Soils and Foundations Handbook

2026

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Chapter 1: Introduction

The purpose of this handbook is to provide Geotechnical Engineers with established procedures for performing geotechnical activities for the Florida Department of Transportation. Specifically, this handbook defines the tasks involved in performing a subsurface investigation and the geotechnical aspects of the design and construction of roadways and roadway structures.

As each project presents unique considerations and requires engineering judgment based on a thorough knowledge of the individual situation, the scope of services in the contract for each project supersedes the minimum scope of work outlined in this handbook. The scope of services dictates the specific practices which are to be used on a particular project. Additionally, the scope defines the required interaction between the Department's Geotechnical Engineer and those performing the geotechnical work.

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

This handbook is designed to present information in the same sequence, as it would occur during project development for a design-bid-construct project. A general outline of the tasks, which should be performed by a Geotechnical Engineer during a project, is shown in Sections 1.1.1 through 1.1.4. The details of these tasks are discussed and amplified in subsequent chapters. Chapter 11 discusses the process for a design-build project. A general outline of the tasks, which should be performed by a Geotechnical Engineer for a design-build project, is shown in Sections 11.1 through 11.3.

Finally, it should be noted that this is not intended as an all-encompassing or comprehensive procedural handbook. Methods of subsurface investigation and of analyzing data and solving problems are not discussed in detail. The lists of references at the end of each chapter are but a few of the many sources of information that will provide the engineer with greater insight into investigation procedures and analysis and problem solving techniques. Clarification regarding the content of this Handbook is available from the District Geotechnical Engineer, the State Geotechnical Materials Engineer in Gainesville, and the State Geotechnical Engineer ~~and State Construction Geotechnical Engineer~~ in Tallahassee.

1.1 Geotechnical Tasks in Typical Highway Projects

1.1.1 Planning, Development, and Engineering Phase

- Prepare geotechnical scope of services for consultant projects.
- Assist in corridor and route selection.
- Review existing information.
- Review the Public Soil Boring Viewer (fdot.gov) for previous borings in the area.
- Perform field reconnaissance of site and existing structures.
- Plan and supervise field investigation program, field and laboratory testing.

2.2.3 Geological Maps and Reports

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

2.2.4 Natural Resources Conservation Service Surveys

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

2.2.5 Potentiometric Surface Map

The potentiometric surface elevation shown on the map (see Figure 2-1) can supplement and be correlated with what was found in the field by the drillers. The Potentiometric Surface map can be obtained from the local Water Management District office.

2.2.6 Adjacent Projects

Data may be available on nearby projects from the Department, or county or city governments. Review the Public Soil Boring Viewer ~~(fdot.gov)~~ for previous borings in the area, however, when using this data please note that when the borings were performed, the boring location may be more approximate than is required for borings performed today. The Department may have as-built drawings and pile driving records for the final structure. This data is extremely useful in setting preliminary boring locations and depths and in predicting problem areas. Maintenance records for existing nearby roadways and structures may provide additional insight into the subsurface conditions. For example, indications of differential settlement or slope stability problems may provide the engineer with valuable information on the long-term characteristics of the site.

2.3 Field Reconnaissance

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

1. Nearby structures should be inspected to ascertain their foundation performance and potential to damage from vibration or settlement from foundation installation. Also, the structure's usages must be looked at to check the impact the foundation installation may have (i.e. a surgical unit, printing company, etc.).

2. On water crossings, banks should be inspected for scour and the streambed

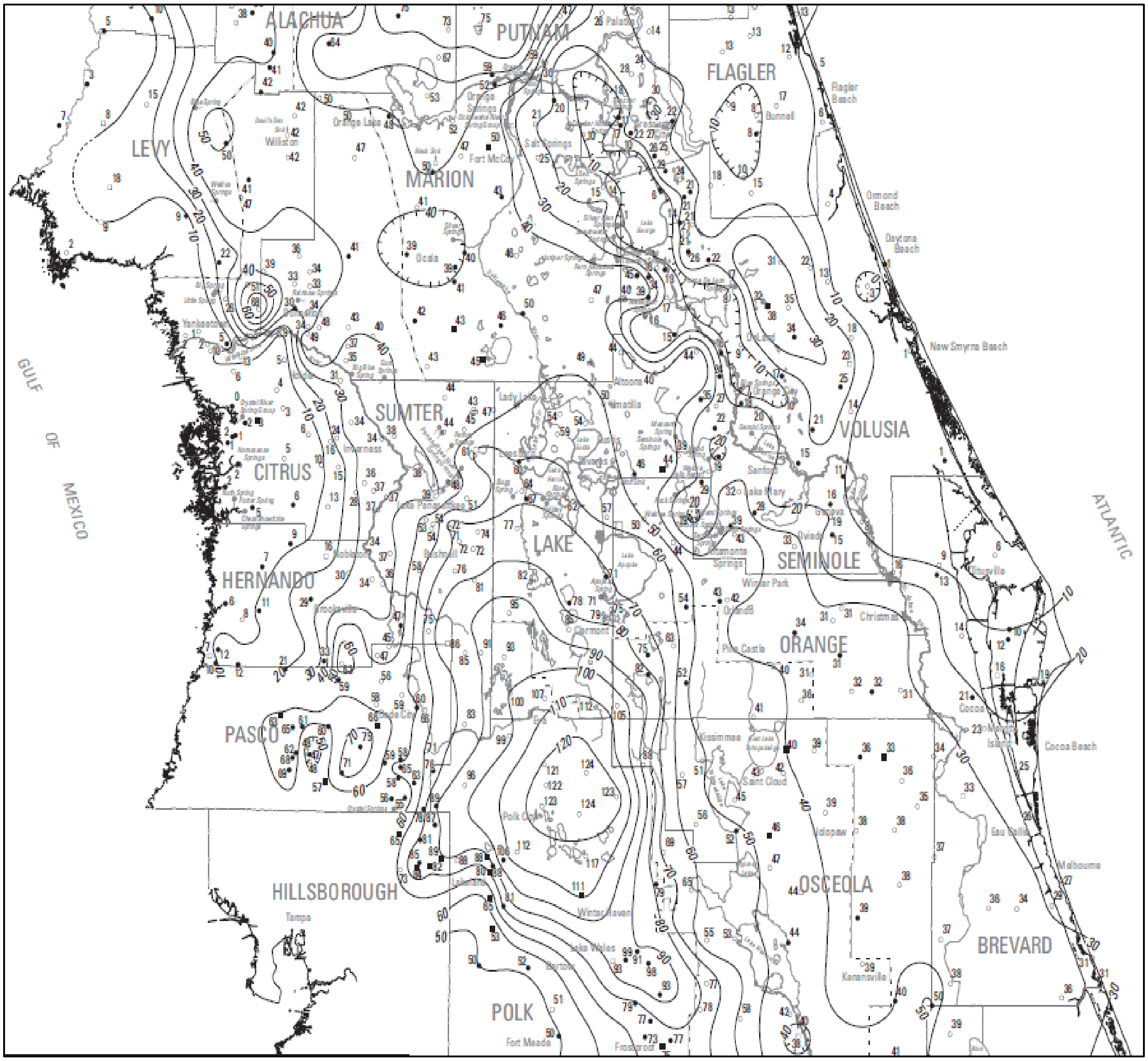


Figure 2-1~~2-12-12-1~~: ~~Excerpt from the~~ Potentiometric Surface Map of the Upper Floridan Aquifer~~St. Johns River Water Management District and Vicinity, Florida, September 1993 map~~

Note: Above figure is an excerpt from the potentiometric surface of the upper Floridan aquifer in the St. Johns River Water Management District and Vicinity, Florida, May 2009
Source: USGS.gov

The actual elevation and location of each boring and sounding including the Station, Offset, Latitude and Longitude, shall be determined by the project surveyor either before or after the boring or sounding is performed. Corrosion testing must be performed for each site unless the structure is designed for the most aggressive conditions.

3.2.2.1 Bridges

1. Minimum frequency of Bridge Foundation Borings (increase boring frequency for highly variable sites). For straddle piers, consider each column as a separate pier:
    a. Spread Footings –
        i. Footings < 70 feet wide - at least one boring per footing
        ii. Footings ≥ 70 feet wide - at least two borings per footing
    b. Driven Piles –
        i. for all bridges without test piles ensure at least one boring confirming the bearing materials is within 50 feet of every pile;
        ii. for bridges with test piles & spans ≥ 60’
            - Bents/pier foundations (pile groups) < 70’ wide
                - at least one boring per bent/pier foundation per structure within 25 feet of each bent/pier footing;
            - Bents/pier foundations (pile groups) ≥ 70’ wide
                - at least two evenly spaced borings within 25 feet of each footing for each bent/pier foundation per structure;
        iii. for bridges with test piles & spans < 60’
            - Bents/pier foundations (pile groups) < 70’ wide
                - at least one boring within 25 feet of every other bent/pier foundation per structure
            - Bents/pier foundations (pile groups) ≥ 70’ wide
                - at least two evenly spaced borings within 25 feet of 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. Nonredundant Drilled Shafts – at least one per shaft (See Item

No. 13 in Section 3.2.2.1~~12~~)

e. Auger Cast Piles (ACP) –

  i. Bents/pier foundations < 70 feet wide - at least one boring per bent/pier per structure within 25 feet of each bent/pier footing;

  ii. 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;

  iii. 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.

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. Boring depths must consider the most likely foundation type for the bridge.

   a. Borings for shallow foundations shall be continued to a depth below the foundation of :

      i. 2B where L< 2B,

      ii. 5B where L > 5B

      iii. Interpolate depth for L between 2B and 5B

      where B is the diameter of a circular foundation or the smaller dimension of a rectangular foundation, and L is the larger dimension of a rectangular foundation.

b. Borings for driven pile foundations tipped in soil shall be continued until all unsuitable foundation materials have been penetrated and the predicted stress from the equivalent footing loading is less than 10% of the original overburden pressure (see Figure 3-1). For pile foundations tipped in rock (with core $q_u \geq$ 550 psi or N=100), continue borings to at least 10 feet below the foundation tip elevations. For piles tipped in weaker materials, continue borings to at least 20 feet below the foundation tip elevations.

Commentary: For typical pile resistances, borings to at least 25 feet of competent bearing material (generally N-values of 50 or greater) will usually satisfy the above.

c. Borings for rock socketed drilled shafts shall continue through competent materials for at least ~~two~~ three shaft diameters below the expected shaft tip elevation (See Item No. 6 in Section 3.2.2.1). Borings for non-rock socketed drilled shafts shall continue through competent materials for at least two times the width of the shaft group below the expected shaft tip elevation. (Scour and lateral requirements must be satisfied.) For nonredundant drilled shafts see additional requirements below.

d. Borings for rock socketed ACP shall continue through competent materials for at least 10 feet below the expected pile tip elevation (See Item No. 6 in Section 3.2.2.1). Borings for non-rock ACP shall continue through competent materials for at least two times the width of the pile group below the expected pile tip elevation. (Scour and lateral stability requirements must be satisfied.)

4. When using the Standard Penetration Test, split-spoon samples shall be obtained at a maximum interval of 2.5 to 3.0 feet and at the top of each stratum. Continuous SPT sampling in accordance with ASTM D-1586 is required in the top 15 feet unless the material is obviously unacceptable for shallow foundations.

5. When cohesive soils are encountered, undisturbed samples shall be obtained at 5-foot intervals in at least one boring. Undisturbed samples shall be obtained from more than one boring where possible.

6. When rock is encountered during the preliminary site investigation typically associated with preparation of the Bridge Development Report (BDR), successive core runs shall be made with the objective of obtaining the best possible core recovery. **SPT's shall be performed between core runs, typically at 5-foot intervals.**

7. When rock is encountered on projects where a rock-socketed drilled-in-

place deep foundation such as Drilled Shafts, Auger Cast piles or Micropiles has been selected for bridge support, the final subsurface investigation shall include a full depth SPT boring to at least three shaft diameters below the anticipated tip elevation, with an adjacent borehole for coring (within 10 feet) at each Pier or Bent, as a minimum. At each Pier/Bent the SPT boring shall be performed first, to delineate the zones where rock coring is required, followed by rock coring of the intervals of interest.

8. In addition to the requirements of Step 7, ~~F~~for bridges (including pedestrian bridges) to be supported by nonredundant drilled shaft foundations (See Section 8.2.3 Drilled Shafts~~.~~), perform at least one SPT boring at each drilled shaft location during the design phase.

9. In situ vane, pressuremeter, or dilatometer tests (See Chapter 4) are recommended where soft clays are encountered.

10. Corrosion series tests (see Chapter 4) are required on all new bridge projects designed for less than the most aggressive conditions. The soil and the water shall be tested. If inland locations are identified to have extremely aggressive environments which do not seem to represent the field conditions, the engineer should obtain three additional samples per project to confirm an extremely aggressive test result and contact the Corrosion Section of the State Materials Office (SM-corrosionsection@dot.state.fl.us).

11. In the case of a water crossing, samples of streambed materials and each underlying stratum shall be obtained for determination of the median particle diameter, D_{50}, needed for scour analysis. Sample and test materials above the maximum probable depth of scour. Consult the Drainage Engineer as necessary when determining this depth.

12. For piers designed for ~~large ship~~ vessel impact loads, pressuremeter tests are recommended to profile the material from the scour elevation to at least seven (7) foundation element diameters below the deepest scour elevation at the pier location.

13. For nonredundant drilled shafts:

    The minimum number of borings required to be evenly spaced at each nonredundant drilled shaft location will be dependent on the shaft size as follows:

Maximum Shaft Diameter (feet)	Minimum Borings/Shaft	Minimum Borings/ Pier
For fairly uniform sites:		
≤ 8	1	1
9 to 10	1	2
For variable sites or karstic areas:		
≤ 7	1	1
8 to 10	2	2

Commentary: Variable sites include those in known variable geologic areas and those determined to be variable (difficult to predict based on other borings) during the subsoil exploration program.

