Soils and Foundations Handbook
20~~21~~22
Summary of Changes

State Materials Office
Gainesville, Florida

Chapter 4

4 In-situ Testing

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

4.1 Standard Penetration Test (SPT)

This test is probably the most widely used field test in the United States. It has the advantages of simplicity, the availability of a wide variety of correlations for its data, and the fact that a sample is obtainable with each test. A standard split barrel sampler is advanced into the soil by dropping a 140-pound safety or automatic hammer on the drill rod from a height of 30 inches. (Note: Use of a donut hammer is not permitted). The sampler is advanced a total of 18 inches. The number of blows required to advance the sampler for each of three 6-inch increments is recorded. The sum of the number of blows for the second and third increments is called the Standard Penetration Value, or more commonly, N-value (blows per foot). Perform all Standard Penetration Tests in accordance with ASTM D 1586 (AASHTO T 206).

Note the type of hammer (safety or automatic) on the boring logs, since this will affect the actual input driving energy. Only one type of hammer may be used in each SPT boring. **Because of the substantial increase in consistency, automatic SPT hammers are required for all SPT borings performed using truck and all-terrain vehicle mounted drilling equipment;** safety hammers will be permitted only for borings requiring specialty and/or unique drilling equipment that cannot support an automatic hammer (i.e., small amphibious rigs, tripod, small barge, etc.) Use of safety hammers requires the approval of the District Geotechnical Engineer.

When Standard Penetration Tests (SPT) are performed in soil layers containing shell or similar materials, the sampler may become plugged. A plugged sampler will cause the SPT N-value to be much larger than for an unplugged sampler and, therefore, not a representative index of the soil layer properties. In this circumstance, a realistic design requires reducing the N-value used for design to the trend of the N-values which do not appear distorted. (See **Figure 5** and **Reference 3**) However, the actual N-values should be presented on the Report of Core Borings Sheet.

During design, the N-values may need to be corrected for overburden pressure. A great many correlations exist relating the corrected N-values to relative density, angle of internal friction, shear strength, and other parameters. Design methods are available for using N-values in the design of driven piles, embankments, spread footings and drilled shafts. However, when using FB-Deep or GeoStat, the N-values should not be corrected since the design methodology is based on uncorrected N-values.

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, c~~C~~ompile 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.

4.10.1 pH of Soils

a) FM 5-550

4.10.2 pH of Water

a) FM 5-550

4.10.3 Chloride Ion in Water

a) FM 5-552

4.10.4 Chloride Ion in Soil

a) FM 5-552

4.10.5 Sulfate Ion in Brackish Water

a) FM 5-553

4.10.6 Sulfates in Soil

a) FM 5-553

4.10.7 Electrical Resistance of Water

a) FM 5-551

4.10.8 Electrical Resistance of Soil

a) FM 5-551

4.11 Grout Plug Pull-out Test

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

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

4.13 Specifications and Standards

Subject	ASTM	AASHTO	FM
Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils	D 1586	T 206	-
Standard Test Method for Field Vane Shear Test in Cohesive Soil	D 2573	T 223	-
Standard Test Method for Infiltration Rate of Soils in Field Using Double-Ring Infiltrometer	D 3385	-	-
Standard Test Method (Field Procedure) for Withdrawal and Injection Well Tests for Determining Hydraulic Properties of Aquifer Systems	D 4050	-	-
Standard Test Method for Energy Measurement for Dynamic Penetrometers	D 4633	-	-
Standard Test Methods for Prebored Pressuremeter Testing in Soils	D 4719	-	-
Standard Practices for Preserving and Transporting Rock Core Samples	D 5079	-	-
Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils	D 5778	-	-
Standard Practice for Using the Electronic Piezocone Penetrometer Tests for Environmental Site Characterization and Estimation of Hydraulic Conductivity	D 6067		
Standard Test Method for Field Measurement of Hydraulic Conductivity Using Borehole Infiltration	D 6391		
Standard Test Method for Performing the Flat Plate Dilatometer Test	D 6635	-	-
Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications	D 6951	-	-
Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method	G 57	-	-
Standard Test Method for pH of pH of Soil and Water	-	-	5-550
Standard Test Method for Resistivity of Soil and Water	-	-	5-551
Standard Test Method for Sulfate in Soil and Water	-	-	5-553
Standard Test Methods for Chloride in Soil and Water	-	-	5-552

