Soils and Foundations Handbook

2021

Summary of Changes

State Materials Office
Gainesville, Florida

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. 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 (O.D.) split barrel which is driven into the soil with a 140-pound hammer dropped 30 inches. After it has been driven 18 inches, it is withdrawn and the sample removed.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 (O.D.) by 30 inches 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).

When materials are too weak to be retained by a Shelby tube, a piston type of sampler should be used.

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 and organics. 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 and organic soils.

2.4.5.5 Rock Core Sampling

Rock cores shall be obtained in accordance with ASTM D 2113 Standard Practice for Diamond Core Drilling for Site Excavation using a double or triple wall core barrel 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 core barrels for 2.4 inch cores generally provide lesser quality samples than triple tube barrels, and shall only be used for core samples larger than 3.5 inches. Triple tube core barrels are required for core samples smaller than 3.5 inches and are described below. (Note: face discharge bits generally provide better return in Florida limestone). 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

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. Munfakh, George, Arman, Ara, Samtani, Naresh, and Castelli, Raymond, Subsurface Investigations, FHWA-HI-97-021, 1997.

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

Subject	ASTM	AASHTO
Standard Practice for Soil Investigation and Sampling by Auger Borings	D 1452	-
Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils	D 1586	T 206
Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes	D 1587	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~~	-
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 not currently valid

Modification for Non-Conventional Projects:
Delete the previous paragraph and insert the following: The following general standards apply as outlined herein to all investigation programs, except as otherwise described in the RFP:

1. Preliminary exploration depths should be estimated from data obtained during field reconnaissance, existing data, local geology 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 horizontal and vertical location shall be determined for each boring, sounding, and test pit as follows:

   Offshore borings should be referenced to mean sea level with the aid of a tide gauge. (Note: There are two vertical datums. They are the 1929 datum and the 1988 datum; ensure that the proper one is being referenced.)
5. Locate bridge borings by survey; use survey methods or a field Global Positioning System (GPS) unit with a manufacturer's rated accuracy of ±10 feet to locate the Longitude and Latitude coordinates of roadway, pond and miscellaneous structure borings, and the boundaries of muck probe areas.
6. A sufficient number of samples, suitable for the types of testing intended, should be obtained within each layer of material.
7. Water table observation within each boring or test pit should be recorded after sufficient time (typically 24 hours) has elapsed for the water table to stabilize. ~~Refer to ASTM D 4750.~~ Other groundwater observations (artesian pressure, etc.) should also be recorded.
8. 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

- Bents/pier foundations ≥ 70 feet wide - at least two evenly spaced borings within every other bent/pier foundation (or one boring at alternating ends of every bent/pier foundation) per structure

c. Redundant Drilled Shafts - at least one per bent/pier foundation in consistent soil conditions; in variable soil conditions, ensure at least one boring is within 20 feet of each shaft.

d. Non-redundant Drilled Shafts – at least one per shaft (See 12)

e. Auger Cast Piles (ACP) –

- Bents/pier foundations < 70 feet wide - at least one boring per bent/pier per structure within 25 feet of each bent/pier ~~foundation~~footing;
- Bents/pier foundations ≥ 70 feet wide - at least two evenly spaced borings per bent/pier foundation per structure, with at least one boring within 25 feet of each end of each bent/pier footing;
- All bridges with ACP foundations require static load tests. Perform at least one boring within 5 feet of the location of the static load test pile.

Commented [LJ1]: Revised to match SDG terminology.

Commented [LJ2]: Revised to match SDG terminology.

For structure widenings, the total number of borings may be reduced depending on the information available for the existing structure.

When practical, perform each 2.5-inch minimum diameter SPT boring at each pier or abutment location during the design phase. The hole pattern should be staggered so that borings occur at the opposite ends of adjacent piers.

2) If pier locations are unknown, a Phase I Investigation including borings spaced approximately every 500 feet, or as directed by the District Geotechnical Engineer, may be performed to provide sufficient information for the structural engineer to complete the Bridge Development Report process and determine the locations of the bridge piers. Perform the pier specific borings during a Phase II Investigation after the bridge pier locations are determined.

Modification for Non-Conventional Projects:
Delete Item 2) and replace with “2) If pier locations are unknown, perform a Phase I Investigation including borings spaced to provide sufficient information for the structural engineer to complete the Bridge Development Report process and determine the locations of the bridge piers. Perform the pier foundation specific borings during the design phase after the bridge pier locations are determined.”

3.3 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 Soils Mechanics, Department of the Navy, Naval Facilities Engineering Command, 1986.
3. "Checklist and Guidelines for Review of Geotechnical Reports and Preliminary Plans and Specifications," Federal Highway Administration, 1985. Revised 2003.
4. Schmertmann, J.H., Guidelines For Use In The Soils Investigation and Design of Foundations For Bridge Structures In The State Of Florida, Research Report 121-A, Florida Department of Transportation, 1967.
5. Munfakh, George, Arman, Ara, Samtani, Naresh, and Castelli, Raymond, Subsurface Investigations, FHWA-HI-97-021, 1997.
6. Recommended Guidelines for Sealing Geotechnical Exploratory Holes, National Cooperative Highway Research Program, NCHRP Report 378.
7. Rigid Pavement Design Manual, FDOT, (Current version).
8. General Tolling Requirements (GTR) Volume 1, FDOT, (Current Version)

3.4 Specifications and Standards

Subject	ASTM	AASHTO	FM
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~~	~~-~~	~~-~~

coefficients and suggested test methods from Reference 18 are presented in **Figure 14**. Formulas for computing permeability coefficients from constant and variable head tests are included in **Figure 15**. For in-situ variable head tests, see References 17 and 2. Perform laboratory tests according to ASTM D 5856. Perform constant head and falling head borehole permeability tests in accordance with ASTM D 6391.

4.9.1 Constant Head Test

The most commonly used permeability test is the constant head test. However, it may be difficult to perform in materials of either very high or very low permeability since the flow of water may be difficult to maintain or to measure.

4.9.2 Rising Head Test

In a saturated zone with sufficiently permeable materials the rising head test is more accurate than a constant or a falling head test. Plugging of the pores by fines or by air bubbles is less apt to occur in a rising head test. In an unsaturated zone, the rising head test is inapplicable.

4.9.3 Falling Head Test

In zones where the flow rates are very high or very low, the falling head test may be easier to perform than a constant head test. In an area of unknown permeability the constant head and rising head tests should be attempted before a falling head test.

4.9.4 Pumping Test

In large scale seepage investigations or groundwater resource studies, the expense of aquifer or pumping tests may be justified as they provide more accurate and useful data than any other type of test. Pump tests require a test well, pumping equipment, and lengthy test times. Observation wells are necessary. A vast number of interpretive techniques have been published for special conditions.

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 and Reference 17.

4.9.5 Vertical Insitu Permeameter (VIP) Test

The FDOT sponsored a study to develop a field permeability test method using a probe as an alternative to conventional borehole testing methods. The conical probe that was developed can be pushed into the soil using a standard drill rig. It has a vertical injection port to control the outflow of water into the surrounding soil. The result is a mean coefficient of permeability at the depth to which the probe was advanced, and multiple depths can be tested from a single sounding. Tests shall be performed in accordance with FM 5-614.

