Soils and Foundations Handbook

~~2019~~2020

Summary of Changes

State Materials Office
Gainesville, Florida

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

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

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

3.2 Guidelines for Minimum Explorations

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

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

Modification for Non-Conventional Projects:
Delete the first sentence and insert the following: The following are the minimum explorations for various types of projects, except as otherwise described in the RFP:

3.2.1 Roadway Soil Surveys and Rails to Trails/Multi-use Trail Projects

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

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

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

b. In areas of variable soil conditions, additional borings shall be located at each interval considering the following criteria.

    1) For interstate highways, three borings are to be placed at each interval, one within the median and one within each roadway.

    2) For four lane roadways, two borings are to be placed at each interval, one within each roadway.

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

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

e. In all cases, at least three samples per mile 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.

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 is within 50 feet of every pile;
      ~~i.~~ii. for bridges with test piles & spans ≥~~>~~ 60' ~~and all bridges without test piles~~
      - Bents/pier foundations < 70 feet wide - at least one boring per bent/pier foundation per structure;
      - Bents/pier foundations ≥~~>~~ 70 feet wide - at least two evenly spaced borings within each ~~per~~ bent/pier foundation per structure;

      ~~ii.~~iii. for bridges with test piles & spans < 60'
      - Bents/pier foundations < 70 feet wide - at least one

boring at every other bent/pier foundation per structure

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

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

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

e. Auger Cast Piles (ACP) –

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

~~d.~~

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

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

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

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

specific borings during the design phase after the bridge pier locations are determined."

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

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

i. 2B where L< 2B,

ii. 5B where L > 5B

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

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

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

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

c. Borings for rock socketed drilled shafts shall continue through competent materials for at least two shaft diameters below the expected shaft tip elevation (See 6). Borings for non-rock socketed drilled shafts shall continue through competent materials for at least two times the width of the shaft group below the expected shaft tip elevation. (Scour and lateral requirements must be satisfied.) For non-redundant drilled shafts see additional requirements below.

~~c.~~d. Borings for rock socketed ACP shall continue through competent materials for at least 10 feet below the expected pile tip elevation (See 6). Borings for non-rock ACP shall continue through competent materials for at least two times the width of the pile group below the expected pile tip elevation. (Scour and lateral stability requirements must be satisfied.)

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

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

6) When rock is encountered, successive core runs shall be made with the objective of obtaining the best possible core recovery. **SPT's shall be performed between core runs, typically at 5-foot intervals**.

7) For bridges (including pedestrian bridges) to be supported by non-redundant drilled shaft foundations (See Section 8.2.3 Drilled Shafts.), perform at least one SPT boring at each drilled shaft location during the design phase.

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

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

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

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

12) For non-redundant drilled shafts:

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

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

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.

3.2.2.4 Noise Walls

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

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

3.2.2.5 Buildings

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

3.2.2.6 Drainage Structures

1) Borings shall be taken at proposed locations of box culverts. Trenches or hand auger borings may suffice for smaller structures.

2) For box culverts, borings shall extend a minimum of 15 feet below the bottom of the culvert or until firm material is encountered, whichever is deeper.

3) For smaller structures, borings or trenches shall extend at least 5 feet below the bottom of the structure or until firm material is encountered, whichever is deeper.

4) Corrosion testing must be performed for each site unless the structure is

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~~s~~ 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 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.

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.

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).

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

8.2.3 Drilled Shafts

Drilled shafts derive their resistance from direct contact between the surrounding soil and the drilled shaft concrete. As with driven piles, drilled shafts must be designed considering both axial and lateral loads.

8.2.3.1 Design Procedure for Major Structures

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

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

Reference 9 is generally applicable to all conditions except for drilled shafts socketed in Florida limestone. Refer to Appendix A for an approved method of determining the side resistance for drilled shafts socketed in Florida limestone. The normal spacing for drilled shafts is 3D. For rock socketed drilled shaft groups with spacing of 2.5D or greater, a group efficiency factor of 1 may be used for axial loads; for shafts tipped in other materials refer to the current AASHTO LRFD Bridge Design Specification. P-y multipliers for lateral loads are in the Structures Design Guidelines. General foundation analysis considerations are further described below. The computer program FB-Deep is available for assessment of axial design capacity and the computer program FB-Pier is available for assessment of lateral design capacity and shaft group settlement through the **Bridge Software Institute** (BSI). The Help Files for the FB-Deep & FB-Pier programs are both recommended references.