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-Pier is available for assessment of lateral design capacity and shaft group settlement
- GeoStat is available for the evaluation of site variability

The Help Files for the FB-Deep, FB-Pier and GeoStat programs are recommended references.~~The computer program FB-Deep is available for assessment of axial design capacity and the computer program FB-Pier is available for assessment of lateral design capacity and pile group settlement through the **Bridge Software Institute** (BSI). The Help Files for the FB-Deep & FB-Pier programs are both recommended references.~~ Include all materials within 3B of the individual ~~shaft~~ pile tip or 2 times the minimum group dimension below the tip of the ~~shafts~~piles, whichever is deeper.

For foundations tipped on rock or IGM, ensure the bearing layer thickness below the tip elevation is sufficient to prevent punching failure into the weaker stratum below the bearing stratum for the end bearing resistance included in the design (See 8.3.1 Rock Fracture). 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 and/or dynamic 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 are normally driven closed end. In extremely aggressive conditions they may be used only if 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.

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 (Ref 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 the **Bridge Software Institute** (BSI):

- FB-Deep is available for assessment of axial design capacity
- ~~and the computer program~~ FB-Pier 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

~~through the **Bridge Software Institute** (BSI).~~ The Help Files for the FB-Deep, ~~&~~ FB-Pier and GeoStat programs are ~~both~~ recommended references.

Non-redundant drilled shaft bridge foundations have special design requirements as follows:

1. All shafts in non-redundant 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 Rock Fracture).

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

8.2.3.2 Considerations

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

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

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

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

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

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

8.2.3.3 Design Procedure for Miscellaneous Structures

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

When laterally loaded drilled shafts for miscellaneous structures are founded through compacted select fill berm or slope, include the portion of the pile with less than 2.5D horizontal soil cover (face-of-pile to face-of-slope) in the unsupported length, and design the portion of the pile with more than 2.5D soil cover as though founded in level ground.

Commented [LJ1]: Adapted from Appendix B, pg 161

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

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

8.2.4 Auger-Cast Piles

As with driven piles and drilled shafts, auger-cast piles must be designed considering both axial and lateral loads. However, lateral loads typically govern when auger-cast-piles are used for noise wall foundations. See the SDG for restrictions on the use of Auger Cast Piles for bridges and other structures.

8.2.4.1 Design Procedure

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

Generic designs for noise barrier wall foundations on level ground are presented in the Standard Plans for Road and Bridge Construction. When walls are founded through compacted select fill berm, include the portion of the pile with less than 2.5D horizontal soil cover (face-of-pile to face-of-slope) in the unsupported length, and design the portion of the pile with more than 2.5D soil cover as though founded in level ground.

Commented [LJ2]: Copied from Appendix B, pg 161. The repeated language from Appendix B has been deleted.

If the site specific soil conditions are weaker than the values presented in the Standard Plans Instructions (SPI) or if a site specific design is desired, auger-cast piles shall be designed in accordance with the procedure outlined in Appendix B**.** Consult with the District Geotechnical Engineer for local guidelines regarding auger-cast piles.

8.2.4.2 Considerations

Considerations for auger-cast piles supporting precast noise walls are presented in the Standard Plans Instructions, Index 534-200.

8.2.5 Micro Piles

In special cases micro piles may be the preferred foundation system. This would typically be in cases of limited access, 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.

8.2.5.1 Design Procedure

Designs must comply with Section 10.9 of Reference 30, 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

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

Soldier pile and lagging walls usually consist of steel H-piles and horizontal lagging and are primarily used for top-down construction. Soldier pile walls can be cantilevered or anchored, temporary or permanent. For permanent applications in Florida, concrete pile and panel lagging is usually preferred. Soldier Pile/Panel walls should be considered in locations where sheet pile walls are needed, however, sheet pile installation difficulties are expected. Refer to the FDOT Structures Design Guidelines and the FDM for additional requirements.