Contact the State Geotechnical Engineer for exploration requirements for drilled shaft diameters larger than 10 feet (if allowed).

Core the limestone load bearing strata and test the core samples recovered. Borings shall extend to not less than three shaft diameters below the proposed/final shaft tip elevation or to the depth required above in Item 3), whichever is deeper. Pilot holes shall be required as necessary during construction in cases where the original boring depth is insufficient, where shafts are lengthened or shaft locations are modified. Borings shall be located by survey and performed within one (1) foot of the shaft location. If access during the design phase limits the ability to accomplish these borings this close to the drilled shaft locations, perform a preliminary boring no farther than 60 feet from the shaft and include plan notes to require the pilot holes to be taken during construction, unless otherwise authorized by the District Geotechnical Engineer. **However, every effort shall be made to perform these borings and test the cores during the design phase in lieu of the need for pilot holes and rock core testing during construction.**

Note the size of rock core sampled in the boring log. The minimum acceptable rock core diameter is 2.4 inches for general design borings (although 4 inch diameter rock cores are preferable). Rock core samples for drilled shaft specific pilot holes should be 4 inches in diameter or larger in order to increase core recovery, RQD and increase the likelihood of obtaining a better quality core.

3.2.2.2 Approach Embankments

1. At least one boring shall be taken at the point of highest fill; the borings

taken for the bridge abutment will usually satisfy this purpose.

If settlement or stability problems are anticipated, due to the height of the proposed embankment and/or the presence of poor foundation soils, additional borings shall be taken along the alignment. If a boring was not performed at the bridge abutment, the first of these borings shall be no more than 15 feet from the abutment. The remaining borings shall be placed at 100-foot intervals until the height of the fill is less than 5 feet. Borings shall be taken at the toe of the proposed embankment slopes as well as the embankment centerline.

2. Borings shall extend to a depth of twice the proposed embankment height and unsuitable founding materials have been penetrated. In the event suitable founding materials are not encountered, borings shall be continued until the superimposed stress is less than 10% of the original overburden pressure (see Figure 3-2).

3. Sampling and in situ testing criteria are in accordance with ASTM D-1586.

3.2.2.3 Retaining Walls

1. At all permanent and ~~critical~~ temporary ~~retaining~~ critical wall locations borings shall be taken at a maximum interval of one per 150 feet of the wall, as close to the wall alignment as possible. Borings shall be extended below the bottom of the wall a minimum of twice the wall height or at least 10 feet into competent material. This applies to all earth retaining structures, proprietary systems as well as precast and cast-in-place. For sheet pile walls, borings shall be extended below the lower adjacent ground surface to a minimum of twice the wall height or at least 10 feet into competent rock.

2. Sampling and in situ testing criteria are in accordance with ASTM D-1586.

3. When existing MSE walls will be widened or modified, collect samples of the existing reinforced fill materials for corrosion testing at a minimum frequency of three samples per mile per wall of existing wall length. Determine the friction angle of the reinforced backfill at the same or greater frequency using a direct (direct shear tests on bulk samples prepared at 95% of the maximum FM 1-T180 dry density) or indirect method (e.g., See Appendix B) as approved by the District Geotechnical Engineer.

3.2.2.4 Noise Walls

1. Noise Wall Borings shall be taken at a maximum interval of one per 500 feet of the wall, as close to the wall alignment as possible. Extend borings below the bottom of the wall to a depth of twice the wall height or 30 feet whichever is less. Increase the boring frequency in variable locations and areas of suspected weak soils such as wetlands, filled wetlands, etc.

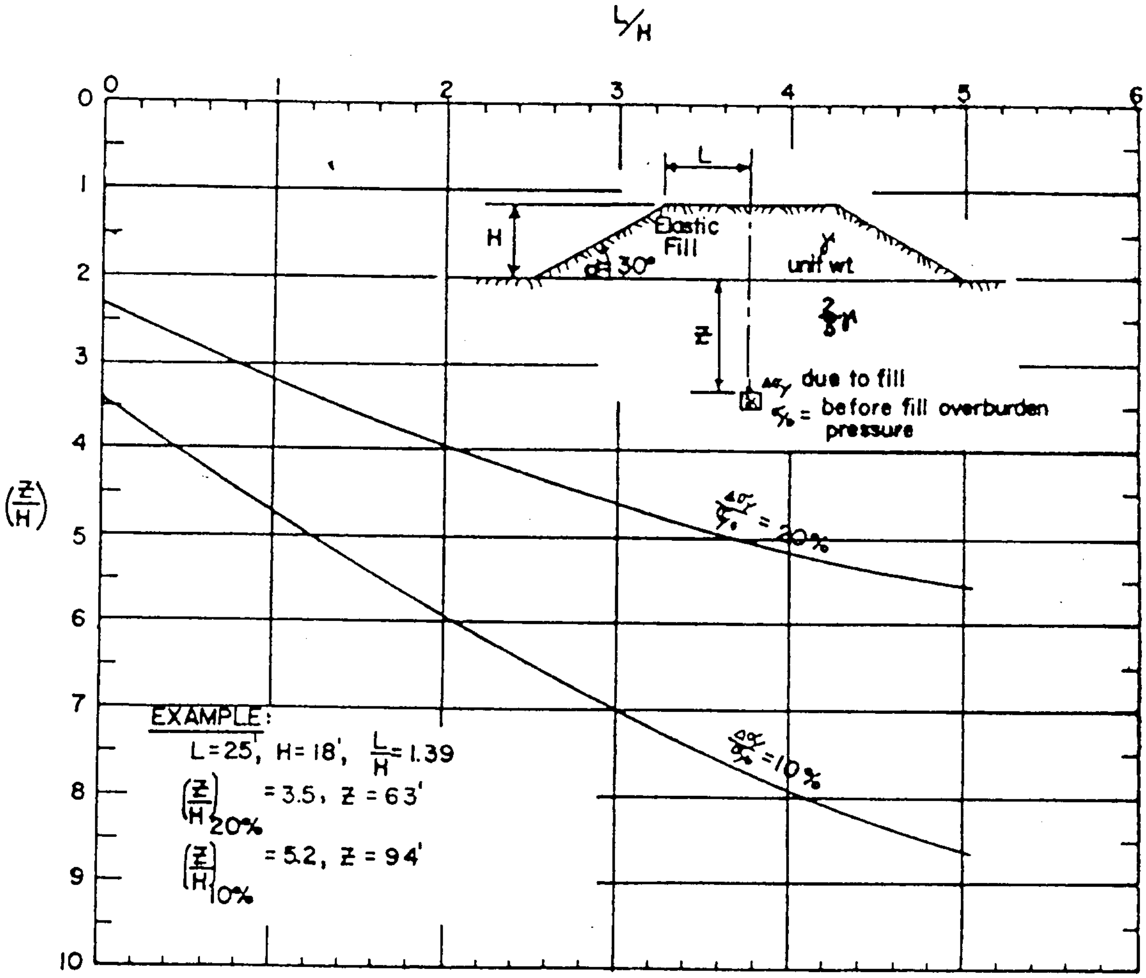


Figure 3-2~~3-23-23-2~~: Determining the Maximum Depth of Significant Increase in Vertical Stress in the Foundation Soils Resulting from an Infinitely Long Trapezoidal Fill

Note in Figure 3-2, native soil and embankment fill ~~both fill and foundation~~ are assumed homogeneous, isotropic and elastic. (After Schmertmann, 1967)

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	D1586	T 206	-

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 D4050 and Reference 17.

4.9.5 Vertical In Situ 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, resistivity, chloride content, and sulfate content. (Refer to the latest Structures Design Guidelines, for the criteria, which defines each class). For roadway drainage systems, test results for each stratum are presented for use in determining alternate culvert materials.

Testing shall be performed in the field and/or the laboratory according to the standard procedures listed below. Once the project's corrosion test results have been reviewed by the District Geotechnical Office, compile the sample data and results into the "Corrosion Series Test Results_SMO.xlsx" Excel form on the Geotechnical Engineering webpage, and email the completed form to SM-corrosionsection@dot.state.fl.us.

Description	FM
pH of Soils	FM 5-550
pH of Water	
Chloride Ion in Water	FM 5-552
Chloride Ion in Soil	
Sulfate Ion in Brackish Water	FM 5-553
Sulfates in Soil	
Electrical Resistance of Water	FM 5-551
Electrical Resistance of Soil	

4.11 Grout Plug Pull-Out Test

This test is performed when the design of drilled shafts in rock is anticipated. However, the values obtained from this test should be used carefully.

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

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

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.

The test is performed by ~~C~~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.

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.

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~~,~~ and 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.

5.1.14 Resilient Modulus Test (Dynamic)

This test is used to determine the dynamic elastic modulus of a base or subgrade soil under conditions that represent a reasonable simulation of the physical conditions and stress states of such materials under flexible pavements subjected to wheel loads. A prepared cylindrical sample is placed in a triaxial chamber and conditioned under static or dynamic stresses. A repeated axial stress is then applied at a fixed magnitude, duration, and frequency. The resilient modulus, M_r, is calculated by dividing the deviator stress by the resilient axial strain. This value is used in the design and evaluation of pavement systems. Tests shall be performed in accordance with AASHTO T 307.

5.2 Rock Cores

Laboratory tests on rock are performed on small samples of intact cores. However, the properties of in ~~-~~situ rock are often determined by the presence of joints, bedding planes, etc. It is also important that the rock cores come from the zone that the constituents can be included in the description using the terminology of ASTM D-2488 through the use of terms such as trace (<5%), few (5-10%), little (15-25%), some (30- 45%) and mostly (50-100%).

6.1.1.3 Grading

6.1.1.3.1 Coarse-Grained Soils

Coarse-grained soils are defined as the following~~either~~:

Well-Graded	Soil contains a good representation of all particle sizes from largest to smallest.
Poorly-Graded	Soil contains particles about the same size. A soil of this type is sometimes described as being uniform.
Gap-Graded	Soil does not contain one or more intermediate particles sizes. A soil consisting of gravel and fine sand would be gap graded because of the absence of medium and coarse sand sizes.

6.1.1.3.2 Fine-Grained Soil

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

6.1.1.4 Relative Density and Consistency

Relative density refers to the degree of compactness of a coarse-grained soil. Consistency refers to the stiffness of a fine-grained soil. When evaluating subsoil conditions using correlations based on safety hammer SPT tests, SPT-N values obtained using an automatic hammer should be increased by a factor of 1.24 to produce the equivalent safety hammer SPT-N value. However, only actual field recorded (uncorrected) SPT-N values shall be included on the **Report of Core Borings Sheet.**

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

Table 6-1: Relative Density or Consistency

Granular Materials		
Relative Density	Safety Hammer SPT N-Value (Blow/Foot)	Automatic Hammer SPT N-Value (Blow/Foot)
Very Loose	Less than 4	Less than 3
Loose	4 – 10	3 – 8
Medium Dense	10 – 30	8 – 24
Dense	30 – 50	24 – 40
Very Dense	Greater than 50	Greater than 40

Silts and Clays		
Consistency	Safety Hammer SPT N-Value (Blow/Foot)	Automatic Hammer SPT N-Value (Blow/Foot)
Very Soft	Less than 2	Less than 1
Soft	2 – 4	1 – 3
Firm	4 – 8	3 – 6
Stiff	8 – 15	6 – 12
Very Stiff	15 – 30	12 – 24
Hard	Greater than 30	Greater than 24

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

The pocket penetrometer and torvane devices may be used in the field as an index of the remolded undrained shear strength of clay samples. See Section 5.1.5.4~~5.15.4~~.

6.1.1.5 Friction Angle vs. SPT-N

Various published correlations estimate the angle of internal friction, φ, of cohesionless soils based on SPT-N values and effective overburden pressure. Some of these correlations are widely accepted whereas others are more likely to overestimate triaxial test data. In the absence of laboratory shear strength testing, φ estimates for cohesionless soils, based on SPT-N, shall not exceed the values proposed by Peck, 1974 (see Figure 6-1). These values are based on SPT-N values obtained at an effective overburden pressure of one ton per square foot. The correction factor, C_N, proposed by Peck, 1974 (see Figure 6-2) may be used to "correct" N values obtained at overburden pressures other than 1 tsf.

6.1.1.6 Moisture Content

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

6.1.1.7 Particle Angularity and Shape

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

6.1.1.8 Organic Content

The organic content of materials can greatly alter its engineering properties. In general, materials with an organic content greater than 5% are considered unsuitable for use in roadway embankments. In some instances, materials with lesser organic contents are desired. Classify organic soils as follows:

- Organic Material = O.C. > 5% but < 20%
- Highly Organic Material = O.C. > 20 but < 75%; highly organic materials are often referred to as “muck” in other FDOT documents.
- Peat = O.C. > 75% (which is defined in ASTM D-4427)

6.1.1.9 Additional Descriptive Terms

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

The term “clean sand” is commonly used to describe A-3 sand which is free of organics, debris, clay lumps, etc.

6.1.1.10 Classification

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

6.1.1.10.1 Unified Soil Classification System (USCS)

This system is used primarily for engineering purposes and is particularly useful to the Geotechnical Engineer. Therefore, they should be used for all structural-related projects; such as bridges, retaining walls, buildings, etc. Precise classification requires that a grain size analysis and Atterberg Limits tests be performed on the sample. The method is discussed in detail in ASTM D-2487 and a summary is reprinted in Figure 6-3 ~~**Figure 18** and **Figure 19**~~ for convenience.

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 AASHTO M 145 (ASTM D-3282), and a summary is reprinted in Figure 6-4~~**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, and Additional Descriptive Terms.

