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2027 Geotechnical Design Manual

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Introduction

Geotechnical Design Manual

PURPOSE

The purpose of the Geotechnical Design Manual (GDM) is to establish minimum standards that provide efficient design and reliable performance of geotechnical assets as part of the Department's vision to deliver a safe and reliable transportation network. The GDM outlines the requirements for conducting geotechnical field investigations, laboratory testing, and office-level design based on the results of field and laboratory work. It is the responsibility of the project geotechnical engineer to ensure that their design conforms with Department policies, procedures, standards, guidelines, the American Association of State Highway Transportation Officials (AASHTO) Design Specifications, and sound engineering practices.

AUTHORITY

Sections 20.23(4)(a) and 334.048(3), Florida Statutes.

SCOPE

The GDM is written to the Geotechnical Engineer of Record (GEOR) and the Geotechnical Foundation Design Engineer of Record (GFDEOR) for Conventional and Non-Conventional projects, respectively, unless specified otherwise. The principal users of this manual include District Materials and Research Offices (DMROs), District Construction Offices (DCOs), Resident Construction Offices (RCOs), Operations Centers, the State Materials Office (SMO), the State Construction Office (SCO), the Structures Design Office (SDO), Geotechnical Engineers, Specialty Engineers, and all others involved in geotechnical design and related construction activities.

PROCEDURE

The criteria in this manual represent requirements for the State Highway System which must be met for the design of Department projects unless approved Design Exceptions or Design Variations are obtained in accordance with the ***FDOT Design Manual (FDM).***

This manual provides the minimum requirements for field and laboratory work necessary to develop an acceptable basis for geotechnical design and addresses specific construction requirements as they relate to geotechnical work.

Special requirements for Non-Conventional Projects, e.g., Design-Build Projects and all Non-Design-Bid-Build Projects, will be shown in a "Modification for Non-Conventional Projects" box as shown in the following example:

Modification for Non-Conventional Projects:
Delete the previous paragraph and replace with: Provide a summary of the analysis and recommendations for the preferred foundation.

These boxes are located after a section, paragraph, or table which is to be modified. The requirements listed within these boxes are only applicable to Non-Conventional Projects.

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 acceptable scope of work outlined herein.

Note that this document is not an all-encompassing or comprehensive procedural manual, and it is not intended to replace geotechnical textbooks or engineering judgement. Methods of subsurface investigation, data analysis, and problem-solving are not discussed in detail in this manual. The lists of references at the end of each chapter represent only a few of the many sources of information that can provide engineers with the necessary details of investigation procedures, analysis, and problem-solving techniques. For clarification regarding the content of this manual, engineers may contact the District Geotechnical Engineer (DGE), the State Geotechnical Engineer (SGE) in Tallahassee, or the State Geotechnical Materials Engineer (SGME) in Gainesville.

DISTRIBUTION

The SDO maintains the Geotechnical Design Manual which is available through the Internet website at the following address: ***Geotechnical Office Webpage***.

ORGANIZATION

The GDM is a single volume document that is subdivided into five parts organized in a manner that follows the typical progress of events in roadway and bridge projects. Part I contains chapters one through seven and covers subsurface investigation procedures and guidelines for highway and related structures, in situ testing methods, laboratory tests, material classification and logging, and field instrumentation. Part II contains chapters eight and nine and covers analysis and design and requirements for the geotechnical report. Part III contains chapter ten covering load and integrity testing, and Part IV contains chapter eleven which addresses geotechnical considerations for non-conventional projects. Part V includes the Appendices created to provide clarity on design topics where local practice may differ from other states.

REVISIONS AND ADDITIONS

GDM users are encouraged to submit comments and suggestions for updates to the manual to the SDO. Suggestions will be reviewed and if necessary, coordinated with other offices affected by the proposed change. Immediate items warranting change will be made with the approval of the Director of Design, the State Structures Design Engineer

(SSDE), and the SGE in the form of a Design Bulletin.

Chapter 1 Terminology and Abbreviations

1.1 Terminology

AASHTO Code: In this manual, refers to the AASHTO LRFD Bridge Design Specifications, 10th Edition.

Allowable Stress Design (ASD): Design methodology where unfactored service loads are compared against material strength divided by a single factor of safety.

Atterberg Limits: Critical water content limits for fine-grained soils. Refer to Liquid Limit and Plastic Limit.

Bulk Specific Gravity: The ratio of the weight of a porous material to the weight of an equal volume of water. Used to determine the unit weight of the material.

Consistency: In this manual, used to describe the stiffness of fine-grained soils.

Coring/Core: Drilling method that extracts a solid, cylindrical sample of subsurface bedrock by means of a core barrel that is advanced through rock by the application of downward pressure during rotation.

D_{50}: The median particle size diameter where 50% of the soil is composed of particles smaller than that size and 50% of the soil is composed of particles larger than that size.

Department: The Florida Department of Transportation.

Downdrag: The downward movement (settlement) of a deep foundation element in response to drag load.

Drag Load (DR): The load applied to a deep foundation element due to settlement of the adjacent ground relative to the deep foundation element.

Dynamic Load Test: A type of field test used to determine the resistance of deep foundations using the concepts of wave mechanics.

Earthwork: In this manual, the activities related to excavation of soil, embankment fill, and roadway subgrade and base.

Earthwork Factors: Adjustment factors used to estimate earthwork quantities to account for soil and rock volume changes in density. Includes swell, shrinkage, and load factors.

Engineer of Record (EOR): Refer to the ***Standard Specifications for Road and Bridge Construction***.

Factored Bearing Resistance: The nominal resistance multiplied by a resistance factor. See Nominal Bearing Resistance.

Gap-Graded: Grading used to describe coarse-grained soils that do not contain one or more intermediate particles sizes. (e.g. a soil consisting of gravel and fine sand without medium and coarse sand sizes). See also Well-Graded and Poorly-Graded.

Geophysical Testing: Non-invasive nondestructive exploratory method used to map the subsurface.

Infiltration Rate: The maximum rate at which water can enter the soil from the surface under specified conditions.

Intermediate Geomaterial (IGM): A material that is transitional between soil and rock in terms of strength and compressibility, such as residual soils or very weak rock.

Invert Elevation: The elevation of the lowest inside point (bottom) of a pipe or conduit.

Liquid Limit (LL): The moisture content of a soil at the boundary between the liquid and plastic states.

Load and Resistance Factor Design (LRFD): Design methodology that accounts for uncertainty in the anticipated applied loads and resistances by independently applying load and resistance factors (as applicable).

Minimum Tip Elevation: The minimum required elevation a deep foundation element's tip must achieve. Sometimes referred to as minimum tip.

Modified Special Provision: Revisions to the Standard Specifications setting forth conditions varying from or additional to the Standard Specifications applicable to a specific project.

Monitoring Well: Also known as observation well; used to monitor groundwater levels or to provide ready access for sampling to detect groundwater contamination and consists of a perforated section of pipe or well point attached to a riser pipe, installed in a sand-filled borehole.

Nominal Bearing Resistance (NBR): The unfactored bearing resistance. See Factored Bearing Resistance.

Non-Redundant Foundation: A foundation in which the underperformance of one of the elements may cause underperformance of the entire deep foundation group.

Overconsolidation Ratio: The ratio of a soil's maximum past effective vertical stress (preconsolidation pressure) to its current effective vertical stress.

Percent Recovery (REC): The total length of rock samples successfully recovered from a borehole, divided by the total length of the core run drilled.

Percolation Rate: The rate at which the water moves through near surface soils. Common tests to determine this involve filling a hole in the soil to a prescribed water level and measuring the drop in water level over a set time.

Permeability: Also known as hydraulic conductivity—is a soil's capacity to allow fluids, primarily water, to pass through its interconnected pore spaces***.***

Piezometers: Instrumentation used to measure the amount of water pressure within the saturated pores of a specific zone of soil.

Plans: In this manual, Plans refer to the construction plans for a given project.

Plasticity Index (PI): Range of moisture contents over which the soil deforms plastically. Obtained by finding the difference between the Liquid Limit and Plastic Limit.

Plastic Limit (PL): The moisture content at the boundary between the plastic and semi-solid states.

Poorly-Graded: Grading used to describe coarse-grained soils containing particles that are about the same size. A soil of this type is sometimes described as being uniform. See also Well-Graded and Gap-Graded.

Preformed Pile Hole: A hole drilled prior to pile driving used as a penetration aid to achieve required minimum tip elevation.

Production Drilled Shaft/Auger-Cast-Pile/Micropile: All elements not requiring a load test.

Production Pile: For driven piles, all piles not designated as test piles.

Relative Density: Measure of the density of a soil in its loosest and densest state. In this manual, relative density refers to the degree of compactness of a coarse-grained soil.

Resilient (Elastic) Modulus: A measure of the elastic response (e.g. how well the soil is able to return to its original state after being stressed). A measure of the material's stiffness.

Rock Quality Designation (RQD): The sum of the lengths of successfully recovered sound rock core segments that are equal to or greater than 4 inches in length, divided by the total length of the core run drilled.

Settlement Platform/Plate: Settlement monitoring instrumentation consisting of a square wooden platform or steel plate placed on the existing ground surface prior to embankment construction. A reference rod and protecting pipe are attached to the platform. As fill operations progress, additional rods and pipes are added. See ***Figure 7-2***.

Shaft Inspection Device (SID): Equipment used to inspect the bottom cleanliness of drilled shafts prior to placement of concrete through the use of an inspection bell which houses a high resolution video camera. See ***Figure 10-6***.

Shear Strength: The maximum shear stress the soil structure can resist before failure.

Shrinkage Limit (SL): The maximum water content at which a reduction in water content will not cause a decrease in volume of the soil mass.

Sieve Analysis: A direct measurement of the particle size distribution of a soil determined by passing a sample through a series of wire screens with progressively smaller openings of known size and weighing the amount of material retained on each sieve.

Soundings: Method of exploration in which either a static or dynamic force is used to cause a rod tipped with a testing device to penetrate soils.

Specific Gravity (G_s): The ratio of the mass in air of a given volume of soil particles to the mass in air of an equal volume of gas free distilled water at a stated temperature (typically 68° F).

Specifications: Includes Standard Specifications, Supplemental Specifications. Special Provisions, Technical Special Provisions, and Developmental Specifications. Refer to the ***Standard Specifications for Road and Bridge Construction*** for definitions.

Splitting Tensile Strength: A mechanical property measuring a material's ability to resist tensile stresses.

Splitting Tensile Strength Test: Laboratory test where a compressive line load is continuously applied parallel to a sample core's axis until failure occurs. Used to determine the split tensile strength of rock and IGM.

SPT N-value: In Standard Penetration Testing, the uncorrected sum of the number of blows for the second and third 6-inch increments reported as blows per foot.

SPT N_{60}: SPT N-value corrected for the hammer energy efficiency.

SPT $N_{1\text{-}60}$: SPT N-value corrected for overburden and hammer energy efficiency.

Standard Penetration Test (SPT): Type of in situ test where a standard split barrel sampler is advanced into the soil by dropping a 140-pound safety or automatic hammer on the drill rod from a height of 30 inches and advanced a total of 18 inches in three, 6-inch increments. The number of blows required to advance the sampler for each 6-inch increment is recorded.

Standard Plans: In this manual, the applicable version of the Florida Department of Transportation's ***Standard Plans for Road and Bridge Construction***.

Static Load Test: A type of field test used to determine the load-deformation characteristics and ultimate resistance of foundations by applying a slow steady load. See ***Figure 10-1***.

Statnamic Load Tests: A type of field test used to determine the resistance of foundations using a larger wavelength load application as compared to dynamic testing. See ***Figure 10-2*** and ***Figure 10-3***.

Test Drilled Shaft/Auger-Cast-Pile/Micropile: Element designated for load testing.

Test Pile: For driven piles, a pile that is exploratory in nature and is longer, driven harder, and to a greater bearing resistance than other piles.

Test Pit/Trench: Excavations performed by hand, backhoe, or dozer with hand excavations often performed with posthole diggers or hand augers.

Unconfined Compression Test: Strength test consisting of subjecting a cylindrical sample to an axial load under no confinement until failure.

Unconfined Compressive Strength (q_u): Maximum axial stress a specimen can withstand under no confining pressure.

Undrained Shear Strength (s_u): The maximum shear stress a saturated soil structure can resist before failure without water escaping the soil.

Unit Weight: The weight of a sample divided by the total volume. Types include bulk (total), dry, saturated, and effective unit weight.

Vertical In Situ Permeameter: Field permeability test consisting of a conical probe pushed into the soil using a standard drill rig. A vertical injection port controls the outflow of water into the surrounding soil. The result is a mean coefficient of permeability at the depth to which the probe is advanced. Refer to ***FM 5-614.***

Well-Graded: Grading used to describe coarse-grained soils that contain a good representation of all particle sizes. See also Poorly-Graded and Gap-Graded.

1.2 Acronyms

AASHTO	American Association of State Highway and Transportation Officials
ACP	Auger Cast Pile(s)
APL	Approved Products List
AREMA	American Railway Engineering and Maintenance-of-Way Association
ASD	Allowable Stress Design
BDR	Bridge Development Report
BOE	Basis of Estimates
BSI	Bridge Software Institute
CFSP	Correction Factor for Sheet Pile Proximity
CPAM	Construction Project Administration Manual
CPT	Cone Penetration Test
CSI	Cost Saving Initiative
CSL	Crosshole Sonic Logging
DCO	District Construction Office
DEP	Department of Environmental Protection
DGE	District Geotechnical Engineer
DMT	Dilatometer Test
DMRO	District Materials and Research Office
EDC	Embedded Data Collector
EOR	Engineer of Record
EPS	Expanded Polystyrene
FDL	Factored Design Load
FDM	FDOT Design Manual
FDOT	Florida Department of Transportation
FHWA	Federal Highway Administration
FM	Florida Method of Test
GDM	Geotechnical Design Manual
GEOR	Geotechnical Engineer of Record
GFDEOR	Geotechnical Foundation Design Engineer of Record
GPC	Goble Pile Check
GPI	Grout Pressure Index
GPR	Ground Penetrating Radar

GPS	Global Positioning System
GRS	Geosynthetic Reinforced Soil
GSI	Geological Strength Index
HDPE	High-Density Polyethylene
IGM	Intermediate Geomaterial
LBR	Limerock Bearing Ratio
LRFD	Load and Resistance Factor Design
MQN	Match Quality Number
MSE	Mechanically Stabilized Earth
MSP	Modified Special Provision
MVDS	Microwave Vehicle Detection Systems
MWD	Measuring While Drilling
NBR	Nominal Bearing Resistance
NCHRP	National Cooperative Highway Research Program
N_GAPA	Nguyen_Goble Automated Pile Analysis
NRCS	Natural Resources Conservation Service
O.C.	Organic Content
OCR	Overconsolidation Ratio
PDA	Pile Driving Analyzer
PMT	Pressuremeter Test
PSI	Pounds per square inch
PVC	Polyvinyl Chloride
RCO	Resident Construction Office
REC	Recovery
RETA	Rotating Erosion Test Apparatus
RFC	Released for Construction
RFP	Request for Proposal
RMR	Rock Mass Rating
RQD	Rock Quality Designation
SCO	State Construction Office
SDG	Structures Design Guidelines
SDM	Structures Detailing Manual
SDO	Structures Design Office

SGE	State Geotechnical Engineer
SGME	State Geotechnical Materials Engineer
SID	Shaft Inspection Device
SMO	State Materials Office
SPI	Standard Plans Instructions
SPT	Standard Penetration Test
SSDE	State Structures Design Engineer
TCM	Tip Capacity Multiplier
TEB	Toll Equipment Building
TSF	Tons per square feet
TSP	Technical Special Provision
USCGS	United States Coast and Geodetic Survey
USCS	Unified Soil Classification System
USDA	United States Department of Agriculture
USGS	United States Geological Survey
VIP	Vertical In Situ Permeameter

Chapter 2 Subsurface Investigation Procedures

Given the varying complexity of projects and soil conditions, a rigid format to be followed in conducting each subsurface investigation is not reasonable. However, the steps summarized in this and subsequent chapters are considered minimum requirements, unless approved otherwise by the SGE, the SGME or the DGE, depending on the nature of the variation and the project category.

This chapter discusses review of existing data and commonly used methods for performing field explorations. Guidelines for minimum investigations for various types of projects are presented in ***Chapter 3***. Field and laboratory test methods are discussed in ***Chapter 4*** and ***Chapter 5***, respectively. Refer to ASTM D5434.

2.1 Review of Project Requirements

Complete a thorough review of the project requirements including necessary information such as the project location, alignment, structure locations, structure loads, approximate bridge span lengths and pier locations, and cut and fill area locations. The typical section, plan and profile sheets, and cross sections with a template for the proposed roadway showing cuts and fills should be obtained to aid in planning the investigation and minimize expensive and time-consuming supplemental fieldwork.

2.2 Review of Available Data

Collect all relevant information available on the project site to aid in understanding the geology, geography, and topography of the area and assist in laying out the field explorations and locating potential problems. Contact the DGE for assistance in obtaining sources of this available data. Existing data may be available from the following sources:

1. Topographic Maps.
2. Arial Photographs.
3. Geological Maps and Reports.
4. Natural Resources Conservation Service Surveys.
5. Potentiometric Surface Map (see ***Figure 2-1*** for an example).
6. Adjacent Projects.
    a. Review the ***Public Soil Boring Viewer*** for previous borings in the area.
7. Other relevant sources.

2.3 Field Reconnaissance

Following the review of the existing data, visit the project site to gain first-hand knowledge of field conditions and correlate this information with previous data. The form shown in ***Figure 2-2*** indicates the type of information to look for. At a minimum, complete the following during the field reconnaissance:

1. Inspect nearby structures to ascertain their foundation performance and potential to damage from vibration or settlement from foundation installation.

Check the structures' usage to determine the impact the foundation installation may have (i.e. a surgical unit, printing company, etc.)

2. On water crossings, inspect banks for scour and inspect the streambed for evidence of soil deposits not previously indicated.
3. Note any feature that may affect the boring program, such as accessibility, structures, overhead utilities, signs of buried utilities, or property restrictions.
4. Note any feature that may assist in the engineering analysis, such as the angle of any existing slopes and the stability of any open excavations or trenches.
5. Note any drainage features, including signs of seasonal water tables.
6. Note any features that may need additional borings or probing such as areas containing layers of muck or wetlands.

2.4 Field Exploration Methods

Obtain access and utility clearances prior to beginning field explorations. For drilling operations involving Measuring While Drilling (MWD), refer to ***FM 5-625***. Common methods of field exploration include but are not limited to:

1. Test pits and trenches for delineation of muck or deleterious material.
2. Borings (Auger Borings, Hollow-Stem Auger Borings, and Wash Borings). Backfill all boreholes in accordance with applicable Department of Environmental Protection (DEP) and Water Management District regulations. In many cases this will require full-depth grouting.
3. Coring. Refer to ***2.4.1.4*** for details on the rate of advancement.
4. Soundings. See ***Chapter 4*** for details of the cone penetrometer.
5. Geophysical Methods (Seismic Refraction and Reflection, Resistivity, Ground Penetrating Radar [GPR], and 2D and 3D Seismic Waveform). Geophysical explorations must be accompanied by conventional borings. Refer to ASTMs D6429, D5777, D4428, G57, D6431, and D6432 and U.S. Army Corps of Engineers ***Engineering Manual EM-1110-1-1802***.

2.4.1 Soil Sampling

Properly preserve all samples and carefully transport to the laboratory such that sample properties and integrity are maintained. Refer to ASTM D4220.

2.4.1.1 Bulk Samples

Obtain bag bulk samples for classification, moisture-density, Limerock Bearing Ratio (LBR), corrosivity tests, and other tests where disturbed samples are permitted. Ensure a portion of each sample is placed in a sealed container for moisture content determination. A sufficiently large sample shall be provided to perform the required testing.

2.4.1.2 Split-Barrel

Immediately examine, log, and place split barrel samples in sample jars for storage. These are disturbed samples and are not to be utilized for strength or consolidation testing. Refer to ASTM D1586 (AASHTO T 206).

2.4.1.3 Shelby Tube

Shelby tube samples are suitable for strength and consolidation testing provided they are undisturbed. Ensure samples are immediately sealed upon withdrawal and have sufficient cohesion to remain in the tube during withdrawal. Refer to ASTM D1587 (AASHTO T 207).

Use a piston type sampler for materials too weak to be retained by a Shelby tube. Limit the use of floating piston samplers to stiff or hard cohesive materials.

2.4.1.4 Rock Core Sampling

Obtain rock cores in accordance with ASTM D2113 *Standard Practice for Rock Core Drilling and Sampling of Rock for Site Exploration* (AASHTO T 225) using a double or triple wall core barrel equipped with diamond or tungsten-carbide tipped bits. The use of single tube core barrels is not allowed. For core samples smaller than 3.5 inches, use only triple tube core barrels. For core samples larger than 3.5 inches, the use of double or triple tube core barrel is permitted. Refer to ASTM D5079 for practices of preserving and transporting rock core samples.

Note: Face discharge bits generally provide better return in Florida limestone.