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,

4.13 Specifications and Standards

Subject	ASTM	AASHTO	FM
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 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 Energy Measurement for Dynamic Penetrometers	D 4633	-	-
Standard Test Methods for Prebored 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 Field Measurement of Hydraulic Conductivity Using Borehole Infiltration	D 6391		
Standard Test Method for Performing the Flat Plate Dilatometer Test	D 6635	-	-
Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications	D 6951	-	-
Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method	G 57	-	-
Standard Test Method for pH of pH of Soil and Water	-	-	5-550
Standard Test Method for Resistivity of Soil and Water	-	-	5-551
Standard Test Method for Sulfate in Soil and Water	-	-	5-553
Standard Test Methods for Chloride in Soil and Water	-	-	5-552
Standard Test Method for Determination of Mean Permeability in the Field Using the Vertical Insitu Permeameter (VIP)			5-614

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.

This chapter is limited to a brief description of the tests, their purpose and the uses of the resulting data. Detailed instructions on test procedures will be found in the References and Specifications and Standards listed at the end of the chapter. Tests shall be performed and results presented as described in the listed References and Specifications and Standards except as stated herein. Not every test outlined below is applicable to every project. Engineering judgment 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 (~~AASHTO T 27 or AASHTO T 311 (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 (ASTM D 2216).

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 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 at the rate of 2 drops/second. Tests shall be performed in accordance with ~~ASTM D 4318 (~~AASHTO T 89 (ASTM D 4318).

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 in diameter without crumbling. Tests shall be performed in accordance with ~~ASTM D 4318 (~~AASHTO T 90 (ASTM D 4318).

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). 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 (ASTM D 854). This method is used for soil samples composed of particles less than the No. 4 U.S. standard sieve (0.187 inch). For particles larger than this sieve, use the procedures for Specific Gravity and Absorption of Coarse Aggregate (~~ASTM C 127 or~~ AASHTO T 85 (ASTM C 127).

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 (ASTM D 2166).

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 (ASTM D 2850).

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. Refer to ASTM D 7181

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 FM 3-D3080~~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 and doubling each increment up to 16 tsf. After each load application the change in sample height is monitored incrementally for, generally, 24 hours or more to clearly identify t_{100} and creep consolidation characteristics. 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 (ASTM D 2435).

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.

For high organic materials (organic content greater than 50%), FDOT-sponsored studies have shown that end of primary consolidation occurs quickly in the laboratory and field, and that a major portion of the total settlement is due to secondary and tertiary compression (creep). As a result, differentiating between primary consolidation and creep settlement on the individual loading's settlement versus time plots can be very difficult and generate misleading results. To analyze results from one-dimensional consolidation tests for these types of materials, use the Square Root (Taylor) Method to identify the end of primary consolidation for each load sequence. ~~In addition, e~~Each load sequence must be maintained for ~~at least~~ 24 hours or more to identify ~~a~~ the slopes of the secondary and tertiary compression portions of the settlement versus time plot.

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 (ASTM D 2974).

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 4943.

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 (ASTM D 4546).

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 preferred for use with coarse-grained soils with $k>10^{-3}$ cm/sec (Bowles 1984). Tests shall be performed in accordance with ~~ASTM D 5856 or ASTM D 2434 (~~AASHTO T 215 (ASTM D 5856 or ASTM D 2434).

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 often considered more economical for tests of long duration, such as tests on fine-grained soils with k between $5x10^{-5}$ and 10^{-3} cm/sec (Bowles 1984). Tests shall be performed in accordance with FM 5-513~~ or ASTM D 5856~~.

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 the **Environmental Corrosion Tests** section in **Chapter 4** for a list of test procedures. Corrosion testing must be performed for each site unless the most aggressive conditions are assumed.

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 rammer dropped from a height of 12 inches. The sample is compacted in three layers. Tests shall be performed in accordance with FM 1-T 099~~ASTM D 698 (AASHTO T 99)~~.

5.1.11.2 Modified Proctor

This test method uses a 10-pound rammer dropped from a height of 18 inches. The sample is compacted in five layers. Tests shall be performed in accordance with FM 1-T 180~~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 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. 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 in Florida.

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. 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 diameter piston to penetrate the soil at a constant rate and to a depth of 0.5 inches. A load-penetration curve is plotted for each sample and the LBR corresponding to 0.1-inch 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.

5.4 Specifications and Standards

Subject	ASTM	AASHTO	FM
Standard Test Method for Coefficient of Permeability - Falling Head	-	-	5-513
Standard Test Method for Limerock Bearing Ratio (LBR)	-	-	5-515
Standard Test Method for Determining the Resilient Modulus of Soils and Aggregate Materials	-	T 307	-
Standard Test Methods for 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 Sieve Analysis of Fine and Coarse Aggregate	C 136	T 27	
Standard Test Method for Grain-Size Analysis of Granular Soil Materials		T 311	
Standard Test Method for Particle-Size Analysis of Soils	~~D 422~~-	T 88	-
Standard Test Method for Shrinkage Factors of Soils by the Wax Method	D 4943	-	-
Standard 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~~	~~-~~1-T 099
Standard Test Method for Specific Gravity of Soils	D 854	T 100	-
Standard 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~~1-T 180
Standard Test Method for Unconfined Compressive Strength of Cohesive Soil	D 2166	T 208	-
Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock	D 2216	T 265	-
Standard Test Method for Permeability of Granular Soils (Constant Head)	D 2434	T 215	-
Standard Test Method for One-Dimensional Consolidation Properties of Soils	D 2435	T 216	-
Standard Test Method for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression	D 2850	T 296	-
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~~-	~~-~~3-D3080

Subject	**ASTM**	**AASHTO**	**FM**
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 & 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~~ -	-
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 Test Method for Measurement of Hydraulic Conductivity of Porous Material Using a Rigid-Wall, Compaction-Mold Permeameter	D 5856	-	-
Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures	D 7012	-	-
Standard Test Method for Consolidated, Drained Triaxial Compression Test for Soils	D 7181	-	-

6.1.1.10.2 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 D 3282 or~~ AASHTO M 145 (ASTM D 3282), and a summary is reprinted in **Figure 20** and **Figure 21** 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
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. Do not include subjective descriptions of rock hardness. Include only the objective indicators of the rock hardness (SPT-N values, excessive drilling time and down pressure, results of core testing, etc.) that would lead others to your subjective conclusions.

sufficient horizontal reaction to resist moments in shafts embedded at least seven shaft diameters below the design ground surface.

Various drilled shaft sizes should be analyzed to achieve an optimum design. For water crossings, depth of scour must be considered. 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. References 6, 7 & 30 are recommended for analyzing group effects. See Appendix C for a step by step design procedure for the analysis of downdrag. For foundations tipped on rock or IGM, ensure the bearing layer thickness below the tip elevation is sufficient to prevent punching failure into the weaker stratum below the bearing stratum for the end bearing resistance included in the design (See 8.3.1 Rock Fracture).

Prudent design considers that estimated side shear resistance requires sufficient surface area of the shaft to interact with the socket. Design values are based on statistical techniques; some portions of the rock are likely weaker than others due to normal geologic variability. Furthermore, undetected construction flaws could reduce load transfer. Therefore, the minimum rock socket length shall be 8 feet or 1.5 times the shaft diameter, whichever is longer. When the total socket length must be broken into layers, each layer should be at least one shaft diameter.

