Soils and Foundations Handbook

~~2022~~ 2024

Summary of Changes

Effective January 1, 2024

Chapter 1

1 Introduction

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

As each project presents unique considerations and requires engineering 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.
- Analyze all data available.
- Prepare preliminary geotechnical report summarizing available data and providing recommendations.
- Identify potential needs for the design investigation to address construction requirements and anticipate problems (high groundwater issues, preforming requirements, vibration and noise impacts).

1.1.2 Project Design Phase

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

1.1.3 Construction Phase

- Establish construction criteria for geotechnical portions of project.
- Inspect construction procedures to assure compliance with design and specifications.
- Design, install, perform, monitor, and evaluate load test programs and/or instrumentation systems.
- Solve unforeseen foundation and/or roadway soils problems.

2.2.2 Aerial Photographs

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

2.2.3 Geological Maps and Reports

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

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

Chapter 3

3 Subsurface Investigation Guidelines for Highways and Related Structures

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

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

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

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

Modification for Non-Conventional Projects:
Delete the first paragraph and insert the following: A subsurface investigation should be performed at the site of all new structure, roadway construction, widenings, extensions, trails and rehabilitation locations as outlined herein, except as otherwise described in the RFP.

3.1 General Requirements

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

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

1. Preliminary exploration depths should be estimated from data obtained during field reconnaissance, existing data, local geology and local experience. The borings should penetrate unsuitable founding materials (organic soils, soft clays, loose sands, etc.) and terminate in competent material. Competent materials are those suitable for support of the foundations being considered.
2. All borings shall be extended below the estimated scour depths.
3. Each boring, sounding, and test pit should be given a unique identification number for easy reference.
4. The horizontal and vertical location shall be determined for each boring, sounding, and test pit as follows:

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

3.2 Guidelines for Minimum Explorations

Following is a description of the recommended minimum explorations for various types of projects. It is stressed that these guidelines represent the minimum extent of exploration and testing anticipated for most projects and must be adapted to

c. For roadway widenings that provide an additional lane, one boring shall be placed within the additional lane at each interval.

d. In areas of cut or fill, where stability analysis is anticipated, a minimum of two additional borings shall be placed at each interval near the outer reaches (toe) of the sloped areas.

e. In all cases, at least three samples per mile or 3 per project whichever is greater shall be obtained for each stratum encountered. Each of the samples representing a particular stratum shall be obtained from a different location, with sampling locations spread out over each mile. Samples should be of adequate size to permit classification and moisture content testing.

f. For new construction, three 100 lb. samples per mile per stratum or 5 per project whichever is greater, of all materials within 4 feet below the proposed base elevation and considered 'Select' in accordance with Standard Plans, Index 120-001 shall be obtained and delivered to the State Materials Office in Gainesville for Resilient Modulus (M_R) testing. Samples of all strata located in excavation areas (i.e., water retention areas, ditches, cuts, etc.), which can be used in accordance with Standard Plans, Index 120-001 shall also be obtained for M_R testing when fill below paved areas will be required.

g. Corrosion series samples shall be obtained (unless no structures are to be installed) on a frequency of at least one sample per stratum per 1,500 feet of alignment.

h. When a rigid pavement is being considered for design, obtain sufficient samples to perform laboratory permeability tests based upon the requirements given in the Rigid Pavement Design Manual.

i. Borings in areas of little or no grade change shall extend a minimum of 5 feet below grade, drainage pipe or culvert invert level whichever is deeper. For projects with proposed buried storm sewer systems, one boring shall be extended to a nominal depth of 20 feet below grade every 500 feet along the alignment of the storm sewer system; project specifics may dictate adjustments. For projects with proposed regular light poles, one boring shall be extended to a nominal depth of 10 feet below grade every 500 feet along the alignment if borings for buried storm sewer systems are not performed; project specifics may dictate adjustments. Borings may or may not include Standard Penetration Tests (SPT), depending on the specific project requirements and its location.

j. In areas of cut, borings shall extend a minimum of 5 feet below the proposed grade, drainage pipe or culvert invert level whichever is deeper. If poor soil conditions are encountered at this depth, borings shall be extended to suitable materials or to a depth below grade equal to the depth of cut, whichever occurs first. Bag, SPT, undisturbed and core samples shall be obtained as appropriate for analyses.

k. In areas of fill, borings shall extend to firm material or to a depth of twice the embankment height, whichever occurs first. Bag, SPT, and undisturbed samples shall be obtained as appropriate.

l. Delineate areas of deleterious materials (muck, plastic soils, trash fill, buried slabs or pavements, etc.) to both the vertical and the horizontal extents.

