Soils and Foundations Handbook

~~2018~~2019

Summary of Changes

State Materials Office
Gainesville, Florida

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 designed for the most aggressive conditions. When testing is performed, material from each stratum above the invert elevation and any standing water shall be tested. For drainage systems parallel to roadway alignments, tests shall be performed at 1,500-feet (or smaller) intervals along the alignment.

3.2.2.7 High Mast Lighting, and Overhead Sign Structures

1) One boring shall be taken at each designated location; ensure each shaft is within 20 feet of a boring.

2) Borings shall be 40 feet into suitable soil or 10 feet into competent rock with 15 feet minimum total depth. Deeper borings may be required for cases with higher than normal torsional loads.

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 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.

3.2.2.8 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

4.9.1 Constant Head Test

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

4.9.2 Rising Head Test

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

4.9.3 Falling Head Test

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

4.9.4 Pumping Test

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

Permeability calculations are made based on the rate of pumping, the measured draw down, and the configuration of the test hole and observation wells. Refer to ASTM D 4050 and Reference 17.

4.9.5 Vertical Insitu Permeameter (VIP) Test

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

4.10 Environmental Corrosion Tests

These tests are carried out on soil and water at structure locations, on structural backfill materials and on subsurface materials along drainage alignments to determine the corrosion classification to be considered during design. For structures, materials are classified as slightly, moderately, or extremely aggressive, depending on their pH, resistivity, chloride content, and sulfate content. (Refer to the latest Structures Design Guidelines, for the criteria, which defines each class). For roadway drainage systems, test results for each stratum are presented for use in determining alternate culvert materials. Testing shall be performed in the field and/or the laboratory according to the standard procedures listed below. Compile the sample data and results into the "Corrosion Series Test Results_SMO.xlsx" Excel form on the Geotechnical Engineering webpage, and email the completed form to SM-corrosionsection@dot.state.fl.us.

Chapter 8

8 Analysis and Design

Once all exploration and testing have been completed, the Geotechnical Engineer must organize and analyze all existing data and provide design recommendations. The scope of the analysis will of course depend upon the scope of the project and the soils involved.

This chapter will discuss the major factors, which must be considered during the analysis and design phase and possible methods of solving potential problems. **Table 2** and **Table 3** present FHWA guidelines regarding analyses which should be performed. The references cited in the text provide suggested methods of analysis and design. A list of computer programs, which are approved for use by the Department to aid analysis, is ~~given in **Tables 4 through 12**.~~available on the Geotechnical Engineering webpage.

8.1 Roadway Embankment Materials

The suitability of in-situ materials for use as roadway embankment is determined by analysis of the results of soil survey explorations. Embankment materials must comply with Standard Plans, Index 120-001.

The subsurface materials identified during soil survey explorations should be classified, usually according to the AASHTO classification system, and stratified. Soils must be stratified such that similar soils are contained within the same stratum. Stratifications shall be based upon the material removal and utilization requirements of Standard Plans, Indexes 120-002 & 120-001. If testing identifies dissimilar types within the same stratum, additional sampling and testing may be required to better define or restratify the in-situ materials.

Once stratified, each stratum must be analyzed to define characteristics that may affect the design. Such characteristics include:

8.1.1 Limits of Unsuitable Materials

The limits of all in-situ materials considered unsuitable for pavement embankments should be defined and the effect of each material on roadway performance should be assessed. Refer to Standard Plans, Index 120-002 for requirements on excavation and replacement of these materials. In areas where complete excavation is not feasible but the potential for problems exists, possible solutions to be considered include stabilization with lime, cement, or flyash, placement of geotextile, surcharging, and combinations of these and other methods.

8.1.2 Limerock Bearing Ratio (LBR) (When Allowed)

When LBR testing is permitted by the State Materials Office for design purposes, the LBR value should be determined based on test results and the stratification of subsurface materials. The design value should be representative of actual field conditions. Two methods are applied to the LBR test data to account for variability in materials, moisture contents and field versus laboratory

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).