8.2.3.2 Considerations

When estimating drilled shaft resistance from side shear and end bearing (for shafts tipped in rock or IGM), ensure the resistance limits the end bearing to 1/3 of the ~~strain compatible~~ultimate value.

In sand, drilled shafts with pressure grouted tips should be considered. Pressure grouted tips are most effective in loose to medium dense sands. Guidance for the design of drilled shafts with pressure grouted tips may be found in Appendix D and in Reference 9.

Load tests on test shafts should be specified when necessary to verify capacity and/or constructability. Reinforced method shafts (test holes) are always required for bridges, and their locations shall be specified in the plans. Load tests should not be performed on method shafts. Method shafts should be the depth of the deepest shafts on the project, whereas the load test shafts should verify the resistance of the most economical bearing zone. Refer to the Structures Design Guidelines for additional considerations.

Drilled shafts may be constructed using temporary or permanent casing, however, the drilled shaft design methods are applicable only for computing the resistance of the uncased portions of the shaft. Portions of the shaft constructed with temporary casing will most commonly have reduced side shear resistance versus constructing the same portion of the shaft using slurry.

All resistance must be strain compatible. Peak side shear in rock will normally occur well before peak side shear in soil. The difference in the deformation required to mobilize skin friction in soil and rock versus what is required to mobilize end bearing shall be considered when estimating axial compressive resistance of shafts embedded in rock. (See References 9 and 30)

Verify the bearing strata will support the drilled shaft(s) without the risk of punching shear failure.

8.2.3.3 Design Procedure for Miscellaneous Structures

Drilled shafts for miscellaneous structures must be designed considering both axial and lateral loads, however the design for lateral loads will normally govern. The controlling loading condition for miscellaneous structures is normally due to wind loading during the design storm event after several days of continuous rain would have occurred. Therefore, the design groundwater level is normally at the ground surface. When drilled shafts for miscellaneous structures will be founded in limestone, the guidelines in Appendix B for rock may be used. An example lateral load analysis using Broms' Method for a cable barrier end terminal is presented in Appendix G.

When borings indicate water levels will not be encountered during drilled shaft construction, add the following note to the Plans:

- The Dry Method of drilled shaft construction may be appropriate for this foundation.

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 govern when auger-cast-piles are used for noise wall foundations. See the SDG for restrictions on the use of Auger Cast Piles for bridges and other structures.

8.2.4.1 Design Procedure

Design Auger Cast Piles for Bridges (when allowed) using the same design procedures as for side shear resistance of drilled shafts. For side shear resistance of rock or cohesive IGM materials, use the design procedures outlined in Appendix A. Unit side shear values for all foundations must be strain compatible; this is particularly important for auger cast pile bridge foundations. Therefore, for design of rock or IGM socketed auger cast piles supporting bridges, the side shear resistance from the overburden soil is neglected unless strain compatible values are determined by site specific load tests.

Generic designs for noise barrier wall foundations are presented in the Standard Plans for Road and Bridge Construction.

8.2.6.2 Considerations

Limitations and considerations are presented in Sections 3.12.12 and 3.13.4 of the Structures Design Guidelines, and in the Instructions for Developmental Standard Plans, Index D6025.

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 in the geotechnical report

8.3.1 Rock Fracture

For shallow foundations and the end bearing component of deep foundations supported on layered profiles where limestone or IGM bearing materials are underlain by weaker materials such as those depicted in **Figure 28**, ensure the bearing layer thickness below the bearing elevation is sufficient to prevent punching failure into a weaker stratum below the bearing stratum. Perform this check as part of the bearing analysis for the strength limit state. For spread footings use a trapezoidal pressure distribution.

Because the RMR (Rock Mass Rating) & GSI (Geological Strength Index) methods in AASHTO are ~~unproven~~ unreliable for nearly all Florida limestone ~~or~~ and IGM materials, estimate the shear resistance within the limestone ~~or~~ and IGM lenses using the method outlined in Appendix A for determining "f_s." The sample set may be limited to the borings closest to each foundation in order to best estimate the bearing conditions.

Commentary: The McVay method applied in Appendix A is based on the shaft socket interface being sufficiently rough that the failure surface is entirely within the rock or IGM in which the shaft is socketed. Therefore f_s is the rock shear strength. For details see Reference 37. See Reference 41 for a discussion of the applicability of RMR & GSI to Florida limestone.

When the limestone/IGM material has not been cored and tested, the shear resistance of the material below the tip elevation may be estimated using Standard Penetration Test (SPT) blow count using the following equation:

$$q_{frac} = 0.1 \text{ tsf} * N_{60} \leq 5 \text{ tsf}$$

where N_{60} is the corrected (for energy) SPT blow count.

The resistance factor, ϕ, for this check is taken from the Table 3.6.3-1 of the Structures Design Guidelines (SDG) as appropriate for redundant or nonredundant drilled shafts. For piles, use the resistance factor for redundant shafts including end bearing from Table 3.6.3-1 of the SDG. For spread footings, use the resistance factors in AASHTO Table 10.5.5.2.2-1

Consider the effects on foundations constructed in sandy soils due to temporary cofferdams or sheet piles in close proximity. The installation of displacement pile groups within a temporary cofferdam becomes difficult as the degree of soil confinement increases with each pile installed. This confinement generally increases the driving resistance only while the cofferdam or sheet pile remains in place; extraction of the cofferdam or sheet pile (particularly vibratory extraction) may dramatically reduce the final pile resistance from the tip elevation of the cofferdam or sheet pile up to the ground surface, even with non-displacement piles. In addition to ensuring the minimum tip elevation of the foundation is well below the cofferdam or sheet pile, consider the following:

1. Set check perimeter piles after extraction of the cofferdam or sheet pile. If any pile set checked does not meet the required resistance, set check all piles in the group.
2. If seepage is not an issue, use braced sheets with short penetrations.
3. Use preformed pile holes to reduce resistance above the tip elevation of the cofferdam or sheet pile, and ensure NBR is achieved from only the soil and/or rock below the tip elevation of the cofferdam or sheet pile.
4. Instrument all piles to ensure the NBR is achieved after excluding all the temporary resistance from materials above the tip of cofferdam and sheet pile.
5. Ensure the top of rock socket for the drilled shaft is deeper than the tip elevation of the cofferdam or sheet pile.
6. Evaluate the effect of vibrations during extraction on spread footing bearing materials. Ensure the cofferdam or sheet pile tip is above a 1:2 control line (1V:2H) extended from the bottom edge of the spread footing or other shallow foundation.

Cofferdam design, should consider seepage flow and seepage pressure to determine sheet pile penetration depth.

8.4 Embankment Settlement/Stability

The completed embankment must provide sufficient support for value added pavement. (See Specification Sections 338 & 355) ~~These factors~~Embankment settlement and global stability should be addressed concurrently, as various options to solve settlement problems will also impact or be impacted by stability.

8.4.1 Settlement

Settlement calculations should be based on the results of consolidation tests performed on high-quality samples.

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 and included in the report. For compressible clay and organic materials, base total settlement estimates on primary consolidation, and secondary compression (creep) settlements over the design life of the roadway. In these cases, creep estimates must be based on coefficients of secondary compression values obtained from laboratory consolidation test results. Include time rate of settlement estimates; basing these estimates on laboratory or field tests is recommended.