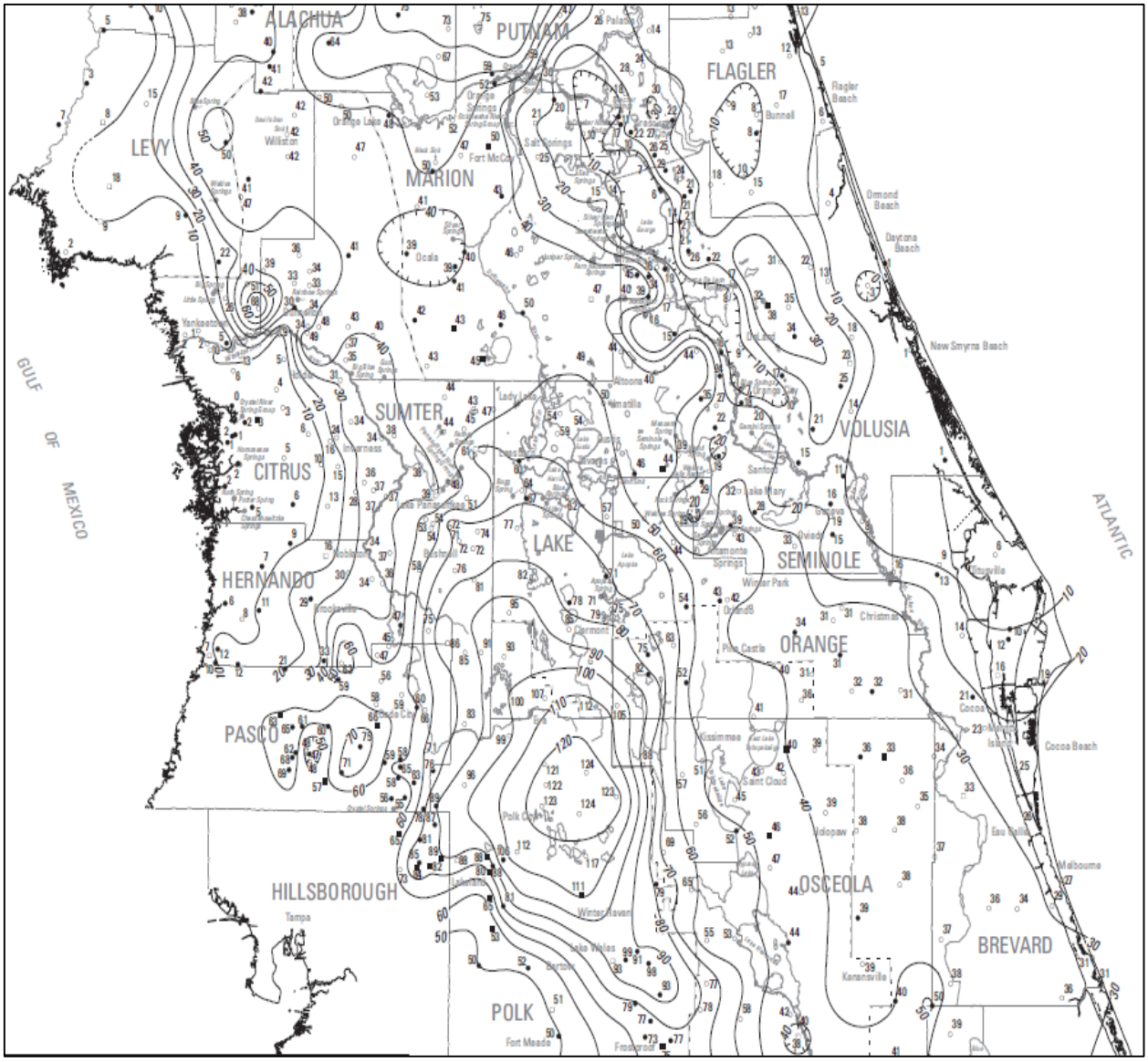
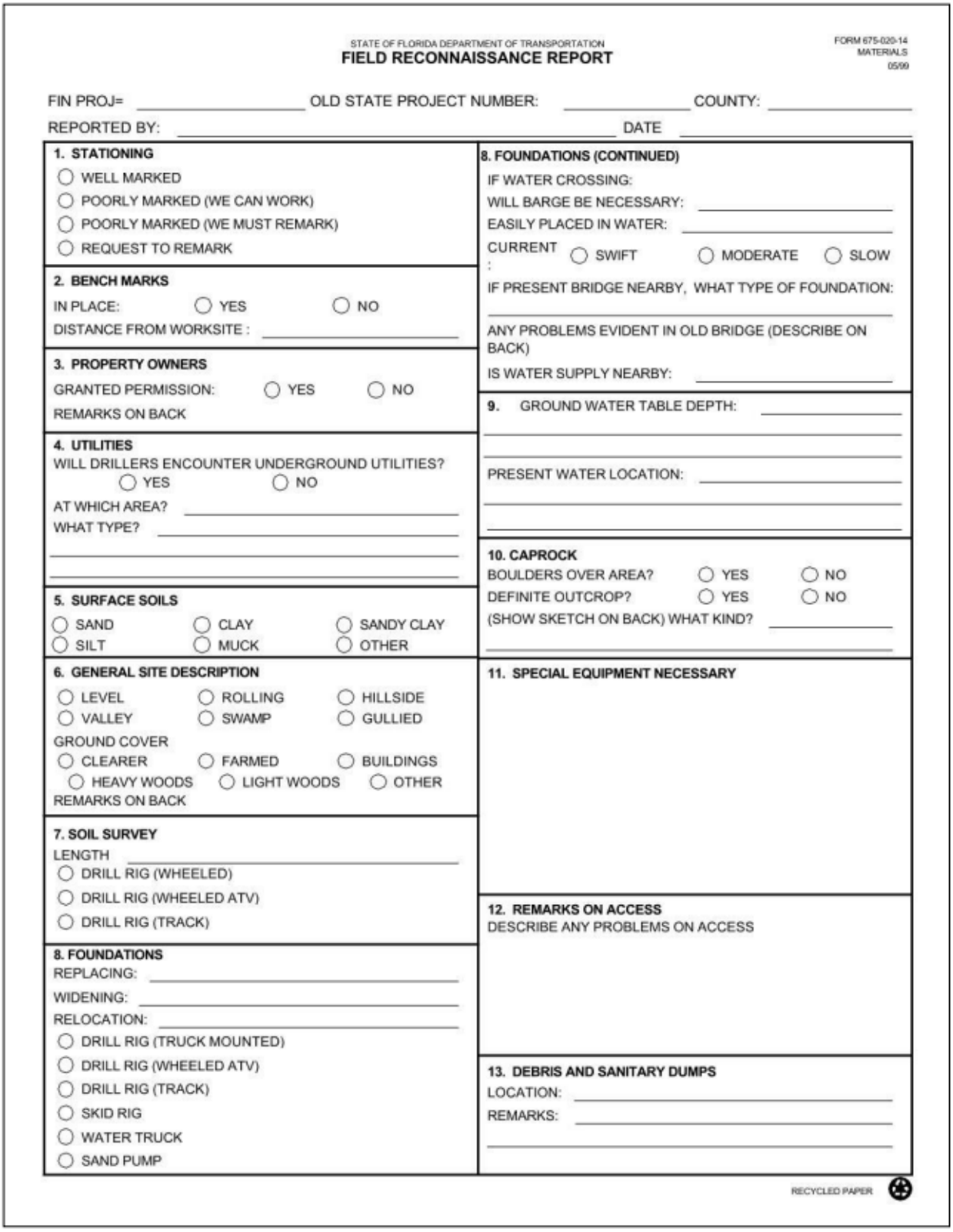


Figure 2-1 Potentiometric Surface Map of the Upper Floridan Aquifer

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



STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION
FIELD RECONNAISSANCE REPORT

FORM 675-020-14
MATERIALS
05/99

FIN PROJ= _____ OLD STATE PROJECT NUMBER: _____ COUNTY: _____

REPORTED BY: _____ DATE _____

1. STATIONING
- ○ WELL MARKED
- ○ POORLY MARKED (WE CAN WORK)
- ○ POORLY MARKED (WE MUST REMARK)
- ○ REQUEST TO REMARK

2. BENCH MARKS
IN PLACE: ○ YES ○ NO
DISTANCE FROM WORKSITE : _____

3. PROPERTY OWNERS
GRANTED PERMISSION: ○ YES ○ NO
REMARKS ON BACK

4. UTILITIES
WILL DRILLERS ENCOUNTER UNDERGROUND UTILITIES?
○ YES ○ NO
AT WHICH AREA? _____
WHAT TYPE? _____

5. SURFACE SOILS
○ SAND ○ CLAY ○ SANDY CLAY
○ SILT ○ MUCK ○ OTHER

6. GENERAL SITE DESCRIPTION
○ LEVEL ○ ROLLING ○ HILLSIDE
○ VALLEY ○ SWAMP ○ GULLIED
GROUND COVER
○ CLEARER ○ FARMED ○ BUILDINGS
○ HEAVY WOODS ○ LIGHT WOODS ○ OTHER
REMARKS ON BACK

7. SOIL SURVEY
LENGTH _____
- ○ DRILL RIG (WHEELED)
- ○ DRILL RIG (WHEELED ATV)
- ○ DRILL RIG (TRACK)

8. FOUNDATIONS
REPLACING: _____
WIDENING: _____
RELOCATION: _____
- ○ DRILL RIG (TRUCK MOUNTED)
- ○ DRILL RIG (WHEELED ATV)
- ○ DRILL RIG (TRACK)
- ○ SKID RIG
- ○ WATER TRUCK
- ○ SAND PUMP

8. FOUNDATIONS (CONTINUED)
IF WATER CROSSING:
WILL BARGE BE NECESSARY: _____
EASILY PLACED IN WATER: _____
CURRENT : ○ SWIFT ○ MODERATE ○ SLOW
IF PRESENT BRIDGE NEARBY, WHAT TYPE OF FOUNDATION: _____
ANY PROBLEMS EVIDENT IN OLD BRIDGE (DESCRIBE ON BACK)
IS WATER SUPPLY NEARBY: _____

9. GROUND WATER TABLE DEPTH: _____
PRESENT WATER LOCATION: _____

10. CAPROCK
BOULDERS OVER AREA? ○ YES ○ NO
DEFINITE OUTCROP? ○ YES ○ NO
(SHOW SKETCH ON BACK) WHAT KIND? _____

11. SPECIAL EQUIPMENT NECESSARY

12. REMARKS ON ACCESS
DESCRIBE ANY PROBLEMS ON ACCESS

13. DEBRIS AND SANITARY DUMPS
LOCATION: _____
REMARKS: _____

RECYCLED PAPER

Figure 2-2 Field Reconnaissance Report

2.5 References

2-1. AASHTO, Manual on Subsurface Investigations, Washington DC, 1988.

2-2. Cheney, R. S., Chassie, R. G., Soils and Foundations Workshop Manual – Second Edition, FHWA HI-88-009, 1993.

2-3. Department of Army, ***Engineering Manual 1110-1-1802 – Geophysical Exploration for Engineering and Environmental Investigations***, U.S. Army Corps of Engineers, 1995.

2-4. Department of the Navy, ***NAVFAC DM-7.1 – Soil Mechanics***, Naval Facilities Engineering Command, 1982.

2-5. Fang, H.Y., Foundation Engineering Handbook Second Edition, Van Nostrand Reinhold Company, New York, 1990.

2-6. Hannigan, P.J., Rausche, F., Likins, G.E., Robinson, B.R. Becker, M.L., Design and Construction of Driven Pile Foundations ***Volume I*** and ***Volume II***, FHWA-NHI-16-009, 2016.

2-7. Mayne, P., Christopher, B., Dejong, J., ***Subsurface Investigations – Geotechnical Site Characterization***, FHWA-NHI-01-031, 2002.

2-8. National Cooperative Highway Research Program, ***Recommended Guidelines for Sealing Geotechnical Exploratory Holes***, NCHRP Report 378, 1995.

2.6 Specifications and Standards

Subject	ASTM	AASHTO
Standard Practice for Soil Investigation and Sampling by Auger Borings	D1452	-
Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils	D1586	T 206
Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes	D1587	T 207
Standard Practice for Diamond Core Drilling for Site Investigation	D2113	T 225
Standard Practices for Preserving and Transporting Soil Samples	D4220	-
Standard Test Methods for Crosshole Seismic Testing of Soil and Rock	D4428	-
Standard Practices for Preserving and Transporting Rock Core Samples	D5079	-
Standard Guide for Field Logging of Subsurface Explorations of Soil and Rock	D5434	-

Subject	ASTM	AASHTO
Standard Guide for Using the Seismic Refraction Method for Subsurface Investigation	D5777	-
Standard Practice for Using Hollow-Stem Augers for Geotechnical Exploration and Soil Sampling	D6151	-
Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method	G57	-
Standard Guide for Selecting Surface Geophysical Methods	D6429	-
Standard Guide for Using the Direct Current Resistivity Method for Subsurface Investigation	D6431	-
Standard Guide for Using the Surface Ground Penetrating Radar Method for Subsurface Investigation	D6432	-

Chapter 3 Subsurface Investigations for Highways and Related Structures

Perform a subsurface investigation for all new structures, roadway construction, widenings, extensions, trails, and rehabilitation locations as directed by the DGE or project scope.

This chapter presents the minimum requirements 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.

As a minimum, the investigation should provide sufficient data to recommend the most efficient design.

Comprehensive subsurface investigation programs may include both conventional borings and other specialized field investigatory or testing methods.

Modification for Non-Conventional Projects:
Delete the first paragraph and replace with the following: Perform a subsurface investigation for all new structures, roadway construction, widenings, extensions, trails, and rehabilitation locations as outlined herein, except as otherwise described in the Request for Proposal (RFP) or Contract Documents.

3.1 General Requirements

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

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 or Contract Documents:

1. Estimate preliminary exploration depths from data obtained during field reconnaissance, existing data, local geology, and local experience. The borings shall 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. Extend all borings below the estimated scour depths.
3. Provide a unique identification number for each boring, sounding, and test pit for easy reference.
4. Determine the horizontal and vertical location for each boring, sounding, and test pit. Reference offshore borings to mean sea level with the aid of a tide gauge.

Note: Ensure the proper datum (1929 datum or the 1988 datum) is 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.
6. Obtain a sufficient number of samples suitable for the types of testing intended within each layer of material.
7. Record the water table observation within each boring or test pit after sufficient time (typically 24 hours) has elapsed for the water table to stabilize. Record other groundwater observations (signs of seasonal high, artesian pressure, etc.)
8. Unless serving as an observation well, backfill or grout each borehole, sounding, and test pit according to applicable environmental guidelines (Reference ***3-6***).

3.2 Guidelines for Minimum Explorations

The guidelines herein are minimum requirements for the exploration and testing anticipated for most projects and must be adapted to the specific requirements of each individual project. Consult the DGE 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 DGE and verify that the Federal Highway Administration (FHWA) minimum criteria are met (Reference ***3-3***). The guidelines below consider the use of Standard Penetration Testing (SPT), auger borings, and test pits/trenches as applicable.

Commentary: While SPT borings are the most common type of exploration, it may be deemed 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 guidelines herein are the minimum requirements for Standard Penetration Testing (SPT) and auger borings, as applicable, 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

Complete a soil survey exploration along the proposed roadway alignment to define the subsurface materials for use in the design of the pavement section. Define the limits of unsuitable materials and any remedial measures to be taken and determine the stability of cut or fill slopes.

The 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 to 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. Place at least one boring 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. Locate additional borings as necessary to define the limits of any undesirable materials or to better define soil stratification.

b. In areas of variable soil conditions, locate additional borings at each interval considering the following criteria:

    1) For interstate highways, place three borings at each interval, one within the median and one within each roadway.

    2) For four lane roadways, place two borings at each interval, one within each roadway.

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

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

e. In all cases, obtain at least three samples per mile or three per project, whichever is greater, for each stratum encountered. Obtain each of the samples representing a particular stratum from a different location, with sampling locations spread out over each mile. Ensure samples are of adequate size for classification and moisture content testing.

f. For new construction, obtain three 100-pound samples per mile per stratum or five per project, whichever is greater, of all materials within four feet below the proposed base elevation and considered ‘Select’ in accordance with ***Standard Plans*** Index 120-001. Deliver the samples to the SMO in Gainesville for Resilient Modulus (M_R) testing. Obtain 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 for M_R testing when fill below paved areas will be required.

g. Obtain corrosion series samples (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. In areas of little or no grade change, extend borings a minimum of five feet below grade, drainage pipe, or culvert invert level whichever is deeper. For projects with proposed buried storm sewer systems, extend

one boring 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, extend one boring 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 SPTs, depending on the specific project requirements and its location.

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

k. In areas of fill, extend borings to firm material or to a depth of twice the embankment height, whichever occurs first. Obtain bag, SPT, and undisturbed samples 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.

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 following general criteria should provide sufficient information about the subsurface materials to permit design of the structure foundations and related geotechnical construction on most projects; however, ensure that appropriate explorations are carried out for each specific project.

All structure borings must include 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. See Reference ***3-3***.

The actual elevation and location of each boring and sounding including the Station, Offset, Latitude, and Longitude, must 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. Ensure the minimum frequency of Bridge Foundation Borings is met. Increase the boring frequency for highly variable sites. For straddle piers, consider each column as a separate pier:

a. Spread Footings –

   i. Footings < 70 feet wide - minimum one boring per footing.

   ii. Footings ≥ 70 feet wide - minimum two borings per footing.

b. Driven Piles –

   i. For all bridges without test piles ensure a minimum one boring confirms the bearing materials are within 50 feet of every pile.

   ii. For bridges with test piles and spans ≥ 60 feet:

   - Bents/pier foundations (pile groups) < 70 feet wide:
     - A minimum of one boring per bent/pier foundation per structure within 25 feet of each bent/pier footing.
   - Bents/pier foundations (pile groups) ≥ 70 feet wide:
     - A minimum of two evenly spaced borings within 25 feet of each footing for each bent/pier foundation per structure.

   iii. For bridges with test piles and spans < 60 feet:

   - Bents/pier foundations (pile groups) < 70 feet wide:
     - A minimum of one boring within 25 feet of every other bent/pier foundation per structure.
   - Bents/pier foundations (pile groups) ≥ 70 feet wide:
     - A minimum of 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 – A minimum of one boring per bent/pier foundation in consistent soil conditions. In variable soil conditions, ensure a minimum of one boring is within 20 feet of each shaft.

d. Nonredundant Drilled Shafts – A minimum of one boring per shaft (see Item No. 13 in ***3.2.2.1***).

e. Auger Cast Piles (ACP) –

   i. Bents/pier foundations < 70 feet wide – A minimum of one boring per bent/pier per structure within 25 feet of each bent/pier footing.

   ii. Bents/pier foundations ≥ 70 feet wide – A minimum of two evenly spaced borings per bent/pier foundation per structure, with at least one boring within 25 feet of each end of each bent/pier footing.

   iii. All bridges with ACP foundations require static load tests.

Perform a minimum of one boring within five (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 is to 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 DGE, may be performed to provide sufficient information for the structural engineer to complete the Bridge Development Report (BDR) 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 or altered, perform a Phase I Investigation including borings spaced to provide sufficient information for the structural engineer to complete the Bridge Development Report (BDR) 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 as follows:

      i. 2B where L< 2B,

      ii. 5B where L > 5B

      iii. Interpolate the 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. Continue borings for driven pile foundations tipped in soil until all unsuitable foundation materials have been penetrated and the predicted stress from the equivalent footing loading is less than 10% of the original overburden pressure (see ***Figure 3-1***). For pile foundations tipped in rock (with core samples with $q_u \geq 550$ pounds per square inch [psi] or N-value = 100), continue borings a minimum of 10 feet below the foundation tip elevations. For piles tipped in weaker materials, continue borings a minimum of 20 feet below the foundation tip elevations.

      Borings for driven concrete piles must be deep enough to satisfy the

maximum pile driving resistance outlined in ***Structures Design Guidelines (SDG)*** Table 3.5.13-1. Coordination with the drilling crew during field operations will be required. Use FB-Deep software to determine this depth.

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. Continue borings for rock socketed drilled shafts through competent materials for at least three shaft diameters below the expected shaft tip elevation (see Items No. 6 and 7 in ***3.2.2.1***). For non-rock socketed drilled shafts, continue borings through competent materials for at least two times the width of the shaft group below the expected shaft tip elevation. Scour and lateral requirements must be satisfied. For nonredundant drilled shafts see additional requirements below.

d. Continue borings for rock socketed ACP through competent materials to a minimum of 10 feet below the expected pile tip elevation (see Items No. 6 and 7 in ***3.2.2.1***). For non-rock socketed ACP, continue borings 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 SPT, obtain split-spoon samples at a maximum interval of 2.5 feet and at the top of each stratum. Continuous SPT sampling in accordance with ASTM D1586 is required in the top 15 feet unless the material is unacceptable for shallow foundations.

5. When cohesive soils are encountered, obtain undisturbed samples at five-foot intervals in at least one boring. Obtain undisturbed samples from more than one boring where possible.

6. When rock is encountered during the preliminary site investigation typically associated with preparation of the BDR, make successive core runs with the objective of obtaining the best possible core recovery. **Perform SPTs between core runs, typically at five-foot intervals.**

7. When rock is encountered on projects where a rock-socketed drilled-in-place deep foundations such as Drilled Shafts, ACP, or Micropiles has been selected for bridge support, the final subsurface investigation must include a full depth SPT boring to at least three shaft diameters below the anticipated tip elevation, with an adjacent borehole for coring (within 10 feet) at each Pier or Bent, as a minimum. At each Pier/Bent, perform the SPT boring first to delineate the zones where rock coring is required, followed by rock coring of the intervals of interest.

8. In addition to the requirements of Step 7, for bridges (including pedestrian bridges) to be supported by nonredundant drilled shaft foundations (see ***8.2.3***), perform a minimum of one SPT boring at each drilled shaft location during the design phase.

9. In situ vane, pressuremeter, or dilatometer tests (see ***Chapter 4***) are

recommended where soft clays are encountered.

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

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

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

13. For nonredundant drilled shafts, the minimum number of borings required to be evenly spaced at each nonredundant drilled shaft location will be dependent on the shaft size as follows:

Table 3-1 Minimum Number of Borings Required for Non-Redundant Drilled Shafts

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

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

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

Core the limestone load bearing strata and test the core samples recovered at all locations along the shaft. Extend borings to not less than three shaft

diameters below the proposed/final shaft tip elevation or to the depth required above in item 3, whichever is deeper. Pilot holes are required as necessary during construction in cases where the original boring depth is insufficient, where shafts are lengthened, or shaft locations are modified. Locate borings by survey and perform within one (1) foot of the shaft location. If access during the design phase limits the ability to accomplish these borings this close to the drilled shaft locations, perform a preliminary boring no farther than 60 feet from the shaft and include Plan notes to require the pilot holes to be taken during construction, unless otherwise authorized by the DGE. **However, every effort must be made to perform these borings and test the cores during the design phase in lieu of the need for pilot holes and rock core testing during construction.**

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

3.2.2.2 Approach Embankments

1. A minimum of one boring taken at the point of highest fill; the borings taken for the bridge abutment will usually satisfy this purpose.

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

2. Extend borings 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, continue borings until the superimposed stress is less than 10% of the original overburden pressure (see ***Figure 3-2***).

3. Sample and perform in situ testing in accordance with ASTM D1586.

3.2.2.3 Retaining Walls

1. At all wall locations, perform borings at a maximum interval of one per 150 feet of the wall, as close to the wall alignment as possible. Extend borings 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, extend borings below the lower adjacent ground surface to a minimum of twice the wall height or at least 10 feet into competent rock.