Color	As with soils, the description should be limited to two predominant colors.
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.
Weathering	The degree of weathering should be described. Classical classification systems do not apply to Florida rock.
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. In historic documents "soft limerock" sometimes referred to materials containing limestone or limerock fragments with SPT-N less than or equal to 50 blows per foot.
Formation	Include the name of the geologic formation in parentheses after the description of the sample.
Additional Description Terms	Use any additional terms that will aid in describing the type and condition of the rock being described. Terms such as fossiliferous, friable, indurated, and micaceous are to be used where applicable.

6.2 Logging

The standard boring log included as Figure 6-5~~22~~, or its equivalent as approved by the District Geotechnical Engineer, shall be used for all borings and test pits. A sample completed log is included as Figure 6-6~~Figure 23~~. The majority of information to be included on this form is self-explanatory. Information that should be presented in the remarks column includes:

6.2.1 Comments on Drilling Procedures and/or Problems

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

6.2.2 Test Results

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

6.2.3 Rock Quality Designation (RQD)

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

Chapter 7: Field Instrumentation

7.1 Instrumentation

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

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

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

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

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

7.1.1 Inclinometers (Slope Indicators)

These instruments are used to monitor embankment or cut slope stability. An inclinometer casing consists of a grooved metal or plastic tube that is installed in a borehole. The bottom of the tube must be in rock or dense material, which will not experience any movement, thereby achieving a stable point of fixity. A sensing probe is lowered down the tube and deflection of the tube is measured. Successive readings can be plotted to provide the engineer with information about the rate of subsurface movement with depth (see Figure 7-1~~Figure 24~~). Refer to ASTM D-6230.

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

To monitor embankment construction, inclinometers should be placed at or near the toes of slopes of high-fill embankments where slope stability or lateral squeeze is considered a potential problem. The casing should penetrate the strata in which problems are anticipated. Readings should be taken often during embankment construction. Fill operations should be halted if any sudden increase in movement rate is detected. The applicable portions of the technical special provision T120, T141, T144, T442: Instrumented Surcharge Embankment with Wick Drains should be modified for site conditions, other usable pore-pressure transducer types and instruments, and included in the contract package.

7.1.2 Settlement Indicators

Settlement instruments simply record the amount and rate of the settlement under a load; they are most commonly used on projects with high fill embankments where significant settlement is predicted. The simplest form is the settlement platform or plate, which consists of a square wooden platform or steel plate placed on the existing ground surface prior to embankment construction. A reference rod and protecting pipe are attached to the platform. As fill operations progress, additional rods and pipes are added. (See Figure 7-2~~**Figure 25**~~ or **Standard Plans, Index 141-T01**). Settlement is evaluated by periodically measuring the elevation of the top of the reference rod. Benchmarks used for reference datum shall be known to be stable and remote from all possible vertical movement. It is recommended to use multiple benchmarks and to survey between them at regular intervals.

Settlement platforms should be placed at those points under the embankment where maximum settlement is predicted. On large jobs two or more per embankment are common. The platform elevation must be recorded before embankment construction begins. This is imperative, as all future readings will be compared with the initial reading. Readings thereafter should be taken periodically until the embankment and surcharge (if any) are completed, then at a reduced frequency. The settlement data should be plotted as a function of time.

The Geotechnical Engineer should analyze this data to determine when the rate of settlement has slowed sufficiently for construction to continue. The technical special provision T141: Settlement Plates should be modified for site conditions and included in the contract package.

A disadvantage to the use of settlement platforms is the potential for damage to the marker pipe by construction equipment. Also, care must be taken in choosing a stable survey reference which will not be subject to settlement. If the reference is underlain by muck, other soft soils or, is too close to construction activities, it may also settle with time.

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

7.1.3 Piezometers

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

The simplest type of piezometer is an open standpipe extending through the fill, but its use may be limited by the response time lag inherent in all open standpipe piezometers. More useful and common in Florida are the vibrating wire and the pneumatic piezometers. Pneumatic piezometers consist of a sensor body with a flexible diaphragm attached. This sensor is installed in the ground and attached to a junction box with twin tubes. The junction box outlet can be connected to a readout unit. Pressurized gas is applied to the inlet tube. As the applied gas pressure equals and then exceeds the pore water pressure, the diaphragm deflects allowing gas to vent through the outlet tube. The gas supply is then turned off and the diaphragm returns to its original position when the pressure in the inlet tube equals the pore water pressure. This pressure is recorded (see Figure 7-3). Refer to AASHTO T 252. Vibrating wire piezometers are read directly by the readout unit. Electrical resistance piezometers are also available; however, the use of electrical resistance piezometers is generally limited to applications where dynamic responses are to be measured.

Piezometers should be placed prior to construction in the strata in which problems are most likely to develop. If the problem stratum is more than 10 feet (3 m) thick, more than one piezometer should be placed, at varying depths. The junction box should be located at a convenient location but outside the construction area if possible, however, the wire leads or pneumatic tubing need to be protected from excessive strain due to settlements.

The pore water pressure should be checked often during embankment construction. After the fill is in place, it can be monitored at a decreasing frequency. The data should be plotted (as pressure or feet (meters) of head) as a function of time. A good practice is to plot pore water pressure, settlement, and embankment elevation on the same time-scale plot for comparison. The technical special provision T144: Pore-Pressure Transducers (Piezometers) should be modified for site conditions and included in the contract package.

7.1.4 Tiltmeters

Tiltmeters measure the inclination of discreet parts of structures from the norm. They are most commonly used to monitor tilting of bridge abutments and decks or retaining problems due to the presence of rock, etc. The effect of these characteristics on roadway performance should be assessed.

8.2 Foundation Types

As an absolute minimum for Design-Bid-Build projects, GRS abutments, spread footings, driven piles, auger cast piles and drilled shafts should be considered as potential foundation types during the preliminary or Bridge Development Report (BDR) phase for each bridge structure. For noise barrier walls, auger-cast piles may be the preferred foundation. On some projects, one or more of these alternatives will be obviously not feasible for the subsurface conditions present. Analysis of design capacity should be based on SPT and/or cone penetrometer results, laboratory and/or in-situ strength tests, consolidation tests, and the results of instrumentation programs, if available. Consider the need for additional field tests based on the variability of the conditions observed. After the foundation type has been selected in the BDR phase, only the selected foundation type needs to be evaluated further if the final geotechnical investigation confirms it is suitable for the entire structure.

Analyze all foundations in accordance with the latest requirements of the *AASHTO LRFD Bridge Design Specifications* except where specific requirements have been superseded by the Structures Design Guidelines or those contained herein. Particular attention shall be paid to deflections in the service limit state, especially for drilled shafts where large deflections may be required to satisfy the strength limit state.

Evaluate the foundation depths and conditions of all existing structures close enough to likely be affected by construction activities. Ensure the selection of the proposed foundation type will not affect the integrity and stability of the existing foundations. Some existing bridges and retaining walls may be particularly vulnerable to certain foundation construction procedures, such as:

a. Shallow foundations and short piles founded on soils that may settle due to construction vibrations.

b. Piles driven near existing piles on widening projects. The existing piles may experience lateral and downdrag forces during the driving of a displacement pile that advances in very close proximity. Particular attention must be placed on piles that were installed with a batter angle and existing piles accepted based on soil set-up (soil freeze) during original construction.

c. Drilled shafts constructed using vibratory methods for the casing installation and removal, in proximity of shallow foundations, short piles, and MSE walls.

d. Drilled shafts using casing that may not be long enough to support the excavation when sidewall caving could affect the stability of adjacent shallow foundations, short piles, or MSE walls.

FDOT existing structures damaged due to foundation settlements and failures are typically very costly to repair and will potentially create: extended delays, safety concerns, and bridge/roadway closures during construction.

8.2.1 Spread Footings

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

8.2.1.1 Design Procedure

References 3, 5, and 22 offer good methods. Provide the minimum foundation elevation and the anticipated bearing material. Estimate settlements, including the amount of total settlement, rate of settlement, and the potential for differential settlement.

For spread footings on rock or IGM, ensure against punching failure into the weaker stratum below the bearing stratum (See 8.3.1 ~~Rock Fracture~~). Evaluate the effect of excavation aids such as continuous sheet pile which could compromise the continuity of the bearing layer (See 8.3.6).

8.2.1.2 Considerations

Varying depths of footings should be considered to achieve maximum economy of design. For water crossings, depth of scour will be a controlling factor, which may preclude consideration of spread footings. Difficult conditions for dewatering and preparation of foundation soils shall be addressed when applicable. Ground improvement methods which permit the use of spread footings in otherwise marginal cases (grouting, vibratory compaction, etc.) may be considered where their use might be more economical than deep foundations.

8.2.2 Driven Piles

Driven piles must be designed for axial and lateral loading conditions as applicable. The following types of driven piles are considered acceptable for supporting structural loads on permanent FDOT structures (depending on environmental restrictions): Steel H-piles, Steel Pipe Piles, Prestressed Concrete Piles 18" square and larger, and Concrete Cylinder Piles of 54" or 60" diameter. 14" square Prestressed Concrete Piles may be used for pedestrian bridges if there are no environmental restrictions. Timber piles may be used for temporary bridges, however, steel piles are chosen more often by contractors. Other pile types and sizes may be considered for design-build projects and contractor's Cost Savings Initiative (CSI) submittals.

8.2.2.1 Design Procedure

The following computer programs are available the **Bridge Software Institute** (BSI):

- FB-Deep is available for assessment of axial design capacity.
- FB-MultiPier is available for assessment of lateral design capacity and pile group settlement.
- GeoStat is available for the evaluation of site variability.

The Help Files for the FB-Deep, FB-MultiPier, and GeoStat programs are recommended references. Include all materials within 3B of the individual pile tip or 2 times the minimum group dimension below the tip of the piles, whichever is deeper. Unless otherwise approved by the Director of Design, all driven pile bridge foundations require 100% dynamic testing.

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). Address pile group effects, settlement and downdrag as applicable. References 5, 6, 7 & 30 are recommended for analyzing group effects and settlement potential. See Appendix C for a step-by-step design procedure for the analysis of downdrag.

8.2.2.2 Considerations

Various pile types and sizes should be analyzed to achieve an optimum design. For water crossings, depth of scour must be considered for both axial and lateral load analyses. Test pile locations should be recommended and the need for static load testing addressed. Consider the drivability of the piles. See the Structures Design Guidelines for load limits for driving of different pile sizes. In FB-Deep and GeoStat analyses, code sand layers containing 30% (“Some” by ASTM D-2488) or greater quantities of shell as soil type 4.

On FDOT projects, steel pipe piles may be used in extremely aggressive conditions only if driven closed-end and filled with a cast-in-place concrete core in accordance with SDG 3.1.F.2 (See SDG 3.1.F & SDG Table 3.1-1 for additional information).

8.2.3 Drilled Shafts

Drilled shafts derive their resistance from direct contact between the surrounding soil and the drilled shaft concrete. As with driven piles, drilled shafts must be designed considering both axial and lateral loads. Proximity to embankments, retaining walls, or other sources of stress increase that could affect the stability of the excavation during drilled shaft construction shall be addressed during design, and may require the need to specify temporary and/or permanent casing.

8.2.3.1 Design Procedure for Major Structures

Resistance factors and associated design methods for geotechnical resistance of drilled shafts are in SDG Table 3.6.3-1. It is implicitly shown in the table that the resistance factors for drilled shafts tipped in sand or clay are based on side shear design methods only (i.e. FHWA alpha method in clay and FHWA beta method in sand). Note also that the beta method for side shear resistance in sand refers to the method developed by O’Neil & Reese (Reference 9), not the beta method described in FHWA's GEC 10.

Because tip movements on the order of several inches are generally required to mobilize tip resistance in sand or clay, methods to pre-mobilize tip resistance must be incorporated to include tip resistance in these materials. Methods to pre-mobilize tip resistance include: pressure grouted tips, rim cell devices and bi-directional load test jacks.

Reference 9 is generally applicable to all conditions except for drilled shafts socketed in Florida limestone. Refer to Appendix A for an approved method of determining the side resistance for drilled shafts socketed in Florida limestone. The normal spacing for drilled shafts is 3D. For rock socketed drilled shaft groups with spacing of 2.5D or greater, a group efficiency factor of 1 may be used for axial loads; for shafts tipped in other materials refer to the current AASHTO LRFD Bridge Design Specification. P-y multipliers for lateral loads are in the Structures Design Guidelines. General foundation analysis considerations are further described below.

The following computer programs are available from the **Bridge Software Institute** (BSI):

- FB-Deep is available for assessment of axial design capacity
- FB-MultiPier is available for assessment of lateral design capacity and shaft group settlement
- GeoStat is available for the evaluation of site variability and minimum number of laboratory tests required to develop side shear design correlations

The Help Files for the FB-Deep, FB-MultiPier, and GeoStat programs are recommended references.

Nonredundant drilled shaft bridge foundations have special design requirements as follows:

1. All shafts in nonredundant bridge foundations shall be a minimum of four feet in diameter.
2. Consider the effects of combined axial loads and moment to properly evaluate the geotechnical bearing resistance of the shaft and the effect on the distribution of the stresses across the shaft bottom. There is often 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).

   Prudent design considers that estimated side shear resistance requires sufficient surface area of the shaft to interact with the socket. Design

8.2.5 Microp~~P~~iles

In special cases such as low over-head, vibration sensitive sites, zones with utility conflicts or retrofit, micro-piles may be the preferred foundation system. ~~This would typically be in cases of limited access, close proximity to settlement sensitive structures and foundations to be strengthened.~~ See the SDG for restrictions on the use of micropiles for bridges and other structures. When used for bridge support in rock-socket applications, Appendix A shall be used for design.