~~-~~For high organic content materials (organic content greater than 50%), base total settlement estimates ~~should be based~~ on primary consolidation, ~~and~~ secondary and tertiary compression (creep) settlements over the design life of the roadway. In these cases, creep estimates must be based on coefficients of secondary and tertiary compression values obtained from laboratory consolidation test results.

If excessive settlement due to compressible clays or organic materials is predicted 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.

~~Design lightweight fill embankments to provide a factor of safety ≥ 1.20 against buoyancy, and lateral movement due to the 500 year storm. Provide details for a PVC or HDPE liner to protect lightweight fills such as EPS geofoam which may be damaged by accidental exposure to chemical or petroleum spills.~~ If surcharge is required, maintain surcharge load until at least 90% of the total expected settlement due to the fill with surcharge has occurred. Design the surcharge loading such that 100% of primary consolidation plus ~~part~~at least half of the ~~secondary~~creep consolidation for non-surcharged embankment has completed before the surcharge is removed.

Design lightweight fill embankments to provide a factor of safety ≥ 1.20 against buoyancy, and lateral movement due to the 500 year storm. Provide details for a PVC or HDPE liner to protect lightweight fills such as EPS geofoam which may be damaged by accidental exposure to chemical or petroleum spills.

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 to be surcharged.
6. Excavate soft compressible material and backfill with granular soil.
7. Ground modification such as stone columns, dynamic compaction, deep soil mixing, etc. (See References 38, 39 and GeoTechTools)
8. Combinations of some of the above.

8.4.2 Stability

Commented [LJ3]: Per AASHTO LRFD BDS 11.6.3.6, slope stability is now performed in the strength limit state.

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.

In the ~~Service~~ Strength Limit State, LRFD slope stability analyses shall be based on a resistance factor of 0.75 when the geotechnical parameters are well defined and reasonably consistent, or based on worst case conditions. When the geotechnical parameters are highly variable, a resistance factor of 0.65 shall be used. ~~at any time the slope will support or impact traffic.~~ Analyses for slopes supporting structures shall include all factored bearing loads from the supported structure~~be based on a resistance factors of 0.65 or lower~~ in accordance with the current AASHTO LRFD Bridge Design Specifications. Analyses are required for all slopes steeper than 2H to 1V. ~~The Department may require a~~Analyses may be needed for flatter slopes depending on soil and site conditions.

8.4.2.1 Design Procedure

References 3, 13, ~~and~~ 18 and 30 are recommended. References 3, 13 and 18 are based on Factor of Safety or Service Limit State analyses which may be helpful, but will need to be modified. Various computer programs are available to assist in the analysis. Identify required reinforcement materials as R-2 or R-3 Geosynthetics when required for Embankments Over Soft Soils or Reinforced Slope applications, respectively.

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.) for the service limit state. 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

Table 2, Geotechnical Engineering Analysis Required in Reference 1 for Embankments, Cut Slopes, Structure Foundations and Retaining Walls

TABLE 3

REQUIRED GEOTECHNICAL ENGINEERING ANALYSIS

Soil Classification			Embankment and Cut Slopes		Structure Foundations (Bridges and Retaining Structures)		Retaining Structures (Conventional, Crib and MSE)	
Unified	AASHTO[1]	Soil Type	Slope Stability[2] Analysis	Settlement Analysis	Bearing Capacity Analysis	Settlement Analysis	Lateral Earth Pressure	Stability Analysis
GW GP GM GC SW SP SM SC	A-1-a A-1-a A-1-b A-2-6 A-2-7 A-1-b A-3 A-2-4 A-2-5 A-2-6 A-2-7	GRAVEL Well-graded GRAVEL Poorly-graded GRAVEL Silty GRAVEL Clayey SAND Well-graded SAND Poorly-graded SAND Silty SAND Clayey	Generally not required if cut or fill slope is 1.5H to 1V or flatter, and underdrains are used to draw down the water table in a cut slope. Erosion of slopes may be a problem for SW or SM soils.	Generally not required except possibly for SC soils. For FDOT Projects, analysis is required for all slopes steeper than 2H to 1V. See 8.4.2	Required for spread footings, pile or drilled shaft foundations. Spread footings generally adequate except possibly for SC soils	Generally not needed except for SC soils or for large, heavy structures. Empirical correlations with SPT values usually used to estimate settlement	GW, SP, SW & SP soils generally suitable for backfill behind or in retaining or reinforced soil walls. GM, GC, SM & SC soils generally suitable if have less than 15% fines. Lateral earth pressure analysis required using soil angle of internal friction.	All walls should be designed to provide minimum F.S. = 2 against overturning & F.S. = 1.5 against sliding along base. External slope stability considerations same as previously given for cut slopes & embankments.
ML	A-4	SILT Inorganic silt Sandy	Required unless non-plastic. Erosion of slopes may be a problem.	Required unless non-plastic.	Required. Spread footing generally adequate.	Required. Can use SPT values if non-plastic.	These soils are not recommended for use directly behind or in retaining or reinforced soil walls.	
CL	A-6	CLAY Inorganic Lean Clay	Required	Required				
OL	A-4	SILT Organic	Required	Required				

[1] This is an approximate correlation to Unified (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 is similar or rock gives required slope angles.

34. Adams, M., Nicks, J., Stabile, T., Wu, J., Schlatter, W. and Hartmann, J., Geosynthetic Reinforced Soil Integrated Bridge System, Interim Implementation Guide, FHWA-HRT-11-026, 2011

35. Adams, M., Nicks, J., Stabile, T., Wu, J., Schlatter, W. and Hartmann, J., Geosynthetic Reinforced Soil Integrated Bridge System, Synthesis Report, FHWA-HRT-11-027, 2011

36. Gray, Kathy M., *Central Florida Sinkhole Evaluation,* http://www.dot.state.fl.us/geotechnical/publications.shtm

37. McVay, M.C., Townsend, F.C., and Williams, R.C., 1992, "Design of Socketed Drilled Shafts in Limestone" ASCE Journal of Geotechnical Engineering, Vol. 118, No. 10, October, 1992.

38. Elias, V., Welsh, J., Warren, J., Lukas, R., Collin, J., and Berg, R., Ground Improvement Methods - Vols 1 & 2, FHWA NHI-06-019 & 020, 2006

39. Bruce, M.E., Berg, R., Collin, J., Filz, G., Terashi, M., and Yang, D., Deep Mixing for Embankment and Foundation Support, FHWA-HRT-13-046, 2013

40. Nam, B.H., Shamet, R., Soliman, M., Wang, D., Yun, H.B., Development Of A Sinkhole Risk Evaluation Program, FDOT Research Report BDV24-977-17, 2018

~~40.~~41. McVay, M.C., Song, X, Wasman, S., Nguyen, T., and Wang, K., Strength Envelopes for Florida Rock and Intermediate Geomaterials, FDOT Research Report BDV31-977-51, 2019

9.1.5 Appendix

All roadway soil survey reports shall include an appendix, containing the following information:

a. Soil conservation (NRCS/USDA) and USGS maps, depicting the project location.

b. Boring location plan, plots of boring logs and/or cone soundings

c. Results of roadway soil survey borings performed.

d. Any other pertinent information.

e. Analysis of the geotechnical information.