2. Sample and perform in situ testing in accordance with ASTM D1586.
3. When existing Mechanically Stabilized Earth (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 either direct shear tests on bulk samples prepared at 95% of the maximum dry density as determined by ***FM 1-T 180*** or an indirect method (see ***Appendix B***) as approved by the DGE.

3.2.2.4 Noise Walls

1. Perform Noise Wall borings 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. Sample and perform in situ testing in accordance with ASTM D1586.

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. Perform borings at proposed locations of box culverts. Trenches or hand auger borings may suffice for smaller structures.
2. For box culverts, extend borings a minimum of 15 feet below the bottom of the culvert or until firm material is encountered, whichever is deeper.
3. For smaller structures, extend borings or trenches at least five (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, test material from each stratum above the invert elevation and any standing water. For drainage systems parallel to roadway alignments, perform tests at 1,500 feet (or smaller) intervals along the alignment.

3.2.2.7 High Mast Lighting and Overhead Sign Structures

1. Perform one boring at each designated location ensuring each shaft is within 20 feet of a boring.
2. Borings must reflect 40 feet of embedment into suitable soil or 10 feet into competent rock with 15 feet minimum total depth. Deeper borings may be required for cases with large torsional loads.

3. Sample and perform in situ testing in accordance with ASTM D1586.
4. Corrosion testing may be omitted when the structure is designed for the most aggressive conditions, unless otherwise directed by the DGE.

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

3.2.2.8 Mast Arm Assemblies and MVDS Poles and Strain Poles

For Mast Arm Assemblies and Microwave Vehicle Detection Systems (MVDS) Poles and Strain Poles the following applies:

1. Perform one boring to 25 feet into suitable soil or 10 feet into competent rock with 15 feet minimum total depth (Auger, SPT, or Cone Penetration Testing [CPT]) 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 Plans***. 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 DGE.

Modification for Non-Conventional Projects:
Delete 4 and insert the following: 4. 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. Perform one boring at each designated location. 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. Extend borings 20 feet into suitable soil below prevailing grade or 25 feet below the top of embankment. 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 DGE, the foundation depth is expected to be deeper.
3. Sample and perform in situ testing in accordance with ASTM D1586.
4. Corrosion testing may be omitted and the structure designed for the most

aggressive conditions unless otherwise required by the DGE.

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

3.2.2.10 Cable Barriers

1. Perform one boring to 35 feet into suitable soil or 15 feet into competent rock (Auger, SPT, or CPT) in the area of each designated location for cable barrier end anchorages.
2. For Standard Cable Barrier End Anchorages, verify that the soil strength properties at the foundation locations meet or exceed the soil strength properties assumed in ***Developmental Specification*** DEV540-968HTCB. A site-specific design must be performed for those sites having weaker strength properties.
3. In addition to the soil borings at the end anchorages, perform a geotechnical assessment of the soils along the cable barrier alignment between the anchor locations. This may be done using any of the normal preliminary investigation methods (topographic maps, aerial photos, geological maps and reports, etc.) as well as original Roadway Plans. As a minimum, a visual assessment in the field is required. Investigate areas that appear to be wetlands, have high organic content, or that are saturated for extended periods by taking site specific borings.
4. Corrosion testing may be omitted and the structure designed for the most aggressive conditions unless otherwise required by the DGE.

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

3.2.2.11 Tunnels

Due to the greatly varying conditions under which tunnels are constructed, investigation criteria for tunnels will be established by the DGE for each project on an individual basis.

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

3.2.2.12 Other Structures

Contact the DGE 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. Obtain samples to permit classification, moisture, compaction, permeability test, LBR, Resilient Modulus (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)

Perform two auger borings (SPT borings with continuous sampling may be substituted) per acre of pond area, with a minimum depth of five feet below the deepest elevation of the pond, or until a confining layer is encountered or local Water Management District criteria are satisfied. Perform a minimum of two field permeability tests per pond or a minimum of one test per acre, whichever is greater.

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

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

Perform one auger boring (SPT borings with continuous sampling may be substituted) per acre of pond, with a minimum depth of five feet below the deepest elevation of the pond, and one SPT boring per acre of pond, with a minimum depth of two times the proposed water height or until local Water Management District criteria are satisfied. Perform a minimum of two field permeability tests per pond or a minimum of one test per acre, whichever is greater.

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

3.2.6 Exfiltration Trenches/French Drains

Perform one auger boring (SPT borings with continuous sampling may be substituted) per 1,000 feet of continuous exfiltration trench, with a minimum depth of 20 feet. Perform a minimum of one field permeability test per 1,000 feet of continuous exfiltration trench.

If rock is to be excavated or expected to be encountered, perform sufficient SPT borings to estimate the depth, volume, and hardness of the rock to be encountered.

3.2.7 Roadside Tolling Cabinets

Perform one SPT boring per site. The boring must be located within 20 feet of any proposed slab and advanced to a minimum depth of 20 feet below the slab bottom elevation. When an SPT boring of the required depth is performed for the gantry within

20 feet of the slab, a separate boring is not required.

3.2.8 Toll Equipment Building Foundation

Perform one SPT boring per toll equipment building (TEB). The boring must be located within 20 feet of any proposed TEB and advanced to a minimum depth of 20 feet below the bottom-of-footing elevation. When an SPT boring of the required depth is performed for the gantry within 20 feet of the TEB, a separate boring is not required.

3.2.9 Toll Gantry Foundation

Perform a subsurface investigation in accordance with the requirements of ***3.2.2.7*** to a minimum depth of 10 feet below the design pile/shaft tip elevation.

Modification for Non-Conventional Projects:
Submit a foundation installation plan and a certification package for all gantry foundations following the requirements of ***FDOT's Design-Build Specifications*** SP4550000DB.

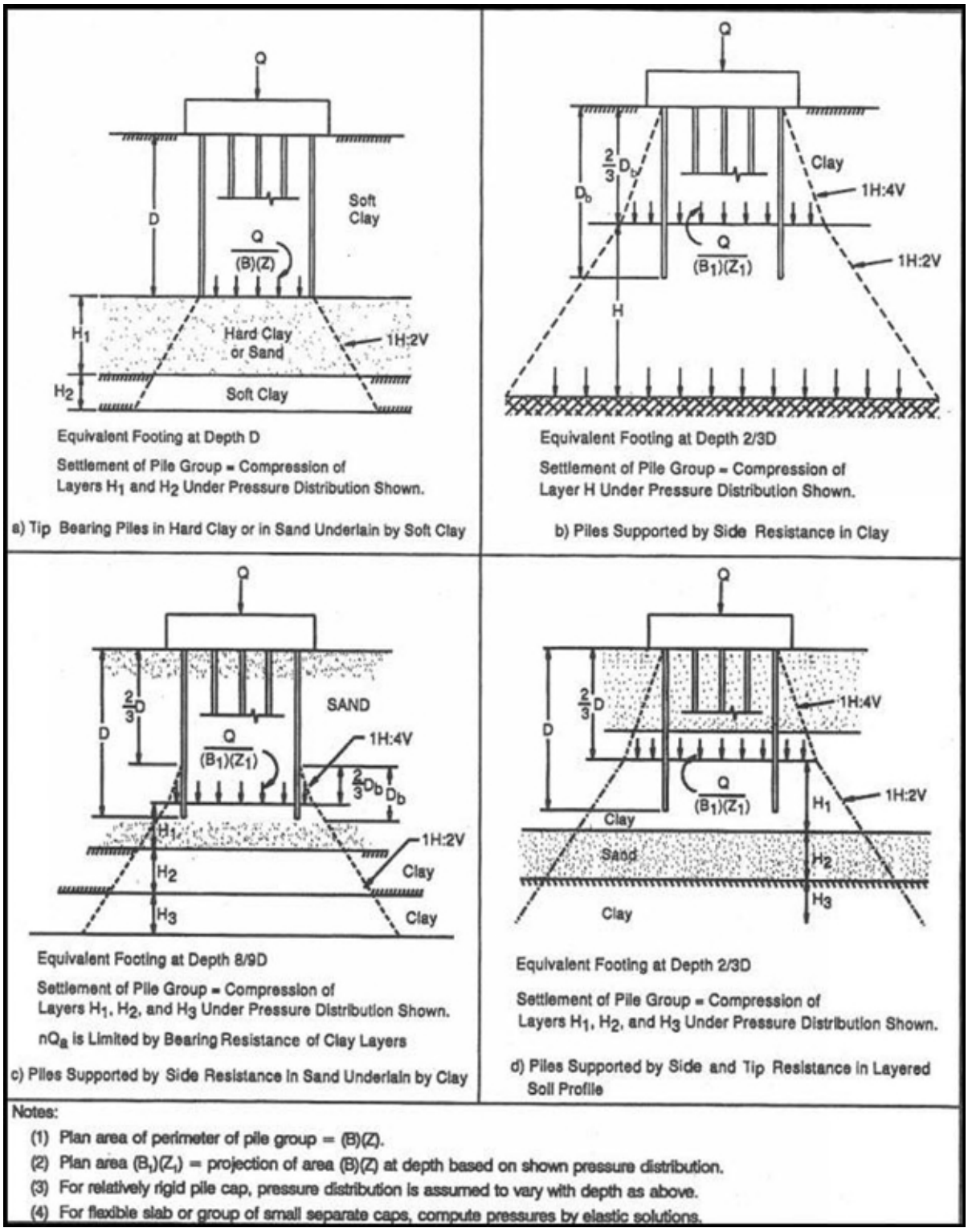


Figure 3-1 Stress Distribution Below Equivalent Footing for Pile Group (AASHTO 2024)

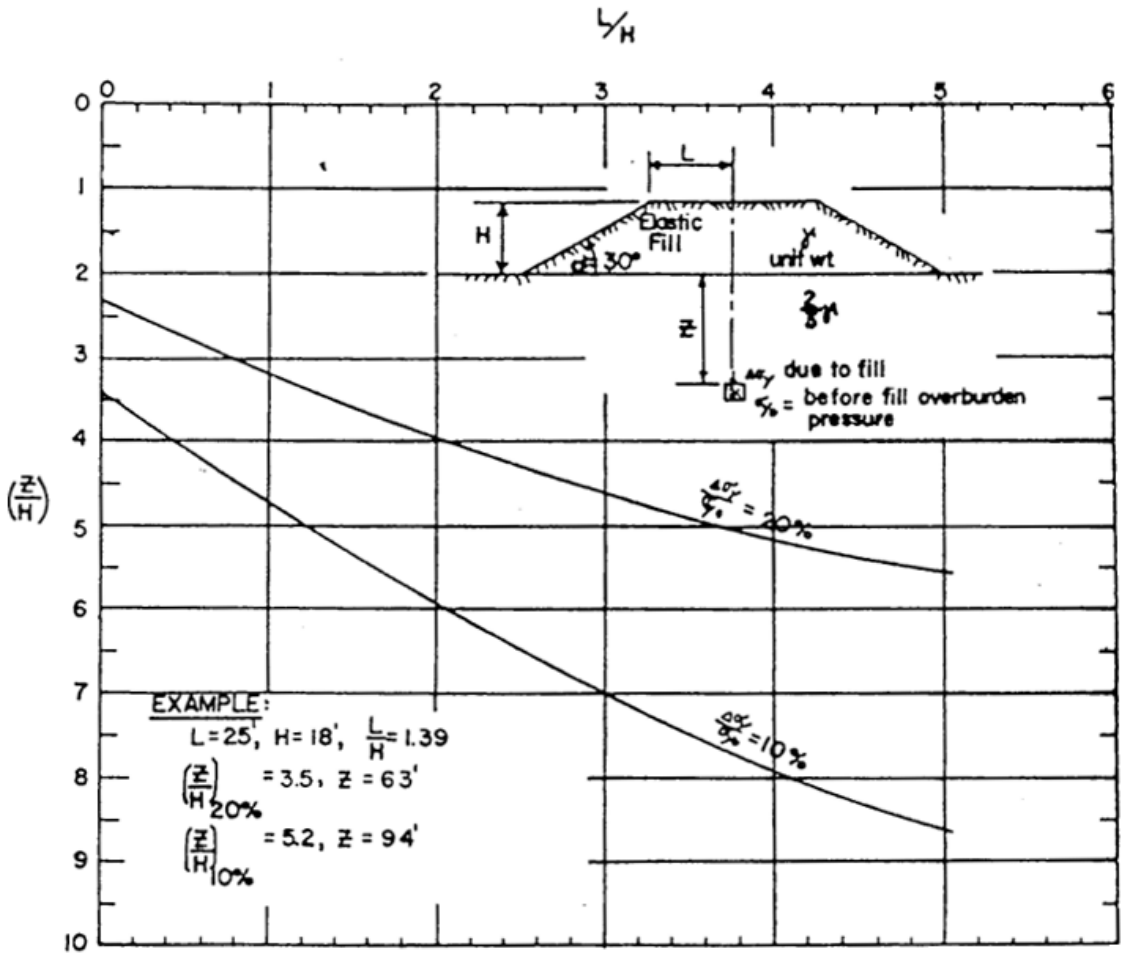


Figure 3-2 Determining the Maximum Depth of Significant Increase in Vertical Stress in the Foundation Soils Resulting from an Infinitely Long Trapezoidal Fill

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

3.3 References

3-1. Cheney, R. S. & Chassie, R. G., Soils and Foundations Workshop Manual – Second Edition, FHWA HI-88-009, 1993.

3-2. Department of the Navy, ***NAVFAC DM-7.1 – Soil Mechanics***, Naval Facilities Engineering Command, 1982.

3-3. Federal Highway Administration, ***Checklist and Guidelines for Review of Geotechnical Reports and Preliminary Plans and Specifications***, 1985. Revised 2003.

3-4. Florida Department of Transportation, ***General Tolling Requirements (GTR) Volume 1***, (Current Version).

3-5. Florida Department of Transportation, ***Rigid Pavement Design Manual***, (Current version).

3-6. Mayne, P., Christopher, B., Dejong, J., ***Subsurface Investigations – Geotechnical Site Characterization***, FHWA-NHI-01-031, 2002.

3-7. National Cooperative Highway Research Program, ***Recommended Guidelines for Sealing Geotechnical Exploratory Holes***, NCHRP Report 378, 1995.

3-8. Schmertmann, J.H., Guidelines for Use in the Soils Investigation and Design of Foundations for Bridge Structures in the State of Florida, Research Report 121-A, Florida Department of Transportation, 1967.

3.4 Specifications and Standards

Subject	ASTM	AASHTO	FM	STD. INDEX
Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils	D1586	T 206	-	
Moisture Density Relations of Soils Using a 10-lb. (4.54kg) Rammer and an 18-in. (457mm) Drop	-	-	1-T 180	
Embankment Utilization	-	-	-	120-001

Chapter 4 In Situ Testing

This chapter provides testing requirements for soil and rock parameters determined in situ. This is important on all projects, especially those involving soft clays, loose sands, and/or sands below the water table due to the difficulty of obtaining representative samples suitable for laboratory testing.

4.1 Standard Penetration Test (SPT)

Perform SPT borings using automatic hammers as outlined below. The use of a donut hammer is not permitted. Perform all SPT borings in accordance with ASTM D1586 (AASHTO T 206).

Automatic SPT hammers are required for all SPT borings performed using truck and all-terrain vehicle mounted drilling equipment; safety hammers are permitted only for borings requiring specialty and/or unique drilling equipment that cannot support an automatic hammer (i.e., small amphibious rigs, tripod, small barge, etc.) Note the type of hammer (safety or automatic) on the boring logs. Only one type of hammer may be used in each SPT boring. Use of safety hammers requires the approval of the DGE.

When SPTs are performed in soil layers containing shell or similar materials, the sampler may become plugged causing SPT N-values to be much larger than for an unplugged sampler. In these instances, a reduction in the N-value used for design is required. Refer to ***Figure 4-1*** and Reference ***4-13*** for reducing distorted N-values to the trend of the N-values that do not appear distorted. Report the actual N-values obtained in the field on the Report of Core Borings Sheet.

Do not correct the N-values when using FB-Deep or GeoStat, as the design methodology is based on uncorrected N-values.

Commentary: SPT values should not be used indiscriminately as they are sensitive to the fluctuations in individual drilling practices and equipment. Studies have also indicated that the results are more reliable in sands than clays. Although extensive use of this test in subsurface exploration is recommended, it should always be augmented by other field and laboratory tests, particularly when dealing with clays.

Follow the procedure for measuring the energy during SPT borings as described in ASTM D4633.

Most design correlations and FDOT design programs are based on N_{60}, therefore, when N-values are obtained using an automatic hammer, convert to an equivalent N_{60} using the following relationship:

$$N_{60} = \xi \times N_{auto}$$

where:

N_{AUTO} = Automatic Hammer N-value

ξ = Equivalent Safety Hammer Conversion Factor

N_{60} = Equivalent Safety Hammer N-value corrected for efficiency

Use a value of 1.24 for ξ in the above relationship. No other multiplier shall be used to

convert automatic hammer N-values to equivalent safety hammer N-values without written concurrence from the SGE. Consultants desiring to use their own rig specific conversion factor must perform annual calibrations in accordance with ASTM D4633.

Plot the field-collected SPT N-values on the soil profiles depicted in the Report of Core Boring sheets for inclusion in the Plans.

4.2 Cone Penetration Test (CPT)

Perform CPT soundings using penetration rates between 0.4 and 0.8 in/sec and standard cone dimensions of a 60-degree tip angle and a 10 cm^2 (1.55 in^2) projected end area. Perform tests in accordance with ASTM D5778 for electronic friction cones and piezocones.

Present the raw penetrometer data showing the end-bearing resistance, the friction resistance, and the friction ratio (friction resistance divided by end bearing resistance) vs. depth. Include pore pressures, if measured. Present the results in tabular form indicating the interpreted results of the raw data. See ***Figure 4-2*** and ***Figure 4-3.***

Note: The log for a standard CPT sounding would only include the first three plots: tip resistance, local friction, and friction ratio as shown in ***Figure 9-5.***

Generally, soil samples are not obtained with soundings, therefore, augment CPT soundings with SPT borings or other boring-types that include soil sampling.

4.3 Dynamic Cone Penetrometer Test

Perform shallow tests in accordance with ASTM D6951. For deeper tests, the equipment, testing procedure and interpretation of the results should be based upon the manufacturer's recommendations.

4.4 Dilatometer Test (DMT)

Perform tests in accordance with ASTM D6635. See ***Figure 4-4*** and ***Figure 4-5***.

4.5 Pressuremeter Test (PMT)

Perform tests in accordance with ASTM D4719. See ***Figure 4-6***.

4.6 Field Shear Vane Test

Perform tests in accordance with ASTM D2573 (AASHTO T 223). Use of this method is limited to measurement of shear strength in soft clays and organic deposits and is not to be used in stiff and hard clays. Results can be affected by the presence of gravel, shells, roots, or sand layers. In highly plastic clays, shear strength may be overestimated and requires a correction factor to be applied. See ***Figure 4-7***.

4.7 Infiltration Test

Perform tests in accordance with ASTM D3385. In the case of differing velocities for the inner ring and the annular space, use the maximum velocity from the inner ring.

4.8 Permeability Tests

Permeability tests include constant head, rising head, falling head, pumping, and Vertical In Situ Permeameter (VIP) tests. Measurement of permeability is highly sensitive to natural and test conditions. When performing these tests, use care to minimize sources of error and correctly interpret and compensate for deviations from ideal test conditions.

Factors such as position of the water level, type of material (rock or soil), depth of the test zone, permeability of the test zone, and heterogeneity and anisotropy of the test zone influence the performance and applicability of permeability tests. To account for this, test zones must be isolated. See References ***4-7*** and ***4-11*** for methods to isolate test zones, ***Figure 4-8*** for typical ranges of permeability coefficients, and ***Figure 4-9*** for formulas for computing permeability coefficients from constant and variable head tests. Perform constant head and falling head borehole permeability tests in accordance with ASTM D6391, VIP tests in accordance with ***FM 5-614,*** and pumping tests in accordance with ASTM D4050 (see Reference ***4-11***).

4.9 Environmental Corrosion Tests

Perform corrosion testing 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 version of ***SDG*** for the criteria, defining each class. For roadway drainage systems, present test results for each stratum for use in determining alternate culvert materials.

Perform tests in the field and/or the laboratory in accordance with the standard procedures listed below. Once the project's corrosion test results have been reviewed by the DMRO, compile the sample data and results into the "Corrosion Series Test Results_SMO.xlsx" Excel form on the ***Geotechnical Office Webpage,*** and email the completed form to ***SM-corrosionsection@dot.state.fl.us***.

Table 4-1 Corrosion Series Tests

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

4.10 Grout Plug Pullout Test

Grout Plug Pullout tests can be performed in conjunction with static load tests to confirm unit skin friction values for rock-socketed drilled shafts.

