well (Observation Well)	D 4750
Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils	D 4767
Standard Test Method for High-Strain Dynamic Testing of Piles	D 4945
Standard Practices for Preserving and Transporting Rock Core Samples	D 5079
Standard Test Method for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter	D 5084
Standard Guide for Field Logging of Subsurface Explorations of Soil and Rock	D 5434
Standard Guide for Using the Seismic Refraction Method for Subsurface Investigation	D 5777
Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils	D 5778
Standard Test Method for Low Strain Integrity Testing of Piles	D 5882
Standard Practice for Using Hollow-Stem Augers for Geotechnical Exploration and Soil Sampling	D 6151
Standard Practice for the Use of Metric (SI) Units in Building Design and Construction	E 0621
Standard Test Method for Measuring pH of Soil for Use in Corrosion Testing	G 51
Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method	G 57
Provisional Guide for Selecting Surface Geophysical Methods	PS 78
Standard for Use of the International System of Units (SI): The Modern Metric System	SI-10

AASHTO

<u>Subject</u>	<u>AASHTO</u>
Standard Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes	M 145
Standard Test Method for Specific Gravity and Absorption of Coarse Aggregate	T 85
Standard Test Method for Particle-Size Analysis of Soils	T 88
Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils	T 89
Test Method for Shrinkage Factors of Soils by the Mercury Method	T 92
Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft ³ (600 kN-m/m ³))	T 99
Standard Test Method for Specific Gravity of Soils	T 100
Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft ³ (2,700 kN-m/m ³))	T 180
Standard Practice for Soil Investigation and Sampling by Auger Borings	T 203
Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils	T 206
Standard Practice for Thin-Walled Tube Geotechnical Sampling of Soils	T 207
Standard Test Method for Unconfined Compressive Strength of Cohesive Soil	T 208
Standard Test Method for Permeability of Granular Soils (Constant Head)	T 215
Standard Test Method for One-Dimensional Consolidation Properties of Soils	T 216
Standard Test Method for Field Vane Shear Test in Cohesive Soil	T 223
Standard Practice for Diamond Core Drilling for Site Investigation	T 225
Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions	T 236
Standard Practice for Using Hollow-Stem Augers for Geotechnical Exploration and Soil Sampling	T 251
Pore Pressure	T 252
Standard Test Method for Rock Mass Monitoring Using Inclinometers	T 254
Standard Test Methods for One-Dimensional Swell or Settlement Potential of Cohesive Soils	T 258

<u>Subject</u>	<u>AASHTO</u>
Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock	T 265
Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils	T 267
Resilient Modulus – Soil	T 294
Standard Test Method for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression	T 296
Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils	T 297
Standard Test Method for High-Strain Dynamic Testing of Piles	T 298

Florida Test Method

<u>Subject</u>	<u>FM</u>
Chloride Content - Soil (Retaining wall backfill)	5-556
Standard Test Method for Sulfate Ion in Brackish Water, Seawater, and Brines	5-553
Standard Test Methods for Chloride Ion In Water	5-552
Standard Test Methods for Electrical Conductivity and Resistivity of Water	5-551
Standard Test Method for Measuring pH of Soil for Use in Corrosion Testing	5-550
Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft ³ (600 kN-m/m ³))	5-525
Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft ³ (2,700 kN-m/m ³))	5-521
Florida Bearing Value	5-517
Limerock Bearing Ratio	5-515
Permeability - Falling Head	5-513
Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils	1-T 297
Standard Test Method for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression	1-T 296
Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils	1-T 267
Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock	1-T 265
Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions	1-T 236
Standard Test Method for One-Dimensional Consolidation Properties of Soils	1-T 216
Standard Test Method for Permeability of Granular Soils (Constant Head)	1-T 215
Standard Test Method for Unconfined Compressive Strength of Cohesive Soil	1-T 208
Standard Practice for Thin-Walled Tube Geotechnical Sampling of Soils	1-T 207
Standard Test Method for Specific Gravity of Soils	1-T 100
Test Method for Shrinkage Factors of Soils by the Mercury Method	1-T 092

Subject**FM**

Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils	1-T 090 & 1-T-089
Standard Test Method for Particle-Size Analysis of Soils	1-T 088
Standard Test Method for Specific Gravity and Absorption of Coarse Aggregate	1-T 085
Standard Test Method for Density of Bentonitic Slurries	8-RP13B-1
Viscosity of Slurry	8-RP13B-2
Standard Test Method for Sand Content by Volume of Bentonitic Slurries	8-RP13B-3
pH of Slurry	8-RP13B-4

Appendix C

Reference List

AASHTO

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TRB

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FDOT

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