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 technical 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, local geology and any assumptions related to the project.

b. Vicinity map, including potentiometric map, USGS and soil survey maps (NRCS/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. See **Figure 32** and **Figure 33** for examples of Report of Core Borings and Report of Cone Soundings sheets. Provide the longitude and latitude of each boring or sounding below the station, offset and elevation, and the depth of temporary casing used to perform the boring on the Report of Core Borings and Report of Cone Soundings sheets. Use the standard soil type symbols shown in **Figure 34** as described in **Table 6** when plotting boring logs. Note the size of rock core sampled. Provide the make and model of the GPS unit used to determine the Latitude and Longitude coordinates of borings, bulk samples, muck probe

4. Estimated short and long term settlements assuming spread footings are constructed in accordance with Specification 455.
5. Soil improvement method(s).
6. If soil material needs to be over excavated and replaced, recommend plan notes specifying the depth of excavation. Provide recommendations for technical special provisions for footing construction, including compaction requirements and the need for particular construction methods such as dewatering or proof rolling in addition to the requirements in Specifications 125 and 455. Estimate the reduction in settlements anticipated resulting from these special requirements.
7. Sinkhole potential.

9.2.5.3 Driven Piles

1. Suitable pile types and reasons for design selections and exclusions.
2. Plotted design curves of soil resistance for selected pile size alternates. Plotted curves should present the Davisson capacity, ultimate skin friction and mobilized end bearing versus pile tip elevation for the existing soil profile. The Davisson capacity is equivalent to the LRFD nominal resistance (R_n).

   Provide ~~Separate~~ separate pile analyses for each recommended pile size, ~~s are to be performed~~ for each SPT boring and/or CPT sounding. Provide ~~A~~ 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, show the corresponding pile capacity curves ~~are to be shown~~together on the same plot and establish the lower bound ~~relationship established~~ for that pile group.
3. Estimated elevation of consistent bearing layer suitable for providing the required nominal resistance without the risk of punching shear failure.
4. Recommendations for pile length or bearing elevation to minimize post-construction settlements in soil layers or punching shear failure of rock or hard layers.
5. Recommendations for pile length or bearing elevation to ~~resist~~ provide the nominal uplift resistance. (The resistance factor for uplift is determined by the Construction QC method used to verify uplift resistance, see Structures Design Guidelines Table 3.5.6-1).
6. Estimated pile settlement and pile group settlement for the minimum tip elevation.
7. Effects of scour, downdrag~~,~~ and lateral squeeze, if applicable.
8. Estimated maximum pile driving resistance to be encountered in reaching the minimum tip elevation. If the FB-Deep Davisson bearing capacity computed at or above the minimum tip elevation exceeds the Maximum Pile Driving Resistance defined in Table 3.2 of the Structures Design

Guidelines, determine the preforming or jetting elevations required to reduce the driving resistance to an acceptable magnitude. Provide additional capacity curves required by the FDOT Structures Design Guidelines on separate pages.

9. Recommended limitations on predrilling/preforming operations to prevent impacts from observed or expected artesian conditions.

10. Recommended locations of test piles.

11. 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: ~~for LRFD designs,~~ the greater of 2 times the factored design load or the design nominal resistance)

12. 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**.

13. Recommendations and special techniques to address the effects of temporary cofferdams or sheet piles on the pile capacity; see Section 8.3.6.

14. Present recommendations for information to be placed in the Pile Data Table shown in the SPI for FDOT ~~Structures Design Guidelines~~Standard Plans Index 455-001.

15. Present soil parameters to be used for lateral analysis accounting for installation techniques and scour. The Geotechnical Engineer shall check the final lateral load analyses for correct soil property application.

16. On small projects with reasonably predictable bearing layers, provide the production pile lengths in the Pile Data Table for each bent rather than test pile lengths.

17. Sinkhole potential and its implications for pile installation and performance.

9.2.5.4 Drilled Shafts

1. Include plots of resistance versus tip elevation for selected alternate shaft sizes. Plots should be developed for both factored (Q_r) and nominal (Q_n) resistance and should show end bearing, skin friction and total resistance (end bearing shall not be discounted). Depths of scour analyzed should be included.

2. Unless otherwise specified, separate shaft analyses for the recommended shaft sizes are to be performed for each SPT boring and/or CPT sounding. Provide resistance versus tip elevation curves for each analysis. When more than one boring is taken at a shaft group or when it is appropriate to otherwise generalize the soil strata, the corresponding resistance versus tip elevation curves are to be shown on the same plot

and a recommended relationship established for that particular structure(s). Indicate the unit skin friction and end bearing values used for the analyses. Ensure socket lengths are sufficient to prevent punching shear failure in cases where the foundation is anticipated to tip in a strong layer underlain by weaker layer.

3. Provide 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).
4. Minimum shaft spacing or influence of group effects on capacity.
5. Effects of scour, downdrag~~,~~ and lateral squeeze, if any.
6. Estimate drilled shaft settlement and shaft group settlement.
7. Recommend 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 (for LRFD designs, the greater of 2 times the factored design load or the nominal resistance).
8. Evaluate the need for technical 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**.
9. Present recommendations for information to be placed in the Drilled Shaft Data Table shown in the FDOT Structures Design Guidelines.
10. Include the potentiometric Surface Map information.
11. Present soil/rock 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/rock property application.
12. Sinkhole potential and its implications for drilled shaft construction and performance.

9.2.5.5 Auger Cast Piles

1. Suitable pile sizes.
2. Plotted design curves of soil resistance for selected pile size alternates. The ultimate skin friction capacity is equivalent to the LRFD nominal resistance (R_n).

   Provide separate pile analyses for each recommended pile size, for each SPT boring and/or CPT sounding. Provide a corresponding pile capacity curve for each analysis. When more than one boring is taken at a pile group or when it is appropriate to otherwise generalize the soil strata, show the corresponding pile capacity curves together on the same plot and establish the lower bound for that pile group.
3. Estimated elevation of consistent bearing layer suitable for providing the required nominal resistance.

4. Recommendations for providing the nominal uplift resistance (see Structures Design Guidelines Table 3.5.20-2).
5. Estimated pile settlement and pile group settlement for the recommended tip elevation.
6. Effects of scour, downdrag and lateral squeeze, if applicable.
7. Recommended locations of demonstration piles and load test piles.
8. The ultimate load for the load test must be shown in the plans (the greater of 2 times the factored design load or the design nominal resistance).
9. Recommendations and special techniques to address the effects of temporary cofferdams or sheet piles on the pile capacity; see Section 8.3.6.
10. Present recommendations for information to be placed in the Drilled Shaft Data Table shown in the FDOT Structures Detailing Manual (Change the title of the table to “Auger Cast Pile Data Table”).
11. Present soil parameters to be used for lateral analysis accounting for installation techniques and scour. The Geotechnical Engineer shall check the final lateral load analyses for correct soil property application.
12. Sinkhole potential and its implications for pile installation and performance.

9.2.6 Roadway and Approach Embankment~~s~~ 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.
3. ~~If surcharge is required, maintain surcharge load until at least 90% of the total expected settlement due to the fill with surcharge has occurred. Design the surcharge loading such that 100% of primary consolidation plus part of the secondary consolidation for non-surcharged embankment has completed before the surcharge is removed.~~

9.2.9 Technical Special Provisions

Technical Special Provisions (TSP's) shall be used to change the Standard Specifications for a project only when extraordinary, project specific conditions exist.