~~l.~~m. Identify the seasonal high groundwater elevation at least every 500 feet along the alignment of the roadway and in the lowest pavement elevations identified between these borings.

3.2.2 Structures

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

All structure borings shall include Standard Penetration Testing (SPT) at regular intervals unless other sampling methods and/or in-situ testing (as defined in **Chapter 4**) are being performed. Extend borings sufficiently below the shallow foundation, or deep foundation tip elevations in accordance with the FHWA minimum criteria to determine the adequacy of the bearing soils and the long term settlement behavior of the foundation. Refer to Reference 3.

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 feet 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 feet 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 feet wide - at least one boring within 25 feet of~~at~~ every other bent/pier foundation per structure
- Bents/pier foundations (pile groups) ≥ 70 feet 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 12)

e. Auger Cast Piles (ACP) –

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

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.

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 4**).

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 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.

~~collect field bulk samples of the existing reinforced fill materials for modified Proctor density tests, direct shear tests and corrosion testing at a minimum frequency of three bulk samples per mile per wall of existing wall length. Bulk samples shall be collected at different elevations along existing wall alignment to represent potential different reinforced fill sources used in the original construction. Particular attention must be given to identify suitable field sampling locations using original MSE wall shop drawings to prevent conflicts with and/or damage to existing soil reinforcements. Proposed bulk sample locations and elevations to be approved by the District Geotechnical Engineer.~~

~~2)~~

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

Corrosion testing must be performed for each foundation unless the structure is designed for the most aggressive conditions.

3.2.2.8 Mast Arms Assemblies, Microwave Vehicle Detection Systems (MVDS) Poles and Strain Poles

1) One boring to 25 feet into suitable soil or 10 feet into competent rock with 15 feet minimum total depth (Auger, SPT or CPT) shall be taken in the area of each designated location (for uniform sites one boring can cover more than one foundation location).

2) For Standard Mast Arm Assemblies, verify that the soil strength properties at the foundation locations meet or exceed the soil strength properties assumed for the Standard Mast Arm Assemblies in the Standard Indices. A site-specific design must be performed for those sites having weaker strength properties.

3) For mast arm assemblies not covered in the standards an analysis and design must be performed.

4) Corrosion testing may be omitted and the structure designed for the most aggressive conditions unless otherwise required by the District Geotechnical Engineer.

Modification for Non-Conventional Projects:
Delete 4) and insert the following: Corrosion testing must be performed for each foundation unless the structure is designed for the most aggressive conditions.

3.2.2.9 CCTV Poles

1) One boring shall be taken at each designated location~~; ensure each shaft is within 20 feet of a boring~~. If the pole location is subsequently moved, perform another boring as close as practical to the new location if the site is variable or if weaker soils are suspected.

2) Borings shall be 20 feet into suitable soil below prevailing grade or 25 feet below the top of embankment. ~~or~~The boring may terminate at 10 feet into competent rock with 15 feet minimum total depth. Deeper borings may be required for cases with higher than normal loads or where in the opinion of the District Geotechnical Engineer, the foundation depth is expected to be deeper.

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

4) Corrosion testing may be omitted and the structure designed for the most aggressive conditions unless otherwise required by the District

Modification for Non-Conventional Projects:
Delete this paragraph and see the RFP for requirements.

3.2.2.12 Other Structures

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

Modification for Non-Conventional Projects:
Delete this paragraph and see the RFP for requirements.

3.2.3 Borrow Areas

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

3.2.4 Open Retention Ponds (Stormwater Ponds with a positive outlet)

Two auger borings (SPT borings with continuous sampling may be substituted) shall be taken per 40,000 feet^2 of pond, with a minimum depth of 5 feet below the deepest elevation of the pond, or until a confining layer is encountered or local Water Management District criteria are satisfied. A minimum of two field permeability tests per pond shall be performed, with this number increasing for larger ponds.

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

3.2.5 Closed Retention Ponds (Stormwater Ponds without a positive outlet)

One auger boring (SPT borings with continuous sampling may be substituted) shall be taken per 40,000 feet^2 of pond, with a minimum depth of five feet below the deepest elevation of the pond, and one SPT boring per 40,000 feet^2 of pond, with a minimum depth of two times the proposed water height or until local Water Management District criteria are satisfied. A minimum of two field permeability tests per pond shall be performed, with this number increasing for larger ponds.