8.2.5.1 Design Procedure

Designs must comply with Section 10.9 of Reference 30 for soil, and Appendix A for rock and Intermediate Geomaterial. However, all side shear resistance in the casing plunge length shall be disregarded. References 26 and 30 are recommended for background information. Static Load Tests are required to verify the design.

8.2.6 GRS Abutments

GRS abutments are part of FHWA's Every Day Counts (EDC) initiative to reduce bridge construction time and cost. Bridge projects constructed using this technology were considered cost effective, are performing well, and the lessons learned during those projects led to the GRS Guide (Reference 34). GRS abutments are permitted for simply supported spans as described in the Structures Manual; for additional background, see References 34 and 35.

8.2.6.1 Design Procedure

Designs must comply with Appendix C of Reference 34, except as otherwise indicated in Sections 3.12.12 and 3.13.4 of the Structures Design Guidelines.

Present GRS abutments in the Plans. The Plans may or may not utilize Developmental Standard Plans, Index D6025, however, the same information needs to be presented. GRS abutments shall be constructed using Developmental Specification 549. The District Specifications Office needs to file the request for Developmental Specification 549 to be incorporated into the specifications package.

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 8-1~~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 unreliable for nearly all Florida limestone and IGM materials, estimate the shear resistance within the limestone 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:

$q_{frac} = 0.1 \text{ tsf x } N_{60} \leq \cancel{5}\ 4 \text{ tsf}$

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.

8.3.2 Lateral Loads

Lateral load analyses for deep foundations shall be performed on all retaining structures and almost all bridges permitting navigation. The Structural Engineer using soil parameters provided by the Geotechnical Engineer shall perform the analyses for bridges. The Geotechnical Engineer shall check the final lateral load analysis for correct soil property application. The associated minimum tip elevations requirement (elevation where structure stability is achieved plus 5 feet) must be reviewed. Designs may need to be changed if lateral deflection is excessive. Reference 10 is recommended.

8.3.3 Scour

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

8.3.4 Downdrag

For piles driven through a soil layer(s) subject to settlement, a load transfer (negative total expected settlement due to the fill with surcharge has occurred. Design the surcharge loading such that 100% of primary consolidation plus at least half of the 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

Stability analyses are conducted using data from ~~performed based on the results of~~ in ~~-~~situ strength tests and/or laboratory strength tests performed on high quality samples. A range of possible material strengths is typically~~often~~ considered, ~~thus providing the~~ allowing engineers to evaluate a spectrum of ~~with a range of~~ soil resistance and assess slope ~~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.~~The overall stability of the retaining wall, retained slope, and foundation soil or rock shall be evaluated using limit equilibrium methods of analysis in accordance with the AASHTO LRFD Bridge Design Specifications. The analysis shall provide an ASD equivalent minimum ~~safety~~ factor of safety of 1.5.

~~In the 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. Analyses for slopes supporting structures shall include all factored bearing loads from the supported structure in accordance with the current AASHTO LRFD Bridge Design Specifications.~~

Slope stability analyses are generally not required when slopes are constructed entirely of select fill on level ground and are 2.5H:1V or flatter. However, analyses are required for all slopes that support structures, contain non-select fill, constructed on sloping ground, or slopes steeper than 2.5H:1V. ~~When constructed entirely of select fill on level ground, analyses are generally not required for 2H to 1V or flatter slopes. Analyses are required for all slopes supporting structures, containing non-select fill or constructed on sloping ground. Analyses are required for all slopes steeper than 2H to 1V.~~

8.4.2.1 Design Procedure

References 3, 13, 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 problem is identified early in the planning phase.
3. Flatten slope (Right of way requirements?).
4. Staged construction, to allow soft soil to gain strength through consolidation.
5. Excavate and replace soft soils.
6. Include geotextile or geogrid within the embankment.
7. Place berm at toe.
8. Use lightweight fills.
9. Ground modification such as stone columns, dynamic compaction, deep soil mixing, etc. (See References 38, 39 and GeoTechTools).
10. Using obstructions to keep vehicles from parking on or approaching the crest of the slope.
11. Installing an underdrain system to depress the phreatic surface in the slope.
12. Constructing a trench at the top of the slope to divert surface water from the slope face.

13. Combinations of the above.

8.5 Retaining Wall Design

All retaining walls; including gravity walls, cantilever walls, crib walls, ~~and~~ mechanically stabilized earth (MSE) walls, and soil nail walls; must be designed in accordance with the current AASHTO LRFD Bridge Design Specifications (except as noted in the FDOT Structures Manual, Volume I, ~~FDOT Structures Design Guidelines~~ Structures Design Guidelines (SDG) and the FDOT Design Manual (FDM)) with adequate soil resistance against bearing, sliding, overturning, eccentricity and overall stability. A design analysis is still required when standard index walls are used on a project.

~~The design water elevation for all walls is the flood stage elevation of the 100 yr or 500 yr storm event, whichever controls the design. Consider also that the retained fill and surcharging materials may be saturated or submerged during this period.~~

~~For coastal walls, designing for a more severe storm event may be required, and the rapid drawdown water differential is the maximum wave height.~~

8.5.1 Gravity Walls

8.5.1.1 Design Procedure

Reference 17 is recommended.

8.5.1.2 Considerations

All gravity walls including those taken from the Standard Plans for Road and Bridge Construction should be checked for stability. These walls are sensitive to differential settlements so they must be carefully checked. Refer to the FDOT SDG and the FDM for procedures on design of walls.

8.5.2 Counterfort Walls

8.5.2.1 Design Procedure

References 30 and 17 are recommended for Counterfort walls.

8.5.2.2 Considerations

This type of wall is typically not as economical as an MSE wall, but it is competitive with other walls. It can be used in extremely aggressive environments. Speed of construction is another advantage in congested areas.

Refer to the FDOT SDG and the FDM for procedures on design of walls.

8.5.3 MSE Walls

8.5.3.1 Design Procedure

References 30 and 13 are recommended for design of MSE walls.

8.5.3.2 Considerations

Refer to the Structures Manual, Volume I – SDG, Section 3.13.2 for water elevation design requirements.

The use of proprietary MSE wall systems is growing more common as rights-of-way become limited and congestion grows. FDOT maintains standard indices of wall systems pre-approved for use as permanent and temporary critical ~~temporary~~ walls.

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

8.5.3.3 Widening Existing Walls

Widening of a roadway supported by MSE walls may require the MSE facing to be moved outward (widened) some distance from its present position. This process may or may not include adding additional height to the wall.

When existing MSE walls in good condition and performing well are to be widened, evaluate the remaining service life (internal stability) of the existing MSE wall based on the minimum density, friction angle, and maximum corrosivity of the existing reinforced fill. From bulk samples of the reinforced fill collected per 3.2.2.3 for direct measurements, at locations and elevations approved by the District Geotechnical Engineer, perform tests for:

a. Modified Proctor Density (FM1-T180)

b. Direct Shear (FM3-D3080)

c. Environmental Corrosion Tests (See Section 4.10)

Refer to the original shop drawing submittal to determine the original configuration and dimensions of the metallic reinforcement, and the assumed corrosion rates per SDG 3.13.2. Use these corrosion rates to determine the remaining service life for the existing reinforcement.

If the remaining service life of the existing reinforcement is less than the design life of widened wall after construction, use the corrosion reduced strength of the existing reinforcement in evaluating the internal and external stability of the widened MSE wall.

References 30 and 13 are recommended for evaluating existing MSE walls.

8.5.4 Sheet Pile Walls

8.5.4.1 Design Procedure

Reference 17 is recommended for sheet pile walls.

8.5.4.2 Considerations

The engineer is responsible for all permanent and temporary critical sheet pile walls~~ and all temporary sheet pile walls considered critical~~. When coatings will be used on wall panels, ensure the friction between the coated wall panel and the soil is properly considered; assume zero friction when a bitumen coating is used.

Steel sheet piles are normally installed using a vibratory hammer; concrete sheet piles are installed by jetting. It is important to alert the contractor to soil or rock layers that will make sheet pile installation difficult by providing appropriate information in the Plans. Consider preforming and other installation effects on the resulting friction between the wall panel and the soil or rock.

Consider the effects of cofferdams or sheet piles constructed near (within 5D of) foundations, particularly when the foundations are supported fully or partially on sandy soils. Reference 42 is recommended for evaluating the effects of sheet pile proximity on driven foundation piles. (See also Section 8.3.6).

8.5.5 Soil Nail Walls

8.5.5.1 Design Procedure

References 17 and 23 are recommended for soil nail walls.

8.5.5.2 Considerations

Soil nail walls in sand may require large movements to mobilize soil resistance, and vertical excavations may not be achievable.

8.5.6 Soldier Pile/Panel Walls

8.5.6.1 Design Procedure

The analysis and design of soldier piles is different from sheet pile walls because the failure of individual pile elements is different from continuous walls. The failure mechanism of the individual pile is analogous to a bearing failure in front of the pile; the total resistance force assumes the pile has an effective width of 3B (or three times the width of the pile) for all types of soil. The bearing resistance pressures for cohesive soils are considered to be uniform with a magnitude of 2c (two times the cohesion) neglecting the soil resistance of 1.5 times the pile width (B) from the bottom of excavation. For granular soils, determine K_p with or without wall friction and neglect the soil resistance to a depth equal to one B below the bottom of excavation. When wall friction is considered, the interface angle δ must not exceed the value given in Table 1 of Reference 5 for the applicable soil and soldier pile materials. References 17 and 30 are recommended for Soldier Pile/Panel walls.

8.5.6.2 Considerations

the information required in 9.2.3.

d. Any other pertinent information.

e. Analysis of the geotechnical information. Include correlations and calculations.

9.1.6 Geotechnical Data for NexGen Plans

The Final Geotechnical Data deliverable for a project is a xml file generated from the FDOT Geotechnical Data Manager (GDM) application. This application is delivered with the FDOT Connect software and is located in the C:\FDOTConnect\Organization-Civil\FDOT\Apps\GeotechDataManager folder.

The Geotechnical Data file should be named XXXXXXXXXXX- modifier.xml where XXXXXXXXXX is the project number, the modifier is optional but can be used to differentiate data files if more than one is delivered with the project. The xml file should be stored in the project's Geotechnical discipline folder.

The GDM application provides the ability to manually create a deliverable xml file from scratch.

If a database system is used for storing Geotechnical data, the data can be exported out in Excel format that the GDM software can convert to the required xml format. Excel formatted templates are provided to show how the exported data shall be formatted for conversion by GDM. These spreadsheets are provided in the project's Geotechnical folder. The existing spreadsheet format should not be modified in any way. Once the database data is exported, it should then be imported into GDM then exported out to the final deliverable xml data file.

Training for this process can be found at the following YouTube location: https://www.youtube.com/watch?v=L944Hj2eJ98

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 9-4 and Figure 9-5 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 9-6 as described in Table 9-2 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 areas, etc.

   These sheets are included in the final plans; see the **Core Borings** section of the FDOT Structures Manual, Volume II, *Structures Detailing Manual* for additional requirements for these sheets.

d. Estimated depths of scour (usually determined by the Hydraulics Engineer), if applicable.

e. Environmental class for both substructure and superstructure, based on results of corrosivity tests. This information is also reported on the Report of Core Borings sheet. For extremely aggressive classification note what parameter placed it in that category.

f. Summary table of soil parameters determined from field and laboratory testing.

g. Table of soil parameters to use with computer modeling (such as ~~the FB-Pier or~~ FB-MultiPier program). These parameters can be broken up into zones across the bridge length. Include all correlations used to develop the table.

h. Recommendations and considerations for any proposed walls. MSE or cast-in-place wall recommendations.

i. Discussion of undesirable conditions observed in the borings and undesirable conditions present in the geologic formation(s) encountered at the site, together with any impact on proposed construction.

Modification for Non-Conventional Projects:
Add j: Discussion of anticipated procedures for mitigating undesirable conditions observed in the borings or expected due to the geologic formation(s) encountered at the site.

9.2.4 Existing Structures Survey and Evaluation

Existing structures to be protected may include sensitive sites, such as those listed in FDM Chapter 117. The Roadway Design Office has determined the Roadway Engineer will generally determine whether there are sensitive sites, such as those listed in 117.3 in proximity to the project. The Department will make a final determination whether revised thresholds of settlement and vibration are warranted.

When requested by the EOR:

1. The geotechnical design effort should evaluate these structures and confirm monitoring during construction is warranted based on the anticipated soil type, building characteristics (type, use, condition, etc.), proximity and the proposed construction operations.
2. Assist the EOR in developing mitigation strategies and evaluating whether limits on vibration limits and settlements more stringent than those specified in Specification ~~section~~ 108 should be required for these structures.
3. Recommend and discuss with the Department the potential need of specifying different movement thresholds.
4. Prepare a Modified Special Provision to specify the revised thresholds of vibration and settlement identifying the sensitive sites where these thresholds shall apply.

Where there is a potential impact on existing structures in the surrounding area, the geotechnical report should include the structure's address, type of construction, the estimated vibration level that may cause damage, the usage (storage building, hospital, etc.), what the potential problem may be and what actions should be taken to minimize the impact. Ensure that settlement and vibration monitoring are specified in the plans for the sites requiring these revised thresholds

Where construction dewatering may create a potential impact on existing structures in the surrounding area, the report should include the structure's address, type of construction, the degree of dewatering that may cause damage, the usage, what the potential problem may be, and what actions are recommended to minimize the impact.

Ensure settlement monitoring of existing foundations including those of FDOT owned structures is specified in the plans when potentially vulnerable to vibrations, pile driving, and excavations as discussed in Section 8.2.

9.2.5 Structure Foundation Analysis and Recommendations

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

9.2.4.

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

15. Present recommendations for information to be placed in the Pile Data Table shown in the SPI for FDOT Standard Plans Index 455-001.

16. 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.

17. 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.