The Department has available a small number of Technical Special Provisions for various items of work tailored to previous projects. These Technical Special Provisions can be obtained from the District Geotechnical Engineer or http://www.fdot.gov/geotechnical/publications.shtm.

TSP's obtained from the Department were tailored to reflect the specific needs of a previous project, and they will need to be updated and revised to reflect the needs of your specific project.

9.2.10 Appendix

All structure investigation reports shall include an appendix, containing the following information:

a. Report of Core Borings Sheet. (See **Figure 32**) (Note the FDOT Geotechnical CADD Standard menu is available ~~for MicroStation~~.)

b. Color photographs of rock cores indicating boring and core elevation.

c. Report of Cone Sounding Sheet. (See **Figure 33**)

d. Data logs or reports from specialized field tests.

e. 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, Splitting Tensile Strength, Unconfined Compressive Strength, Strain at 50% of Unconfined Compressive Strength, Strain at Failure and Corrected Tangent Modulus (adjust the origin to eliminate seating stresses; use the adjusted origin and the slope of the linear portion of the Stress vs. Strain curve).

   2. Rock core data reduction and statistical analyses obtaining design side resistance for drilled shaft socket in rock, if applicable, according to Appendix A of this Handbook.

   3. Gradations: Location, elevation, test results.

   4. Corrosion Tests: Location, elevation, test results.

   ~~4.~~5. Consolidation Tests: plots of e vs. log p' and displacement vs. time (both sqrt time and log time), and index properties of tested materials.

f. Engineering analyses (bearing resistance, lateral stability, group effects, settlement, global stability, punching shear, downdrag, scour, and other applicable analyses).

g. Recommended plan notes.

h. FHWA checklist.

i. Copies of actual field boring logs with all drillers' notes and hand written refinements, if any (not typed logs).

j. 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.
6. Comments issued on the preliminary Report by the District Geotechnical Office and the State Geotechnical Office (if applicable) and the corresponding responses.

9.4 Signing and Sealing

~~Unless plans~~Submittals are required to be electronically signed and sealed~~,~~; 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 5, Signing and Sealing Placement

Geotechnical Report	First page of official copy
Technical Special Provisions	First page of official copy
Roadway Soils Survey Sheet	~~Title Block~~Signature Sheet of the Plans
Report of Core Borings Sheet	~~Title Block~~Signature Sheet of the Plans
Report of Cone Soundings Sheet	~~Title Block~~Signature Sheet of the Plans
Other Geotechnical Sheets	~~Title Block~~Signature Sheet of the Plans

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. See Section 130 of the FDOT Design Manual.

~~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 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 Drainage Engineer.
4. District Structural Design Section.
5. Roadway Design Section.
6. State Geotechnical Engineer (for Category II structures).

Modification for Non-Conventional Projects:
Delete the above distribution list and see the RFP for requirements.

9.6 Plan and Specification Review

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

A marked up set of prints from the Quality Control Review, signed by the geotechnical reviewer, shall be submitted with each phase submittal. The responsible Professional Engineer performing the Quality Control review shall provide a signed statement certifying the review was conducted.

FDOT Standard and Supplemental Specifications shall not be changed except in rare cases, then only with the approval of the District Geotechnical Engineer. The Specifications Office requires a Mandatory Special Provision for all project specific changes to the FDOT Standard and Supplemental Specifications.

Modification for Non-Conventional Projects:
Delete the last Paragraph and insert the following: FDOT Standard. ~~and~~ Supplemental and Developmental Specifications shall not be changed except in rare cases~~,~~ ; then only with the approval of the Engineer.

9.7 Electronic Files

The consultant shall submit an electronic copy of:

1. ~~t~~The final approved geotechnical report in MS Word format. Include the boring log sheets in DGN format, and include the input files used in the analysis programs (FB-Deep, FB-Pier, etc.).

~~1.~~2. The completed Excel spreadsheet with soil boring information for the FDOT GIS Soil Boring Database together with the boring profiles in PDF format.

If the consultant uses a computer program in the design process that is not listed for use in this handbook, the following additional items shall be included in the report submittal:

1. Example hand calculations verifying the results of the consultant's computer programs shall be included in the calculations package.

2. An electronic copy of the geotechnical Consultant's program and the computer input data files.

9.8 Unwanted

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

1. Do not summarize or retype standard test methods or FDOT specifications into the report. Specifications and test methods should be referenced by number, and the reader can look them up if needed.
2. Do not change the Standard Specifications without valid justification. (For example, do not change the MSE wall backfill gradation; base your design on the backfill material required in the Standard Specifications.)
3. Do not include long verbal descriptions when a simple table will be more clear.
4. Do not bury the capacity curves in printed computer output files.

~~9.10 Specifications and Standards~~

~~**Subject**~~	~~**ASTM**~~	~~**AASHTO**~~	~~**FM**~~
~~Standard Practice for the Use of Metric (SI) Units in Building Design and Construction~~	~~E 621~~	-	-

Appendix F

Determination Of Acceptance~~Blow Count~~ Criteria For Driven Piles
&
Determining the Capacity of a Pile from an Instrumented Set-Check

Determination Of ~~Blow Count~~Acceptance Criteria For Driven Piles

Piles must be installed to not less than the Nominal Bearing Resistance (NBR) in the Plans. For details on the computation of NBR refer to the SDG, chapter 3.

The potential effect of nearby construction activities on pile capacity shall be evaluated using acceptable theoretical methods and engineering judgment. For example, the influence of jetting concrete sheet pile or vibratory installation/removal of steel sheet pile in the vicinity of foundation piles shall be considered, when evaluating foundation performance. Confirmation of pile resistance through set-checks after completion of nearby construction is the preferred alternative. When set-checks are not feasible, potential reductions in pile resistance due to nearby construction can be addressed by implementing revisions (increases) to the NBR, minimum tip elevation, or applicable Plan notes.

The following construction quality control methods may be used to determine pile resistance in the field (see SDG Chapter 3 Table 3.5.6-1 for an exhaustive list):

1. Standard pile driving criteria with dynamic monitoring equipment such as the Pile Driving Analyzer (PDA) monitored test pile(s) or monitored production pile(s) in projects without test piles connected to external instruments, signal matching software such as CAPWAP, and Wave Equation Analysis. The dynamic monitoring equipment will normally utilize a program, such as the PDA's PDIPlot program described in this appendix, for viewing the results. (The discussions on this method below use the terms 'PDA', 'CAPWAP' and 'PDIPlot' for simplicity.)
2. Embedded Data Collector (EDC)~~EDC~~ monitoring of all Test Piles and all Production Piles (100%), using tip and top gauges, or a combination of piles with top and tip gauges and piles with only top gauges. A percentage of the piles in each bent/pier must be analyzed with the FDOT Method post-processing software.
3. PDA monitoring of all Test Piles and all Production Piles (100%), with CAPWAP analyses of the percentage of the piles in each bent/pier required in the Specification.