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

Chapter 5

5 Laboratory Tests

As with other phases of a subsurface investigation program, the laboratory testing must be intelligently planned in advance but flexible enough to be modified based on test results. The ideal laboratory program will provide the engineer with sufficient data to complete an economical design, yet not tie up laboratory personnel and equipment with superfluous testing. The cost for laboratory testing is insignificant compared to the cost of an over-conservative design. After July 1, 2024, all laboratory testing must be performed in a qualified laboratory meeting the requirements of the Department's Laboratory Qualification Program outlined in the Department's Standard Specification for Road and Bridge Construction Section 105.

This chapter is limited to a brief description of the tests, their purpose and the uses of the resulting data. Detailed instructions on test procedures will be found in the References and Specifications and Standards listed at the end of the chapter. Tests shall be performed and results presented as described in the listed References and Specifications and Standards except as stated herein. Not every test outlined below is applicable to every project. Engineering judgment must be exercised in setting up a testing program that will produce the information required on each specific project.

5.1 Soils

5.1.1 Grain-Size Analysis

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

5.1.1.1 Sieve Analysis

This test provides a direct measurement of the particle size distribution of a soil by causing the sample to pass through a series of wire screens with progressively smaller openings of known size. The amount of material retained on each sieve is weighed. See AASHTO T 27 or AASHTO T 311 (ASTM C 136).

5.1.1.2 Hydrometer

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

Results are reported on a combined grain size distribution plot as the percentage of sample smaller than, by weight, versus the log of the particle diameter. These data are necessary for a complete classification of the soil.

conditions under which the rock exists in the field. Tests shall be performed in accordance with ASTM D 7012.

5.2.5 Unit Weight of Sample

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

5.2.6 Rock Scour Rate Determination

A rotating erosion test apparatus (RETA) was developed during research sponsored by the Department to measure the erosion of intact 4 inch long by 2.4 inch or 4 inch diameter rock core samples. Results from these tests can be used to model the erodibility of cohesive soils and soft rock and estimate scour depths. When reduced scour susceptibility is suspected, contact the District Geotechnical Engineer ~~to determine~~regarding the availability of RETA ~~scour~~ testing for site-specific hard clay or rock scour ~~applications~~determination.

5.3 References

1. Lambe, T. William, Soil Testing for Engineers, John Wiley & Sons, Inc. New York, NY, 1951.
2. NAVFAC DM-7.1 - Soil Mechanics, Department of the Navy, Naval Facilities Engineering Command, 1986.
3. Munfakh, George, Arman, Ara, Samtani, Naresh, and Castelli, Raymond, Subsurface Investigations, FHWA-HI-97-021, 1997.
4. Bowles, J. E., "Engineering Properties of Soils and Their Measurement", 3rd ed., McGraw Hill Book Company, New York, 1986

8.1.6 Earthwork Factors

Truck and fill adjustment factors used in estimating earthwork quantities should be estimated based on local experience. See **Borrow Excavation (Truck Measure)** in the **FDOT Design Manual (FDM)** for example calculations using these factors.

8.1.7 Other Considerations

Other characteristics which can be detected from soil survey explorations and which can affect the roadway design include expansive soils, springs, sinkholes (References 36 & 40 provide helpful insights into Florida sinkhole issues), potential grading problems due to the presence of rock, etc. The effect of these characteristics on roadway performance should be assessed.

8.2 Foundation Types

As an absolute minimum for Design-Bid-Build projects, GRS abutments, spread footings, driven 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-Pier 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-Pier 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 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~~ 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 ~~are normally driven closed end. In~~ may be used in extremely aggressive conditions ~~they may be used~~ 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.

8.2.3.1 Design Procedure for Major Structures

Resistance factors and associated design methods for geotechnical

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 for soil, and Appendix A for rock and Intermediate Geomaterial. ~~however~~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.

8.3.3 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 ~~consolidation~~settlement, a load transfer (negative skin friction) occurs due to the ~~consolidating~~ soil settling around the pile. The downward forces created by this process are known as downdrag. The results of downdrag can be either excessive settlements or overstressing the pile if it is an end bearing pile.

To minimize the downdrag forces: (a) place the embankment fill and allow the compressible soil(s) to consolidate prior to driving, or (b) use a polyethylene wrap around the pile within the embankment fill placed after driving, or (c) bitumen coatings may be used to reduce the load transferred by the adjacent soil(s), but a means for protecting this coating during driving must be used. The Geotechnical Engineer shall provide the downdrag values along with recommended methods to reduce the effect of downdrag. See **Appendix C** or Reference 32 for a step-by-step design procedure for the analysis of downdrag.