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

19. Include a list of pay items and their associated quantities in the Geotechnical report and ensure they are implemented in the plans. Refer to the Specifications 455 for driven pile pay items and the BOE for details.

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. Recommendations for special notes and possible MSP to avoid impacts of potentially damaging installation procedures on existing foundations including FDOT owned structures as discussed in Section 8.2.

9. 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.

10. Present recommendations for information to be placed in the Drilled Shaft Data Table shown in the FDOT SDM.

11. Include the potentiometric Surface Map information.

12. Present soil/rock parameters to be used for lateral analysis accounting for installation techniques and scour.

13. The Geotechnical Engineer shall check the final lateral load analysis for correct soil/rock property application. 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). Rock socketed auger cast piles for structures, other than miscellaneous, shall be designed in accordance with Appendix A.

   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 SDG 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

f. Verify stability for fully saturated conditions.

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 from the following website: 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 (or Report of SPT Borings) Sheet. (See Figure 9-4) (Note the FDOT Geotechnical CADD Standard menu is available.)

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

c. Report of Cone Sounding Sheet. (See Figure 9-5)

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.

   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). Include correlations and calculations.

g. Recommended plan notes.

i. Protection of Existing Structures Checklist

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

k. 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

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 9-1: Signing and Sealing Placement

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

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 Modified 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. 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. 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-MultiPier, etc.).
2. The completed Excel spreadsheet with soil boring and location 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~~clearer.
4. Do not bury the capacity curves in printed computer output files.

Crosshole Sonic Logging has been used to determine the integrity of drilled shafts and slurry walls. The test involves lowering probes to the bottom of water-filled access tubes, and recording the compression waves emitted from a source probe in one tube by a receiver in another tube at the same or different (offset) elevations. The probes are pulled back to the surface at the same rate, and this procedure is repeated at various test configurations in order to obtain a profile of the entire depth of the shaft. Potential defects are indicated by delays in the signal arrival time and lower energies at a given test depth. This test method is limited to detecting defects between the access tubes used during each test. Since access tubes are needed for this test, the design of the reinforcement cage must take the total number and location of these tubes into account. Concrete mixtures producing large amounts of bleed-water have caused CSL tests to indicate zones with apparently poor-quality concrete. Refer to ASTM D-6760.

10.4.3 Gamma-Gamma Density Logging

Gamma-gamma density logging is performed using a radioactive source and receiver within the same access tube. It is used to measure changes in uniformity of the cylindrical zone surrounding the outside of the access tube. The radius of the tested zone is dependent on the equipment used. This test method can be used to detect anomalies outside the cage of reinforcing steel.

10.4.4 Thermal Integrity Testing of Drilled Shafts (TITDS)

Thermal integrity testing uses the heat of the hydrating concrete to differentiate concrete from soil. It can scan the shaft concrete both inside and outside the reinforcing cage within 1 to 2 days after the shaft is poured. As the temperature profiles obtained from embedded thermal wires or ~~logging~~ access tubes are matched to 3-D thermal modeling information, the configuration of the completed shaft is determined. Refer to ASTM D-7949.

10.5 Drilled Shaft Construction

Using the wet method during construction of a drilled shaft, slurry is used to maintain a positive head inside the open shaft in order to keep the hole open prior to placement of concrete. In order to ensure the slurry shall meet the requirements to perform properly, the following control tests shall be performed: density, viscosity, sand content, and pH of the slurry. Refer to FM 3-D4380~~8-R13B-1~~, 8-R13B-2, 8-R13B-3, and 8-R13B-4, respectively.

In order to evaluate the quality of the rock directly below the shaft excavation, rock cores shall be taken to a minimum depth of 5 feet and up to 20 feet below the bottom of the drilled shaft excavation of redundant drilled shafts or three shaft diameters below the bottom of the drilled shaft excavation for nonredundant shafts. Coring shall be performed in accordance with ASTM D-2113 using a double wall or triple wall core barrel. The core barrel shall be designed to provide core samples 4 inches in diameter or larger, and allow the cored material to be removed in an undisturbed state. Refer to ASTM D-2113 and ASTM D-5079.

Guidance for the interpretation of drilled shaft tip grouting results can be found in the research publication "Load and Resistance Factors Design (LRFD) Resistance Factors for Tip Grouted Drilled Shafts, BDV25-977-37", Final Report, 2019.

10.6 Shaft Inspection Device (SID)

SID is ~~A~~a piece of equipment that is used to inspect the bottom cleanliness of drilled shafts prior to placement of concrete through the use of an inspection bell which houses a high resolution video camera (See Figure 10-6). The inspection bell is lowered from a service platform to the bottom of the shaft, and the operator can view the condition of the bottom via the camera. The bell is fitted with a depth gage to indicate the thickness of debris on the shaft bottom. Sufficient views of the shaft bottom are used to inspect a statistically significant portion of the shaft bottom. The Shaft Inspection Device uses pressurized nitrogen to overcome the static head of the drilling fluids, purge the fluids from the camera bell, and provide an unobstructed view of the shaft. A small reduction in air pressure would allow drilling fluid to slowly enter the bell.

When the shaft bottom is flat (as required in Specifications) and the bell is plumb, a layer of water or drilling fluid in the bell can be used to measure the thickness of sediments mounds "away" from the sediment depth gauge. When the fluid rises to the ½" pin on the gauge, the percentage of the view covered with sediment deposits thicker than ½" may be estimated; these sediments are above the fluid level. When the ½" depth pin is missing the first mark at ~~(~~1.0 cm~~)~~ depth must be used. The same procedure may also be used to determine whether any portion of the view contains sediments in excess of 1-½" [4.0 cm] thick. Special care must be used to ensure the fluid does not erode the sediment as it enters the bell, especially if the operator attempts to fill the bell with water using the water jets intended for flushing these sediments, instead of filling the bell with drilling fluid as described above.

10.7 Field Instrumentation Monitoring

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

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

10.8 Troubleshooting

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

Chapter 11: Design-Build Projects

A ~~sufficient number of geotechnical borings needs to be attached to the~~subsurface exploration that satisfies the requirements of this document shall be completed for inclusion in the RFP to ~~give~~ provide the Design-Build ~~DB~~ Teams an understanding of the geotechnical conditions for the project. When considered necessary by the District Geotechnical Engineer or th~~es~~ State Geotechnical Engineer~~possible~~, a more extensive geotechnical investigation ~~should~~ will be performed ~~for Design-build projects than for normal design-bid-construct projects. The total effort may exceed 120% of a normal investigation in order~~ to assist the Teams in offering their most cost effective solution for the project. During the design and construction phase, the Design-build team may still need to perform~~s~~ additional exploration to finalize their project-specific design~~the design specific investigation~~. The Design-build team shall be responsible for its own analysis of any and all data used ~~by the team~~for design and construction.

11.1 Planning and Development Phase:

11.1.1 Department's Geotechnical Engineer Responsibilities

The Department's geotechnical engineer performs a geotechnical investigation to fully support the RFP Concept Plans. It is necessary to perform as complete a geotechnical field and laboratory investigation as access permits and provide the data to the Design-build teams for their use in preparing preliminary designs and technical proposals. Upon completion of the preliminary subsurface investigation, the information obtained must be signed & sealed, and compiled in a format, which will present the data collected to the various design-build teams. The evaluation of the subsurface data should establish the limits of areas of relative uniformity for load testing. The results of the geotechnical investigation performed to support the RFP Concept Plans are provided to prospective teams as Attachments to the RFP~~.~~. Preliminary geotechnical reports prepared by the Department for use by design-build teams should not include analysis of the geotechnical information or any suggestions for handling any potential problems.

11.1.2 Design-build Team Responsibilities

Design-build teams are not yet selected at this time. Potential teams submit letters of interests from which a short list is determined.

11.2 Technical Proposals & Bidding Phase

11.2.1 Department's Geotechnical Engineer Responsibilities

The Department's geotechnical engineer answers questions from the design-build team through the project manager, reviews technical proposals and provides recommendations to other technical reviewers regarding the completeness and appropriateness of proposed supplemental field testing, ground modification, and load testing programs, etc.

11.2.2 Design-Build Team Responsibilities

Short listed design-build teams perform analyses of the geotechnical data and any additional data they gather independently. The teams determine the appropriate design and construction methods based on their approach/equipment, the requirements provided in this document and the Request For Proposals for the

Appendix A Determination of Design Side Shear Resistance from Rock Core Test Data for Drilled Shafts, ~~&~~ Auger Cast Piles, and Micropiles Socketed in ~~the~~ Florida Limestone and Cohesive IGM~~Based on Rock Core Testing~~

~~DETERMINATION OF DESIGN SIDE SHEAR RESISTANCE FROM TEST DATA TO DESIGN PARAMETERS~~
~~FOR~~
~~DRILLED SHAFTS~~, ~~& AUGER CAST PILES~~, ~~AND MICROPILES~~ ~~SOCKETED IN FLORIDA LIMESTONE BASED ON ROCK CORE TESTING~~

~~Remark: This article is an updated version of the papers Peter Lai presented in the 1996 &1998 Design Conferences, as well as the inclusion in the Appendix of the FDOT's Soils and Foundation Handbook, 2002. This update is to clarify the contents that are most often misinterpreted by engineers and present an example.~~

A.1. Introduction

Characterization of the design strength of Florida limestone and well-cemented Marl that classifies as Intermediate Geomaterial, requires a substantial amount of laboratory testing to develop the required statistics for each layer. Thick layers may need to be sub-divided to account for variability in the vertical direction and to avoid averaging errors. In addition, the project site may also need to be sub-divided to account for horizontal variability, and a separate set of statistics developed for each zone. ~~The variable strength properties of the Florida limestone formation always prompted the question of what design side shear resistance should be used for a drilled shaft socketed in it. Some engineers even decide that doing any tests on rock cores obtained from the project site is senseless because of the uncertainties associated with a spatial variability of the limestone. This presentation provides a method for determining a reasonable design side shear resistance value~~The resistance factors associated with bridge foundation design of drilled in-place deep foundations embedded in rock or competent cohesive IGM are associated to the procedure outlined in this appendix, and require ~~from a~~ a statistically significant number of ASTM D-7012 (Method D) unconfined compression and ASTM D-3967 (with t/D ≥ 1.0) splitting tensile tests.

A.2. Design Method

On the basis of the study ~~done~~ performed by the University of Florida, the following method proposed by Prof. McVay ~~seems to be the most~~is appropriate for design of drilled foundations in ~~the~~ Florida limestone~~s~~ and cohesive geomaterial. The ultimate side shear resistance for the portion socketed in the rock is expressed as:

$$f_{su} = \frac{1}{2} \times \sqrt{q_u} \times \sqrt{q_t} \quad (1)$$

where,

f_{su} is the ultimate side shear resistance,

q_u is the unconfined compression strength of rock core, and

q_t is the splitting tensile strength (McVay, 1992).

$$(f_{su})_{DESIGN} = REC \times f_{su} \quad (2)$$

To ~~consider the spatial variations of the rock qualities~~address discontinuities in the rock formation, the ~~average~~ ***REC*** (% recovery in decimal) is applied to the ultimate unit side shear resistance, f_{su}, and the product is used as the design ultimate side shear resistance, after the process outlined below has been implemented.

This method has been used by ~~the~~ Department engineers for several years ~~now~~ and ~~it~~ has generally yielded ~~provided~~ reasonable estimates of design side shear resistance when ~~as~~ compared to static ~~with~~ load test data. ~~However, there are some uncertainties of how to obtain the q_u, q_t, and **REC** values.~~

A.3. Rock Sampling and Laboratory Testing

~~A critical component of this design method work is the quality of the rock cores.~~ The rock core sample quality is dependent upon the sampling techniques as well as the size and type of the core barrel used. Due to the porous nature of ~~the~~ Florida limestone, the larger diameter samplers are ~~more~~ favorable ~~than the smaller diameter samplers~~ since they will provide more representative specimens. ~~Therefore, in the FDOT's 'Soils and Foundation Manual', a~~The minimum core barrel size ~~of~~ allowed is 61 mm (2.4") I.D. ~~is required~~ and a 101.6 mm (4")

I.D. core barrel is recommended ~~for better evaluation of the Florida limestone properties~~. Furthermore, ~~the manual also requires~~ a triple or double barrel must be used ~~as a minimum~~ to have ~~better~~ accurate percentages of recovery ~~as well as~~ and rock quality designation (RQD) ~~RQD depending on the core size~~. After obtaining ~~the better~~ quality core samples, the engineer can select ~~more~~ representative specimens for laboratory unconfined compression and splitting tensile tests. ~~Thus better shear strength test data can be obtained for more accurate design side shear resistance.~~

A.4. Variability

The variability of ~~the~~ Florida limestone formations ~~is~~ can be very large. To obtain representative design values ~~for drilled shafts~~, ~~one has to obtain a lot of~~ a large number of rock core samples is typically required. The number of tested specimens needed for the design depends on the desired level of confidence. The following relationship identifies the ~~amount of~~ standard error (E) in terms of the number of laboratory specimens tested (n), the desired confidence level (t), and the standard deviation of the strength test being evaluated (σ) ~~can be expressed as~~ (Smith, 1986),

$$E = \frac{t\sigma}{\sqrt{n}} \quad (3)$$

This equation is useful to gauge the number of core specimens needed for ~~the~~ a particular design confidence level. ~~, however, since the variability of the rock strengths is so big that the mean value of the samples cannot be used for design most of the time. As an aid, plotting a~~ Generating a frequency distribution ~~(histogram)~~ of the rock/IGM core test results (both the q_u and q_t results individually) ~~can assist~~ may prove useful to the designer ~~in~~ when evaluating the need for additional sampling and testing, and ~~determining a sufficient number of tests in order to identify~~ identifying ~~a clear~~ the type of distribution of the data (~~i.e.~~ e.g., normal, log-normal, etc.)