1. Standard Driving Criteria with PDA Test Piles or monitored indicator production pile(s) in projects without test piles**, CAPWAP and Wave Equation Analysis**

In this method dynamic load tests are initially performed on test piles or indicator production piles and a resistance factor (ϕ) of 0.65 may be used in the computation of the required NBR. Dynamic Load tests are performed in accordance with Specification 455. Dynamic data are collected on PDA sensors connected at the top of the pile throughout the entire drive for every impact blow. The purpose of this method is to establish a "calibrated" model that predicts the number of blows per foot and stroke combination to achieve a desired resistance. The Driving Criteria based on PDA testing involves the following steps:

a. Estimation of production pile tip elevation based on PDA results, and preparation of selected blow for CAPWAP analysis
b. CAPWAP analysis to confirm PDA results
c. Wave Equation calibration and final wave equation analysis

stopping in the higher damping soil without bearing. This may be feasible when the test pile shows an increase in capacity with depth and a conservative criteria does not result in unreasonably long production pile lengths.

3. Establish a different criteria for the upper layers to account for the increased damping value of those soils. One criteria will be applicable above a predetermined elevation, and the other will be applicable below that elevation.

2. Embedded Data Collector (EDC) monitoring of Test Piles and Production Piles (100%)

EDC is an approved method for using embedded sensors to monitor pile driving. In this method, dynamic load tests are performed on test piles and all production piles with the EDC system. Sensors are embedded in the pile in accordance with Standard Plans Index 455-003. Test piles may be driven to determine production pile lengths. A resistance factor (ϕ) of 0.75 may be used with this method. No driving criteria are required because achieving the NBR, without exceeding the allowable stress limits, will be determined in the field by EDC monitoring in accordance with either **a.** or **b.** below.

a. EDC monitoring of all Test Piles and all Production Piles, using 100% t~~i~~op and t~~o~~ip gauges.

All EDC piles are monitored in the field using Smart Structures' UF Method. Smart Structures' FDOT Method post-processing software will be used to verify the UF Method results of at least 10% of all piles in ~~all~~each bent~~s~~ and pile footing~~s~~ (minimum one per bent/group) including all test piles. In unique soil conditions such as extreme scour, large uplift loads or high variability soils a higher percentage FDOT Method analyses is required.

b. EDC monitoring of all Test Piles and all Production Piles, using a combination of top & tip gauges and top only gauges.

1. Use top and tip gauges in at least 10% of the piles (minimum one per bent/group) and top only gauges in the remaining piles. All test piles shall contain top and tip gauges. Test piles are included in the 10% minimum. In unique soil conditions such as extreme scour, large uplift loads or high variability soils a higher percentage FDOT Method analyses is required, therefore, a higher percentage of piles with top and tip gauges is also required.

2. In the field, use the UF Method during driving and confirm pile resistance with the FDOT Method after driving is complete for the piles instrumented with top and tip gauges. Use the Fixed Jc/Case Method with back computed/selected Jc value (as described in the below points) for piles instrumented with top only gauges.

3. For the piles instrumented with top and tip gauges, review the FDOT Method results for at least the first 10 blows in the six inches of the drive qualifying the pile for acceptance and use the Fixed Jc/Max Case Method equation to back compute the damping (Jc) value from the ~~known~~ FDOT Method capacity for the representative blow.

4. In the event the back computed Jc value using FDOT method appears to be out of an acceptable range (<0.1 or greater than 1.0), use the UF method capacity and good engineering judgment to determine Jc.

5. When more than one pile in a bent/group must be analyzed, select the highest Jc value of the analyzed piles for the bent/group and/or good engineering judgement to determine which production piles will be based on which Jc value.

6. When the need for set checks is anticipated, the Jc value for set check conditions will be higher than for initial driving. Therefore, the above procedure must be repeated on a set checked pile at the required set-up periods with top & tip gauges to determine the Jc value for set checking a top sensor only pile. When this is not possible use prudent engineering judgement in consultation with and approval by the District Geotechnical Engineer.

~~**2. EDC monitoring of all Test Piles and all Production Piles (100%), using tip and top gauges.**~~

~~In this method dynamic load tests are performed on test piles and all production piles with the Embedded Data Collector system. Test piles are driven first to determine production pile lengths. With this method a resistance factor (ϕ) of 0.75 may be used. No driving criteria are required as satisfaction of achieving the NBR, without exceeding the allowable stress limits, will be determined in the field by EDC monitoring of all piles.~~

3. PDA monitoring of all Test Piles and all Production Piles (100%), with CAPWAP analysis of the percentage of the piles in each bent/pier required in the Specification.

In this method dynamic load tests are performed on test piles and all production piles. Test piles (when required) are driven first to determine production pile lengths, or in cases when the Contractor has chosen to order ~~ordered~~ production piles in advance, to verify that the ordered length is adequate. With this method, a resistance factor (ϕ) of 0.75 may be used in the computation of the required NBR. No driving criteria are required as achieving the NBR, without exceeding the allowable stress limits, will be determined in the field by PDA and CAPWAP. CAPWAP analyses are required on at least 10% of the piles in each bent or pile footing to confirm that the proper damping value, Jc, is used to estimate static resistance of the remaining piles. In unique soil conditions such as extreme scour, large uplift loads or high variability soils a higher percentage of CAPWAP analyses is required. In addition, piles that meet the criteria at significantly different elevations from where CAPWAP was performed, or tip on a different material type, will require separate CAPWAP analysis.

Finally, at least one additional CAPWAP analysis is required for an instrumented re-drive, if this has a different set-up time than other piles evaluated in the pier.

Determining the Capacity of a Pile from an Instrumented Set-Check

In accordance with section 455-5.10.4, the pile capacity to be reported from an instrumented set-check will be the lowest of:

a. The highest capacity recorded in the set-check
b. The average capacity of the five consecutive blows following the highest capacity blow divided by 0.95
c. The lowest capacity of the remainder of the blows (if any, after the blows in b above) in the set-check divided by 0.90

Note, disregard the last blow, which is typically a low energy blow after hammer was shut down.

Example 1, instrumented set-check w/ minimum blows:	Example 2, instrumented set-check and advance pile:
Blow # Capacity, kips	Blow # Capacity, kips
1. 450	1. 450
2. 600	2. 600
3. 590	3. 590
4. 585	4. 585
5. 580	5. 580
6. 575	6. 575
7. 570	7. 570
8. 277	8. 400
	9. 550
	10. 530
	11. 528
	12. 520
	13. 513
	14. 509
	15. 501
	16. 494
	17. 478
	18. 461
	19. 216
Answer: a. Highest capacity recorded= 600 kips	Answer: a. Highest capacity recorded= 600 kips
b. Average of next 5 blows/0.95 = [(590+585+580+575+570)/5]/0.95= 580 kips/ 0.95= 610 kips	b. Average of next 5 blows/0.95 = [(590+585+580+575+570)/5]/0.95= 580 kips/ 0.95= 610 kips
Answer=600 kips	c. Lowest capacity of the following blows (excluding the last one)= 461/.90= 512 kips
	Answer=512 kips