8.3.5 Construction Requirements

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

8.3.6 Cofferdams & Sheet Piles

Consider the effect of cofferdams penetrating rock layers that may terminate above the tip of the cofferdam or sheet pile. Consider whether continuous sheet piles should be replaced with soldier pile type cofferdams or walls.

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

~~1. Set check perimeter piles after extraction of the cofferdam or sheet pile. If any pile set checked does not meet the required resistance, set check all piles in the group.~~

~~2.~~1. If seepage is not an issue, use braced sheets with short penetrations.

~~3.~~2. Use preformed pile holes to reduce resistance above the tip elevation of the cofferdam or sheet pile, and ensure NBR is achieved from only the soil and/or rock below the tip elevation of the cofferdam or sheet pile.

~~4.~~3. Instrument all piles to ensure the NBR is achieved after excluding all the temporary resistance from materials above the tip of cofferdam and sheet pile.

~~5.~~4. Ensure the top of rock socket for the drilled shaft is deeper than the tip elevation of the cofferdam or sheet pile.

5. Evaluate the effect of vibrations during extraction on spread footing bearing materials. Ensure the cofferdam or sheet pile tip is above a 1:2 control line (1V:2H) extended from the bottom edge of the spread footing or other shallow foundation.

6. When the pile footing can be formed without the sheet piles, set check the perimeter piles after extraction of the cofferdam or sheet pile. If any pile set checked does not meet the required resistance, set check all piles in the group.

~~6.~~7. 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.

Cofferdam design should consider seepage flow and ~~seepage~~ hydrostatic pressure ~~to determine~~in determining the seal thickness and sheet pile penetration depth.

8.4 Embankment Settlement/Stability

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

8.4.1 Settlement

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

8.4.1.1 Design Procedure

References 3 and 11 are recommended.

8.4.1.2 Considerations

8.4.2 Stability

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

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.

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. ~~Analyses may be needed for flatter slopes depending on soil and site conditions.~~

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 Design Guidelines (SDG) and the FDOT Design Manual (FDM)) with adequate soil resistance against bearing, sliding, overturning, and overall stability. A design analysis is still required when standard index walls are used on a project.

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 Structures Design Guidelines 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 Structures Design Guidelines 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

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 critical temporary walls.

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

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 ~~307~~117. The Roadway Design Office has determined the Roadway Engineer will generally determine whether there are sensitive sites, such as those listed in ~~34.1~~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 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, and drilled shafts. An explanation should be included for any of these alternates judged not profile. The Davisson capacity is equivalent to the LRFD nominal resistance (R_n).

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

3. Estimated elevation of consistent bearing layer suitable for providing the required nominal resistance without the risk of punching shear failure.

4. Recommendations for pile length or bearing elevation to minimize post-construction settlements in soil layers or punching shear failure of rock or hard layers.

5. Recommendations for pile length or bearing elevation to provide the nominal uplift resistance. (The resistance factor for uplift is determined by the Construction QC method used to verify uplift resistance, see Structures Design Guidelines Table 3.5.6-1).

6. Estimated pile settlement and pile group settlement for the minimum tip elevation.

7. Effects of scour, downdrag and lateral squeeze, if applicable.

8. Estimated maximum pile driving resistance to be encountered in reaching the minimum tip elevation. If the FB-Deep Davisson bearing capacity computed at or above the minimum tip elevation exceeds the Maximum Pile Driving Resistance defined in Table 3.2 of the Structures Design Guidelines, determine the preforming or jetting elevations required to reduce the driving resistance to an acceptable magnitude. Provide additional capacity curves required by the FDOT Structures Design Guidelines on separate pages.

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

10. Recommended locations of test piles.

11. Selection of load test types, locations and depths where applicable. For static, Statnamic or Osterberg load testing, the ultimate load must be shown in the plans: the greater of 2 times the factored design load or the design nominal resistance)

~~11.~~12. 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**.

~~12.~~13. Recommendations for special provisions for pile installation (special needs or restrictions). Special construction techniques may be needed to minimize the effects of foundation installation discussed in **Section 9.2.4**.

~~13.~~14. Recommendations and special techniques to address the effects of temporary cofferdams or sheet piles on the pile capacity; see Section 8.3.6.