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

Various drilled shaft sizes should be analyzed to achieve an optimum design. For water crossings, depth of scour must be considered. Any anticipated construction problems should be considered. The method of construction (dry, slurry, or casing) should be addressed, as this will affect the side friction and end bearing values assumed during design. Both the unit side

8.7 Computer Programs used in FDOT

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

~~Table 4, Driven Piles~~

~~FB-Deep~~	~~Bridge Software Institute~~ ~~http://bsi-web.ce.ufl.edu/~~	~~Computes static pile capacities based on SPT or CPT data. Used for precast concrete, concrete cylinder, steel H- or steel pipe piles, and drilled shafts.~~
~~WEAP~~		~~Dynamic analysis of pile capacity and drivability.~~

~~Table 5, Drilled Shafts~~

~~FB-Deep~~	~~Bridge Software Institute~~ ~~http://bsi-web.ce.ufl.edu/~~	~~Computes static drilled shaft and driven pile capacities based on SPT or CPT data.~~

~~Table 6, Lateral Loads~~

~~FB-Pier~~ ~~FB-MultiPier~~	~~Bridge Software Institute~~ ~~http://bsi-web.ce.ufl.edu/~~	~~The Lateral Pile Group Structural Analysis Program is a 3-D nonlinear substructure analysis program.~~
~~COM624P~~	~~COM624P - Laterally Loaded Pile Analysis Program for the Microcomputer, Version 2.0, FHWA-SA-91-048, 1993.~~ ~~http://www.fhwa.dot.gov/bridge/software.HTM~~	~~Computes deflections and stresses for laterally loaded piles and drilled shafts.~~
~~LPile~~	~~Ensoft~~	~~Computes deflections and stresses for laterally loaded piles and drilled shafts.~~

~~Table 7, Spread Footings~~

~~CBEAR~~	~~CBEAR Users Manual, FHWA-SA-94-034, 1996.~~	~~Computes ultimate bearing capacity of spread or continuous footings on layered soil profiles.~~

~~Table 8, Sheet Piling~~

~~CWALSHT~~	~~Dawkins, William P., Users Guide: Computer Program For Design and Analysis of Sheet Pile Walls by Classical Methods, Waterways Experiment Station, 1991.~~	~~Design and analysis of either anchored or cantilevered sheet pile retaining walls. Moments, shear, and deflection are shown graphically. Analysis of anchored walls does not follow AASHTO requirements.~~
~~Shoring~~	~~Civil Tech, CT-SHORING~~ ~~http://civiltech.com/software/shoring.php~~	~~Excavation supporting system design and analysis.~~
~~SPW 911~~	~~Pile Buck International, Inc.~~ ~~P.O. Box 64-3609~~ ~~Vero Beach, FL, 32964-3299~~ ~~http://www.pilebuckinternational.com/product/spw911-sheet-pile-design-software/~~	~~Care must be exercised to ensure analyses are in accordance with the AASHTO code earth pressure diagrams. Program may mix methods when inappropriate values are changed. Use Coulomb method.~~

~~Table 9, Slope Stability~~

~~PCSTABL~~	~~PC-STABL6 Purdue University.~~	~~Calculates factor of safety against rotational, irregular, or sliding wedge failure by simplified Bishop or Janbu, or Spencer method of slices.~~

~~RSS~~	~~RSS Reinforced Slope Stability A Microcomputer Program User's Manual, FHWA-SA-96-039, 1997~~ ~~http://www.fhwa.dot.gov/bridge/software.HTM~~	~~A computer program for the design and analysis of reinforced soil slopes (RSS Reinforced Slope Stability). This program analyzes and designs soil slopes strengthened with horizontal reinforcement, as well as analyzing unreinforced soil slopes. The analysis is performed using a two-dimensional limit equilibrium method.~~
~~Visual Slope~~	~~Visual Slope, Inc.~~ ~~http://www.visualslope.com/index.html~~	~~Visual Slope uses drawing procedures similar to AutoCAD to help users establish analytical models, which allow detailed modeling of a complicated project.~~
~~XSTABL~~	~~Interactive Software Designs, Inc., http://xstabl.com/index.htm~~	~~Program performs a two dimensional limit equilibrium analysis to compute the factor of safety for a layered slope using the modified Bishop or Janbu methods.~~

~~Table 10, Embankment Settlement~~

~~FOSSA~~	~~http://www.geoprograms.com/fossaindex.htm~~	~~Calculates compression settlement due embankment loads.~~
~~Settle 3D~~	~~http://www.rocscience.com/products/7/Settle3D~~	~~Analysis of vertical consolidation and settlement under foundations, embankments and surface loads.~~