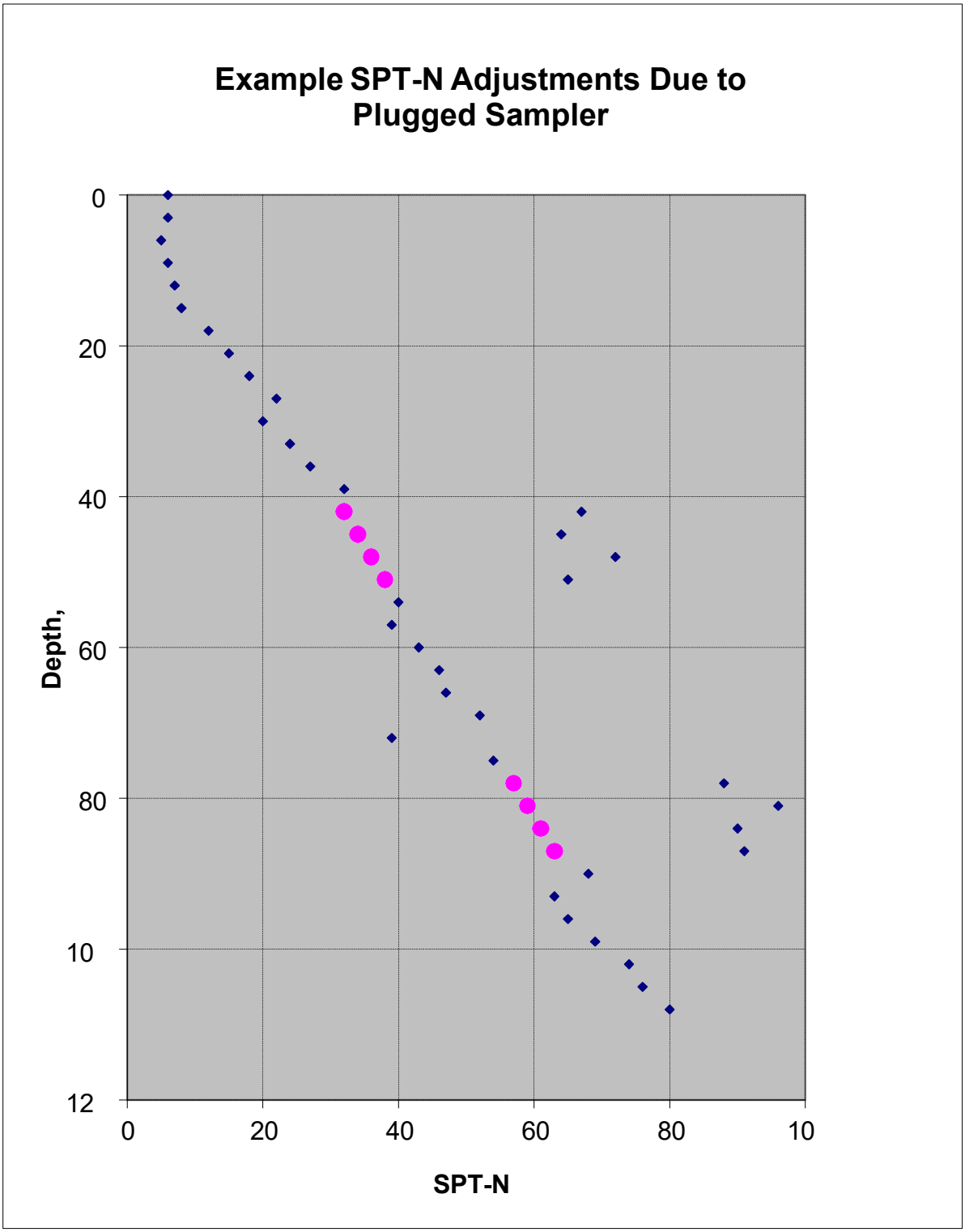


Figure 4-1 Example SPT N Adjustments Due to Plugged Sampler

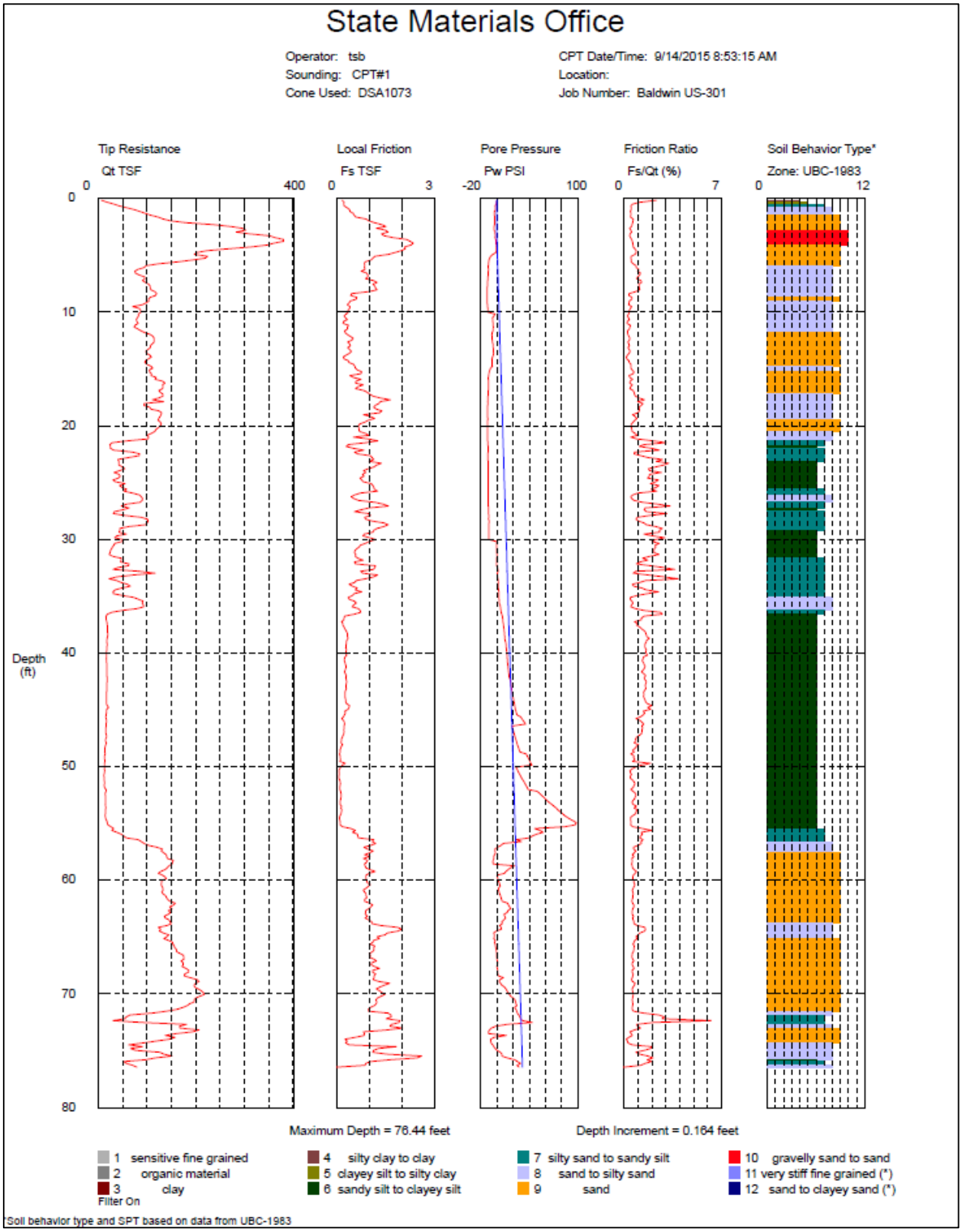


Figure 4-2 Typical Log from Electric Piezocone

FLORIDA D.O.T MATERIALS OFFICE

CPT DATE :04/28/88 9:00 ENGINEER :BLANTON
LOCATION :301+59 6 m LT CL Cone Used :283
Job No. :46090-3511 Water table (meters) : 1.5
Tot. Unit Wt. (avg) : 510 N/m^3

DEPTH (meters)	Qc (avg) (MN/m^2)	Fs (avg) (kN/m^2)	Rf (avg) (%)	SIGV' (MPa)	SOIL BEHAVIOUR TYPE	Eq - Dr (%)	PHI deg.	SPT N	Su (MPa)
0.50	53.24	0.16	0.31	0.04	sand to silty sand	>90	>48	13	UNDEFINED
1.00	139.74	0.56	0.40	0.13	sand	>90	>48	27	UNDEFINED
1.50	205.64	0.71	0.35	0.22	sand	>90	>48	39	UNDEFINED
2.00	312.59	1.77	0.57	0.28	gravelly sand to sand	>90	>48	50	UNDEFINED
2.50	341.26	1.77	0.52	0.31	gravelly sand to sand	>90	>48	>50	UNDEFINED
3.00	262.08	1.08	0.41	0.35	gravelly sand to sand	>90	>48	42	UNDEFINED
3.50	236.04	1.00	0.42	0.38	sand	>90	>48	45	UNDEFINED
4.00	173.89	0.67	0.39	0.42	sand	>90	46-48	33	UNDEFINED
4.50	92.91	0.41	0.44	0.45	sand to silty sand	70-80	44-46	22	UNDEFINED
5.00	17.01	0.13	0.79	0.49	sandy silt to clayey silt	UNDFND	UNDFD	7	1.07
5.50	6.64	0.01	0.21	0.52	sensitive fine grained	UNDFND	UNDFD	3	.38
6.00	10.38	0.03	0.32	0.55	sandy silt to clayey silt	UNDFND	UNDFD	4	.62
6.50	16.33	0.10	0.60	0.59	sandy silt to clayey silt	UNDFND	UNDFD	6	1.01
7.00	15.86	0.10	0.61	0.62	sandy silt to clayey silt	UNDFND	UNDFD	6	.97
7.50	14.86	0.09	0.63	0.66	sandy silt to clayey silt	UNDFND	UNDFD	6	.90
8.00	10.37	0.06	0.61	0.69	sandy silt to clayey silt	UNDFND	UNDFD	4	.60
8.50	13.54	0.09	0.67	0.73	sandy silt to clayey silt	UNDFND	UNDFD	5	.80
9.00	22.86	0.16	0.70	0.76	silty sand to sandy silt	<40	34-36	7	UNDEFINED
9.50	29.15	0.21	0.71	0.80	silty sand to sandy silt	<40	36-38	9	UNDEFINED
10.00	35.88	0.26	0.72	0.83	silty sand to sandy silt	<40	36-38	11	UNDEFINED
10.50	39.31	0.28	0.71	0.87	silty sand to sandy silt	40-50	36-38	13	UNDEFINED
11.00	53.59	0.30	0.56	0.90	sand to silty sand	50-60	38-40	13	UNDEFINED
11.50	58.47	0.30	0.52	0.94	sand to silty sand	50-60	38-40	14	UNDEFINED
12.00	92.18	0.64	0.69	0.97	sand to silty sand	60-70	40-42	22	UNDEFINED
12.50	94.25	0.44	0.47	1.01	sand to silty sand	60-70	40-42	23	UNDEFINED
13.00	125.46	1.04	0.83	1.04	sand to silty sand	70-80	40-42	30	UNDEFINED
13.50	50.89	1.15	2.26	1.08	sandy silt to clayey silt	UNDFND	UNDFD	19	3.24
14.00	51.81	0.39	0.76	1.11	silty sand to sandy silt	40-50	36-38	17	UNDEFINED
14.50	68.95	0.32	0.46	1.15	sand to silty sand	50-60	38-40	17	UNDEFINED
15.00	154.16	0.41	0.27	1.18	sand	70-80	42-44	30	UNDEFINED
15.50	214.47	0.46	0.22	1.22	gravelly sand to sand	80-90	42-44	34	UNDEFINED
16.00	239.03	0.43	0.18	1.25	gravelly sand to sand	80-90	42-44	38	UNDEFINED
16.50	168.43	0.21	0.13	1.29	sand	70-80	42-44	32	UNDEFINED
17.00	102.13	0.13	0.13	1.32	sand	60-70	38-40	20	UNDEFINED
17.50	101.49	0.22	0.22	1.36	sand	60-70	38-40	19	UNDEFINED
18.00	171.24	0.28	0.16	1.39	sand	70-80	40-42	33	UNDEFINED
18.50	174.32	0.23	0.13	1.43	sand	70-80	40-42	33	UNDEFINED
19.00	191.14	0.25	0.13	1.46	sand	70-80	42-44	37	UNDEFINED

Dr - All sands (Jamiolkowski et al. 1985) PHI - Robertson and Campanella 1983 Su: Nk= 15

**** Note: For interpretation purposes the PLOTTED CPT PROFILE should be used with the TABULATED OUTPUT from CPTINTR1 (v 3.04) ****

Figure 4-3 Typical Interpreted Output from Electric Cone Penetrometer

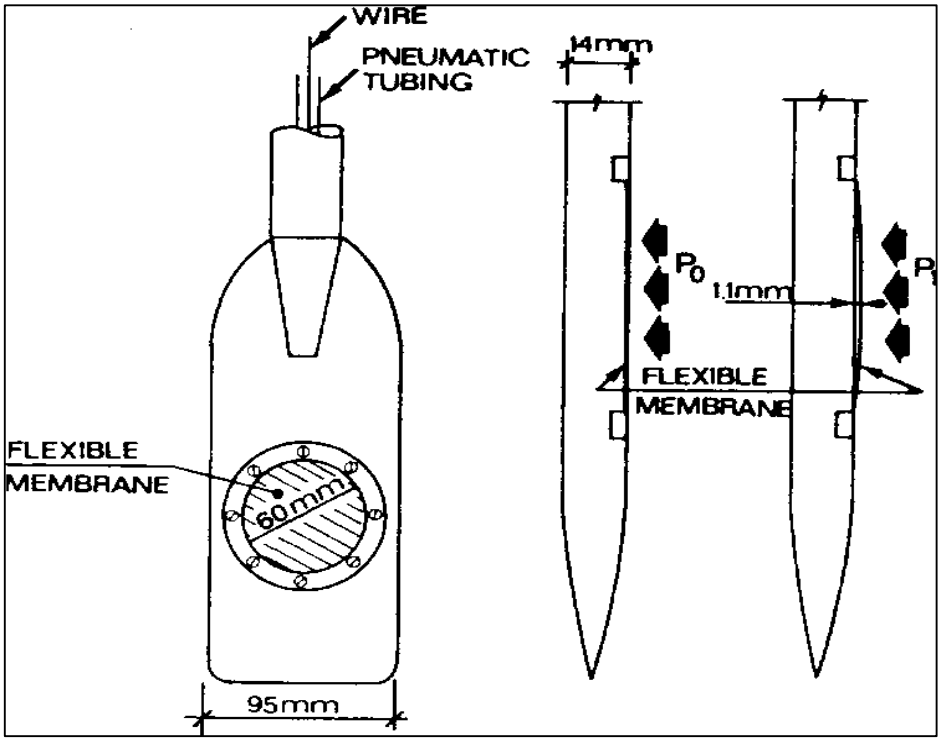


Figure 4-4 Schematic of the Marchetti Flat Dilatometer (After Baldi, et al., 1986)

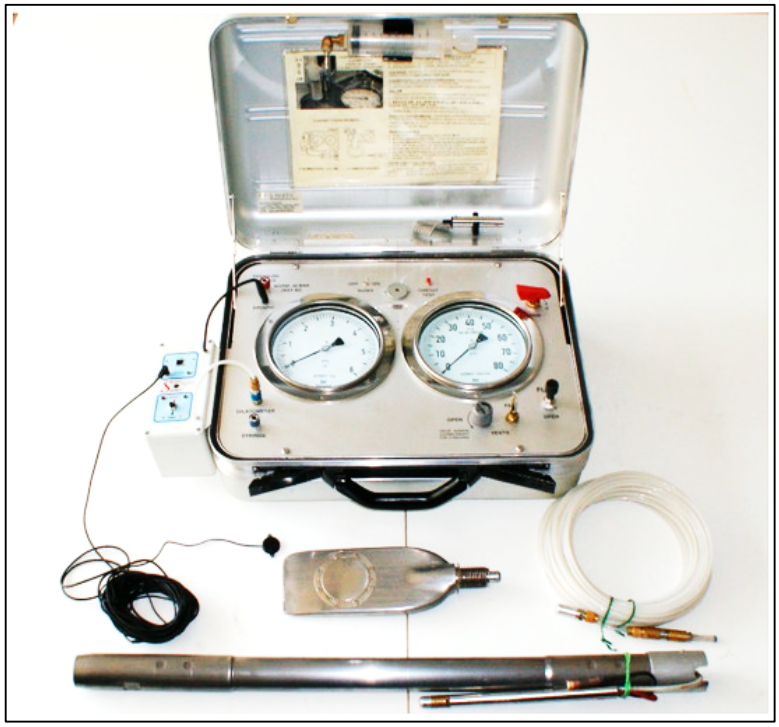
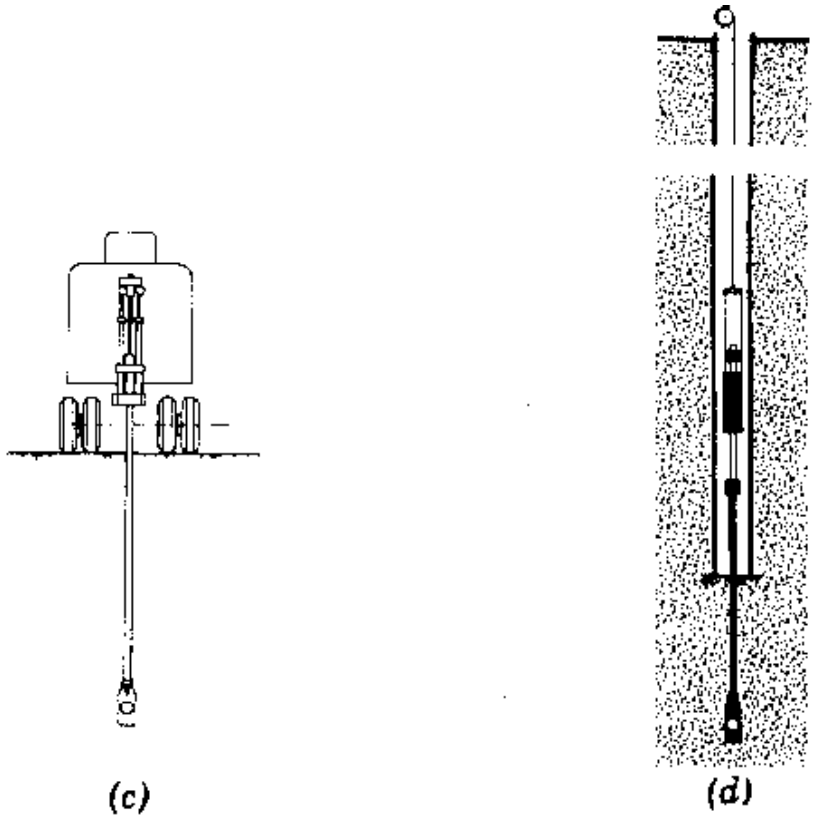


Figure 4-5 Dilatometer (After Marchetti 1980)



Flat Dilatometer: (a) Side View and Front View; (b) Blade, Control Unit and Cable; (c) Dilatometer Being Jacked into Ground; (d) Dilatometer Being Driven by Down-The-Hole Wireline Hammer

Figure 4-5 Continued

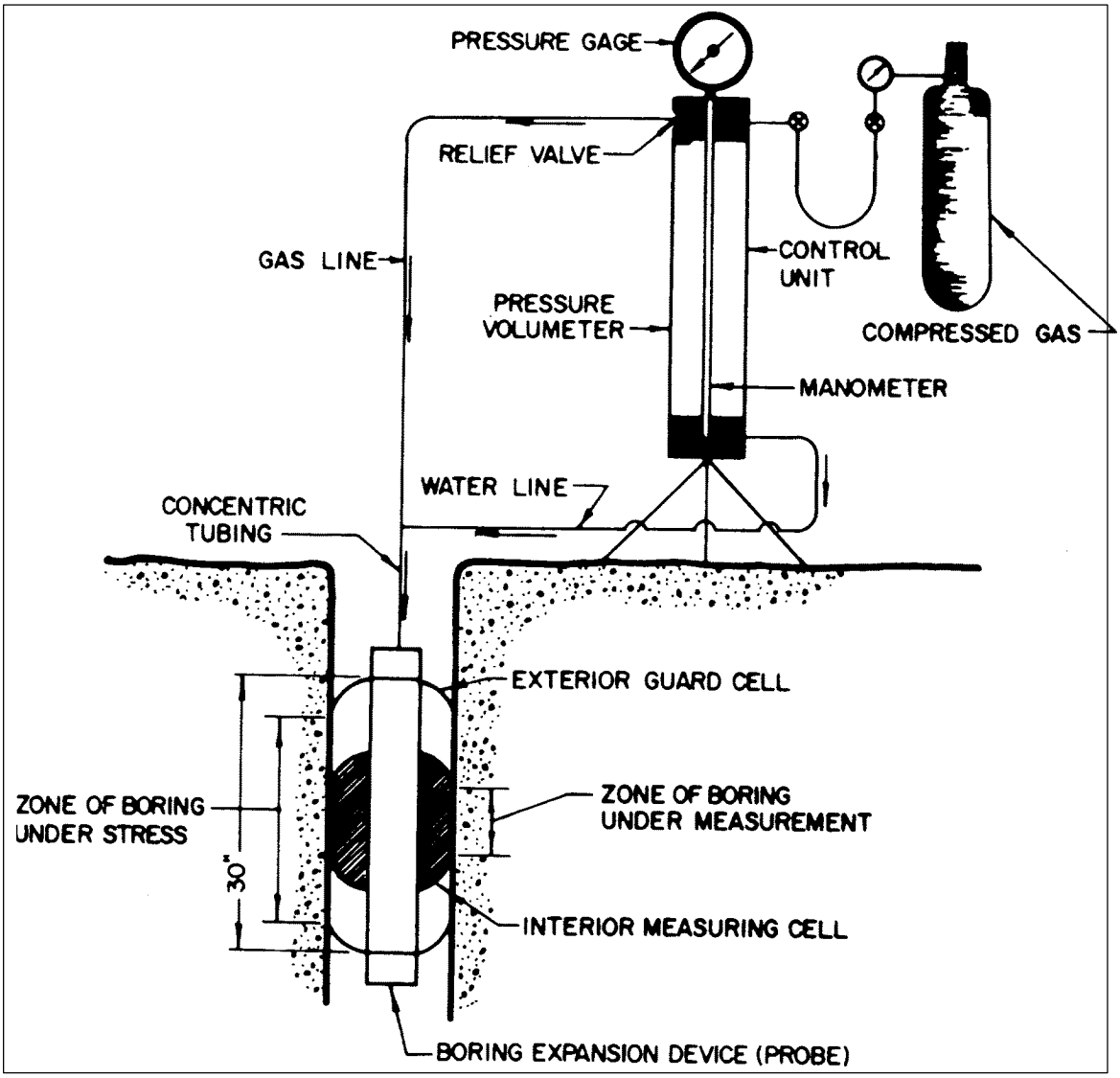


Figure 4-6 Menard Pressuremeter Equipment (After NAVFAC, 1986)