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

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

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

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

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

8.2.4 Auger-Cast Piles

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

8.2.4.1 Design Procedure

Design Auger Cast Piles for Bridges (when allowed) using the same design procedures as for drilled shafts outlined in Appendix A.

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

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

8.2.4.2 Considerations

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

8.2.5 Micro Piles

In special cases micro piles may be the preferred foundation system. This would typically be in cases of limited access, close proximity to settlement sensitive structures and foundations to be strengthened. See the SDG for restrictions on the use of micropiles for bridges and other structures.

8.2.5.1 Design Procedure

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

8.7 Computer Programs used in FDOT

See the listing of Geotechnical Computer Programs used in FDOT on the Geotechnical Engineering webpage.

Hyperlink added

8.8 References

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Chapter 10

10 Construction and Post-Construction

The Geotechnical Engineers' involvement does not end with the completion of the final report; they may also be involved in the preconstruction, construction and maintenance phases of a project.

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

Modification for Non-Conventional Projects:
Delete the first and second paragraphs, and insert the following: During construction, in-situ materials and construction methods for geotechnical elements must be inspected to assure compliance with the design assumptions and the project specifications. Such inspection tasks include subgrade and/or embankment compaction control, assurance of proper backfilling techniques around structural elements, and routine footing, drilled shaft, and piling installation inspection.

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

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

10.1 Dynamic Pile Driving Analysis

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

10.2 Dynamic Monitoring of Pile Driving

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

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

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

The Embedded Data Collector (EDC) system developed under FDOT sponsored research utilizes 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 gages to determine the pile capacity and is considered equivalent to ~~without the need for~~ 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.

diameters below the bottom of the drilled shaft excavation for non-redundant shafts. Coring shall be performed in accordance with ASTM D 2113 using a double wall or triple wall core barrel. The core barrel shall be designed to provide core samples 4 inches in diameter or larger, and allow the cored material to be removed in an undisturbed state. Refer to ASTM D 2113 and ASTM D 5079.

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

10.6 Shaft Inspection Device (SID)

A piece of equipment that is used to inspect the bottom cleanliness of drilled shafts prior to placement of concrete through the use of an inspection bell which houses a high resolution video camera (See **Figure** ~~**Figure**~~ **40**) The inspection bell is lowered from a service platform to the bottom of the shaft, and the operator can view the condition of the bottom via the camera. The bell is fitted with a depth gage to indicate the thickness of debris on the shaft bottom. Sufficient views of the shaft bottom are used to inspect a statistically significant portion of the shaft bottom. The Shaft Inspection Device uses pressurized nitrogen to overcome the static head of the drilling fluids, purge the fluids from the camera bell, and provide an unobstructed view of the shaft. A small reduction in air pressure would allow drilling fluid to slowly enter the bell.

When the shaft bottom is flat (as required in Specifications) and the bell is plumb, a layer of water or drilling fluid in the bell can be used measure the thickness of sediments mounds "away" from the sediment depth gauge. When the fluid rises to the 1/2" pin on the gauge, the percentage of the view covered with sediment deposits thicker than 1/2" may be estimated; these sediments are above the fluid level. When the 1/2" depth pin is missing the first mark (1.0 cm) depth must be used. The same procedure may also be used to determine whether any portion of the view contains sediments in excess of 1-1/2" [4.0 cm] thick. Special care must be used to ensure the fluid does not erode the sediment as it enters the bell, especially if the operator attempts to fill the bell with water using the water jets intended for flushing these sediments, instead of filling the bell with drilling fluid as described above.

10.7 Field Instrumentation Monitoring

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

All field instrumentation should be installed, and have readings taken, by qualified personnel under the supervision of a Geotechnical Engineer. A Geotechnical Engineer should interpret all data and recommend any necessary action. For example, in projects where surcharging or precompression is required to improve the foundation soils, waiting periods are required. It is essential that the Geotechnical Engineer

Appendix A

Determination of Design Side Resistance for Drilled Shafts & Auger Cast Piles Socketed in the Florida Limestone Based on Rock Core Testing

DETERMINATION OF DESIGN SIDE SHEAR RESISTANCE FROM TEST DATA TO DESIGN PARAMETERS FOR DRILLED SHAFTS & AUGER CAST PILES SOCKETED IN FLORIDA LIMESTONE

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

Introduction

The variable strength properties of the Florida limestone formation always prompted the question of what design side shear resistance should be used for a drilled shaft socketed in it. Some engineers even decide that doing any tests on rock cores obtained from the project site is senseless because of the uncertainties associated with a spatial variability of the limestone. This presentation provides a method for determining a reasonable design side shear resistance value from a statistically significant number of ASTM D 7012 (Method D) unconfined compression and ASTM D 3967 (with t/D ≥ 1.0) splitting tensile tests.