~~14.~~15. Present recommendations for information to be placed in the Pile Data Table shown in the SPI for FDOT Standard Plans Index 455-001.

~~15.~~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.

~~16.~~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.

~~17.~~18. Sinkhole potential and its implications for pile installation and performance.

9.2.5.4 Drilled Shafts

1. Include plots of resistance versus tip elevation for selected alternate shaft sizes. Plots should be developed for both factored (Q_r) and nominal (Q_n) resistance and should show end bearing, skin friction and total resistance (end bearing shall not be discounted). Depths of scour analyzed should be included.
2. Unless otherwise specified, separate shaft analyses for the recommended shaft sizes are to be performed for each SPT boring and/or CPT sounding. Provide resistance versus tip elevation curves for each analysis. When more than one boring is taken at a shaft group or when it is appropriate to otherwise generalize the soil strata, the corresponding resistance versus tip elevation curves are to be shown on the same plot and a recommended relationship established for that particular structure(s). Indicate the unit skin friction and end bearing values used for the analyses. Ensure socket lengths are sufficient to prevent punching shear failure in cases where the foundation is anticipated to tip in a strong layer underlain by weaker layer.
3. Provide recommendations for minimum shaft length or bearing elevation, for shaft diameter, and design soil resistance. The minimum socket length should be indicated, if applicable (non-lateral).
4. Minimum shaft spacing or influence of group effects on capacity.
5. Effects of scour, downdrag and lateral squeeze, if any.
6. Estimate drilled shaft settlement and shaft group settlement.
7. Recommend test types, locations and depths. For static, Statnamic or Osterberg load testing, the ultimate load the test should be taken to must be shown in the plans (for LRFD designs, the greater of 2 times the factored design load or the nominal resistance).
8. 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**.

the final lateral load analyses for correct soil property application.

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

9.2.6 Roadway and Approach Embankment Considerations

9.2.6.1 Settlement

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

9.2.6.2 Stability

1. Estimated overall stability using the latest AASHTO LRFD resistance factors.
2. Evaluation of possible treatment alternatives if required resistance is not provided. Recommended treatment based on economic analysis, time and environmental constraints.
3. Verify stability for fully saturated conditions.

9.2.6.3 Construction Considerations

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

9.2.7 Retaining Walls and Seawalls

a. Settlement potential

b. Recommended lateral earth pressure parameters.

c. Recommended wall type according to the FDOT Design Manual (FDM).

d. Factored soil resistance or alternate foundation recommendations.

e. Factored soil resistance and loads with respect to sliding and overturning (including standard index wall designs).

f. Overall stability of walls.

g. The design water elevation.

~~g.~~h. Recommendations for technical special provisions for fill material (except MSE walls) and drainage.

~~h.~~i. Special considerations for tiebacks, geotextiles, reinforcing materials, etc., if applicable.

~~i.~~j. MSE reinforcement lengths required for external stability, if applicable. See the FDOT Structures Design Guidelines and the FDM for details.

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).

g. Recommended plan notes.

h. FHWA checklist.

~~h.~~i. Protection of Existing Structures Checklist

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

~~j.~~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 5, Signing and Sealing Placement

Geotechnical Report	First page of official copy

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-Pier, 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.
4. Do not bury the capacity curves in printed computer output files.

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), Goble Pile Check (GPC) or Embedded Data Collectors (EDCs). Refer to Appendix F and 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.

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.

Chapter 11

11 Design-Build Projects

~~Typically, the compressed procurement schedule for Design-build limits the available time for a full geotechnical investigation to be performed prior to issuing the RFP for a Design-build project.~~ A sufficient number of geotechnical borings needs to be attached to the RFP to give DB Teams an understanding of the geotechnical conditions for the project. When possible, a more extensive geotechnical investigation should 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 performs the design specific investigation. The Design-build team shall be responsible for its own analysis of any and all data used by the team.

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~~gathers data on the conditions at the site sufficient for the design-build team to make a realistic proposal~~. It is ~~preferred~~ necessary to perform as complete a geotechnical field and laboratory investigation as ~~time~~ 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 ~~limited geotechnical data collected prior to bidding is~~ provided to prospective teams ~~for their use and incorporation into their Plans submittals~~ as Attachments to the RFP.~~information only~~. 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 ~~preliminary~~ 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 project; submit technical proposals and bids.

11.3 Design/Construction Phase

11.3.1 Department's Geotechnical Engineer

The Department's geotechnical engineer reviews design and construction methods for compliance with the contract documents and performs verification testing as required.