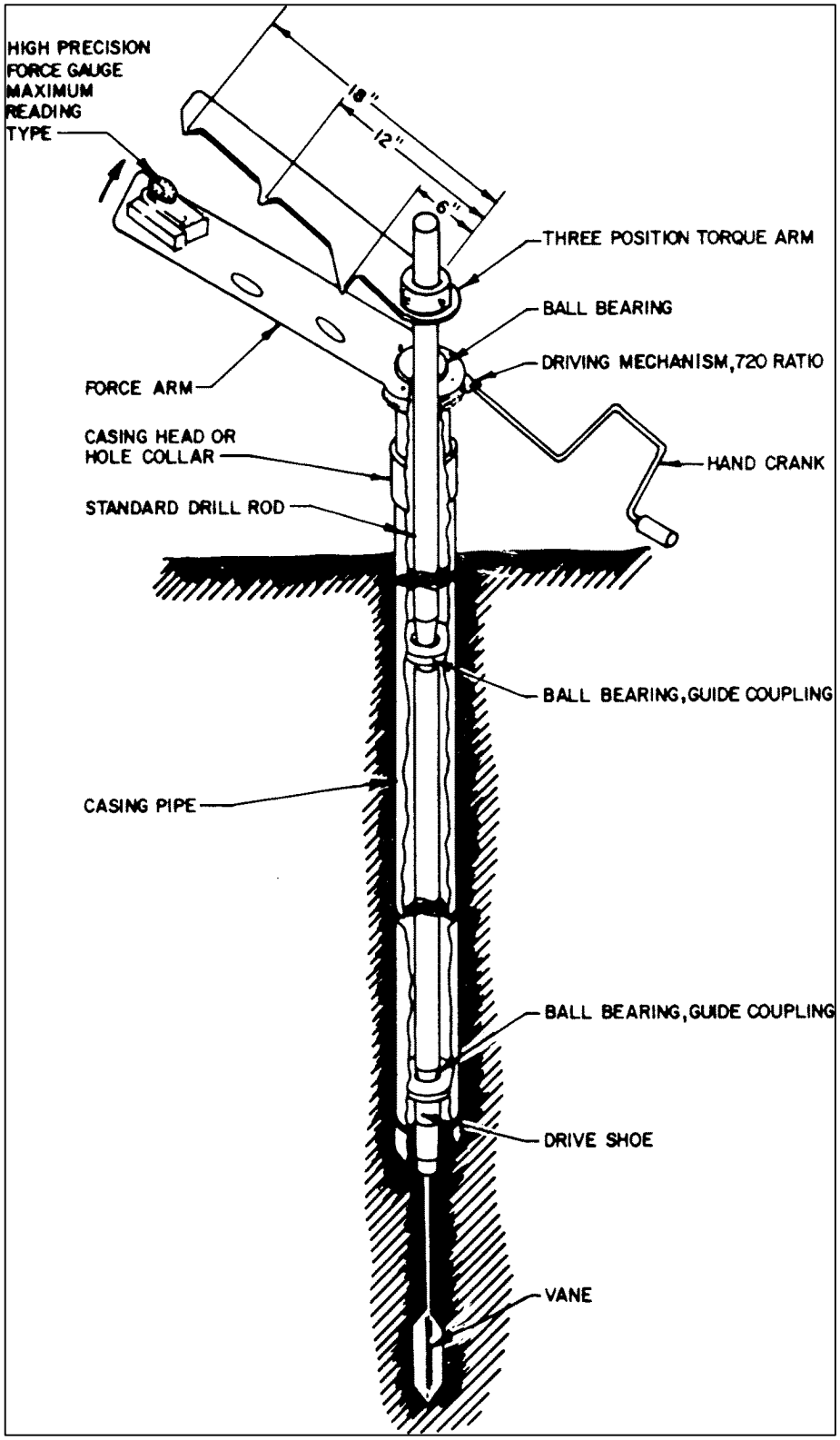


Figure 4-7 Vane Shear Test Equipment (After NAVFAC, 1986)

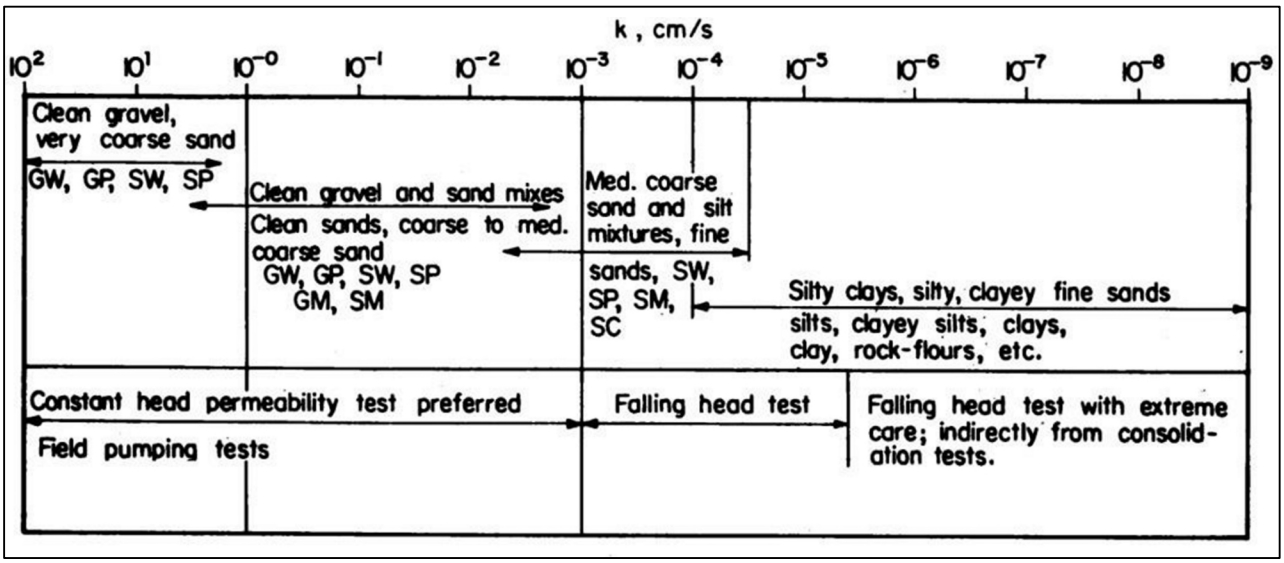
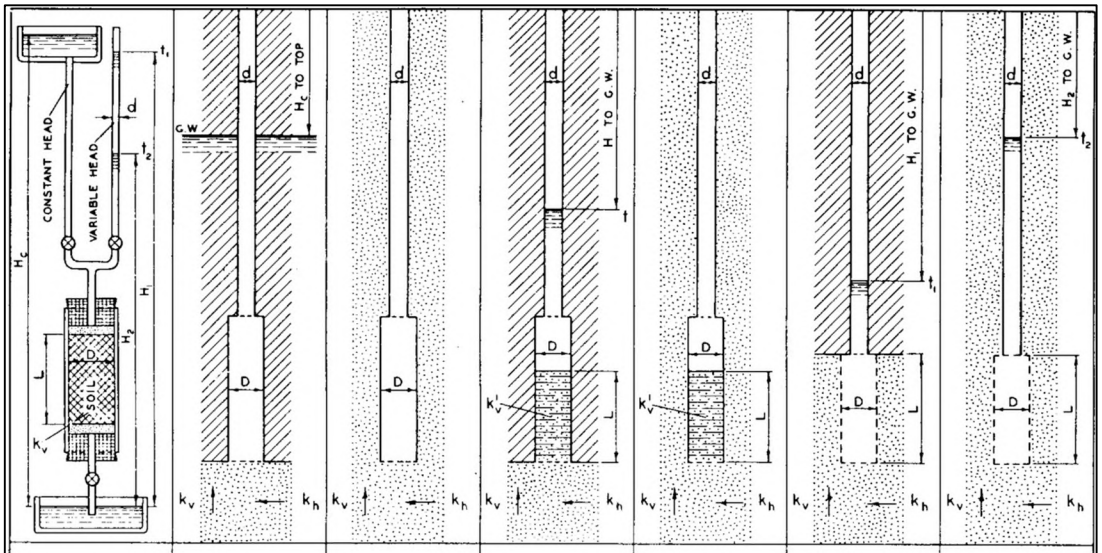


Figure 4-8 Permeability Test Methods (from Bowles, 1984)

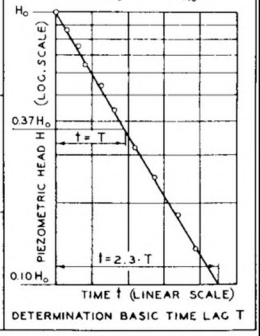


LABORATORY PERMEAMETER (CONSOLIDOMETER) A	FLUSH BOTTOM AT IMPERVIOUS BOUNDARY B	FLUSH BOTTOM IN UNIFORM SOIL C	SOIL IN CASING AT IMPERVIOUS BOUNDARY D	SOIL IN CASING IN UNIFORM SOIL E	WELL POINT–FILTER AT IMPERVIOUS BOUNDARY F	WELL POINT–FILTER IN UNIFORM SOIL G

CASE	CONSTANT HEAD	VARIABLE HEAD	BASIC TIME LAG
A	$k_v = \frac{4 \cdot q \cdot L}{\pi \cdot D^2 \cdot H_c}$	$k_v = \frac{d^2 \cdot L}{D^2 \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_v = \frac{L}{t_2 - t_1} \ln \frac{H_1}{H_2}$ FOR $d = D$	$k_v = \frac{d^2 \cdot L}{D^2 \cdot T}$ $k_v = \frac{L}{T}$ FOR $d = D$
B	$k_m = \frac{q}{2 \cdot D \cdot H_c}$	$k_m = \frac{\pi \cdot d^2}{8 \cdot D \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_m = \frac{\pi \cdot D}{8 \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ FOR $d = D$	$k_m = \frac{\pi d^2}{8 \cdot D \cdot T}$ $k_m = \frac{\pi \cdot D}{8 \cdot T}$ FOR $d = D$
C	$k_m = \frac{q}{2.75 \cdot D \cdot H_c}$	$k_m = \frac{\pi \cdot d^2}{11 \cdot D \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_m = \frac{\pi \cdot D}{11 \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ FOR $d = D$	$k_m = \frac{\pi \cdot d^2}{11 \cdot D \cdot T}$ $k_m = \frac{\pi \cdot D}{11 \cdot T}$ FOR $d = D$
D	$k_v' = \frac{4 \cdot q \cdot \left[\frac{\pi}{8} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m} + L\right]}{\pi \cdot D^2 \cdot H_c}$	$k_v' = \frac{d^2 \cdot \left[\frac{\pi}{8} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m} + L\right]}{D^2 \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_v = \frac{\frac{\pi}{8} \cdot \frac{D}{m} + L}{t_2 - t_1} \ln \frac{H_1}{H_2}$ FOR $k_v' = k_v$, $d = D$	$k_v' = \frac{d^2 \cdot \left[\frac{\pi}{8} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m}\right] + L}{D^2 \cdot T}$ $k_v = \frac{\frac{\pi}{8} \cdot \frac{D}{m} + L}{T}$ FOR $k_v' = k_v$, $d = D$
E	$k_v' = \frac{4 \cdot q \cdot \left[\frac{\pi}{11} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m} + L\right]}{\pi \cdot D^2 \cdot H_c}$	$k_v' = \frac{d^2 \cdot \left[\frac{\pi}{11} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m} + L\right]}{D^2 \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_v = \frac{\frac{\pi}{11} \cdot \frac{D}{m} + L}{t_2 - t_1} \ln \frac{H_1}{H_2}$ FOR $k_v' = k_v$, $d = D$	$k_v' = \frac{d^2 \cdot \left[\frac{\pi}{11} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m} + L\right]}{D^2 \cdot T}$ $k_v' = \frac{\frac{\pi}{11} \cdot \frac{D}{m} + L}{T}$ FOR $k_v' = k_v$, $d = D$
F	$k_h = \frac{q \cdot \ln \left[\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right]}{2 \cdot \pi \cdot L \cdot H_c}$	$k_h = \frac{d^2 \cdot \ln \left[\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right]}{8 \cdot L \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_h = \frac{d^2 \cdot \ln \left(\frac{4mL}{D}\right)}{8 \cdot L \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ FOR $\frac{2mL}{D} > 4$	$k_h = \frac{d^2 \cdot \ln \left[\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right]}{8 \cdot L \cdot T}$ $k_h = \frac{d^2 \cdot \ln \left(\frac{4mL}{D}\right)}{8 \cdot L \cdot T}$ FOR $\frac{2mL}{D} > 4$
G	$k_h = \frac{q \cdot \ln \left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{2 \cdot \pi \cdot L \cdot H_c}$	$k_h = \frac{d^2 \cdot \ln \left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{8 \cdot L \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_h = \frac{d^2 \cdot \ln \left(\frac{2mL}{D}\right)}{8 \cdot L \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ FOR $\frac{mL}{D} > 4$	$k_h = \frac{d^2 \cdot \ln \left[\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right]}{8 \cdot L \cdot T}$ $k_h = \frac{d^2 \cdot \ln \left(\frac{2mL}{D}\right)}{8 \cdot L \cdot T}$ FOR $\frac{mL}{D} > 4$

NOTATION

D = DIAM, INTAKE, SAMPLE, CM
d = DIAMETER, STANDPIPE, CM
L = LENGTH, INTAKE, SAMPLE, CM
H_c = CONSTANT PIEZ. HEAD, CM
H_1 = PIEZ. HEAD FOR $t = t_1$, CM
H_2 = PIEZ. HEAD FOR $t = t_2$, CM
q = FLOW OF WATER, $CM^3/SEC.$
t = TIME, SEC.
T = BASIC TIME LAG, SEC.
k_v' = VERT. PERM. CASING, CM/SEC.
k_v = VERT. PERM. GROUND, CM/SEC.
k_h = HORZ. PERM. GROUND, CM/SEC.
k_m = MEAN COEFF. PERM., CM/SEC.
m = TRANSFORMATION RATIO
$k_m = \sqrt{k_h \cdot k_v}$ $m = \sqrt{k_h / k_v}$
$\ln = \log_e = 2.3 \log_{10}$



DETERMINATION BASIC TIME LAG T

ASSUMPTIONS

SOIL AT INTAKE, INFINITE DEPTH AND DIRECTIONAL ISOTROPY (k_v AND k_h CONSTANT) – NO DISTURBANCE, SEGREGATION, SWELLING OR CONSOLIDATION OF SOIL – NO SEDIMENTATION OR LEAKAGE – NO AIR OR GAS IN SOIL, WELL POINT OR PIPE – HYDRAULIC LOSSES IN PIPES, WELL POINT OR FILTER NEGLIGIBLE

Figure 4-9 Formulas for Determination of Permeability (Hvorslev, 1951)

4.11 References

4-1. Baldi, G., Bellotti R., Ghionna, V., Jamiolkowski, M., Marchetti, S. and Pasqualini, E. Flat Dilatometer Tests in Calibration Chambers in *Use of In situ Tests in Geotechnical Engineering,* ASCE Specialty Conference, Geotechnical Special Publication No. 6, 1986.

4-2. Bowles, J.E., Physical and Geotechnical Properties of Soils, Second Edition, New York: McGraw-Hill Book Company, 1984.

4-3. Briaud, J.L., The Pressuremeter Test for Highway Applications, FHWA-IP-89-008, 1989.

4-4. Briaud, J.L., Miran, J., Manual on the Cone Penetrometer Test, FHWA-SA-91-043, 1992.

4-5. Briaud, J.L., Miran, J., ***Manual on the Dilatometer Test***, FHWA-SA-91-044, 1992.

4-6. Cheney, R. S., Chassie, R. G., Soils and Foundations Workshop Manual – Second Edition, FHWA HI-88-009, 1993.

4-7. Department of the Navy, ***NAVFAC DM-7.1 – Soil Mechanics***, Naval Facilities Engineering Command, 1982.

4-8. Florida Administrative Code, ***Standards for Onsite Sewage Treatment and Disposal Systems***, Rules of the Department of Health and Rehabilitative Services, Chapter 64E-6.

4-9. Florida Department of Transportation, ***Drainage Manual***, (Current version).

4-10. Florida Department of Transportation, ***Structures Design Guidelines***, (Current version).

4-11. Hvorslev, J.M., ***Time Lag and Soil Permeability in Ground-Water Observations***, Bulletin No. 36, United States Army Corps of Engineers Waterways Experiment Station, Vicksburg, 1951.

4-12. Marchetti, S., ***In Situ Tests by Flat Dilatometer***, Journal of the Geotechnical Engineering Division, ASCE, Vol. 106, No. GT3, March 1980.

4-13. Mayne, P., Christopher, B., Dejong, J., ***Subsurface Investigations – Geotechnical Site Characterization***, FHWA-NHI-01-031, 2002.

4-14. Schmertmann, J.H., Guidelines for Use in the Soils Investigation and Design of Foundations for Bridge Structures in the State of Florida, Research Report 121-A, Florida Department of Transportation, 1967.

4-15. Schmertmann, J. H., Guidelines for Cone Penetration Test - Performance and Design, FHWA-TS-78-209, 1978.

4-16. Schmertmann, J.H., Suggested Method for Performing the Flat Dilatometer Test, Geotechnical Testing Journal, ASTM, Vol. 9, No. 2, June 1986.

4-17. U.S. Department of Interior, Bureau of Reclamation, ***Earth Manual***, U.S.

Government Printing Office, Washington, D.C., 1998.

4-18. U.S. Environmental Protection Agency, Land Treatment of Municipal Wastewater - Process Design Manual, 1981.

4.12 Specifications and Standards

Subject	ASTM	AASHTO	FM
Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils	D1586	T 206	-
Standard Test Method for Field Vane Shear Test in Cohesive Soil	D2573	T 223	-
Standard Test Method for Infiltration Rate of Soils in Field Using Double-Ring Infiltrometer	D3385	-	-
Standard Test Method (Field Procedure) for Withdrawal and Injection Well Tests for Determining Hydraulic Properties of Aquifer Systems	D4050	-	-
Standard Test Method for Energy Measurement for Dynamic Penetrometers	D4633	-	-
Standard Test Methods for Prebored Pressuremeter Testing in Soils	D4719	-	-
Standard Practices for Preserving and Transporting Rock Core Samples	D5079	-	-
Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils	D5778	-	-
Standard Practice for Using the Electronic Piezocone Penetrometer Tests for Environmental Site Characterization and Estimation of Hydraulic Conductivity	D6067		-
Standard Test Method for Field Measurement of Hydraulic Conductivity Using Borehole Infiltration	D6391		-
Standard Test Method for Performing the Flat Plate Dilatometer Test	D6635	-	-
Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications	D6951	-	-
Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method	G 57	-	-
Standard Test Method for pH of pH of Soil and Water	-	-	5-550
Standard Test Method for Resistivity of Soil and Water	-	-	5-551
Standard Test Method for Sulfate in Soil and Water	-	-	5-553
Standard Test Methods for Chloride in Soil and Water	-	-	5-552
Standard Test Method for Determination of Mean Permeability in the Field Using the Vertical In situ Permeameter (VIP)	-	-	5-614

Chapter 5 Laboratory Tests

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 Specifications for Road and Bridge Construction*** (hereon after referred to as Standard Specifications) Section 105.

Perform tests and present results as described in the listed References and Specifications and Standards except as stated herein. Engineering judgment must be exercised in setting up a testing program that will produce the information required on each specific project.

5.1 Soils

The following tests are commonly performed on soils. Perform each test in accordance with the below specified standards. For additional information, see the references at the end of the chapter.

Table 5-1 Laboratory Tests for Soils

Test/Property	Specified Test Standard
Sieve Analysis	AASHTO T 27 or AASHTO T 311 [ASTM C136]
Hydrometer Analysis	AASHTO T 88
Moisture Content	AASHTO T 265 (ASTM D2216)
Liquid Limit	AASHTO T 89 (ASTM D4318)
Plastic Limit	AASHTO T 90 (ASTM D4318)
Specific Gravity of Soils	AASHTO T 100 (ASTM D854) and AASHTO T 85 (ASTM C127)
Unconfined Compressive Strength[1]	AASHTO T 208 (ASTM D2166)
Unconsolidated-Undrained (UU) Triaxial Compression[1,2]	AASHTO T 296 (ASTM D2850)
Consolidated-Undrained (CU) Triaxial Compression[1,2]	ASTM D4767
Consolidated-Drained (CD) Triaxial Compression[1,2]	ASTM D7181
Direct Shear[2]	***FM 3-D3080***
Miniature Vane Shear (Torvane)[3]	ASTM D4648
Pocket Penetrometer[3]	ASTM D8605
One-dimensional Consolidation[1]	AASHTO T 216 (ASTM D2435)
Constant Rate of Strain Consolidation[1]	ASTM D4186

Test/Property	Specified Test Standard
Organic Content (Ignition Loss Test)	AASHTO T 267 (ASTM D2974)
Shrinkage	ASTM D4943
Swell	AASHTO T 258 (ASTM D4546)
Constant Head Permeability	AASHTO T 215 (ASTM D2434)
Falling Head Permeability	***FM 5-513***
Flexible Wall Permeability	ASTM D5084
Standard Proctor Compaction	***FM 1-T 099***
Modified Proctor Compaction	***FM 1-T 180***
Maximum and Minimum Index Density	ASTM D4253 & D4254
Limerock Bearing Ratio (LBR)	***FM 5-515***
Resilient Modulus (Dynamic)	AASHTO T 307

1 Must be performed using undisturbed samples.
2 A minimum of three test specimens is required.
3 Used only as an index for s_u and cannot be used in place of other, more direct laboratory testing.

Commentary: For high organic materials (organic content greater than 50%), FDOT-sponsored studies have shown that end of primary consolidation occurs quickly in the laboratory and field, and that a major portion of the total settlement is due to secondary and tertiary compression (creep). As a result, differentiating between primary consolidation and creep settlement on the individual loading's settlement versus time plots can be very difficult and generate misleading results. To analyze results from one-dimensional consolidation tests for these types of materials, use the Square Root (Taylor) Method to identify the end of primary consolidation for each load sequence. Each load sequence must be maintained for 24 hours or more to identify the slopes of the secondary and tertiary compression portions of the settlement versus time plot.

5.1.1 Environmental Corrosion Tests

Refer to **Environmental Corrosion Tests** section in ***Chapter 4*** for a list of test procedures. Corrosion testing must be performed for each site unless the most aggressive conditions are assumed.

5.2 Rock and Intermediate Geomaterials (IGM)

Perform laboratory tests on rock and IGM on intact cores noting the presence of joints, bedding planes, etc. Rock cores must come from the zone that the foundations will be founded in. Consider laboratory test results in conjunction with knowledge of the in situ characteristics of the rock/IGM mass. The following tests are commonly performed on rock and IGM. Perform each test in accordance with the below specified standards. Test the recovered samples at their field moisture content. When necessary, take measures to preserve the field moisture content such as keeping the samples in a moisture room or

climate controlled room, wrapping, etc.