Design Method

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

$$f_{su} = \frac{1}{2} * \sqrt{q_u} * \sqrt{q_t} \qquad (1)$$

where $\boldsymbol{f_{su}}$ is the ultimate side shear resistance,
$\boldsymbol{q_u}$ is the unconfined compression strength of rock core, and
$\boldsymbol{q_t}$ is the splitting tensile strength (McVay, 1992).

$$(f_{su})_{DESIGN} = REC * f_{su} \qquad (2)$$

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

Appendix B

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

GENERAL

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

NOISE BARRIER FOUNDATIONS

See Section 8.2.4.1

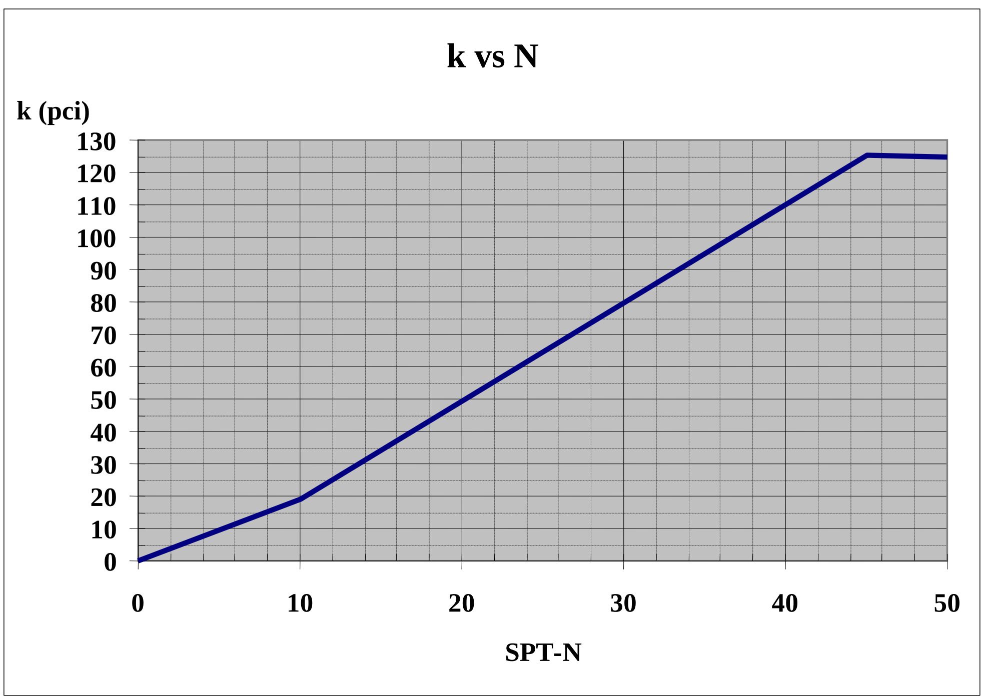
LATERAL LOAD RESISTANCE

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

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

k values in Sands.

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



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

Friction Angles in Sand

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

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

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

Walls founded on berms

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

Clay

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

Rock

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

Rock material with N-values less than 10 blows/foot shall be modeled as sand. Rock material with N-values between 10 and ~~25~~ 20 blows/foot shall be modeled as sandy gravel:

$$\text{Friction Angle, } \varphi = N/4 + 33$$

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

Rock material with N-values of ~~25~~ 20 blows/foot or more:

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

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

Appendix D

Design Method for Drilled Shaft with Pressure Grouted Tip

Design Method for Drilled Shaft with Pressure Grouted Tip

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

~~1. Calculate the 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 the FB-Deep for drilled shaft founded in cohesionless soils, thus, one can use the FB-Deep to calculate q_{tip} = 0.6 x SPT N_{60}, where SPT N_{60} is weighted average at shaft tip (Reese & O'Neill, 1988).~~

1. Calculate the nominal side shear resistance~~,~~ (F_s~~,~~) for the given shaft diameter (D) ~~(D)~~ and total embedded shaft length ~~of shaft~~.
2. Calculate the nominal uplift side shear resistance ($F_{s\ uplift}$);

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

~~2.~~ *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 ~~the~~ FB-Deep for drilled shafts founded in cohesionless soils, thus, one can use the FB-Deep formula (~~to calculate~~ q_{tip} = 0.6 x SPT N_{60}, tsf~~,~~) where SPT N_{60} is weighted average at shaft tip (Reese & O'Neill, 1988).