Appendix H

Specifications and Standards

ASTM

Subject	ASTM
Standard Test Methods for Absorption and Bulk Specific Gravity of Dimension Stone	C 97
Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate	C 127
Standard Test Method for Sieve Analysis of Fine and Coarse Aggregate	C 136
~~Standard Test Method for Particle-Size Analysis of Soils~~	~~D 422~~
Standard Test Methods for Chloride Ion In Water	D 512
Standard 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 Soil Solids by Water Pycnometer	D 854
Standard Test Methods for Electrical Conductivity and Resistivity of Water	D 1125
Standard Test Method for Deep Foundations Under Static Axial Compressive Load	D 1143
Standard Test Methods for pH of Water	D 1293
Standard Practice for Soil Exploration and Sampling by Auger Borings	D 1452
Standard 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 (SPT) and Split-Barrel Sampling of Soils	D 1586
Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes	D 1587
Standard Practice for Rock Core Drilling and Sampling of Rock for Site Exploration	D 2113
Standard Test Method for Unconfined Compressive Strength of Cohesive Soil	D 2166
Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass	D 2216
Standard Test Method for Permeability of Granular Soils (Constant Head)	D 2434
Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading	D 2435
Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System)	D 2487
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 Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils	D 2850
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~~

Subject	ASTM
Standard Practice for 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 Foundations 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 Saturated Cohesive 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
Standard Test Method for Sand Content by Volume of Bentonitic Slurries	D 4381
Standard Classification of Peat Samples by Laboratory Testing	D 4427
Standard Test Methods for Crosshole Seismic Testing	D 4428
Standard Practice for Estimating Peat Deposit Thickness	D 4544
Standard Test Methods for One-Dimensional Swell or Collapse of Cohesive Soils	D 4546
Standard Test Method for Energy Measurement for Dynamic Penetrometers	D 4633
Standard Test Method for Determination of Water (Moisture) Content of Soil by Microwave Oven Heating	D 4643
Standard Test Method for Laboratory Miniature Vane Shear Test for Saturated Fine-Grained Clayey Soil	D 4648
Standard Test Method for Prebored Pressuremeter Testing in Soils	D 4719
Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils	D 4767
Standard Test Method for Shrinkage Factors of Soils by the Wax Method	D 4943
Standard Test Method for High-Strain Dynamic Testing of Deep Foundations	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

Subject	**ASTM**
Standard Guide for Field Logging of Subsurface Explorations of Soil and Rock	D 5434
Standard Guide for Planning and Conducting Borehole Geophysical Logging	D 5753
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 Deep Foundations	D 5882
Standard Test Method for Measurement of Hydraulic Conductivity of Porous Material Using a Rigid-Wall, Compaction-Mold Permeameter	D 5856
Standard Practice for Using Significant Digits in Geotechnical Data	D 6026
Standard Practice for Using Hollow-Stem Augers for Geotechnical Exploration and Soil Sampling	D 6151
Standard Test Method for Field Measurement of Hydraulic Conductivity Using Borehole Infiltration	D 6391
Standard Guide for Selecting Surface Geophysical Methods	D 6429
Standard Test Method for Performing the Flat Plate Dilatometer Test	D 6635
Standard Test Method for Integrity Testing of Concrete Deep Foundations by Ultrasonic Crosshole Testing	D 6760
Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis	D 6913
Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications	D 6951
Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures	D 7012
Standard Test Method for Consolidated Drained Triaxial Compression Test for Soils	D 7181
Standard Test Methods for Laboratory Determination of Density (Unit Weight) of Soil Specimens	D 7263
Standard Test Methods for Axial Compressive Force Pulse (Rapid) Testing of Deep Foundations	D 7383
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
American National Standard for Use of the International System of Units (SI): The Modern Metric System	SI-10

AASHTO

Subject	AASHTO
Standard Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes	M 145
Standard Test Method for Sieve Analysis of Fine and Coarse Aggregate	T 27
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 Method of Test for Determining the Liquid Limit of Soils	T 89
Standard Method of Test for Determining the Plastic Limit and Plasticity Index of Soils	T 90
Standard Method of Test for Moisture-Density Relations of Soils Using a 2.5-kg (5.5-lb) Rammer and a 305-mm (12-in.) Drop	T 99
Standard Test Method for Specific Gravity of Soils	T 100
Standard Method of Test for Moisture-Density Relations of Soils Using a 4.54-kg (10-lb) Rammer and a 457-mm (18-in.) Drop	T 180
Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils	T 206
Standard Method of Test for Thin-Walled Tube 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 Method of Test for Measurements of Pore Pressures in Soils	T 252
Standard Method of Test for Determining Expansive Soils	T 258
Standard Method of Test for Laboratory Determination of Moisture Content of Soils	T 265
Standard Method of Test for Determination of Organic Content in Soils by Loss on Ignition	T 267
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
Standard Test Method for Grain-Size Analysis of Granular Soil Materials	T 311

Florida Test Methods

Subject	FM
~~Standard Test Method for Sulfate in Soil and Water~~	~~5-553~~
~~Standard Test Methods for Chloride in Soil and Water~~	~~5-552~~
~~Standard Test Methods for Resistivity of Soil and Water~~	~~5-551~~
~~Standard Test Method for pH of Soil and Water~~	~~5-550~~
~~Standard Test Method for Moisture Density Relations of Soils Using a 10-lb. (4.54kg) Rammer and an 18-in. (457mm) Drop~~	~~5-521~~
~~Standard Test Method for Limerock Bearing Ratio (LBR)~~	~~5-515~~
Standard Test Method for Coefficient of Permeability - Falling Head	5-513
Standard Test Method for Limerock Bearing Ratio (LBR)	5-515
Standard Test Method for pH of Soil and Water	5-550
Standard Test Methods for Resistivity of Soil and Water	5-551
Standard Test Methods for Chloride in Soil and Water	5-552
Standard Test Method for Sulfate in Soil and Water	5-553
Standard Test Method for Determination Of Mean Permeability In The Field Using The Vertical Insitu Permeameter (VIP)	5-614
Standard Test Method for Specific Gravity and Absorption of Coarse Aggregate	1-T 085
Standard Test Method for Particle-Size Analysis of Soils	1-T 088
Standard Test Method for Determining the Liquid Limit of Soils	1-T-089
Standard Test Method for Determining the Plastic Limit and Plasticity Index of Soils	1-T-090
Standard Test Method for Specific Gravity of Soils	1-T 100
Standard Test Method for Moisture Density Relations of Soils Using a 10-lb. (4.54kg) Rammer and an 18-in. (457mm) Drop	1-T 180
~~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 Determination of Organic Content in Soils by Loss on Ignition~~	~~1-T 267~~
~~Standard Test Method for Laboratory Determination of Moisture Content of Soils~~	~~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 Practice for Thin-Walled Tube Geotechnical Sampling of Soils	1-T 207
Standard Test Method for Permeability of Granular Soils (Constant Head)	1-T 215
Standard Test Method for One-Dimensional Consolidation Properties of Soils	1-T 216

Subject	**FM**
Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions	1-T 236
Standard Test Method for Laboratory Determination of Moisture Content of Soils	1-T 265
Standard Test Methods for Determination of Organic Content in Soils by Loss on Ignition	1-T 267
Standard Test Method for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression	1-T 296
Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils	1-T 297
~~Standard Test Method for Specific Gravity of Soils~~	~~1-T 100~~
~~Standard Test Method for Determining the Liquid Limit of Soils~~	~~1-T-089~~
~~Standard Test Method for Determining the Plastic Limit and Plasticity Index of Soils~~	~~1-T-090~~
~~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 Direct Shear Test of Soils Under Consolidated Drained Conditions	3-D3080
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