A.5. Check the Big Picture

~~First the borings and core recoveries and test results for a project need to be reviewed for~~

~~uniformity.~~ Determine whether the ~~test results are~~available data is reasonably consistent ~~across~~, or if generating vertical and horizontal layers is necessary ~~the project, whether there are different approximate areas or sites~~ (Paikowsky, 2004). ~~within the project, whether there are two or more reasonably different strata, or whether the project is so variable that each boring appears to be from a different site.~~ A histogram of the rock core test results can help identify secondary peaks in the data which could suggest the need for sub-dividing the site. ~~may indicate a secondary distribution exists within the project site. This would indicate that there are significant site variabilities which warrant separating the data into multiple sets to represent different areas or strata within the project.~~

~~When borings show extreme variability, the engineer needs to prudently reconsider whether the drilled shaft design is likely to be appropriate for each and every pier on the~~

~~project.~~ When the ~~project location subsoils~~strength characteristics are so variable that trends cannot be reasonably discerned, the need for more data~~,~~ or a different foundation type~~, is needed~~shall be evaluated.

A.6. Data Reduction Method

The data reduction method outlined herein is designed to yield q_u, q_t, and REC, which are used for estimating side shear resistance in rock formations. This method involves a sequence of analytical steps applied to each area or sub-site within the project boundaries.~~The data reduction method presented here is intended to provide a means to obtain a more reliable q_u, q_t and **REC** values that can provide realistic design side shear resistance for the rock formations. This method involves the following steps of analyses for each area or site within the project limits.~~

1. Find the mean and standard deviations of both the q_u and q_t strength test data sets.
2. Establish the upper and lower limits of each type of strength test data set by using the mean values, ± one standard deviation.
3. Discount all the data in the data sets that are larger or smaller than the established upper and lower limits, respectively.
4. Recalculate the mean and standard deviation of the data within the boundaries of each modified strength test data set computed above.
5. Establish the upper and lower bounds of q_u and q_t by setting the calculated mean value as the design upper bound value and the mean minus one standard deviation as the design lower bound value.
6. Use the q_u and q_t obtained from steps 4 and 5 to calculate the respective upper and lower bounds of the ultimate side shear resistance, f_{su}.
7. Multiply the ultimate side shear resistance f_{su} by the mean ***REC*** (in decimal) to account for the spatial variability. This step is performed to obtain a global design value. However, it needs to be checked against the local recovery at each pier/bent.

~~8.~~ Consider these two design boundaries the global side shear resistance values for the area or site within the project. Select the corresponding resistance factor in accordance with the Structures Manual, Volume I – ~~Structures Design Guidelines~~DG, Table 3.5.20-2,

Table 3.6.4-1 or Table 3.6.4-2, as applicable.~~A resistance factor should be applied to these side shear resistance values depending on the construction method used. The following table may be used as a guide to obtain an appropriate a resistance factor for the Load and Resistance Factor Design (LRFD) method.~~

~~Resistance Factor, φ~~

~~Drilled Shaft Design Basis~~	~~Redundant~~	~~Nonredundant~~
~~Neglecting end bearing~~	~~0.60~~	~~0.50~~
~~Including 1/3 end bearing~~	~~0.55~~	~~0.45~~
~~Static Load Testing*~~	~~0.75~~	~~0.65~~

~~*Number of load tests required depends on the uniformity of the project.~~

~~9.~~8.~~The engineer should then decide which value is appropriate for the design. For a project with uniform subsurface, a few load tests may qualify the use of the resistance factors listed above. However, if the subsurface at the project is erratic, it requires more tests to qualify for the use of these factors because each area or site within the project limits requires separate load tests. If a representative soil profile cannot be obtained, the number of load tests may be as many as the number of various soil profiles that are existing at the project.~~

~~10.~~9. ~~10~~Generate a chart to index SPT N-values to the global side shear resistance boundariesy~~ values~~ determined in Step 8~~ with the boundary SPT N-values performed between core runs~~. ~~The boundary SPT N-values vary from various geological formations. In general, the lower bound N-value range from 20 to 30 blows per foot and the upper bound ranges from 50 to 100 blows per foot. N-values falling within these boundaries~~Use the site-specific correlation ~~can used~~ to reduce the design resistance at elevation intervals where the blow count falls below anticipated values.~~obtain the design side shear resistance values from the chart.~~ ~~Note that the~~These correlations are ~~for~~ site-specific~~ site use only since the SPT N-values are being used as indices~~. ~~See~~ Refer to 3.2 for requirements related to SPT borings and rock coreing ~~requirements during stru~~ for structures~~ borings~~.

~~11.~~10. ~~11~~Check the global ~~Design~~ design values against ~~the shaft based on~~ local (pier or bent specific) boring logs and rock coring results. When N values are absent, use the design lower bound rock strength to design the drilled shaft socket.

The following example is meant to illustrate the analyses outline above. The data, especially the side shear resistance vs. SPT-N-value chart, are not meant for any real design purposes.

Example: *Design a shaft with 48" diameter and in a group of four shafts. Each shaft will support a factored design load of 2,500 kips. Assuming there will not be any load test for the project. ~~Thus, a~~A resistance factor of 0.55 will be used for the design.*

- Steps 1 to 5 Rock test data reduction

The global design ultimate side shear resistance should also account for the spatial variability of the site by multiplying the mean %REC value (in decimal) to the above mean, which based on FDOT experience is more representative as the high end value, and low values respectively and obtain;

Design Upper Boundary $(f_{su})_{DESIGN} = 0.485 \times \cancel{*}48.3 = 23.4\ ksf$

Design Lower Boundary $(f_{su})_{DESIGN} = 0.485 \times \cancel{*}27.1 = 13.1\ ksf$

~~Select the appropriate design resistance factor based on design conditions and whether the design parameters for this site will be verified by a load test.~~Select the corresponding resistance factor in accordance with the Structures Manual, Volume I – ~~Structures Design Guidelines~~DG, Table 3.6.4-1.~~A resistance factor should be applied to these side shear resistance values depending on the construction method used. The following table may be used as a guide to obtain an appropriate a resistance factor for the Load and Resistance Factor Design (LRFD) method.~~

~~Resistance Factor, φ~~

~~Drilled Shaft Design Basis~~	~~Redundant~~	~~Nonredundant~~
~~Neglecting end bearing~~	~~0.60~~	~~0.50~~
~~Including 1/3 end bearing~~	~~0.55~~	~~0.45~~
~~Static Load Testing*~~	~~0.75~~	~~0.65~~

~~*Number of load tests required depends on the uniformity of the project.~~

~~The engineer should then decide which value is appropriate for the design. For a project with uniform subsurface, a few load tests may qualify the use of the resistance factors listed above. However, if the subsurface at the project is erratic, it requires more tests to qualify for the use of these factors because each area or site within the project limits requires separate load tests. If a representative soil profile cannot be obtained, the number of load tests may be as many as the number of various soil profiles that are existing at the project.~~

- Step ~~10~~ 9 Generate a ~~design~~ side shear resistance chart

Using the above calculated global ultimate design shear resistance together with the lower and upper bound SPT N-values of 25 and ~~50~~ Refusal (the minimum and maximum SPT values in the rock core data set being evaluated, for example 50 blows for 6-inches), respectively; the following design chart is generated.

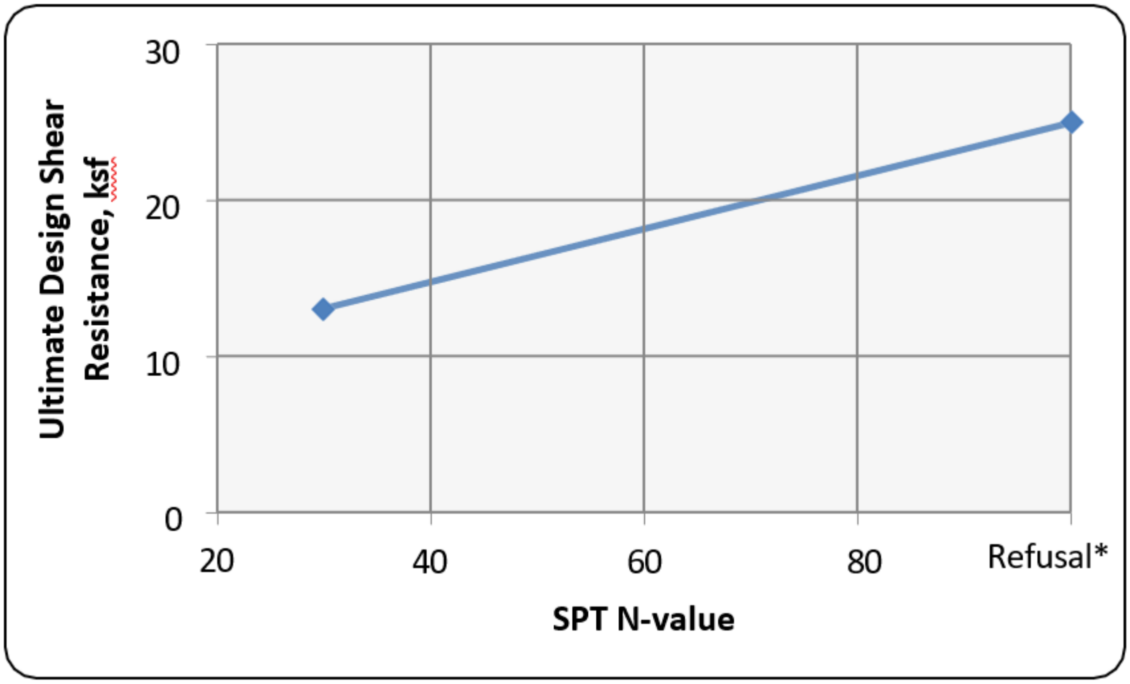
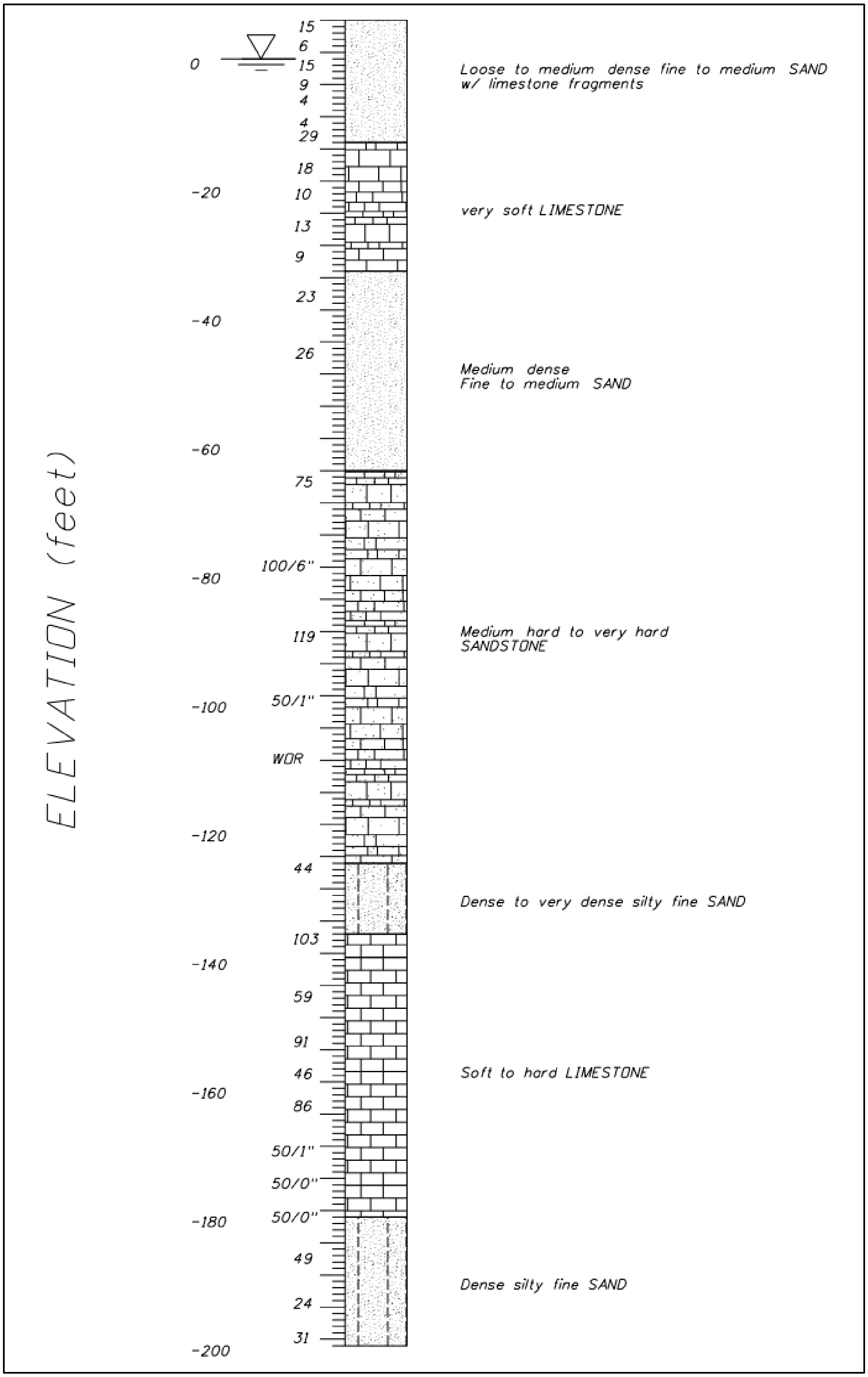


Figure A-3: Ultimate Design Shear Resistance vs. SPT N-value

*Refusal as defined in ASTM D1586.