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

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

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

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

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

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

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

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

~~41.~~42. Chung, J.H., McVay, M.C., Davidson, M.T., Effect of proximity of Sheetpile Walls on the Apparent Capacity of Driven Displacement Piles, FDOT Research Report BDV31-977-26, August 2018.

9.1.5 Appendix

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

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

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

c. Results of roadway soil survey borings performed. For all SPT borings, include the information required in 9.2.3.

d. Any other pertinent information.

e. Analysis of the geotechnical information.

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 XXXXXXXXXXX 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.

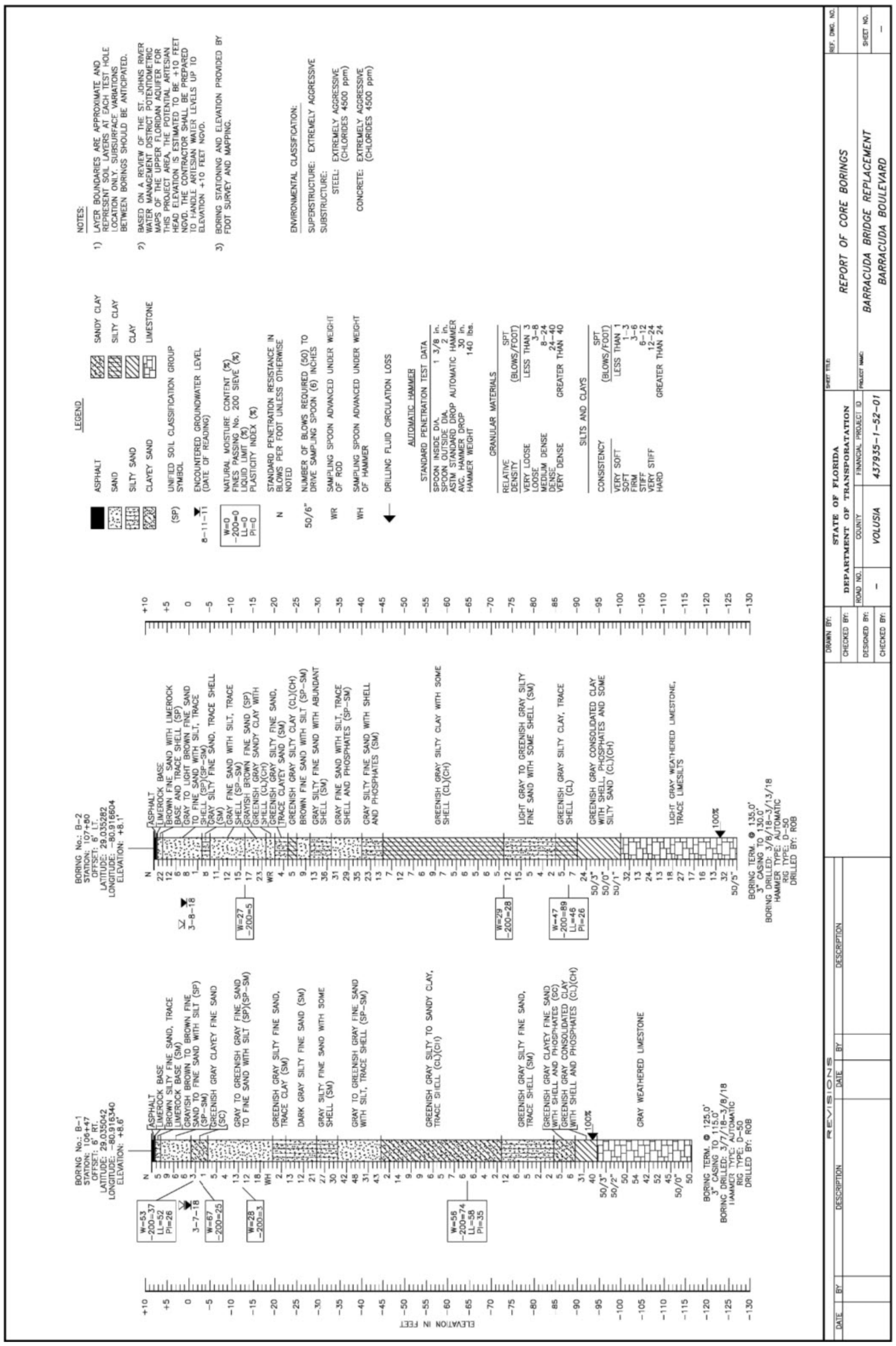
If a database system is used for storing Geotechnical data, the data can be exported out in Excel format that the GDM software can convert to the required xml format. Excel formatted templates are provided to show how the exported data shall be formatted for conversion by GDM. These spreadsheets are 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.