~~3.~~4. Determine the maximum anticipated grout pressure (GP_{max}) ~~–~~by ~~divide~~ dividing the nominal uplift side shear resistance~~,~~ ($F_{s\ uplift}$~~,~~) by the cross-sectional area of the shaft~~,~~ (A);

$$GP_{max} = F_{s\ uplift}/A$$

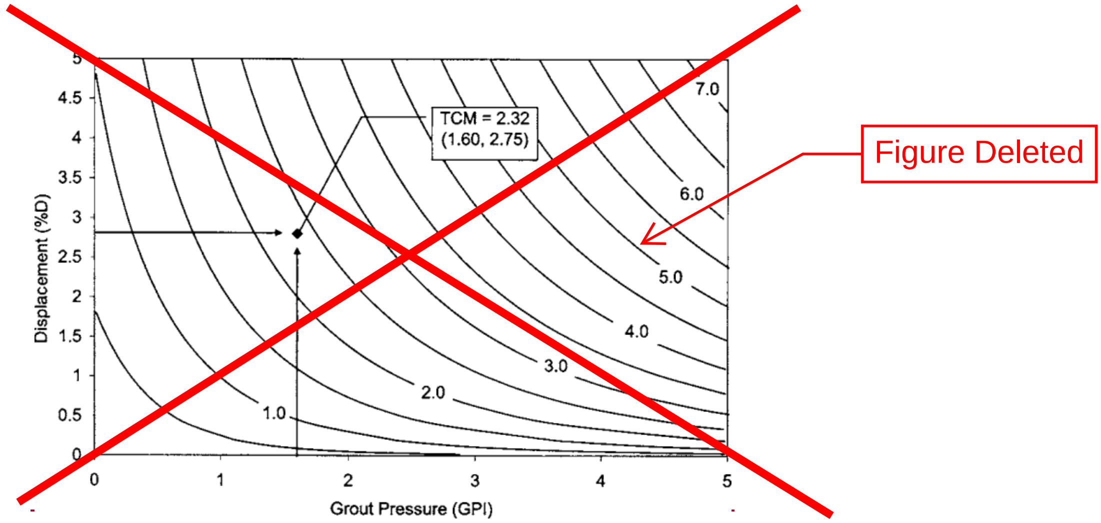
~~4.~~5. Calculate the Grout Pressure Index, *GPI*, as the ratio of the maximum anticipated grout pressure (~~step~~ Step ~~3~~4) to the ungrouted unit tip resistance~~,~~ (~~q_{ult}~~ q_{tip}), (~~step~~ Step ~~1~~3);

$$GPI = GP_{max}/\ q_{tip}$$

~~5. Establish the maximum permissible service displacement as a ratio of the shaft diameter, %*D*.~~

6. Determine the Tip Capacity Multiplier (*TCM*) using the following equation ~~or chart with the Grout Pressure Index (step 4) and the maximum permissible service displacement, %D, from step (5).~~

$$TCM = 0.713(GPI)\cancel{(\%D^{0.364})} + 0.3\cancel{\frac{\%D}{0.4(\%D)+3.0}}$$ ~~or use graph:~~



7. Estimate the grouted unit tip resistance as the product of the Tip Capacity Multiplier (~~step 6~~Step 6) and the ungrouted unit tip resistance ~~,~~ (q_{tip}~~,~~), (~~step1~~Step1).

$$q_{grouted} = (TCM)(q_{tip})$$

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

   ***The tip area of a grouted shaft has been shown to be larger than the shaft diameter due to cavity expansion of the soils beneath the tip. While values less than the constructed shaft diameter have been suggested to account for variability, the constructed diameter of the shaft was used to develop this design method and therefore statistically incorporates variations both larger and smaller than the nominal shaft diameter.

9. Compute the nominal resistance $R_n = R_{n\_side\ shear} + R_{n\_tip}$
10. Compute the factored resistance $R_R = \phi(R_{n\_side\ shear} + R_{n\_tip})$

~~The design of the nominal resistance for post grouted shafts is simply the sum of the ultimate side shear resistance and the grouted tip resistance at some specified allowable shaft displacement.~~ 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.