- Step ~~11~~ 10 Design ~~shaft~~ the cast-in-place foundation using local subsurface information – boring log/rock core at the pier location



The following table is a summary of the boring log and how it will be used for~~the~~ ~~drilled shaft~~ design.

Layer	Soil Description	Elevation (ft)	Thickness (ft)	Avg. N-value	Unit Side Shear (ksf)	Side Resistance (Kips/ft)
1	Sand	5.7 to -13	18.7	9	*	-
2	Soft Limestone	-13 to -23	10	16	**	-
3	Sand	-23 to -64	41	25	*	-
4	Limestone	-64 to -109	45	>Refusal	23.4	294.4

Notes:

*Neglected because of high ground water table and casing may be used.

**The soft limestone layer is very close to the top of the shaft. If casing is used, the rock-casing interface will shatter during the installation. In the second case, if casing is not used, the rock-shaft interface will slip and the deformation will pass the peak strength strain into the residual strength range due to high stress concentration at the top part of the shaft. Thus, in both cases, the upper limestone stratum will behave like granular material and should be designed as such.

Diameter of shaft	D = 48" or 4'
Perimeter area per foot of shaft	$A = \pi \times D = 12.57\ ft^2$
Side resistance per foot of rock socket, kips/ft	R_s = A x unit side shear = 294.4
Nominal Bearing Resistance (kips)~~Factored design load, kips~~ $= \frac{Factored\ Design\ Load}{\varphi}$	$\cancel{Q}NBR = \frac{2500}{0.55} = 4545.5$
Total required socket length, ft	$L = \frac{4545.5}{294.4} = 15.4$

~~Thus the design shaft should socket~~A rock socket embedment of 15.5 feet into the limestone with a tip ~~or tipped at~~ elevation of -79.5' is required for a foundation designed to provide resistance in friction only~~if only side resistance is used~~. ~~Shaft base resistance can also be utilized for design, however, a strain compatible design, such as O'Neal's Design Method for IGM must be used.~~

A.7. References:

1. 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.
2. O'Neill, M., "Drilled Shafts: Construction Procedures and Design Methods" Publication No. FHWA-IF 99-025, August 1999.

B.1. General

In order to accommodate the post supports of noise walls and reinforcement with the required cover, the normal foundation diameter is approximately 30 inches. It is generally desirable and efficient to limit foundation depths to 25 or 30 feet. If the design indicates a 30-inch diameter foundation will need to be longer than 30 feet, a larger diameter foundation should be considered. Refer also to 8.2.4.1.

B.2. Noise Barrier Foundations

See Section 8.2.4.1.

B.3. Lateral Load Resistance

Use a Load Factor in accordance with the latest AASHTO LRFD Bridge Design Specifications.

When required, computer programs such as FB-MultiPier, LPILE, or COM624 may be used to determine the deflections and rotations.

B.3.1. k values in Sands.

For structures subject to lateral loads due to a storm event, k values input into FB-MultiPier, LPILE, or COM624 shall not exceed the following values in pounds per cubic inch, without lateral load tests:

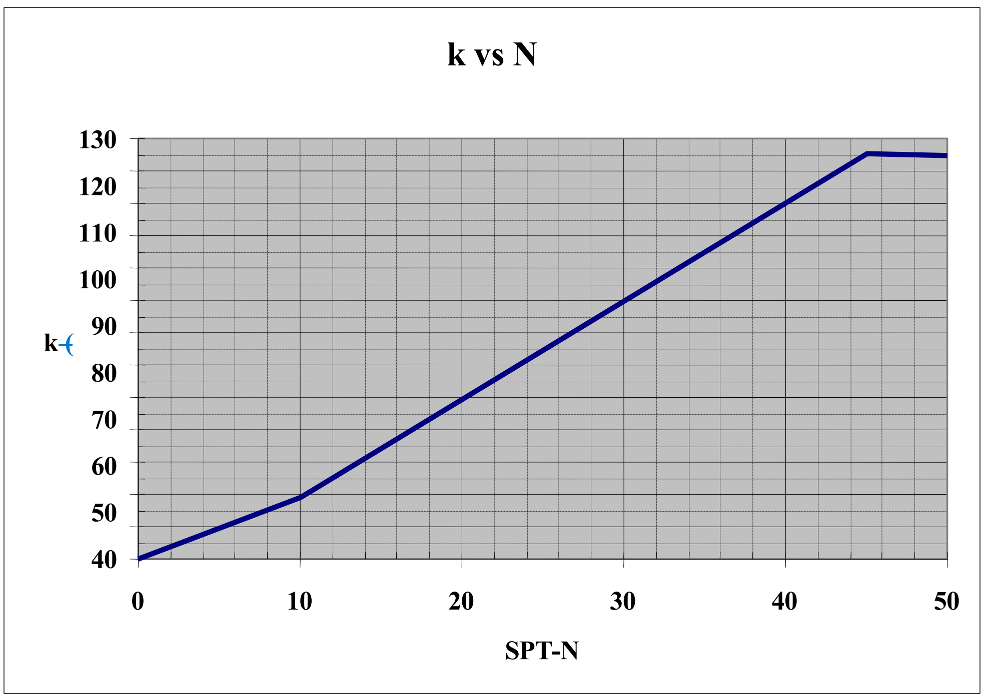


Figure B-1: k vs. N

Note: Since submerged conditions are likely to exist when the design load condition occurs, make no distinction between dry and submerged conditions.

B.4. Friction Angles in Sand

The following typical correlation may be used to estimate the soil friction angle, ϕ:

$$\phi = \frac{N}{4} + 28$$

As an alternative, the procedure described in 6.1.1.5 ~~in 6.1.1.5 Friction Angle vs. SPT-N~~ shall be used. The maximum ϕ value shall be limited to 35 degrees for silty sand (A-2-4) and 38 degrees for clean sand (A-3), unless higher friction angles are statistically supported by laboratory shear strength test results.

B.5. Clay

Use the LPILE or COM624 program guideline to determine k and ε_{50} values. However, limit the properties of clay to stiff clay or weaker (design values for undrained shear strength shall not exceed 2000 psf and the ε_{50} shall not be less than 0.007), unless laboratory stress- strain measurements indicate otherwise.

B.6. Rock

The results of SPT borings are most often used for designing noise wall foundations in shallow limestone strata. Less conservative designs require more vigorous sampling and testing to demonstrate that less conservative design values are appropriate in all locations. In the absence of a comprehensive, vigorous sampling and testing program, the design based on SPT borings shall be as follows:

Rock material with N-values less than 10 blows/foot shall be modeled as sand. Rock material with N-values between 10 and 20 blows/foot shall be modeled as sandy gravel:

Friction Angle, $\phi = \frac{N}{4} + 33$

The maximum friction angle value shall be limited to 40 degrees, unless higher friction angles are statistically supported by laboratory shear strength test results.

Rock material with N-values of 20 blows/foot or more:

- Use the LPILE or COM624 program guideline to model p-y curves of weak rock.

Modeling rock as stiff clay will be acceptable, provided N-values are 10 blows/foot or more and reasonable conservatism in the selection of k and undrained shear strength are adopted.

Note: AXIAL LOAD RESISTANCE (doesn't normally control the design of noise barrier foundations)

B.7. Side Resistance in Sands

Side resistance in cohesionless soils shall be computed by the FHWA Method (Beta Method) specified in the Publication FHWA-IF-99-025 (August, 1999) for drilled shafts as follows:

$f_s = \sigma'_v \ \beta_c$

$\beta_C = \beta \times \frac{N}{15}$ where $\beta_c \leq \beta$

$\beta = 1.5 - 0.135\ (z)^{0.5}$ (z, depth in ft), where $1.2 \geq \beta \geq 0.25$

where, f_s = Ultimate unit side resistance

The maximum value of f_s shall be limited to 2.1 tsf, unless load test results indicate otherwise.

σ'_v = Effective vertical stress

B.8. Side Resistance in Rock:

When limestone and calcareous rock cores are obtained for laboratory testing, the ultimate unit side resistance shall be estimated as discussed in Appendix A.

When rock cores and laboratory testing are not available, use the following approach:

- If SPT N-value in rock is less than 25 blows / foot, assume sand behavior.
- If SPT N-value in rock is greater than or equal to 25 blows / foot, use the following:

  $f_s = 0.1$ N (tsf), where $f_s \leq$ 4~~5~~.0 tsf

B.9. Side Resistance in Clay

Model inorganic clays and silts in accordance with FHWA methods. Shear strength values should be estimated from UU tests, unconfined tests, vane tests, etc. If only SPT tests are available, Consultants are expected to use reasonable judgment in the selection of undrained shear strength from correlations available in the literature.

The shear strength of clay estimated from SPT-N values or CPT results shall not exceed 2000 psf, unless laboratory stress-strain measurements indicate otherwise.

Side resistance shall be computed by the FHWA Method (Alpha Method) specified in the Publication FHWA-IF-99-025 (August, 1999) for drilled shafts as follows:

$f_s = \alpha * S_u$

where, S_u = Design undrained shear strength of clay (psf)

α = A dimensionless correlation coefficient as defined below:

$\alpha = 0$ between 0 to 5 feet depth

$\alpha = 0$ for a distance of B (the pile diameter) above the base

$\alpha = 0.55$ for $1.5 \geq S_u/P_a$

$\alpha = 0.55 - 0.1\ (S_u/P_a - 1.5)$ for $2.5 \geq S_u/P_a \geq 1.5$

for $S_u/P_a > 2.5$, follow FHWA-IF-99-025 Figure B.10

P_a = Atmospheric pressure (2116 psf at 0 ft Mean Sea Level)

B.10. Organic Soils

Neglect any side resistance in soils with an organic content greater than 5.0% by ASTM D 2974.

B.11. End Bearing Resistance

Neglect any end bearing resistance.

B.12. Factors of Safety & Resistance Factors

To compute an allowable axial load, a minimum factor of safety of 2.0 shall be used. The service axial load shall not exceed this allowable load.

For LRFD design, use a Load Factor in accordance with the latest AASHTO LRFD Bridge Design Specifications and a Resistance Factor of 0.6.

B.13. Design Water Table

For structures where the design is controlled by storm related wind loads, the design water table is at the ground surface.

For load conditions not associated with storm related wind loads, the seasonal high water table estimated for the location may be ~~the used~~used for computation of axial capacity and lateral load analysis. If no information is available to determine the seasonal high water table, the designer will assume the water table at the ground surface. Include a justification for the selected design water level in the foundation analysis.

B.14. SPT Energy Corrections

SPT N values from automatic hammers may be corrected to account for higher energy as compared with safety hammer. The energy correction factor shall not exceed 1.24.

B.15. Use of Static Cone Penetrometer Tests

If static cone penetrometer test (CPT) is used in the geotechnical investigation, the cone resistance data shall be converted to SPT N-values. The converted SPT N-values shall in turn be used in the foundation design according to the methods indicated in the previous sections of these design guidelines.

The correlation presented in Figure B-2 shall be used in the conversion of the CPT cone tip resistance, Q_c (tsf) to SPT N-values, based on mean particle size, D_{50} (mm) of the material. The use of design parameters that are less conservative than the values obtained from cone tip resistance to N-value correlations, and other sections of this document, shall be statistically supported by the results of high-quality laboratory tests and/or in situ tests for the specific soil/rock deposits.

C.1. Negative Shaft Resistance or Downdrag

The following is adapted from FHWA HI 97-013 Design and Construction of Driven Pile Foundations (1998)

When piles are installed through a soil deposit which will later settle, the resulting relative downward movement of the soil around piles induces "downdrag" forces on the piles. These "downdrag" forces are also called negative shaft resistance. Negative shaft resistance is the reverse of the usual positive shaft resistance developed along the pile surface. The downdrag force increases the axial load on the pile and can be especially significant on long piles driven through compressible soils. Therefore, the potential for negative shaft resistance must be considered in pile design. Batter piles should be avoided in soil conditions where large soil settlements are expected because of the additional bending forces imposed on the piles, which can result in pile deformation and damage. Settlement computations should be performed to determine the amount of settlement the soil surrounding the piles is expected to undergo after the piles are installed. The amount of relative settlement between soil and pile that is necessary to mobilize negative shaft resistance is about 10 to 12 mm (½ inch). At that movement, the maximum value of negative shaft resistance is equal to the soil-pile adhesion. The negative shaft resistance cannot exceed this value because slip of the soil along the pile shaft occurs at this value. It is particularly important in the design of friction piles to determine the depth at which the pile will be unaffected by negative shaft resistance. Only below that depth can positive shaft resistance forces provide support to resist vertical loads.

The most common situation where large negative shaft resistance develops occurs when fill is placed over a compressible layer immediately prior to, or after piles are driven. Negative shaft resistance can also develop whenever the effective overburden pressure is increased on a compressible layer through which a pile is driven; due to lowering of the ground water table, for example.

STEP BY STEP DESIGN PROCEDURE FOR ANALYSIS OF DOWNDRAG LOADING

1) Establish the simplified soil profile and soil properties for computing settlement.
2) Determine the overburden pressure increase, Δp, versus depth due to the approach embankment fill. There are many methods and computer programs available for this purpose. An acceptable hand method is included at the end of this appendix.
3) Perform settlement computations for the soil layers along the embedded pile length.
    a) Determine the consolidation parameters for ~~each soil~~cohesive layers, preferably from Shelby tube samples that have been tested in the laboratory~~ consolidation test results~~, and the elastic modulus for granular soil layers.
    b) Compute the settlement of each soil layer
    c) Compute the total settlement over the embedded pile length, i.e. the sum of the settlements from each soil layer and partial soil layers. Do not include soil settlements below the pile toe.
4) Determine the pile length that will experience negative shaft resistance. Negative shaft resistance occurs due to the settlement between soil and pile. The amount of settlement between soil and pile necessary to mobilize the negative shaft resistance is about ½ inch. Therefore, negative shaft resistance will occur on the pile shaft in each soil layer or portion

of a soil layer with ½ inch more settlement than the settlement of the pile.