11.3.2 Design-Build Team

The design-build team meets the requirements set forth in the contract documents.

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 ungrouted 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~~Step3).

   $$q_{grouted} = (TCM)(q_{tip})$$
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 Bearing Acceptance Criteria For Driven Piles
&
Determining the Capacity of a Pile from an Instrumented Set-Check

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 tip and top gauges, or a combination of piles with top and tip gauges and piles with only top gauges. A minimum percentage of the piles in each bent/pier must be analyzed with the FDOT Method post-processing software; see Section 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 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 5 of this Appendix.

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

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

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

a. Estimation of production pile tip elevation based on PDA results, and selection of dynamic data for CAPWAP analysis

Based on the field collected dynamic data, estimate the tip elevation where NBR is achieved. Following the recommendations in CAPWAP's manual, select a representative blow of good data quality for signal match analysis. Adjust the blow as required and ensure the wave speed is properly determined, the F (force trace from strain gauges) and V (velocity times impedance trace from accelerometers) forces are proportional, and the final displacement converges to the measured set.

b. CAPWAP Analysis

- Check that the static resistance distribution makes sense, compare with boring logs and pile driving records to ensure reasonable assumptions are being implemented. Do not expect the automatic search feature to provide an accurate resistance distribution.
- **Match Quality number (MQN):** Make every reasonable attempt to obtain a MQN less than three. Make sure good matching is obtained for both wave and force matching analysis.
- **Resistance:** Ensure resistance is not overestimated throughout the entire first 4L/c portion of the record.
- **Match in blow count:** Make every reasonable attempt to match the observed number of blows per foot for the selected interval.

Once the CAPWAP analysis is performed, determine the equivalent Jc (Case damping) value and compare the CAPWAP capacity with the corresponding PDA capacity. The equivalent Jc is the value that produces the same PDA capacity as the one determined by CAPWAP analysis.

Reprocess the PDA and PDIPLOT based on CAPWAP analysis results (using the Jc value from the previous step and the RMX capacity or proper capacity approach), to tabulate the capacity throughout the drive.

c. Wave Equation Calibration

Using the CAPWAP estimated quake, damping and static resistance distribution, establish a WEAP model based on the test pile or indicator production pile length and properties. Perform WEAP analyses to match the following parameters from CAPWAP and PDA: Energy Transferred EMX (within 10%), Compression Stress CSX (within10%), blow count (within 10% but never below the blows/ft measured in the field) for the capacity and stroke evaluated. Some adjustments may be required to the static resistance distribution, hammer efficiency, cushion, thickness, stiffness, etc. to get an acceptable model.

Verify the model: Refer to the corrected PDIPlot and compare at several depths (near the estimated bearing depth) to check whether the model predicts accurate blow counts for this and 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 appropriate. 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. 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).

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 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 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. 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. 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.

2. Standard Driving Criteria with Goble Pile Check (GPC) Test Piles or monitored indicator production pile(s) in projects without test piles**, Nguyen_Goble Automated Pile Analysis (N_GAPA)** and **GPC Wave**

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 (DLT) are performed in accordance with Specification 455. Dynamic data are collected on GPC sensors connected at the top of the pile throughout the entire drive for every impact blow: the early pile driving blows on concrete piles are essential to evaluate wave speed as well as to monitor pile stress. For steel piles, wave speed is a constant and stress limits are high, as such, Specification 455-5.14 allows acceptance based on set-checks or redrives of steel piles. The purpose of this method is to establish a "calibrated" model that predicts the number of blows per foot and stroke combination to achieve the required resistance (establish the driving criteria). The Driving Criteria based on GPC testing involves the following steps:

a. Estimation of production pile tip elevation based on GPC results, and preparation of selected blow for signal match (N_GAPA) analysis
b. Signal match analysis
c. Wave Equation analyses based on the GPC blow and final calibrated GPC wave equation analysis
d. Driving Criteria Letter

a. Estimation of production pile tip elevation based on GPC results, and selection of dynamic data for Signal Match (N_GAPA) analysis

Ensure the material wave speed (WS) is properly determined, the F (force trace from strain gauges) and VZ (velocity times impedance trace from accelerometers) forces are proportional. Do not adjust the replay factor, unless the sensor is incorrectly mounted at an angle on the pile, making the sensor's acceleration smaller than the true pile acceleration.