Design Example

Given: A 3 ft diameter drilled shaft tipped in sand (SPT $N_{60\ tip}$ = 30 and F_s = ~~200~~ 300 tons).

- Calculate the nominal uplift side shear resistance:

$F_{s\ uplift}$ = (0.75)(300 tons)
$F_{s\ uplift}$ = 225 tons

- Calculate the nominal end bearing @ 5%D settlement:

q_{tip} = (0.6)(30)
q_{tip} = 18 tsf

- ~~*~~ Calculate the maximum anticipated grout pressure:

Maximum Grout Pressure = ~~Side Shear Force~~ $F_{s\ uplift}$ / Tip Area
GP_{max} = ~~200~~ (225 tons) / [~~(~~(3 ft)^2 π~~□~~/4]~~)~~
GP_{max} = ~~28.3~~ 31.8 tsf ~~* Calculate the nominal end bearing @ 5%D settlement:~~
~~nominal End Bearing = 0.6 * SPT N60 (Reese & O'Neill, 1988)~~
~~qtip = 0.6 * 30~~
~~qtip = 18 tsf~~

- ~~*~~ Calculate the grout pressure index (GPI):

Grout Pressure Index = ~~Grout Pressure~~ GP_{max} / Ultimate End Bearing
GPI = ~~28.3~~ 31.8 tsf / 18 tsf
GPI = ~~1.57 say 1.6~~ 1.77
~~*Permissible shaft settlement =2.75%~~

- Calculate the Tip Capacity Multiplier (TCM):

TCM = (0.713)(1.77)+0.3
TCM = ~~2.32~~ 1.56

- Calculate grouted unit end bearing capacity

$q_{grouted}$ = (TCM)(q_{tip})= ~~2.32*18=41.7~~ (1.56)(18)=28.1 tsf

Nominal ~~Bearing~~ Side and Tip Resistances after grouting:

$R_{n\ side\ shear}$ = ~~Side Shear Force + (qgrouted)(tip area**) ≤ 2* Side Shear Force~~ 300 tons
$R_{n\ tip}$ = ~~200 tons + (41.7 tsf)[π(2.5ft)²/4] ≤ 2* Side Shear Force~~ ($q_{grouted}$)(A_{tip})
$R_{n\ tip}$ = ~~200 tons + 204 tons ≤ 2* Side Shear Force~~ (28.1 tsf)(3 ft)^2(3.1416/4)
$R_{n\ tip}$ = ~~400~~ 199 tons
R_n = 499 tons

Factored Bearing Resistance after grouting:

$R_R = \phi(R_{n\ side\ shear} + R_{n\ tip})$
R_R = 0.6 (300 tons + 199 tons)
R_R = 299 tons

~~** Note that the tip area will vary from the cross-sectional area of the shaft in a well cleaned shaft excavation to the area of the tip grouting plate in a marginally cleaned shaft. In excavations tipped in loose to medium dense sands, it is very difficult to obtain a well cleaned excavation; this example assumes diligent cleaning effort resulted in only a small 2" to 3" reduction in tip radius. Actual results may vary.~~

Appendix I

Reference List

FHWA-NHI-06-019 and 020 Ground Improvement Methods, Reference Manual Volumes I & II

FHWA-NHI-07-071 Earth Retaining Structures Reference Manual

FHWA-IF-~~10-016~~99-025 Drilled Shafts: Construction Procedures and ~~LRFD~~ Design Methods

FHWA-NHI-10-024 thru 025 Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Volumes I & II

FHWA-HRT-13-046 Deep Mixing for Embankment and Foundation Support

Geotechnical Engineering Circular No. 1	Dynamic Compaction
Geotechnical Engineering Circular No. 2	Earth Retaining Systems
Geotechnical Engineering Circular No. 4	Ground Anchors and Anchored Systems
Geotechnical Engineering Circular No. 5	Evaluation of Soil and Rock Properties
Geotechnical Engineering Circular No. 7	Soil Nail Walls
Geotechnical Engineering Circular No. 8	Design And Construction Of Continuous Flight Auger Piles
~~Geotechnical Engineering Circular No. 10~~	~~Drilled Shafts: Construction Procedures and LRFD Design Methods~~
Geotechnical Engineering Circular No. 11	Mechanically Stabilized Earth Walls and Reinforced Soil Slopes

“Checklist and Guidelines for Review of Geotechnical Reports and Preliminary Plans and Specifications”

Military

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U.S. Environmental Protection Agency, Land Treatment of Municipal Wastewater - Process Design Manual, 1981.