5) Determine magnitude of negative shaft resistance, R_{dd}. The method used to calculate the ultimate negative shaft resistance over the pile length determined in Step 4 is the same method used to calculate the ultimate positive shaft resistance, except that it will act in the opposite direction.

6) Calculate the nominal pile resistance provided by the positive shaft resistance and the toe resistance, R_n. Positive shaft and toe resistances will develop below the depth where the relative pile-soil movements are less than ½ inch. The positive soil resistances can be calculated on the pile length remaining below the negative shaft resistance depth from Step 4 using an appropriate static analysis method for the soil type as described in this chapter.

7) Calculate the net ultimate pile capacity, R_{net} available to resist imposed loads.

$$R_{net} = R_{ult} - R_{dd}$$

8) Calculate the DOWNDRAG ~~value~~ load for inclusion in the Pile Data Table as follows:~~of the plans as~~

DOWNDRAG = R_{dd} + (Driving Resistance of soil contributing to R_{dd})

R_n = (Factored Design Load + Net Scour + Downdrag) / φ

Where: φ is the resistance factor taken from Table 3.1 of the SDG. During initial drive, the driving resistance of the soil contributing to R_{dd} equals about 0.75 times the ultimate skin friction for most sand and silty sand strata; it may be as low as 0.50 times the ultimate skin friction for plastic clayey soils that build-up excess pore water pressures during driving and later exhibit significant soil set-up. The driving resistance will be as high as 1.0 times the ultimate skin friction for clean sands that do not exhibit set-up.

During restrike, the driving resistance of the soil contributing to R_{dd} typically equals 1.0 times the ultimate skin friction because the excess pore pressures that built-up during initial drive will have dissipated.

~~8)~~9) Consider alternatives to obtain higher net ultimate pile capacity such as preloading or surcharging to reduce settlements prior to pile installation, use of lightweight fills to reduce settlements that cause downdrag loads, isolation of pile from consolidating soil, etc.

C.2. Overburden Pressure Determination

Method to determine the overburden pressure increase, Δp, versus depth due to the approach embankment fill.

The overburden pressure increase, Δp, is equal to the pressure coefficient, K_f, determined from the pressure distribution chart presented in Figure C-1, multiplied by the height of fill, h, and the unit weight of fill, γ. The pressure distribution chart provides the pressure coefficient, K_f, at various depths below the bottom of the fill (xb_f), and also at various distances from the centerline of the fill. The depth below the bottom of the fill is given as a multiple of "b,", where b, is the distance from the centerline of the fill to the midpoint of the fill side slope, as shown in the Figure C-1 below.

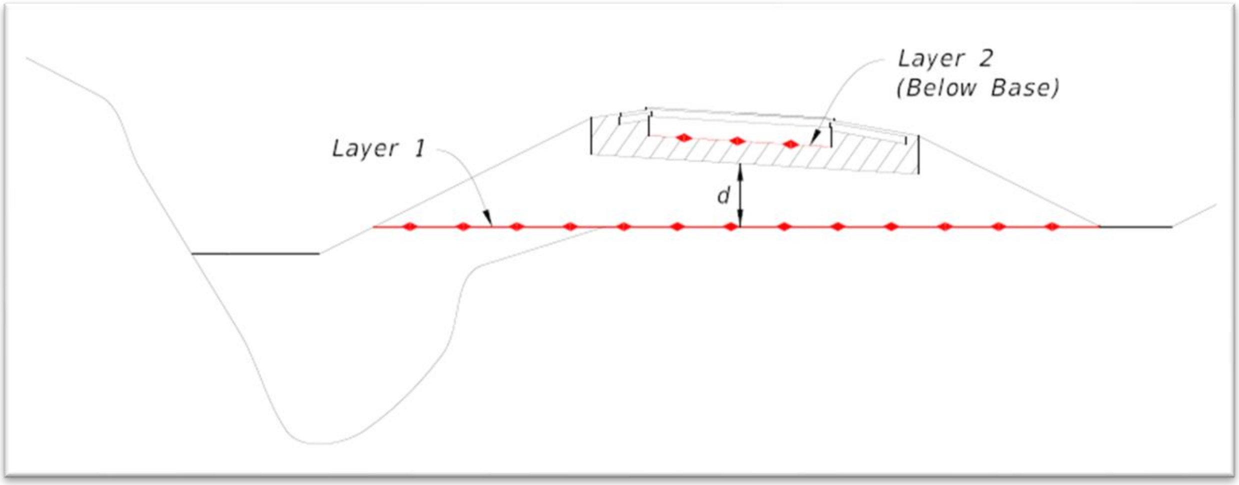


Figure E-1: Reinforced Embankment Design method

Reinforced embankments utilize geosynthetic reinforcement to provide structural support of traffic loads over the life of the pavement. This reinforcement application involves a relatively shallow flexible pavement substructure (embankment/subgrade/base profile) that is constructed over unsuitable soils that are at or near the ground surface. Therefore, the flexible pavement is directly affected by these underlying soft soils. The following design requirements are to be used for the selection of the geosynthetics used in the reinforcement of the roadway embankment system, including both the embankment soils and the aggregate base. Roadway reinforced embankments should be utilized when complete excavation and replacement of unsuitable soils below the proposed pavement system is not economical or desirable.

Step by step instructions are as follows:

1) Loads
   a) Determine construction loads.
   b) Normal highway wheel loads are assumed for this design method.
   c) If wheel loads will exceed legal highway wheel loads, contact a proprietary designer.
2) Strength
   a) Measure strength of in situ soils using Vane Shear, CPT, DMT, PMT, etc.
   b) If S_U design < 250 psf, STOP and use Reinforced Foundation over Soft Soils in FDM Chapter 263
3) Determine **minimum** depth below stabilized subgrade to Layer 1 from Table E-1~~Table 1~~.
4) Determine the required geosynthetic allowable tensile strength (T_R) from Table E-1~~Table 1~~.
5) Determine surcharge requirements.
   a) Use 5 ft minimum surcharge height.
   b) reinforced embankment test sections were surcharged for 6 months
6) Verify global stability.

a) Increase T_R and/or surcharge requirements as required.

7) Design the flexible pavement.

a) Contact the District Materials Office for guidance in selecting the design M_R value for the reinforced structural fill layer.

8) Detail the plans with the required location and T_R of the R-4 geogrid or geotextile. The Contractor will choose an R-4 material from the APL.

$$\frac{T_{ult}}{RF_{ID} RF_D} \geq T_R$$

where,

T_{ult} = Ultimate tensile strength

T_R = Required tensile strength

RF_{ID} = Reduction factor for installation damage

RF_D = Reduction Factor for Durability

Table E-1: Depth Relation using Undrained Shear Strength Relation vs. Required Tensile Strength

S_U (psf)	d (in)	T_R (lb/ft)
750 to 1,500	18	250
500 to 750	18	340
375 to 500	20	340
250 to 375	24	340

F.1. Determination of Bearing 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 with Pile Driving Analyzer (PDA) monitored test pile(s) or monitored production pile(s) in projects without test piles, using 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. Standard pile driving criteria (similar to method 1) with Goble Pile Check (GPC) dynamic monitoring equipment monitored test pile(s) or monitored production pile(s) and N_GAPA signal match analyses.
3. Embedded Data Collector (EDC) monitoring of all Test Piles and all Production Piles (100%), using top~~tip~~ and tip~~top~~ gauges with FDOT method of analysis on all piles, or a combination of piles with top and tip gauges and piles with only top gauges, as outlined in ~~. A minimum percentage of the piles in each bent/pier must be analyzed with the FDOT Method post-processing software; see~~ Section F.1.3~~3~~ of this Appendix.
4. PDA monitoring of all Test Piles and all Production Piles (100%), with CAPWAP analyses on a minimum percentage of the piles in each bent/pier required in Section 4F.1.4 of this Appendix.
5. GPC monitoring of all Test Piles and all Production Piles (100%), with manual N_GAPA signal match analyses on a minimum percentage of the piles in each bent/pier required in Section F.1.5~~5~~ of this Appendix.

F.1.1. Standard Driving Criteria with PDA Test Piles

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 early pile driving blows on

Indicate the minimum stroke or stroke range under which this number of blows must be applied. For more information regarding the driving criteria letter, refer to the Construction Procedures Administration Manual (CPAM, chapter 10.1, sample letters).

e. Additional Considerations

It is important to note that the driving criteria applies to the soil/rock material encountered at the elevation at which Signal Match analysis was performed. Piles that satisfy the driving criteria within different soil/rock strata need to be evaluated to confirm resistance has been achieved. In addition, driving criteria based on initial drive may not be used for set-check (re-strike) conditions. To develop a valid set-check criteria, dynamic load test data must be available for the same driving conditions and time after initial drive was performed, and the same steps (as initial driving criteria development) followed.

In some special conditions, the pile driving log (and GPC Review) may indicate an unusually high blow count in upper layers, even though capacity was not obtained. This may occur because soil properties change with depth. For example, a pile driven through soil with large damping properties will require a larger blow count than low damping soils, for the same capacity. This may also occur when the pile cushion has not been fully compressed. Therefore, in some cases it may be necessary to revise the model to ensure piles will not attain a false bearing (meet the blow count requirement without actually achieving the static resistance). There are three choices:

i. Implement a minimum elevation above which the criteria are not applicable.

ii. Conservatively establish a blow count requirement that is high enough to avoid stopping in the higher damping soil without bearing. This may be feasible when the test pile shows an increase in capacity with depth and the conservative criteria does not result in unreasonably long production pile lengths.

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

F.1.3. Embedded Data Collector (EDC) monitoring of Test Piles and Production Piles (100%)

~~f.~~ 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% top

and tip gauges.

~~All~~ EDC piles are monitored in the field using Smart Structures' UF Method. All piles are subsequently analyzed by the FDOT method to ensure sufficient resistance of the bearing layer.~~Smart Structures' FDOT Method post-processing software will be used to verify the UF Method results of at least 10% of all piles in each bent and pile footing (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. Under this approach, the resistance factor for EDC is 0.75.
2. 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 of FDOT Method analyses is required, therefore, a higher percentage of piles with top and tip gauges is also required.
3. In the field, use the UF Method of analysis 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.
4. 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 FDOT Method capacity for the representative blow.
5. 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.
6. 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.
7. 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.

F.1.4. 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 are driven first to determine production pile lengths, or in cases when the Contractor has chosen to order production piles in advance, the first pile in each bent or pier 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 pier 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.

F.1.5. GPC monitoring of all Test Piles and all Production Piles (100%), with manual N_GAPA 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 are driven first to determine production pile lengths, or in cases when the Contractor has chosen to order production piles in advance, the first pile in each bent or pier 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 GPC and Signal Match analyses. Manual N_GAPA analyses are required on at least 10% of the piles in each bent or pier to confirm GPC results. In unique soil conditions such as extreme scour, large uplift loads or high variability soils, a higher percentage of Signal Match analyses is required. In addition, piles that meet the criteria at significantly different elevations from where Signal Match was performed, or tip on a different material type, will require separate Signal Match analysis. Finally, at least one additional Signal Match analysis is required for an instrumented re-drive if this has a different set-up time than other piles evaluated in the pier.

F.1.6. Determining the Capacity of a Pile from an Instrumented Set-Check

In accordance with section 455-5.1~~0~~1.4, 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

G.1. Cable Barrier Foundation Analysis using Broms' Method

Horizontal Service Load on Foundation, $Q_a = 40$ kips

Effective Unit Weight of Soil, $\gamma = 50$ pcf

Friction angle, $\varphi = 30$ degrees

Cohesion, C = 0

Factor of Safety = 2 (Overturning)

$k_h = 7$ pci

Service Load Deflection = 1 inch

Broms' method is useful for estimating the ultimate lateral capacity of single piles in uniform soils. The method was originally proposed for "short" piles and "long" piles, with and without a rigid pile cap to prevent rotation. Short piles are considered stiff with respect to the surrounding soil and behave like a "fence post" and pivot in response to lateral loading. Long piles remain fixed at depth and the upper portion of the pile bends in response to loading. Generally, finite difference computer programs utilizing p-y methods, such as COM624 or FB-MultiPier are more accurate for long piles, but sometimes do not converge when analyzing short piles.

Broms' method for free-head short piles assumes the pile pivots about the tip, and the resistance is due to the passive earth pressure of 3 times the width of the pile. The method assumes the earth pressure in the direction of the loading does not activate.

$$\Sigma M_{tip} = 0$$

$$\frac{1}{2}\gamma D^2 K_P \frac{1}{3} D 3b - PD = 0$$

where,

$D = Depth\ of\ pile$

$K_P = Coefficient\ of\ passive\ earth\ pressure\ (3.0\ for\ \varphi = 30)$

$b = Width\ of\ pile$

$P = Ultimate\ lateral\ load\ (Service\ Load \text{ x } Factor\ of\ Safety)$

Solving for D:

$$D = \sqrt{\frac{2P}{\gamma K_P b}}$$

For the standard soil and default loading:

Subject	FM
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 In situ Permeameter (VIP)	5-614
Standard Test Method for Specific Gravity and Absorption of Coarse Aggregate	1-T 085
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 Methods for Determination of Organic Content in Soils by Loss on Ignition	1-T 267
Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions	3-D3080
Standard Test Method for Density of Bentonitic Slurries	3-D4380~~8-RP13B-1~~
Viscosity of Slurry	3-D4380~~8-RP13B-2~~
Standard Test Method for Sand Content by Volume of Bentonitic Slurries	3-D4380~~8-RP13B-3~~
pH of Slurry	3-D4380~~8-RP13B-4~~