~~Table 11, Soil Nailing~~

~~SNAP-2~~	~~FHWA:~~ ~~http://www.fhwa.dot.gov/engineering/geotech/software.cfm~~	~~Design and evaluation procedures developed in general accordance with the FHWA guidelines.~~

~~Table 12, Walls and Steepened Slopes~~

~~GEO5~~	~~Bentley GeoStructural Retaining Wall Analysis Software~~ ~~(Ver 19.3)~~	~~Care is needed to update the outdated default load and resistance factors.~~
~~MSEW 3.0~~	~~ADAMA Engineering, Inc.,~~ ~~http://msew.com/msewindex.htm~~	~~The program can be applied to walls reinforced with geogrids, geotextiles, wire mesh, or metal strips. It allows for reduction factors associated with polymeric reinforcement or for corrosion of metallic reinforcement.~~
~~ReSSA 3.0~~	~~ADAMA Engineering, Inc.,~~ ~~http://msew.com/ressaindex.htm~~	~~A computer program for the design and analysis of reinforced soil slopes (RSS Reinforced Slope Stability). This program analyzes and designs soil slopes strengthened with horizontal reinforcement, as well as analyzing unreinforced soil slopes. The analysis is performed using a two dimensional limit equilibrium method.~~

~~MSE LRFD~~	~~FDOT Structures Design Office~~ ~~http://www.dot.state.fl.us/structures/proglib.shtm (See Note 3)~~	~~An Excel spreadsheet for external stability analysis of MSE walls by LRFD methods.~~

~~Cantilever LRFD~~	**~~FDOT Structures Design Office~~** **~~http://www.dot.state.fl.us/structures/proglib.shtm (See Note 3)~~**	**~~An Excel spreadsheet for external stability analysis of cantilever retaining walls by LRFD methods.~~**

~~NOTE:~~

~~1) The programs included in this list are generally available from public sources. Many additional programs, which perform similar tasks, can be obtained from the private sector.~~

~~2) Many of the programs listed are continually updated or revised. It is the user's responsibility to become familiarize with the latest versions.~~

~~3) FDOT's programs are available on the FDOT's Structures Internet site. The address is: http://www.dot.state.fl.us/structures/proglib.shtm . Geotechnical programs are listed below the table of structural engineering/design programs.~~

~~4)~~ **~~Programs not listed require approval from the State Geotechnical Engineer.~~**
~~Requests to use finite element programs for complex geotechnical analyses must include the means to determine input parameters, model calibration and the proposed verification of results.~~

Table 4, Example + 2% of Optimum Method Calculation

TEST NO.	MAXIMUM LBR	LBR AT MOISTURE CONTENTS: (OF OPTIMUM LBR) - 2%	+ 2%
1	165	30	18
2	35	25	25
3	64	60	45
4	35	12	8
5	85	20	45
6	55	45	20
7	33	7	10
MEAN LBR VALUE:	67.42	28.42	24.43
AVERAGE = 26.42 (26) => DESIGN LBR = 26			

b. Evaluate the strength and extent of unsuitable soils within the proposed alignment including their probable effect on roadway performance. Indicate the anticipated horizontal and vertical extent of removal of unsuitable materials. Provide recommendations for special construction considerations, to minimize anticipated problems.

c. Provide estimated soil drainage characteristics and permeability or infiltration rates. In the case of rigid pavement design, include average laboratory permeability values for each stratum based on the requirements given in the Rigid Pavement Design Manual.

d. Provide recommendations for cut or fill sections when seepage, stability or settlements are significant.

e. Provide recommendations and considerations for any proposed walls.

f. Provide recommendations and considerations for any proposed storm water retention ponds.

g. Provide recommendations to minimize the effects of roadway construction (vibratory rollers, utility excavations, sheet pile installation, etc.) on surrounding structures and on the usage of those structures.

9.1.3 Roadway Soils Survey (Report of Tests) Sheet

This sheet presents a material description and results of classification and corrosivity tests for each stratum. Recommendations for material utilization are provided in accordance with Standard Plans, Indexes 120-001 and 120-002. Visual classification of muck is not sufficient; present organic and moisture content test results. The number of lab tests performed for each stratum shall be included for corrosion tests results as well as classification tests. Include the range of result values of all tests performed for each stratum. Round all test values except organic content values less than 10 and pH to the most appropriate whole number; round pH test results and organic content values less than 10 to one decimal place. Include all tests performed, including M_R tests performed by the State Materials Office. The Report of Tests Sheet is included in the report and the construction plans. **Figure 29** is an example of a typical test results sheet.