New Figure

Figure 32, Typical Report of Core Borings Sheet ~~**(Required border may differ)**~~

10.2 Dynamic Monitoring of Pile Driving

Measurements of the dynamic pile response can be obtained during driving by using Embedded Data Collectors (EDCs) or the Pile Driving Analyzer (PDA). Refer to ASTM D 4945 (AASHTO T 298). These measurements are used to determine:

1. Pile capacity
2. Driving stresses and probable damage to the pile
3. Energy transfer to the pile and therefore the efficiency and suitability of the pile driving system.
4. The soil parameters used in wave equation analysis for determining the installation criteria for subsequent piles when applicable.
5. Possible reasons for pile installation problems.

On major projects, dynamic monitoring of pile driving can be used together with static load tests to confirm design-bearing capacities. Quite often, the use of dynamic measurements decreases the number of static load tests required. This will result in time, as well as, cost savings to a load test program. On smaller projects, dynamic measurements alone may serve as the load testing method. The advancement in the design of the PDA system in recent years has made this equipment a reliable tool for the field-testing and inspection of driven piles when combined with signal matching analysis. ~~Refer to ASTM D 4945 (AASHTO T 298).~~

The Embedded Data Collector (EDC) system developed under FDOT sponsored research utilizes internal strain and acceleration measurements at both the top and bottom of the pile. The currently required method of analysis published by Tran et. al. utilizes the data from the top and bottom internal gages to determine the pile capacity and is considered equivalent to signal matching analysis. (Smart Structures, Inc. refers to this method as the FDOT Method because the patent rights are assigned to FDOT) Refer to **Standard Plans, Index 455-003**.

10.3 Load Tests

Many major projects involving driven piles or drilled shafts incorporate load tests to reduce uncertainty and/or increase resistance factors. These tests are conducted to verify that actual pile or shaft response to loading is as assumed by the designer, and to ensure that the measured resistance is not less than the nominal resistance computed during design. The use of resistance factors associated with load testing requires verifying and mobilizing the design side shear and end bearing values during the load test. The project Geotechnical Engineers should be involved in the load testing itself, and the interpretation of the resultant data. They should also be prepared to modify designs if the load tests fail to verify and fully mobilize the design values.

Extrapolating the trend of an under loaded load test does change the measured resistance, and therefore, design values based on such extrapolated trends must not be used with a load testing resistance factor.

Appendix B

Design Guidelines for Auger Cast Piles for Miscellaneous Structures Based on SPT or CPT Values Without Rock Core Tests

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.