Based on the field collected dynamic data, estimate the tip elevation where NBR is achieved. Select a representative blow of good data quality for Signal Match analysis: The selected blow shall have similar stroke height (STK), maximum force (FMX), and transferred energy (EMX) to the average of the blows of that foot. This is to ensure that the set of the selected blow will be similar to the average set of that foot. The average set of the foot is the inverse of the measured blow count per foot. Adjust the blow as necessary so that the final displacement converges to the measured set.

b. Signal Match (N_GAPA) Analysis

- Where the pile template creates friction on the pile, implement the "added damping" at that element to model the non-soil friction.
- Check that the static resistance distribution makes sense, compare with boring logs and pile driving records to ensure reasonable assumptions are being implemented.
- **Match Quality Number (MQN):** Make every reasonable attempt to obtain an MQN less than three.
- **Resistance:** Ensure resistance is not overestimated throughout the first 4L/c portion of the record. The simulated or calculated WU shall not be much larger than the measured WU within this portion.
- **Match in blow count:** Make every reasonable attempt to match the observed number of blows per foot for the selected interval.

Reprocess the GPC Review to produce refined signal match (instant N_GAPA or iN_GAPA) results throughout the drive.

f. Wave Equation Calibration

Import the above Signal Match analysis blow into GPC Wave. The import module in the GPC Wave program will bring in all quake, damping, and static resistance distribution into the Wave Equation Analysis.

Verify the model: Refer to the GPC Review and compare at several depths (near the estimated bearing depth) to check whether the model predicts accurate blow counts for this and other capacities/strokes measurements (use average output per foot or per increment). Refine the model if necessary.

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

g. Driving Criteria Letter

The driving criteria letter provides the inspector direction on when to accept piles. The letter should include the pile acceptance criteria based on blow count vs. stroke height results obtained from Wave Equation analysis, pile cushion details and recommendations regarding the operation of the hammer to avoid damaging the pile while driving. 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).

h. 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.

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

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

a. EDC monitoring of all Test Piles and all Production Piles, using 100% top and tip gauges.

All EDC piles are monitored in the field using Smart Structures' UF Method. Smart Structures' FDOT Method post-processing software will be used to verify the UF Method results of at least 10% of all piles in 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. 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.

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

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

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

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

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

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.

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.

~~Determination Of Acceptance Criteria For Driven Piles~~

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

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

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

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

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

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

a. ~~Estimation of production pile tip elevation based on PDA results, and preparation of~~

~~selected blow for CAPWAP analysis~~

~~b. CAPWAP analysis to confirm PDA results~~

~~c. Wave Equation calibration and final wave equation analysis~~

~~d. Driving Criteria Letter~~

~~a. Estimation of production pile tip elevation based on PDA results, and selection of dynamic data for CAPWAP analysis~~

~~Based on the field collected dynamic data estimate the tip elevation where NBR is achieved. Following the recommendations in CAPWAP's manual select a representative blow of good data quality for signal match analysis. Adjust the blow as required and ensure the wave speed is properly determined, the F (force trace from strain gauges) and V (velocity times impedance trace from accelerometers) forces are proportional and the final displacement converges to the measured set.~~

~~b. CAPWAP Analysis~~

- ~~Check that the static resistance distribution makes sense, compare with boring logs and pile driving records to ensure reasonable assumptions are being implemented. Do not expect the automatic search feature to provide an accurate resistance distribution.~~
- ~~Match Quality number (MQN): Make every reasonable attempt to obtain a MQN less than three. Make sure good matching is obtained for both wave and force matching analysis.~~
- ~~Ensure resistance is not overestimated throughout the entire first 4L/c portion of the record.~~
- ~~Match in blow count. Make every reasonable attempt to match the observed number of blows per foot for the selected interval.~~

~~Once the CAPWAP analysis is performed, determine the equivalent Jc (Case damping) value and compare the CAPWAP capacity with the corresponding PDA capacity. The equivalent Jc is the value that produces the same PDA capacity as the one determined by CAPWAP analysis.~~

~~Reprocess the PDA and PDIPLOT based on CAPWAP analysis results (using the Jc value from the previous step and the RMX capacity or proper capacity approach), to tabulate the capacity throughout the drive.~~

~~c. Wave Equation Calibration~~

~~Using the CAPWAP estimated quake, damping and static resistance distribution, establish a WEAP model based on the test pile or indicator production pile length and properties. Perform WEAP analyses to match the following parameters from CAPWAP and PDA: Energy Transferred EMX (within 10%), Compression Stress CSX (within10%), blow count (within 10% but never below the blows/ft measured in the field) for the capacity and stroke evaluated. Some adjustments may be required~~