Table 5-2 Laboratory Tests for Rock and IGM

Test/Property	Specified Test Standard
Unconfined Compressive Strength[2]	ASTM D7012
Absorption and Bulk Specific Gravity[2]	ASTM C97
Splitting Tensile Strength[1,2]	ASTM D3967
Triaxial Compressive Strength[2]	ASTM D7012
Unit Weight and Moisture Content[2]	ASTM D7263 and D2216
Scour Rate Determination	See ***5.2.1***

1 The minimum t/D (thickness-to-diameter) ratio must be 1.0 when testing.
2 Test sample specimens at the field moisture content.

5.2.1 Rock Scour Rate Determination

When rock or cohesive IGM is encountered within the anticipated scour elevation, coordinate with the hydraulic engineer and the DGE regarding the availability of the Rotating Erosion Test Apparatus (RETA).

Commentary: The RETA measures the erosion of intact 4-inch long by 2.4-inch or 4-inch diameter rock/IGM core samples and is used to model the erodibility of cohesive soils and soft rock and estimate scour depths.

5.3 References

5-1. Bowles, J. E., Engineering Properties of Soils and their Measurement, 3rd ed., McGraw Hill Book Company, New York, 1986.

5-2. Department of the Navy, ***NAVFAC DM-7.1 – Soil Mechanics***, Naval Facilities Engineering Command, 1982.

5-3. Lambe, T. W., Soil Testing for Engineers, John Wiley & Sons, Inc. New York, NY, 1951.

5-4. Mayne, P., Christopher, B., Dejong, J., ***Subsurface Investigations – Geotechnical Site Characterization***, FHWA-NHI-01-031, 2002.

5.4 Specifications and Standards

Subject	ASTM	AASHTO	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 Determining the Resilient Modulus of Soils and Aggregate Materials	-	T 307	-
Standard Test Methods for Absorption and Bulk Specific Gravity of Dimension Stone	C97	-	-
Standard Test Method for Specific Gravity and Absorption of Coarse Aggregate	C127	T 85	1-T 85
Standard Test Method for Sieve Analysis of Fine and Coarse Aggregate	C136	T 27	-
Standard Test Method for Grain-Size Analysis of Granular Soil Materials	-	T 311	-
Standard Test Method for Particle-Size Analysis of Soils	-	T 88	-
Standard Test Method for Shrinkage Factors of Soils by the Wax Method	D4943	-	-
Standard Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft3 (600 kN-m/m3))			1-T 099
Standard Test Method for Specific Gravity of Soils	D854	T 100	-
Standard Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft3 (2,700 kN-m/m3))	-	-	1-T 180
Standard Test Method for Unconfined Compressive Strength of Cohesive Soil	D2166	T 208	-

Subject	ASTM	AASHTO	FM
Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock	D2216	T 265	-
Standard Test Method for Permeability of Granular Soils (Constant Head)	D2434	T 215	-
Standard Test Method for One-Dimensional Consolidation Properties of Soils	D2435	T 216	-
Standard Test Method for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression	D2850	T 296	-
Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils	D2974	T 267	1-T 267
Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions	-	-	3-D3080
Standard Test Method for Splitting Tensile Strength of Intact Rock Core Specimens	D3967	-	-
Standard Test Method for One-Dimensional Consolidation Properties of Soils Using Controlled-Strain Loading	D4186	-	-
Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table	D4253	-	-
Standard Test Method for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density	D4254	-	-
Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils	D4318	T 89 & T 90	-
Standard Test Methods for One-Dimensional Swell or Settlement Potential of Cohesive Soils	D4546	T 258	-
Standard Test Method for Laboratory Miniature Vane Shear Test for Saturated Fine-Grained Clayey Soil	D4648	-	-
Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils	D4767	-	-
Standard Test Method for Shrinkage Factors of Cohesive Soils by the Water Submersion Method	D4943		
Standard Practices for Preserving and Transporting Rock Core Samples	D5079	-	-
Standard Test Method for Measurement of Hydraulic Conductivity of Saturated Porous	D5084	-	-

Subject	ASTM	AASHTO	FM
Materials Using a Flexible Wall Permeameter			
Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures	D7012	-	-
Standard Test Method for Consolidated, Drained Triaxial Compression Test for Soils	D7181	-	-
Standard Test Methods for Laboratory Determination of Density and Unit Weight of Soil Specimens	D7263	-	-
Standard Test Method for Use of the Pocket Penetrometer on Cohesive Soils	D8605	-	-

Chapter 6 Materials Description, Classification, and Logging

During field exploration, a log must be kept of the materials encountered. Record details of the subsurface conditions encountered, including basic material descriptions, and details of the drilling and sampling methods. Refer to ASTM D5434 for guidance.

This chapter outlines the requirements for standardization of the field exploration data. Records of subsurface explorations should follow as closely as possible the standardized format presented herein.

6.1 Materials Description and Classification

Include a detailed description for each material stratum encountered on the log. The extent of detail will depend on the material and purpose of the project. However, the descriptions should be sufficiently detailed to provide an understanding of the material present at the site and permit grouping of similar materials for representative samples for testing.

6.1.1 Soils

In general, soil descriptions must follow the Description and Identification of Soils (Visual - Manual Procedure) of ASTM D2488. A detailed soil description must comply with the following format: color, constituents, grading, relative density or consistency, moisture content, particle angularity and shape, additional descriptive terms, and classification.

6.1.1.1 Color

The color description is restricted to two colors. If more than two colors exist, describe the soil as multi-colored or mottled and the two predominant colors given.

6.1.1.2 Constituents

Constituents are identified considering grain size distribution and the results of the manual tests. In addition to the principal constituent, identify other constituents which may affect the engineering properties of the soil. Secondary constituents are generally indicated as modifiers to the principal constituent (i.e., sandy clay or silty gravel). Include other constituents in the description using the terminology of ASTM D2488 (i.e. trace [<5%], few [5-10%], little [15-25%], some [30- 45%] and mostly [50-100%]).

6.1.1.3 Grading

Define coarse-grained soils as well-graded, poorly-graded, or gap-graded. Refer to ***1.1*** for a description of each.

Do not include grading for descriptions of fine-grained soils.

6.1.1.4 Relative Density and Consistency

Refer to ***1.1*** for a definition of relative density and consistency. Report the field recorded (uncorrected) SPT N-values on the *Report of Core Borings Sheet* (***Figure 9-4***). Refer to ***4.1*** for obtaining N_{60} from field recorded SPT N-values obtained using an automatic hammer.

Use SPT N-values to define the relative density and consistency as follows:

Table 6-1 Relative Density or Consistency

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

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

If SPT data is not available, estimate consistency in the field based on visual-manual examination of the material. Refer to ASTM D2488 for consistency criteria.

The pocket penetrometer and miniature vane shear (torvane) devices may be used in the field as an index of the remolded undrained shear strength of clay samples. Refer to ***5.1***.

6.1.1.5 Friction Angle based on SPT N-value

In the absence of laboratory shear strength testing, friction angle (φ) estimates for cohesionless soils based on SPT N-values must not exceed the values proposed by Peck, 1974 (see ***Figure 6-1***). These values are based on SPT N-values obtained at an effective overburden pressure of one ton per square foot (tsf). Use the correction factor, C_N, proposed by Peck, 1974 (see ***Figure 6-2***) to reflect the effect of horizontal stress on the sampler as a function of depth.

6.1.1.6 Moisture Content

Describe the in situ moisture content of a soil as dry, moist, or wet.

6.1.1.7 Particle Angularity and Shape

Describe coarse-grained soils as angular, sub-angular, sub-rounded, or rounded. Describe gravel, cobbles, and boulders as flat, elongated, or flat and elongated. Descriptions of fine-grained soils do not include particle angularity or shape.

6.1.1.8 Organic Content

In general, materials with organic content (O.C.) greater than 5% are unsuitable for use in roadway embankments. In some instances, less than 5% organic content may be required. Classify organic soils as follows:

- **Organic Material:** O.C. > 5% but < 20%
- **Highly Organic Material:** O.C. ≥ 20 but < 75%

  Note: Highly organic materials are often referred to as "muck" in other FDOT documents.
- **Peat**: O.C. ≥ 75% (as defined in ASTM D4427)

6.1.1.9 Additional Descriptive Terms

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

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

6.1.1.10 Classification

Classify soils to permit relating the soil description to its behavior characteristics. All soils must be classified in accordance to one of the following two systems:

6.1.1.10.1 Unified Soil Classification System (USCS)

Use this system for all structural-related projects such as bridges, retaining walls, buildings, etc. Precise classification requires that a grain size analysis and Atterberg Limits tests be performed on the sample. The method is discussed in detail in ASTM D2487, and ***Figure 6-3***.

6.1.1.10.2 AASHTO Classification System

Use this system in conjunction with roadway soil surveys. Like the USCS, this system requires grain size analysis and Atterberg Limit tests for precise classification. Refer to AASHTO M 145 (ASTM D3282), and ***Figure 6-4***.

6.1.2 Rocks

Use the following format to describe rock and IGM: color, constituents, weathering, grain size, cementation, and additional descriptive terms. Refer to ***Table 6-2***.

Table 6-2 Description for Rock

Color	As with soils, limit the description to two predominant colors.
Constituents	The principal constituent is the rock type that makes up the major portion of the stratum being investigated. Due to rock formations encountered in Florida normally consisting of one rock type, the use of modifying constituents is generally not applicable. However, when more than one rock type is present in any given formation, include both in the description.
Weathering	Describe the degree of weathering. Classical classification systems do not apply to Florida rock.
Hardness	Classical hardness classification systems do not apply to Florida rock. Do not include subjective descriptions of rock hardness. Include only the objective indicators of the rock hardness (SPT N-values, excessive drilling time and down pressure, results of core testing, etc.)
Formation	Include the name of the geologic formation in parentheses after the description of the sample.
Additional Description Terms	Use any additional terms that will aid in describing the type and condition of the rock. Terms such as fossiliferous, friable, indurated, and micaceous are to be used where applicable.

6.2 Logging

Use the standard boring log as shown ***Figure 6-5***, or its equivalent as approved by the DGE for all borings and test pits. A sample completed log is shown in ***Figure 6-6***. Include the following information in the remarks column:

6.2.1 Comments on Drilling Procedures and/or Problems

Report any occurrences which may indicate characteristics of the in situ material such as obstructions, difficulties in drilling such as caving, flowing sands, caverns, loss of drilling fluid, falling drill rods, change in drilling method, and termination of boring above planned depth.

6.2.2 Test Results

Report results of tests performed on samples in the field such as pocket penetrometer or torvane tests. Record results of tests on in situ materials, such as field vane tests.

6.2.3 Rock Quality Designation (RQD)

In addition to the percent recovery, record the RQD for each core run as a percent. RQD is a modified core recovery best used on NX size core or larger (HW is the FDOT minimum size allowed). Refer to ***1.1*** for information on how to determine RQD.

The rock quality is described as follows:

Table 6-3 Rock Quality Description

RQD (%)	Description of Rock Quality
0 - 25	Very poor
25 - 50	Poor
50 - 75	Fair
75 – 90	Good
90 - 100	Excellent

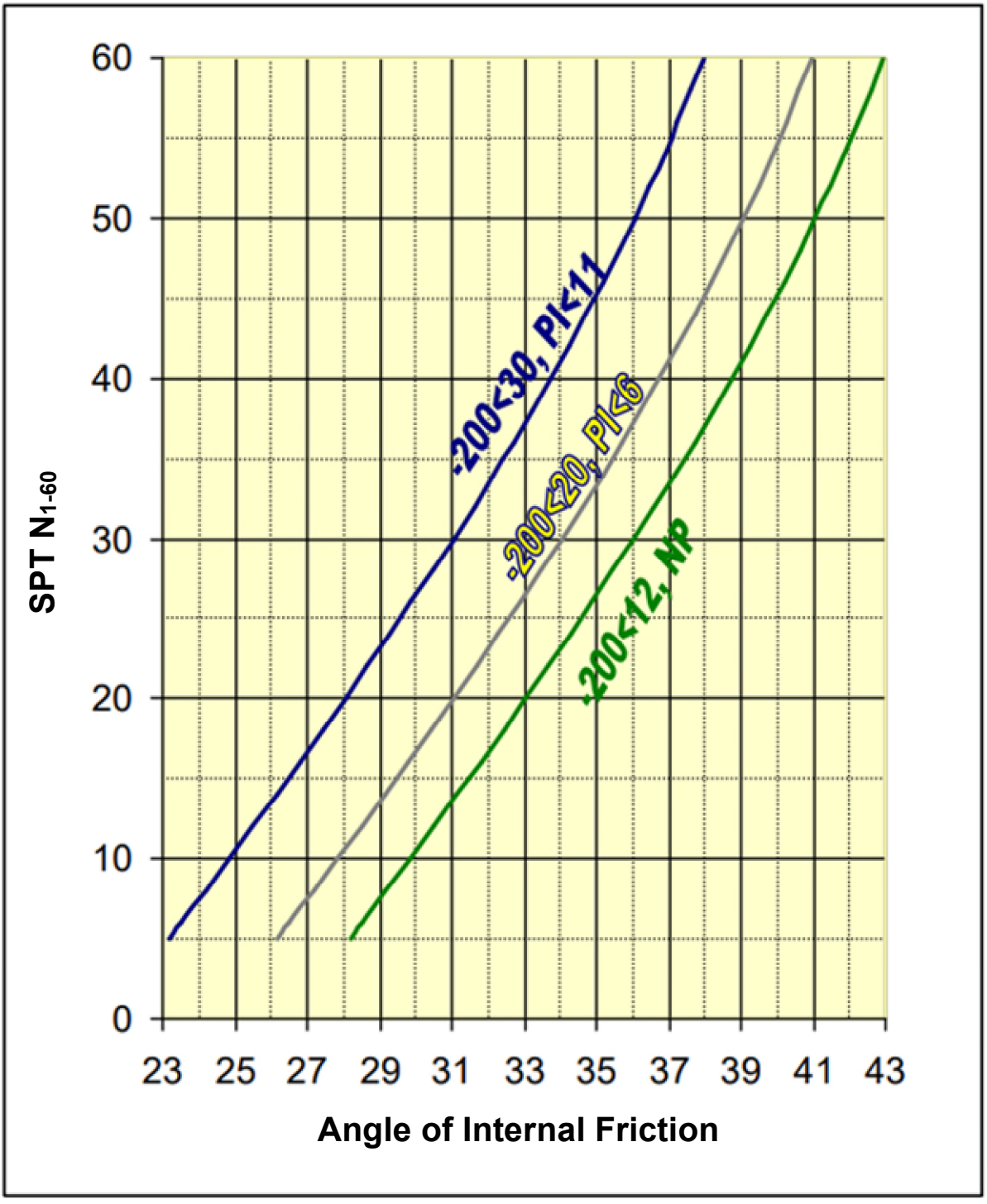


Figure 6-1 Angle of Internal Friction vs. SPT-$N_{1\text{-}60}$ (After Peck, 1974)

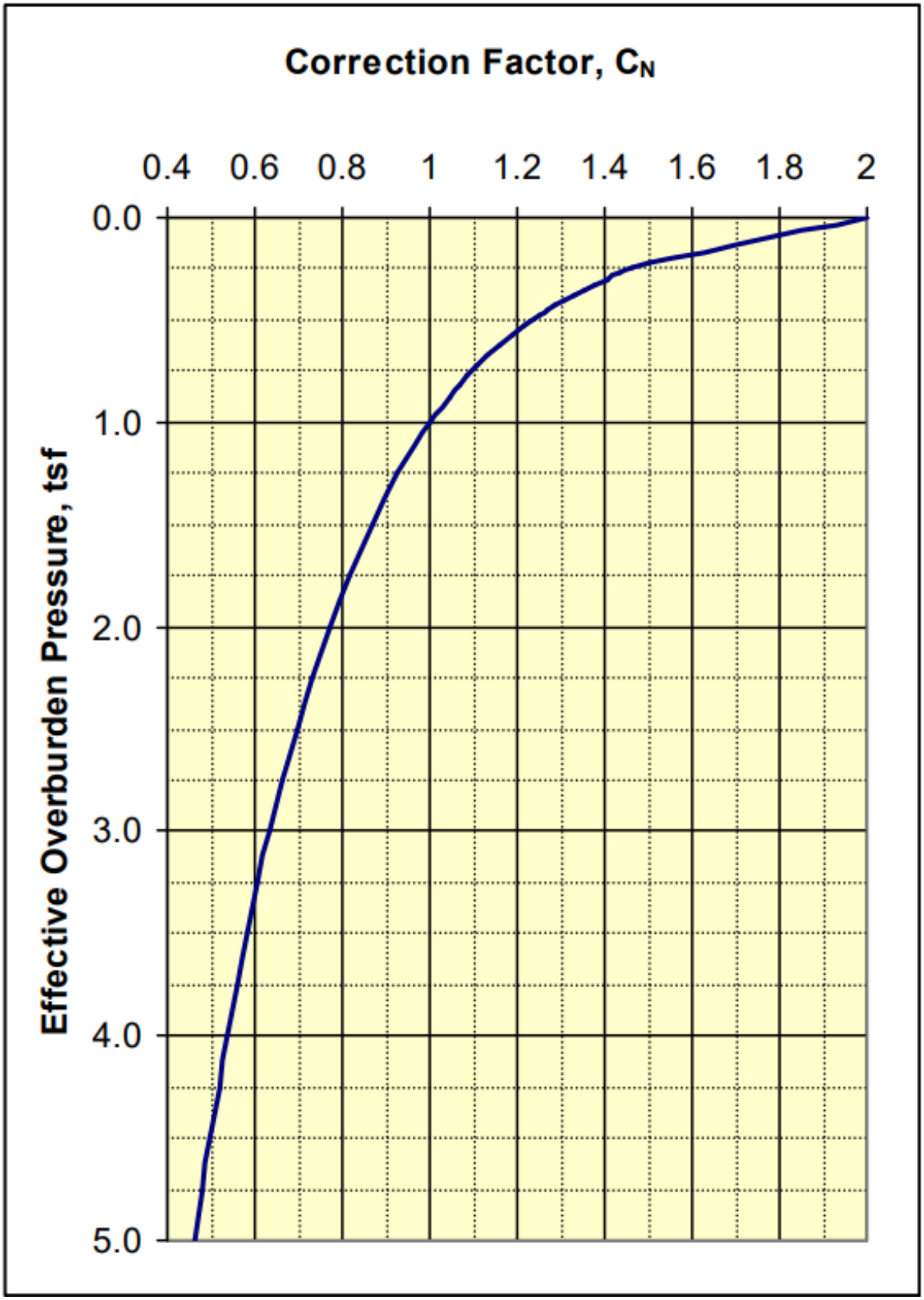


Figure 6-2 C_N vs. Effective Overburden Pressure (After Peck, 1974)

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[A]				Soil Classification	
				Group Symbol	Group Name[B]
COARSE-GRAINED SOILS	Gravels (More than 50 % of coarse fraction retained on No. 4 sieve)	Clean Gravels (Less than 5 % fines[C])	Cu ≥ 4.0 and 1 ≤ Cc ≤ 3.0[D]	GW	Well-graded gravel[E]
			Cu < 4.0 and/or [Cc < 1 or Cc > 3.0][D]	GP	Poorly graded gravel[E]
		Gravels with Fines (More than 12 % fines[C])	Fines classify as ML or MH	GM	Silty gravel[E,F,G]
			Fines classify as CL or CH	GC	Clayey gravel[E,F,G]
More than 50 % retained on No. 200 sieve	Sands (50 % or more of coarse fraction passes No. 4 sieve)	Clean Sands (Less than 5 % fines[H])	Cu ≥ 6.0 and 1.0 ≤ Cc ≤ 3.0[D]	SW	Well-graded sand[I]
			Cu < 6.0 and/or [Cc < 1.0 or Cc > 3.0][D]	SP	Poorly graded sand[I]
		Sands with Fines (More than 12 % fines[H])	Fines classify as ML or MH	SM	Silty sand[F,G,I]
			Fines classify as CL or CH	SC	Clayey sand[F,G,I]
FINE-GRAINED SOILS	Silts and Clays	inorganic	PI > 7 and plots on or above "A" line[J]	CL	Lean clay[K,L,M]
	Liquid limit less than 50		PI < 4 or plots below "A" line[J]	ML	Silt[K,L,M]
		organic	$\frac{\text{Liquid limit – oven dried}}{\text{Liquid limit – not dried}} < 0.75$	OL	Organic clay[K,L,M,N] / Organic silt[K,L,M,O]
50 % or more passes the No. 200 sieve	Silts and Clays	inorganic	PI plots on or above "A" line	CH	Fat clay[K,L,M]
	Liquid limit 50 or more		PI plots below "A" line	MH	Elastic silt[K,L,M]
		organic	$\frac{\text{Liquid limit – oven dried}}{\text{Liquid limit – not dried}} < 0.75$	OH	Organic clay[K,L,M,P] / Organic silt[K,L,M,Q]
HIGHLY ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A]Based on the material passing the 3-in. (75-mm) sieve.
[B]If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
[C]Gravels with 5 to 12 % fines require dual symbols:
GW-GM well-graded gravel with silt
GW-GC well-graded gravel with clay
GP-GM poorly graded gravel with silt
GP-GC poorly graded gravel with clay
[D]$Cu = D_{60}/D_{10}$ $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$
[E]If soil contains ≥15 % sand, add "with sand" to group name.
[F]If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.
[G]If fines are organic, add "with organic fines" to group name.
[H]Sands with 5 to 12 % fines require dual symbols:
SW-SM well-graded sand with silt
SW-SC well-graded sand with clay
SP-SM poorly graded sand with silt
SP-SC poorly graded sand with clay
[I]If soil contains ≥15 % gravel, add "with gravel" to group name.
[J]If Atterberg limits plot in hatched area, soil is a CL-ML, silty clay.
[K]If soil contains 15 to <30 % plus No. 200, add "with sand" or "with gravel," whichever is predominant.
[L]If soil contains ≥30 % plus No. 200, predominantly sand, add "sandy" to group name.
[M]If soil contains ≥30 % plus No. 200, predominantly gravel, add "gravelly" to group name.
[N]PI ≥ 4 and plots on or above "A" line.
[O]PI < 4 or plots below "A" line.
[P]PI plots on or above "A" line.
[Q]PI plots below "A" line.