9.1.4 Roadway (and Pond, etc.) Cross Sections

Stratified boring logs are plotted on the cross section sheets included in the construction plans. Each material stratum is numbered corresponding to the strata on the test results sheet. **Figure 30** is an example of a typical cross sections sheet, and **Figure 31** is a typical example of a generalized soil profile. If cross sections sheets are to be prepared by others, the appropriate subsurface information should be provided. The Geotechnical Engineer shall verify that the data has been correctly incorporated.

The anticipated horizontal and vertical limits for removal of unsuitable materials shall be indicated on the cross sections.

9.1.5 Appendix

All roadway soil survey reports shall include an appendix, containing

STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

DATE OF SURVEY : 2/15/2018-5/1/2018
SURVEY MADE BY : SOIL SURVEY, INC.
SUBMITTED BY : LEIA I. CROSSWELL, P.E.

PROJECT NAME: ______

FINANCIAL PROJECT ID : ______

DISTRICT : 3
ROAD NO : SR 22
COUNTY : BAY

CROSS SECTION SOIL SURVEY FOR THE DESIGN OF ROADS

SURVEY BEGINS STA. : 125+87 SURVEY ENDS STA. : 442+67
REFERENCE: BASELINE SURVEY

	ORGANIC CONTENT		MOISTURE CONTENT		SIEVE ANALYSIS RESULTS % PASS						ATTERBERG LIMITS (%)					CORROSION TEST RESULTS				
STRATUM NO.	NO. OF TESTS	% ORGANIC	NO. OF TESTS	% MOISTURE	NO. OF TESTS	10 MESH	40 MESH	60 MESH	100 MESH	200 MESH	NO. OF TESTS	LIQUID LIMIT	PLASTIC INDEX	AASHTO GROUP	DESCRIPTION	NO. OF TESTS	RESISTIVITY ohms-cm	CHLORIDE ppm	SULFATES ppm	pH
1	0	--	0	--	0	--	--	--	--	--	0	--	N.P.	--	ROCK BASE	0	--	--	--	--
2	0	--	0	--	4	87-98	77-93	59-82	44-55	3-10	0	--	N.P.	A-3	SUBGRADE, GRAY & TAN SAND W/TRACE SILT, LIMEROCK & SHELL	0	--	--	--	--
3	7	3.2-4.4	7	8-20	7	94-100	86-94	65-71	34-45	15-21	0	--	N.P.	A-2-4	FILL, DARK BROWN SAND W/SOME SILT & TRACE LIMEROCK	7	34000-43000	40-60	18-72	6.4-8.3
4	3	1.2-2.2	4	15-25	4	84-100	71-93	60-90	53-82	37-45	4	25-38	5-9	A-4	GRAY AND BROWN SILTY SAND W/TRACE CLAY AND LIMESTONE FRAGMENTS	4	23000-26000	60-120	84-96	8.4-8.9
5	0	--	0	--	3	100	99-100	96-98	75-80	30-34	3	42-44	11-15	A-2-7	TAN AND LIGHT GRAY SILTY SAND W/SOME CLAY AND TRACE SHELL	3	6600-8000	60-120	156-216	7.5-8.2
6	3	22-40	3	47-150	3	--	--	--	--	30-46	3	25-33	10-15	A-8	MUCK, ORGANIC DARK BROWN SILTY SAND W/SOME CLAY	0	--	--	--	--
7	0	--	0	--	3	100	88-92	73-79	60-69	51-55	3	55-61	38-53	A-7-6	YELLOW AND GRAY SILTY SANDY CLAY	0	--	--	--	--
8	3	16-20	3	20-58	3	99-100	97-99	88-97	77-80	10-15	0	--	N.P.	A-2-4	ORGANIC BROWN SAND W/TRACE SHELL	3	2000-3500	120	120	4.6-5.2
9	0	--	0	--	0	--	--	--	--	--	0	--	--	--	NATURAL LIMESTONE	0	--	--	--	--

EMBANKMENT AND SUBGRADE MATERIAL

STRATA BOUNDARIES ARE APPROXIMATE MAKE FINAL CHECK AFTER GRADING

▼ - WATER TABLE ENCOUNTERED

▽ - ESTIMATED SEASONAL HIGH GROUNDWATER LEVEL

GNE - GROUND WATER NOT ENCOUNTERED

The material from Stratum Number 1 is Rock Base under Asphaltic Concrete.