~~to the static resistance distribution, hammer efficiency, cushion, thickness, stiffness, etc. to get an acceptable model.~~

~~**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 blow count requirement without actually achieving the static resistance). There are three choices:~~

1. ~~Implement a minimum elevation above which the criteria is not to be applied.~~
2. ~~Be conservative. Ensure the blow count requirement 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 a conservative criteria does not result in unreasonably long production pile lengths.~~
3. ~~Establish a different criteria for the upper layers to account for the increased damping value of those soils. One criteria will be applicable above a predetermined elevation, and the other will be applicable below that elevation.~~

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

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

~~a. EDC monitoring of all Test Piles and all Production Piles, using 100% top and tip gauges.~~

~~All EDC piles are monitored in the field using Smart Structures' UF Method. Smart Structures' FDOT Method post-processing software will be used to verify the UF Method results of at least 10% of all piles in 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. ~~Use top and tip gauges in at least 10% of the piles (minimum one per bent/group) and top only gauges in the remaining piles. All test piles shall contain top and tip gauges. Test piles are included in the 10% minimum. In unique soil conditions such as extreme scour, large uplift loads or high variability soils a higher percentage FDOT Method analyses is required, therefore, a higher percentage of piles with top and tip gauges is also required.~~

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

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

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

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

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

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

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

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

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

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

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

Example 1, instrumented set-check w/ minimum blows:		Example 2, instrumented set-check and advance pile:	
Blow #	Capacity, kips	Blow #	Capacity, kips
1.	450	1.	450
2.	600	2.	600
3.	590	3.	590
4.	585	4.	585
5.	580	5.	580
6.	575	6.	575
7.	570	7.	570
8.	277	8.	~~400~~572
		9.	550
		10.	530
		11.	528
		12.	520
		13.	513
		14.	509
		15.	501
		16.	494
		17.	478
		18.	461
		19.	216
Answer: a. Highest capacity recorded= 600 kips		Answer: a. Highest capacity recorded= 600 kips	
b. Average of next 5 blows/0.95 = [(590+585+580+575+570)/5]/0.95= 580 kips/ 0.95= 610 kips		b. Average of next 5 blows/0.95 = [(590+585+580+575+570)/5]/0.95= 580 kips/ 0.95= 610 kips	
Answer=600 kips		c. Lowest capacity of the following blows (excluding the last one)= 461/.90= 512 kips	
		Answer=512 kips	

Florida Test Methods

Subject	**FM**
Standard Test Method for Coefficient of Permeability - Falling Head	5-513
Standard Test Method for Limerock Bearing Ratio (LBR)	5-515
Standard Test Method for pH of Soil and Water	5-550
Standard Test Methods for Resistivity of Soil and Water	5-551
Standard Test Methods for Chloride in Soil and Water	5-552
Standard Test Method for Sulfate in Soil and Water	5-553
Standard Test Method for Determination Of Mean Permeability In The Field Using The Vertical Insitu Permeameter (VIP)	5-614
Standard Test Method for Specific Gravity and Absorption of Coarse Aggregate	1-T 085
Standard Test Method for Particle-Size Analysis of Soils	1-T 088
Standard Test Method for Determining the Liquid Limit of Soils	1-T-089
Standard Test Method for Determining the Plastic Limit and Plasticity Index of Soils	1-T-090
Standard Test Method for Specific Gravity of Soils	1-T 100
Standard Test Method for Moisture Density Relations of Soils Using a 10-lb. (4.54kg) Rammer and an 18-in. (457mm) Drop	1-T 180
Standard Practice for Thin-Walled Tube Geotechnical Sampling of Soils	1-T 207
Standard Test Method for Permeability of Granular Soils (Constant Head)	1-T 215
Standard Test Method for One-Dimensional Consolidation Properties of Soils	1-T 216
Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions	1-T 236
~~Standard Test Method for Laboratory Determination of Moisture Content of Soils~~	~~1-T 265~~
Standard Test Methods for Determination of Organic Content in Soils by Loss on Ignition	1-T 267
Standard Test Method for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression	1-T 296
Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils	1-T 297
Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions	3-D3080
Standard Test Method for Density of Bentonitic Slurries	8-RP13B-1
Viscosity of Slurry	8-RP13B-2
Standard Test Method for Sand Content by Volume of Bentonitic Slurries	8-RP13B-3