NOISE BARRIER FOUNDATIONS

See Section 8.2.4.1

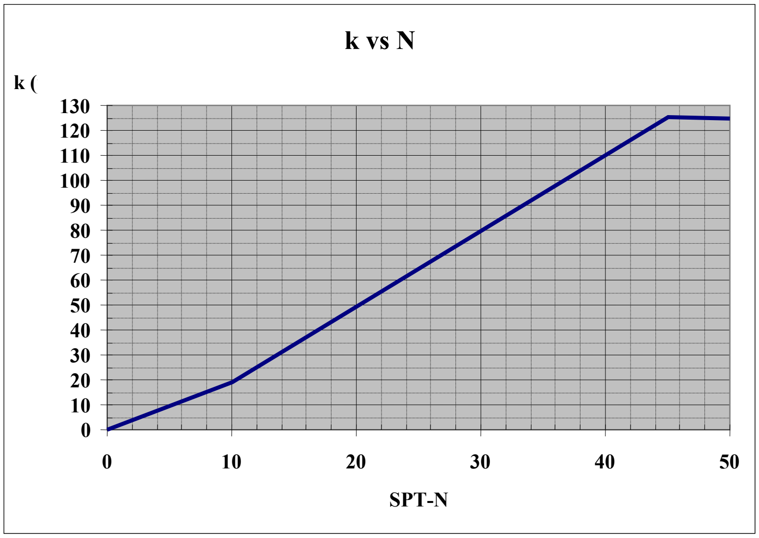
LATERAL LOAD RESISTANCE

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

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

<u>k values in Sands.</u>

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



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

Friction Angles in Sand

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

$$\varphi = N/4 + 28$$

As an alternative, the procedure described 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.

~~**Walls founded on berms**~~

~~When walls are founded through compacted select fill berm, include the portion of the pile with less than 2.5D horizontal soil cover (face-of-pile to face-of-slope) in the unsupported length, and design the portion of the pile with more than 2.5D soil cover as though founded in level ground.~~

Commented [JL4]: Moved to 8.2.4.1

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.

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, $\varphi = N/4 + 33$

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.

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

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:

$f_s = P'_v \ \beta_c$

$\beta_c = \beta * N/15$ 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.

P'_v = Effective vertical stress

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 $f_s \leq 5.0$ tsf

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:

Appendix D

Design Method for Drilled Shaft with Pressure Grouted Tip

Design Method for Drilled Shaft with Pressure Grouted Tip

For a given shaft diameter and anticipated embedment length, the method for estimating the unit tip resistance of grouted shafts in cohesionless soils involves the following steps:

1. Calculate the nominal side shear resistance (F_s) for the given shaft diameter (D) and total embedded shaft length. Using GeoStat software, ensure that a sufficiently large number of laboratory tests are performed to develop side shear design values for rock strata.
2. Calculate the nominal uplift side shear resistance ($F_{s\ uplift}$);

   $$F_{s\ uplift} = (F_s)(Uplift\ Reduction\ Multiplier^*)$$

   *O'Neill cited uplift resistance of shafts to be 0.75 that of compression/downward loading. *O'Neill, M. W. (2001). "Side Resistance in Piles and Drilled Shafts," The Thirty-Fourth Karl Terzaghi Lecture, ASCE J. Geotech. Geoenviron. Eng. 127:3-6.*
3. Calculate the ungrounted nominal unit tip resistance of the shaft ($q_{tip}{}^{**}$) for 5 % Diam. tip settlement as per AASHTO 10.8.2.2.2.

   **The 5% settlement is also the default value used in FB-Deep for drilled shafts founded in cohesionless soils, thus, one can use the FB-Deep formula (q_{tip} = 0.6 x SPT N_{60}, tsf) where SPT N_{60} is weighted average at shaft tip (Reese & O'Neill, 1988).
4. Determine the maximum anticipated grout pressure (GP_{max}) by dividing the nominal uplift side shear resistance ($F_{s\ uplift}$) by the cross-sectional area of the shaft (A);

   $$GP_{max} = F_{s\ uplift}/A$$
5. Calculate the Grout Pressure Index, GPI, as the ratio of the maximum anticipated grout pressure (Step 4) to the ungrouted unit tip resistance (q_{tip}), (Step 3);

   $$GPI = GP_{max}/\ q_{tip}$$
6. Determine the Tip Capacity Multiplier (TCM) using the following equation

   $$TCM = 0.713(GPI) + 0.3$$
7. Estimate the grouted unit tip resistance as the product of the Tip Capacity Multiplier (Step 6) and the ungrouted unit tip resistance (q_{tip}), (Step1).

   $$q_{grouted} = (TCM)(q_{tip})$$
8. Compute the nominal tip resistance $R_{n\ tip} = (q_{grouted})(A_{tip}{}^{***})$

   ***The tip area of a grouted shaft has been shown to be larger than the shaft diameter due to cavity expansion of the soils beneath the tip. While values less than the constructed shaft diameter have been suggested to account for variability, the constructed diameter of the shaft was used to develop this design method and therefore statistically incorporates variations both larger and smaller than the nominal shaft diameter.
9. Compute the nominal resistance $R_n = R_{n\ side\ shear} + R_{n\ tip}$
10. Compute the factored resistance $R_R = \phi(R_{n\ side\ shear} + R_{n\ tip})$

Note that the side shear is assumed to develop with very little displacement, thus allowing for the use of this ultimate value. Care should be taken when specifying maximum allowable shaft uplift during grouting such that the side shear resistance (contributing to the total resistance) is not displaced beyond possible peak strength and into a lower residual value. The Step 6 *TCM* value coincides with the maximum side shear at no more than 1%D tip settlement.