The material from Stratum Number 2 appears satisfactory for use in the embankment when utilized in accordance with Standard Plans, Index 120-001.

The material from Stratum Number 3 appears satisfactory for use in the embankment when utilized in accordance with Standard Plans, Index 120-001. However, this material is likely to retain excess moisture and may be difficult to dry and compact. It should be used in the embankment above the water level existing at the time of construction.
This material may not be used in the subgrade portion of the roadbed due to its organic content.

The materials from Stratum Numbers 4 and 5 are plastic materials and shall be removed in accordance with Standard Plans, Index 120-002. They may be placed above the existing water level at the time of construction, to within 4 feet of the proposed base. They should be placed uniformly in the lower portion of the embankment for some distances along the project rather than full depths for short distances.

The material from Stratum Numbers 6 and 8 is ORGANIC/A-8 material and shall be removed in accordance with Standard Plans, Index 120-002, except where noted in the cross sections.

The material from Stratum Number 7 is Highly Plastic material and shall be removed in accordance with Standard Plans, Index 120-002. It may be used within the project limits as indicated in Standard Plans, Index 120-001 only when excavated within the project limits and is not to be used when obtained from outside the project limits.

The material from Stratum Number 9 is the Natural Limestone Formation. Special tools and equipment may be required to excavate and/or dewater this material.

The " -- " indicates an unmeasured parameter.

LEIA I. CROSSWELL LICENSE No 99995 STATE OF FLORIDA PROFESSIONAL ENGINEER

THIS ITEM HAS BEEN DIGITALLY SIGNED AND SEALED BY
Leia I. Crosswell
2018.10.11 8:31:01 - 4'00'

ON THE DATE ADJACENT TO THE SEAL

PRINTED COPIES OF THIS DOCUMENT ARE NOT CONSIDERED SIGNED AND SEALED AND THE SIGNATURE MUST BE VERIFIED ON ANY ELECTRONIC COPIES.

SOIL SURVEY, INC.
321 EAST 2ND STREET
TALLAHASSEE, FL 32301
CERTIFICATE OF AUTHORIZATION: 11235
LEIA I. CROSSWELL, P.E. NO. 99995

REVISIONS									
DATE	DESCRIPTION	DATE	DESCRIPTION	LEIA I. CROSSWELL, P.E. P.E. NO. 99995 SOIL SURVEY, INC. 321 EAST 2ND STREET TALLAHASSEE, FL 32301 CERTIFICATE OF AUTHORIZATION: 11235	STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION			ROADWAY SOIL SURVEY	SHEET NO.
					ROAD NO.	COUNTY	FINANCIAL PROJECT ID		
					SR 22	BAY	123456-1-52-01		GR-1

Figure 29, Typical Report of Tests Sheet (New Example) 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 ~~UF~~ method of analysis published by Tran et. al. utilizes the data from the top and bottom gages to determine the pile capacity 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 ~~will~~ incorporate~~require the use of~~ 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~~actual ultimate capacities are~~ not less than the nominal resistance computed~~the computed ultimate loads used~~ 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. ~~He~~ They should also be prepared to modify designs if~~where~~ ~~necessary based on~~the load tests fail to verify and fully mobilize the design values ~~data~~.

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.

10.3.1 Static Load Tests

Three types are commonly used based on type of loading: axial compression (refer to ASTM D 1143) (See **Figure 35**), axial tension (refer to ASTM D 3689), or lateral load (refer to ASTM D 3966). In each case, the test typically consists of a jack/load cell system to apply a loading based on the desired application against a reaction system and measuring the resulting displacement.

10.3.2 Statnamic Load Tests

Statnamic applies axial or lateral loads of 30 to over 5,000 tons (0.3 to >44 MN) (See **Figure 36** and **Figure 37** and). The load application is between a static load and a dynamic load. The associated dynamic and rate of loading effects differ by soil type and are subtracted, resulting in the equivalent static load curve. No reaction piles are required. The duration of loading is on the order of 0.1 seconds. The load cell and LVDTs provide direct measurements of load-displacement behavior. Drilled shafts tested by the Statnamic method should be instrumented with electronic resistance strain gauges at various elevations to measure load transfer characteristics. Statnamic produces load versus displacement results immediately on site. ASTM Standard D 7383, Procedure A describes this type of testing.