Figure 6-3 Unified Soil Classification System (After ASTM D2487, 2017)

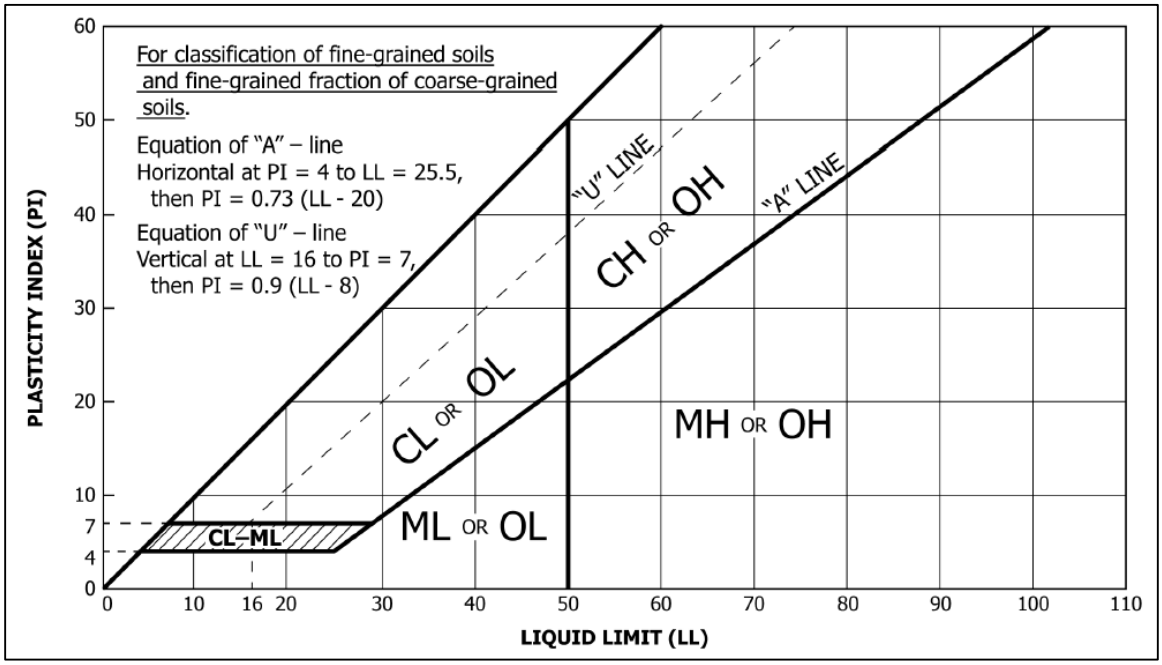


Figure 6-3 Continued

Table 1—Classification of Soils and Soil–Aggregate Mixtures

General Classification	Granular Materials (35 Percent or Less Passing 75 µm)			Silt–Clay Materials (More Than 35 Percent Passing 75 µm)			
Group Classification	A-1	A-3[a]	A-2	A-4	A-5	A-6	A-7
Sieve analysis, percent passing:							
2.00 mm (No. 10)	—	—	—	—	—	—	—
0.425 mm (No. 40)	50 max	51 min	—	—	—	—	—
75 µm (No. 200)	25 max	10 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 0.425 mm (No. 40):							
Liquid limit	—			40 max	41 min	40 max	41 min
Plasticity index	6 max	Nonplastic (NP)	[b]	10 max	10 max	11 min	11 min
General rating as subgrade	Excellent to Good			Fair to Poor			

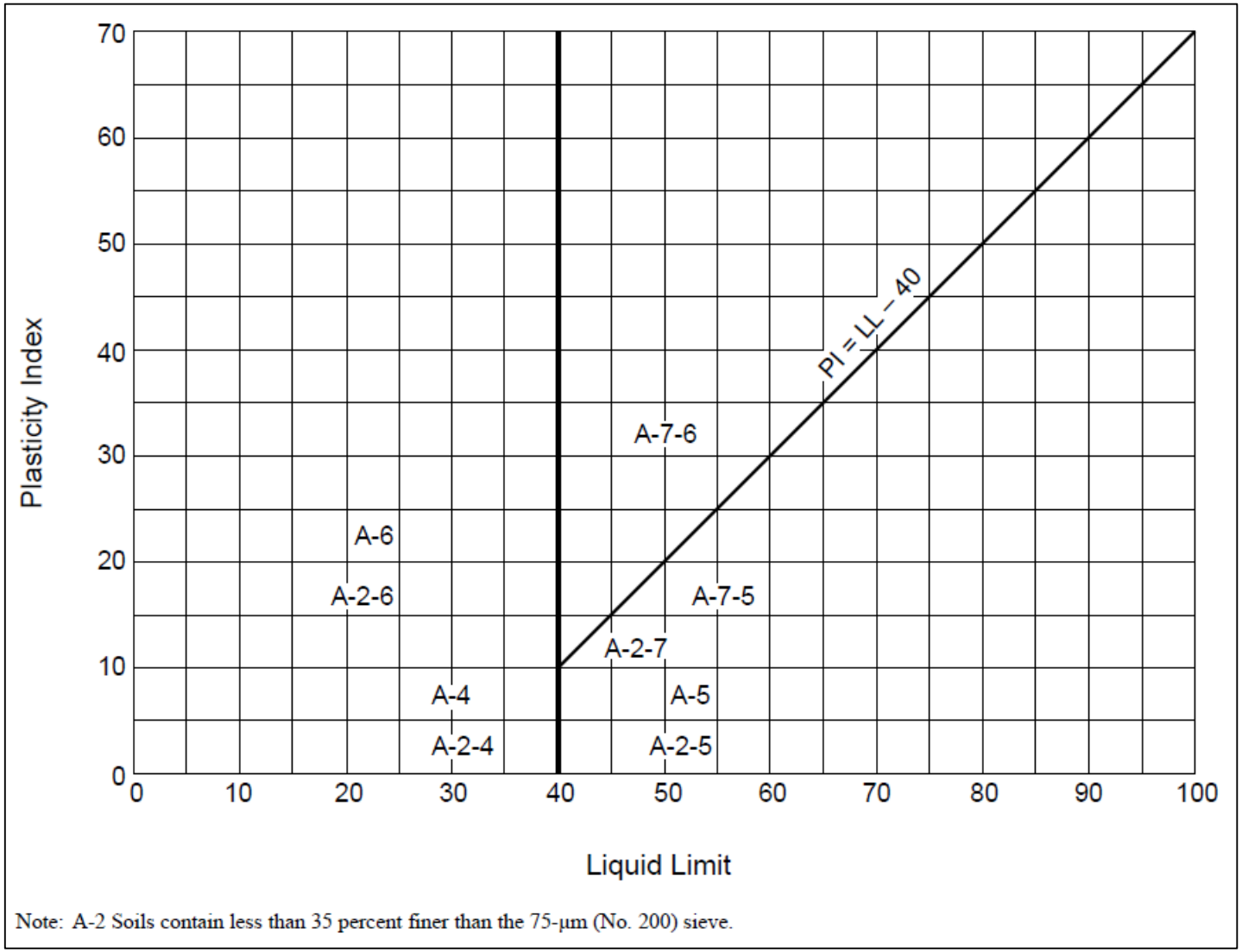
[a] The placing of A-3 before A-2 is necessary in the "left to right elimination process" and does not indicate superiority of A-3 over A-2.
[b] See Table 2 for values.

Table 2—Classification of Soils and Soil–Aggregate Mixtures

General Classification	Granular Materials (35 Percent or Less Passing 75 µm)							Silt–Clay Materials (More Than 35 Percent Passing 75 µm)			
	A-1			A-2							A-7
Group Classification	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7	A-4	A-5	A-6	A-7-5, A-7-6
Sieve analysis, percent passing:											
2.00 mm (No. 10)	50 max	—	—	—	—	—	—	—	—	—	—
0.425 mm (No. 40)	30 max	50 max	51 min	—	—	—	—	—	—	—	—
75 µm (No. 200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 0.425 mm (No. 40):											
Liquid limit	—		—	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity index	6 max		NP	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min[a]
Usual types of significant constituent materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as subgrade	Excellent to Good							Fair to Poor			

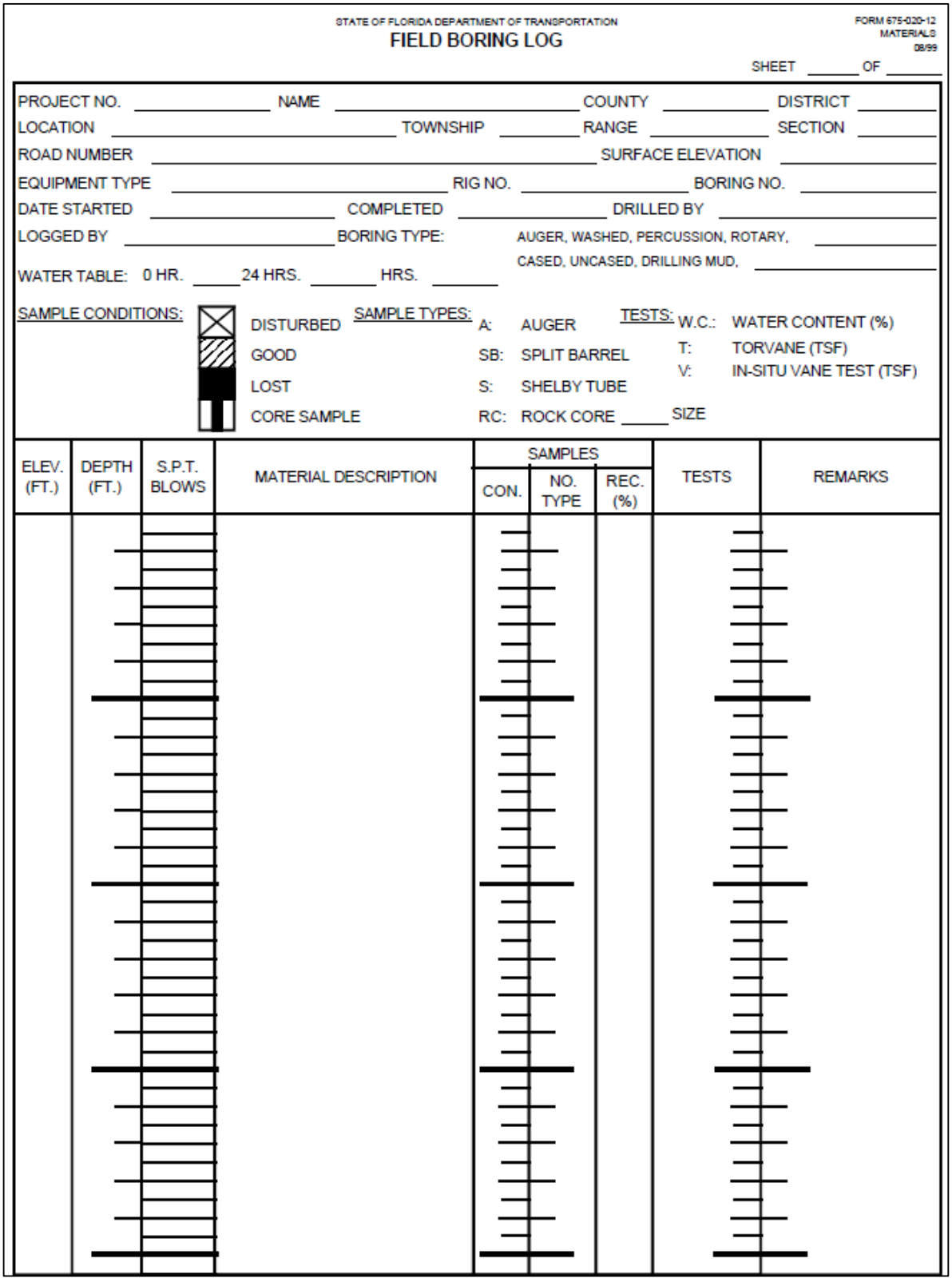
[a] Plasticity index of A-7-5 subgroup is equal to or less than $LL - 30$. Plasticity index of A-7-6 subgroup is greater than $LL - 30$. (See Figure 2.)

Figure 6-4 AASHTO Soil Classification System (AASHTO M 145, 2025)



Note: A-2 Soils contain less than 35 percent finer than the 75-µm (No. 200) sieve.

Figure 6-4 Continued



STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION

FIELD BORING LOG

FORM 675-020-12
MATERIALS
08/99

SHEET _____ OF _____

PROJECT NO. _____ NAME _____ COUNTY _____ DISTRICT _____

LOCATION _____ TOWNSHIP _____ RANGE _____ SECTION _____

ROAD NUMBER _____ SURFACE ELEVATION _____

EQUIPMENT TYPE _____ RIG NO. _____ BORING NO. _____

DATE STARTED _____ COMPLETED _____ DRILLED BY _____

LOGGED BY _____ BORING TYPE: AUGER, WASHED, PERCUSSION, ROTARY, _____

CASED, UNCASED, DRILLING MUD, _____

WATER TABLE: 0 HR. _____ 24 HRS. _____ HRS. _____

SAMPLE CONDITIONS:

DISTURBED

GOOD

LOST

CORE SAMPLE

SAMPLE TYPES:

A: AUGER

SB: SPLIT BARREL

S: SHELBY TUBE

RC: ROCK CORE _____ SIZE

TESTS:

W.C.: WATER CONTENT (%)

T: TORVANE (TSF)

V: IN-SITU VANE TEST (TSF)

ELEV. (FT.)	DEPTH (FT.)	S.P.T. BLOWS	MATERIAL DESCRIPTION	SAMPLES CON.	SAMPLES NO. TYPE	SAMPLES REC. (%)	TESTS	REMARKS

Figure 6-5 Field Boring Log Form

STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION
FIELD BORING LOG
FORM 675-020-12
MATERIALS - 05/94

SHEET 1 OF 1

PROJECT NO. 79100-1523 NAME SR-40 over Tomoka River COUNTY Volusia DISTRICT 5

LOCATION STA 14+80, 25 ft RT CL Survey TOWNSHIP 14S RANGE 31E SECTION 25

ROAD NUMBER SR-40 SURFACE ELEVATION +22.6 ft, NGVD

EQUIPMENT TYPE CME-45, Automatic Hammer RIG NO. 7476 BORING NO. 4

DATE STARTED 8/27/90 COMPLETED 8/28/90 DRILLED BY Jenkins

LOGGED BY Dawson BORING TYPE: (AUGER) WASHED, PERCUSSION, ROTARY,

WATER TABLE: 0 HR. 4.2 ft 24 HRS. 4.2 ft HRS. (CASED) UNCASED, DRILLING MUD, To 14.5 ft

SAMPLE CONDITIONS: DISTURBED, GOOD, LOST, CORE SAMPLE

SAMPLE TYPES: A: AUGER; SB: SPLIT BARREL; S: SHELBY TUBE; RC: ROCK CORE NX SIZE

TESTS: W.C.: WATER CONTENT (%); T: TORVANE (TSF); V: IN-SITU VANE TEST (TSF)

ELEV. (FT.)	DEPTH (FT.)	S.P.T. BLOWS	MATERIAL DESCRIPTION	SAMPLES CON.	SAMPLES NO. TYPE	SAMPLES REC. (%)	TESTS	REMARKS
22.6								
		2 3 5	Light Brown Fine SAND, Poorly Graded, Loose to Compact, Moist to Wet, Sub-Angular (SP)		SB-1	50		
		5 7 8			SB-2	60		
17.6	5	0 0 1	Dark Brown Sandy SILT, some Wood, Very Loose, Wet, Fibrous (Muck) (ML)		SB-3	20		Advanced 12" Under Weight of Hammer
		1 1 2	Reddish-Brown Silty CLAY, Trace Sand and Shell, Soft to Firm, Wet (CL)		SB-4	80		
12.6	10				S-1	100	T = 0.4	
		3 3 4			SB-5	100		
		12 16 25	Tan LIMESTONE, Highly to Moderately Weathered, Soft		SB-6	40		
7.6	15				RC-1	75		RQD = 38% Loss of Water at 17.8 ft
2.6	20	46 50/3"			SB-7	30		Boring Terminated at 20.5 ft Backfilled 8/28/90

Figure 6-6 Typical Boring Log

6.3 References

6-1. Cheney, R. S. & Chassie, R. G., Soils and Foundations Workshop Manual – Second Edition, FHWA HI-88-009, 1993.

6-2. Department of the Navy, ***NAVFAC DM-7.1 – Soil Mechanics***, Naval Facilities Engineering Command, 1982.

6-3. Mayne, P., Christopher, B., Dejong, J., ***Subsurface Investigations – Geotechnical Site Characterization***, FHWA-NHI-01-031, 2002.

6.4 Specifications and Standards

Subject	ASTM	AASHTO	FM
Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System)	D 2487	-	-
Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)	D 2488	-	-
Standard Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes	D 3282	M 145	-
Standard Guide for Field Logging of Subsurface Explorations of Soil and Rock	D 5434	-	-

Chapter 7 Field Instrumentation

7.1 Instrumentation

On projects where analysis has indicated potential problems with embankment or structure settlement or stability, construction must be monitored through the use of field instrumentation. Include the location of such instrumentation in the design of the foundation to assess the settlement rate and evaluate stability as construction proceeds. Determine the location of instrumentation and the minimum frequency of monitoring. Include in the design package special provisions and the hold points, time, or limitations of construction for the Contractor. Many of the special provisions are available from the DGE or SGE.

Use field instrumentation to provide data on existing structures or embankments.

Many field instruments may protrude above ground in the construction area and are susceptible to damage from construction equipment. Work with the Contractor to ensure the protection of these instruments.

The most common types of instrumentation are discussed in this chapter. See Reference ***7-2*** and ***7-3*** for more detail.

7.1.1 Inclinometers (Slope Indicators)

Use inclinometers to monitor embankment or cut slope stability. See ***Figure 7-1*** and refer to ASTM D6230.

Care must be taken when installing the casing to prevent spiraling of the casing that may result in the orientation of the grooves at depth being different than at the surface. Backfill the space between the borehole wall and the casing with a firm grout, sand, or gravel. For installation in highly compressible soils, use telescoping couplings to prevent damage of the casing.

To monitor embankment construction, place inclinometers at or near the toes of slopes of high-fill embankments where slope stability or lateral squeeze is considered a potential problem. Ensure casing penetrates the strata in which problems are anticipated. Take readings often during embankment construction. Halt fill operations if any sudden increase in movement rate is detected. Modify the applicable portions of the Technical Special Provision (TSP) ***T120, T141, T144, T442: Instrumented Surcharge Embankment with Wick Drains*** for site conditions, other usable pore-pressure transducer types, and instruments and include the TSP in the contract package.

7.1.2 Settlement Indicators

See ***Figure 7-2*** or ***Standard Plans***, Index 141-T01.

Place settlement platforms/plates at the points under the embankment where maximum settlement is predicted. Use reference datum benchmarks known to be stable and remote from all possible vertical movement. The use of multiple benchmarks is recommended. Record elevations before embankment construction begins and take readings periodically thereafter until the embankment and surcharge (if any) are completed. Plot the settlement data as a function of time.

Analyze the settlement data to determine when the rate of settlement has slowed sufficiently for construction to continue. Use TSP ***T141: Settlement Plates*** and modify the document for site conditions and include the TSP in the contract package.

7.1.3 Piezometers

Refer to AASHTO T 252 and ***Figure 7-3***.

Place piezometers prior to construction in the strata issues are most likely to develop. If the problem stratum is more than 10 feet (3 m) thick, place more than one piezometer at varying depths. Place any junction boxes outside the construction area if possible and protect the wire leads or pneumatic tubing from excessive strain due to settlements.

Check the pore water pressure often during embankment construction; monitoring frequency can be decreased after the fill is in place. Plot the data as pressure or feet (meters) of head as a function of time.

Commentary: Good practice is to plot pore water pressure, settlement, and embankment elevation on the same time-scale plot for comparison.

Modify TSP ***T144: Pore-Pressure Transducers (Piezometers)*** for site conditions and include the TSP in the contract package.

7.1.4 Tiltmeters

Use tiltmeters to measure the inclination of structures such as tilting of bridge abutments and decks or retaining walls, and to monitor rotational failure surfaces.

7.1.5 Monitoring Wells

Install monitoring wells in conjunction with piezometers to provide a base reference necessary for calculating changes in pore pressure. Place the monitoring well in an unimpacted area of construction to reflect the true static water table elevation. Installation and decommissioning of monitoring wells must be in accordance with local DEP and Water Management District rules and regulations.

7.1.6 Vibration Monitoring

Monitor ground vibrations induced by blasting, pile driving, construction equipment, or traffic when construction is in close proximity to sensitive structures or equipment, which may become damaged if subjected to excessive vibration. When existing bridge structures are within the monitoring distance outlined by the Specifications, ensure that the monitoring protocol requires taking readings of substructure movement. Meet the requirements of ***FDM*** Chapter 117 and ***Standard Specifications*** Section 108. For sensitive sites that warrant a reduction to the peak particle velocity limit outlined in the Specifications, a Modified Special Provision (MSP) is required.

7.1.7 Special Instrumentation

Employ other special instrumentation such as earth pressure cells and strain gauges when required. Experienced personnel is required to install and interpret the data. Consult the SMO for assistance.