Appendix F

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

Verify the model: Refer to the corrected PDIPlot, and compare at several depths (near the estimated bearing depth) to check how the model predicts blow count at other capacities/strokes measurements (use PDIPlot average output per foot or per increment). Refine the model if necessary.

Blow count criteria: On the refined wave equation model, apply production pile lengths and NBR loading conditions to develop a driving criteria. Reduce efficiency for battered piles as required. If the Contractor provides longer piles than the authorized lengths, perform the analysis again to confirm the criteria still applies.

d. Driving Criteria Letter

The driving criteria letter provides the inspector with directions on when to accept piles. The letter should include the pile acceptance criteria based on blow count vs. stroke height results obtained from WEAP analysis, pile cushion details and recommendations regarding the operation of the hammer to avoid damaging the pile while driving. In addition, if the minimum tip elevation is not shown on the Plans, provide a criterion for "firm bearing material" to determine when the minimum pile penetration per 455-5.8 has been achieved. Provide the maximum number of hammer blows that may be applied to pile cushions before they must be replaced and the minimum number of blows a new cushion must be impacted before applying the blow count and refusal criteria. 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 3 and 4).

e. Additional Considerations

It is important to note that the driving criteria applies to the soil/rock material encountered at the elevation at which CAPWAP analysis was performed. Piles that satisfy the driving criteria within different soil/rock strata need to be evaluated to confirm resistance has been attained. In addition, driving criteria based on initial drive may not be used for 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 indicated here should be followed.

In some special conditions, the pile driving log (and PDIPlot) 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. In most cases, a specified elevation above which the criteria does not apply will resolve this issue. However, in some cases it may be necessary to revise the model to ensure piles will not attain a false bearing (meet the

Appendix H

Specifications and Standards

Subject	**ASTM**
Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils	D 4767
Standard Test Method for Shrinkage Factors of Soils by the Wax Method	D 4943
Standard Test Method for High-Strain Dynamic Testing of Deep Foundations	D 4945
Standard Practices for Preserving and Transporting Rock Core Samples	D 5079
Standard Test Method for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter	D 5084
Standard Guide for Field Logging of Subsurface Explorations of Soil and Rock	D 5434
Standard Guide for Planning and Conducting Borehole Geophysical Logging	D 5753
Standard Guide for Using the Seismic Refraction Method for Subsurface Investigation	D 5777
Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils	D 5778
Standard Test Method for Low Strain Integrity Testing of Deep Foundations	D 5882
Standard Test Method for Measurement of Hydraulic Conductivity of Porous Material Using a Rigid-Wall, Compaction-Mold Permeameter	D 5856
Standard Practice for Using Significant Digits in Geotechnical Data	D 6026
Standard Practice for Using the Electronic Piezocone Penetrometer Tests for Environmental Site Characterization and Estimation of Hydraulic Conductivity	D 6067
Standard Practice for Using Hollow-Stem Augers for Geotechnical Exploration and Soil Sampling	D 6151
Standard Test Method for Field Measurement of Hydraulic Conductivity Using Borehole Infiltration	D 6391
Standard Guide for Selecting Surface Geophysical Methods	D 6429
Standard Test Method for Performing the Flat Plate Dilatometer Test	D 6635
Standard Test Method for Integrity Testing of Concrete Deep Foundations by Ultrasonic Crosshole Testing	D 6760
Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis	D 6913
Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications	D 6951
Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures	D 7012
Standard Test Method for Consolidated Drained Triaxial Compression Test for Soils	D 7181