10.3.3 Other Rapid Load Tests

Alternative Axial Compressive Force Pulse (Rapid) Load Tests are described in ASTM Standard D 7383, Procedure B; however, these alternative test methods have not been adequately calibrated to static load test results to determine an appropriate resistance factor for FDOT projects.

10.3.4 Osterberg Load Tests

The Osterberg Load Cell is cast into the bottom of a pile or anywhere in a drilled shaft (See **Figure 38**). The cell expands to jack against the foundation's end bearing capacity so no reaction system is required. The cell can be placed above the bottom of a drilled shaft to equal out the loading. Or multiple cells can be used to isolate various zones. Currently there is no ASTM standard on this type of testing.

10.4 Pile/Drilled Shaft Damage Assessment

Various test methods are available to assess the quality of the in-place deep foundation unit. These quality assurance tests need to be performed by qualified personnel and the results need to be analyzed and interpreted by experienced engineers in order to provide meaningful results.

10.4.1 Pile Integrity Testing

The use of low strain impact non-destructive testing (pulse-echo, etc.) has become common to determine cracks or breaks in driven piles caused by high

Appendix A

Determination of Design Side Resistance for Drilled Shafts Socketed in the Florida Limestone

DETERMINATION OF DESIGN SIDE SHEAR RESISTANCE FROM TEST DATA TO DESIGN PARAMETERS FOR DRILLED SHAFTS 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 C

Step by Step Design Procedure for the Analysis of Downdrag

Negative Shaft Resistance or Downdrag

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

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

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

STEP BY STEP DESIGN PROCEDURE FOR ANALYSIS OF DOWNDRAG LOADING

STEP 1
Establish the simplified soil profile and soil properties for computing settlement.

STEP 2
Determine the overburden pressure increase, Δp, versus depth due to the approach embankment fill. There are many methods and computer programs available for this purpose. An acceptable hand method is included at the end of this appendix.

STEP 3
Perform settlement computations for the soil layers along the embedded pile length.

a. Determine the consolidation parameters for each soil layer, preferably from laboratory consolidation test results.
b. Compute the settlement of each soil layer.

c. Compute the total settlement over the embedded pile length, i.e. the sum of the settlements from each soil layer and partial soil layers. Do not include soil settlements below the pile toe.

STEP 4

Determine the pile length that will experience negative shaft resistance. Negative shaft resistance occurs due to the settlement between soil and pile. The amount of settlement between soil and pile necessary to mobilize the negative shaft resistance is about ½ inch. Therefore, negative shaft resistance will occur on the pile shaft in each soil layer or portion of a soil layer with ½ inch more settlement than the settlement of the pile.

STEP 5

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

STEP 6

Calculate the ~~ultimate~~ nominal pile ~~capacity~~ resistance provided by the positive shaft resistance and the toe resistance, $R_{\cancel{ult}n}$. Positive shaft and toe resistances will develop below the depth where the relative pile-soil movements are less than ½ inch. The positive soil resistances can be calculated on the pile length remaining below the negative shaft resistance depth from Step 4 using an appropriate static analysis method for the soil type as described in this chapter.

STEP 7

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

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

STEP 8

Calculate the DOWNDRAG value for the Pile Data Table of the plans as

DOWNDRAG = $\cancel{Q_{dd}}$ R_{dd} + (Driving Resistance of soil contributing to $\cancel{Q_{dd}}R_{dd}$)

$\cancel{Rn}$ R_n = (Factored Design Load + Net Scour + Downdrag) / φ

Where: φ is the resistance factor taken from Table 3.1 of the Structures Design Guidelines.

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

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

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

Appendix F

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

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

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

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

In this method dynamic load tests are performed on test piles and all production piles. Test piles are driven first to determine production pile lengths, or in cases when the Contractor has ordered production piles in advance, to verify that the ordered length is adequate. With this method a resistance factor (ϕ) of 0.75 may be used in the computation of the required NBR. No driving criteria are required as the NBR, without exceeding the allowable stress limits, will be determined in the field by PDA and CAPWAP. CAPWAP analyses are required to confirm that the proper damping value, Jc, is used to estimate static resistance. In 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