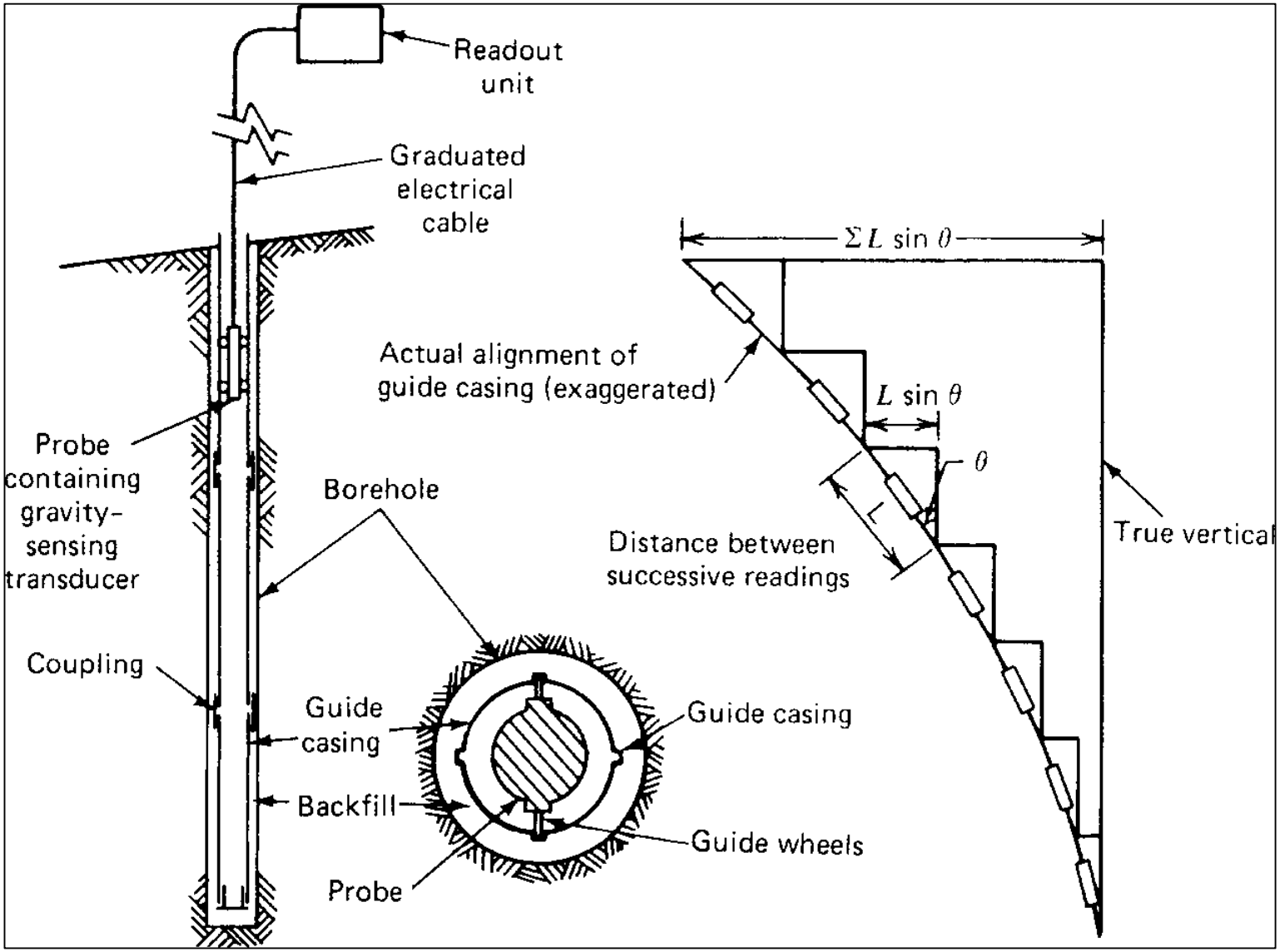


Figure 7-1 Principle of Inclinometer Operation (After Dunnicliff, 1988)

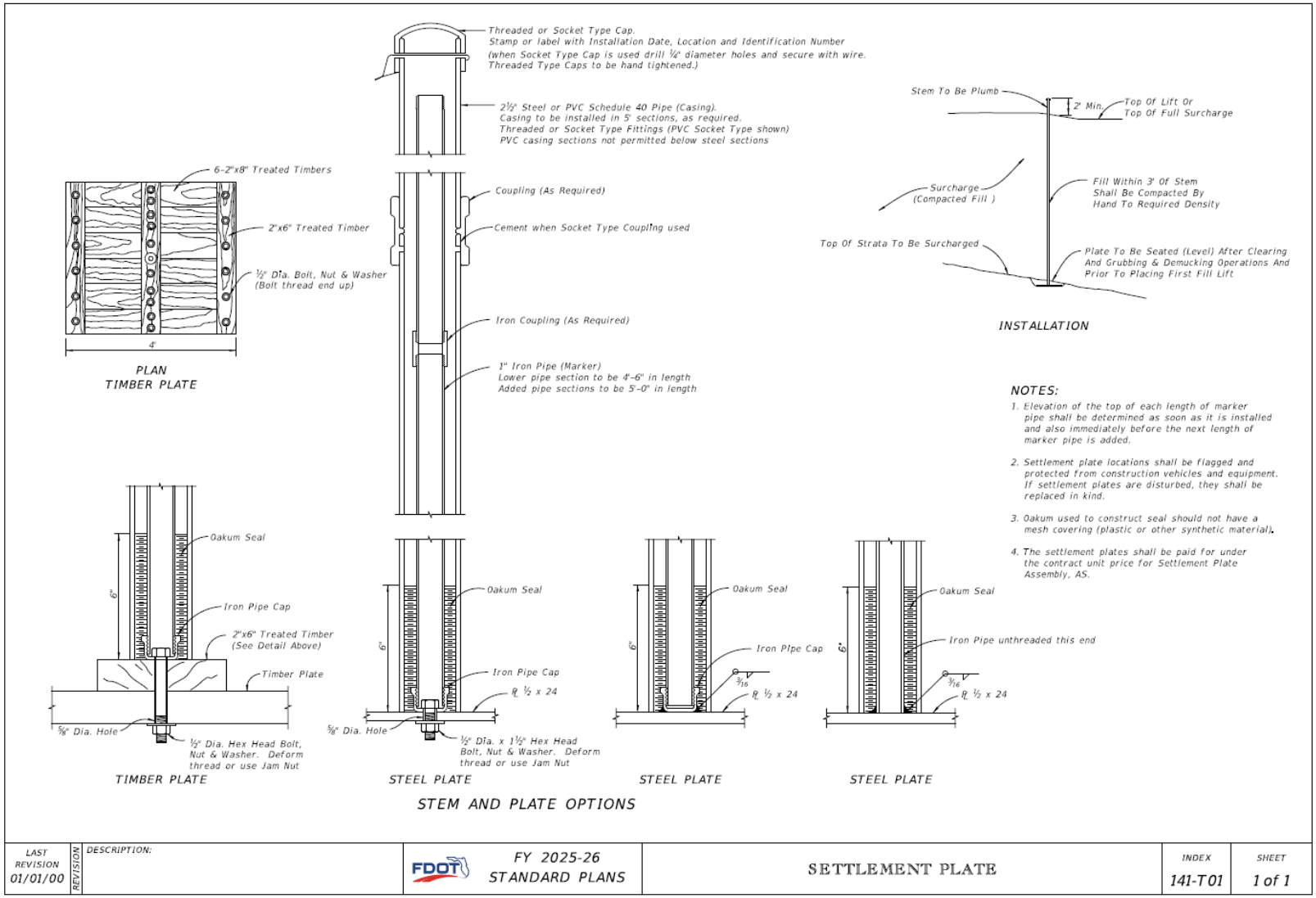


Figure 7-2 Typical Settlement Platform Design (Standard Plans, Index 141-T01)

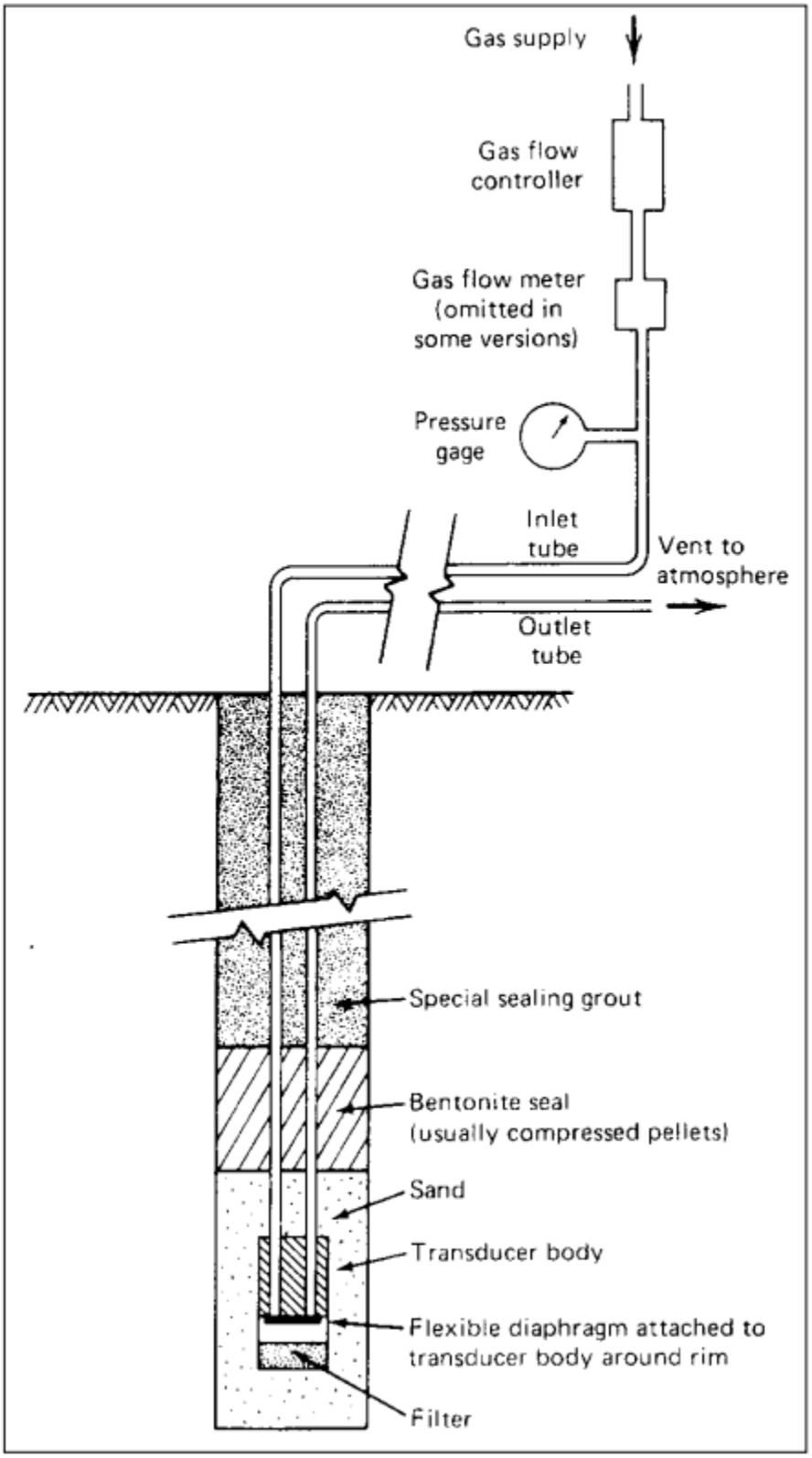


Figure 7-3 Example Schematic of Pneumatic Pore Pressure Transducer

7.2 References

7-1. Cheney, R. S. & Chassie, R. G., Soils and Foundations Workshop Manual – Second Edition, FHWA HI-88-009, 1993.

7-2. Dunnicliff, J., Geotechnical Instrumentation for Monitoring Field Performance, Wiley-Interscience, New York, 1993.

7-3. Dunnicliff, J., ***Geotechnical Instrumentation***, FHWA-HI-98-034, 1998.

7-4. Florida Department of Transportation, ***FDOT Design Manual***, (Current version).

7-5. Florida Department of Transportation, ***Standard Plans for Road and Bridge Construction***, (Current version).

7.3 Specifications and Standards

Subject	ASTM	AASHTO	FM	STD. INDEX
Settlement Platform	-	-	-	540/141-T01
Standard Test Method for Measurements of Pore Pressures in Soils	-	T 252	-	-
Standard Test Method for Monitoring Ground Movement Using Probe-Type Inclinometers	D6230	-	-	-

Chapter 8 Analysis and Design

This chapter discusses the major factors to consider during the analysis and design phase and references some typical methods of solving potential problems. ***Table 8-4*** presents FHWA guidelines for analyses to be performed. A list of computer programs approved for use by the Department to aid analysis are available on the ***Geotechnical Engineering webpage.***

8.1 Roadway

Analyze the results of the soil survey explorations to determine the suitability of in situ materials for use as roadway embankment. Embankment materials must comply with ***Standard Plans*** Index 120-001.

Classify and stratify the subsurface materials identified during soil survey explorations in accordance with the AASHTO classification system.

Soils must be stratified such that similar soils are contained within the same stratum. Stratifications must be based on the material removal and utilization requirements of ***Standard Plans*** Indexes 120-001 and 120-002. If testing identifies dissimilar types within the same stratum, additional sampling and testing may be required to better define or re-stratify the in situ materials.

Once stratified, analyze each stratum to define characteristics that may affect the design such as those outlined below.

8.1.1 Limits of Unsuitable Materials

Define the limits of all in situ materials considered unsuitable for pavement embankments and assess the effect of each material on roadway performance. Refer to ***Standard Plans*** Index 120-002 for requirements on excavation and replacement of these materials. Where complete excavation is not feasible, consider surcharging, column supported embankment, stabilization with lime, cement, or fly ash, placement of geotextile, and combinations of these and other methods.

8.1.2 Limerock Bearing Ratio (LBR) for Design

When LBR is permitted by the SMO for use in design, determine the LBR value based on test results and the stratification of subsurface materials. The design value must be representative of actual field conditions. Apply the below two methods to the LBR test data to account for variability in materials, moisture contents, and field versus laboratory conditions. The design LBR is the lower of the values determined by the two methods.

8.1.2.1 ±2% of Optimum Method

For this method, determine the design LBR by averaging the LBR values corresponding to moisture contents 2% above and 2% below the moisture content of the maximum LBR value (***Table 8-5***).

8.1.2.2 90% Method

For this method, sort the maximum LBR values into ascending or descending order. For each value, calculate the percentage of values which are equal to or greater than that value. Plot the percentages versus the maximum LBR values. The design value is the LBR value corresponding to 90% on the plot. Refer to ***Figure 8-2***.

8.1.3 Resilient Modulus (M_R)

Determine the resilient modulus directly from laboratory testing (AASHTO T 307). For roadway embankment materials, choose a design resilient modulus based on test results at 11 psi bulk stress and the stratification of subsurface materials. The design value must be representative of actual field conditions.

Apply the following method to the M_R test data to account for variability in materials and to provide for an optimum pavement design (see the ***Flexible Pavement Design Manual*** Section 5.2.3 and Reference ***8-50***):

8.1.3.1 90% M_R Method

Sort the M_R values obtained from AASHTO T 307 at 11 psi bulk stress into descending order. For each value, calculate the percentage of values which are equal to or greater than that value. Plot these percentages versus the M_R values. The design M_R value is the value corresponding to 90% on the plot.

8.1.4 Corrosivity

Review the results of field and/or laboratory tests and assess the potential for corrosion of the various structure foundation and drainage system components.

8.1.5 Drainage

Estimate the permeability and infiltration rate of embankment materials and stormwater pond subsurface soils based on test results. Provide this data along with groundwater depth and recommended aquifer parameters such as soil permeability and infiltration rate, seasonal high groundwater level, and aquifer bottom elevation, to the drainage engineer for assessment of the need for and for the design of drainage systems, including pavement underdrains and retention, detention, and infiltration ponds.

8.1.6 Earthwork Factors

Estimate truck and fill adjustment factors used in estimating earthwork quantities based on local experience. Refer to **Borrow Excavation (Truck Measure)** in ***FDM*** for example calculations using these factors.

8.1.7 Other Considerations

Assess any other characteristics which can be detected from soil survey explorations and can affect the roadway design including expansive soils, springs, sinkholes (see References ***8-28*** and ***8-37*** for sinkhole issues), potential grading problems due to the presence of rock, etc.

8.2 Structure Foundations

For Conventional projects, consider Geosynthetic Reinforced Soil (GRS) abutments, spread footings, driven piles, auger cast piles, and drilled shafts as potential foundation types during the preliminary or BDR phase for each bridge structure. For noise barrier walls, ACP may be the preferred foundation. Analyze the design capacity based on SPT and/or CPT 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 requirements of the applicable edition of the *AASHTO Code* except where specific requirements have been superseded by ***SDG*** or those contained herein.

Provide a range of total and differential settlement estimates – both within individual foundation units and between piers/bents – that reflect the variability captured in the data collected for the various layers affected by the anticipated structural loads. Pay particular attention 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 that are likely to be affected by construction activities. Consider the effects of construction of the proposed foundation type(s) on the integrity and stability of existing structures. 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 during the driving of a displacement pile that advances in very close proximity. Pay particular attention to existing battered piles and piles that are not tipped in a competent bearing layer.

c. Drilled shafts constructed using vibratory methods for the casing installation and removal, in proximity to 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.

8.2.1 Spread Footings

8.2.1.1 Design Procedure

See References ***8-14***, ***8-18***, ***8-27, 8-36***, and ***8-43*** for design procedure 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, check against punching shear failure into a weaker stratum underlying the bearing layer (see ***8.3.1***). Evaluate the effect of excavation aids such as continuous sheet pile which could compromise the continuity of the bearing layer (see ***8.3.7***).

8.2.1.2 Considerations

Consider varying depths of footings to achieve the maximum economic design. For water crossings, depth of scour will be a controlling factor, which may preclude consideration of spread footings. Address difficult conditions for dewatering and preparation of foundation soils 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 may be more economical than deep foundations.

8.2.2 Driven Piles

Design driven piles for axial and lateral loading conditions as applicable. The following types of driven piles are acceptable for supporting structural loads on permanent FDOT structures depending on environmental restrictions: Steel H-piles, Steel Pipe Piles, Prestressed Concrete Piles 18-inch square and larger, and 54-inch or 60-inch diameter Concrete Cylinder Piles. 14-inch square Prestressed Concrete Piles may be used for pedestrian bridges if there are no environmental restrictions. Timber piles may be used for temporary bridges. Other pile types and sizes may be considered for non-conventional projects and contractor's CSI submittals.

8.2.2.1 Design Procedure

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

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

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

For foundations tipped on rock or IGM, check that the bearing layer thickness below the tip elevation is sufficient to prevent punching shear failure into a weaker stratum underlying the bearing layer (refer to ***8.3.1***). Address pile group effects, settlement, and downdrag as applicable.

*Note: The AASHTO Code and References **8-8, 8-19**, and **8-29** are recommended for analyzing group effects and settlement potential. See **Appendix C** for a step-by-step design procedure for the analysis of downdrag and drag loads.*

8.2.2.2 Preformed Pile Holes

Preformed pile holes serve as a penetration aid when all other pile installation methods fail to achieve the minimum tip elevation required or to minimize the effects of vibrations on adjacent structures. Anticipate the need for preformed pile holes and include directions in the Geotechnical Report and the Plans.

8.2.2.3 Test Piles

Use test piles to estimate the length of production piles, determine driving resistance characteristics of various soil/rock strata, evaluate the effort required to obtain the minimum required pile penetration, check the ability of the driving system to drive the pile, establish the driving procedures, and confirm the limiting stresses are not exceeded. Drive test piles harder (within the limits of practical refusal), deeper, and to a greater bearing resistance than required for production piling. Include test piles on all projects unless approved otherwise by the DGE.

Modification for Non-Conventional Projects:
Delete the above section and insert the following:
Use test piles to estimate the length of production piles, determine driving resistance characteristics of various soil/rock strata, evaluate the effort required to obtain the minimum required pile penetration, check the ability of the driving system to drive the pile, establish the driving procedures, and confirm the limiting stresses are not exceeded. Drive test piles harder (within the limits of practical refusal), deeper, and to a greater bearing resistance than required for production piling. Include test piles on all projects.

8.2.2.4 Considerations

Analyze various pile types and sizes to achieve an optimum design. For water crossings, consider the depth of scour for both axial and lateral load analyses. Include recommendations for test pile locations and address the need for static load testing. Consider the drivability of the piles. Refer to ***SDG*** for driving limits for different pile sizes. In FB-Deep and GeoStat analyses, code sand layers containing 30% (“some” per ASTM D2488) or greater quantities of shell as soil type 4.

Steel pipe piles may be used in extremely aggressive conditions only if driven closed-end and filled with a cast-in-place concrete core in accordance with ***SDG*** 3.1(G)(2) (See ***SDG*** 3.1(G) and ***SDG*** Table 3.1-1 for additional information).

8.2.3 Drilled Shafts

Design drilled shafts considering both axial and lateral loads. Address during design the proximity of drilled shafts to embankments, retaining walls, or other sources of stress increase that could affect the stability of the excavation during construction and include the need for temporary and/or permanent casing.

8.2.3.1 Design Procedure for Major Structures

Resistance factors and associated design methods for geotechnical resistance of drilled

shafts are in ***SDG*** Table 3.6.4-1. It is implicit 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: The beta method for side shear resistance in sand refers to the method developed by O'Neil & Reese (Reference **8-38**), 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 ***8-38*** 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. Use a spacing of 3D for drilled shafts. 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 drilled shafts tipped in other materials refer to the current *AASHTO Code*. P-y multipliers for lateral loads are in ***SDG***. General foundation analysis considerations are further described below.

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

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

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

Nonredundant drilled shaft bridge foundations have the following special design requirements:

1. All drilled shafts in nonredundant bridge foundations must 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.

   Note: There is often sufficient horizontal reaction to resist moments in drilled shafts embedded at least seven shaft diameters below the design ground surface.

Analyze various drilled shaft sizes to achieve an optimum design. For water crossings, consider the depth of scour. Consider any anticipated construction problems. Address the method of construction (dry, slurry, or casing) as this will affect the side friction and end bearing values assumed during design. Analyze and present both the unit side friction and mobilized end bearing values.

*Note: References **8-8**, **8-12**, and the AASHTO Code are recommended for analyzing*

*group effects. See **Appendix C** for a step-by-step design procedure for the analysis of downdrag*.

For foundations tipped on rock or IGM, ensure the bearing layer thickness below the tip elevation is sufficient to prevent punching failure into weaker stratum below the bearing stratum for the end bearing resistance included in the design (see ***8.3.1***).

The minimum rock socket length must be eight (8) feet or 1.5 times the shaft diameter, whichever is longer, to account for geologic variability. When the total socket length must be broken into layers, each layer must be at least one shaft diameter.

8.2.3.2 Considerations

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

In cohesionless materials, consider drilled shafts with pressure grouted tips. Pressure grouted tips are most effective in loose to medium dense sands. Refer to ***Appendix D*** and Reference ***8-38*** for guidance on the design of drilled shafts with pressure grouted tips.

Specify load tests on test shafts when necessary to verify capacity and/or constructability. Reinforced method shafts (test holes) are required for bridges and their locations must be specified in the Plans. Do not perform load tests on method shafts. Method shafts must be as deep as the deepest shafts on the project, whereas the load test shafts verify the resistance of the most economical bearing zone. Refer to ***SDG*** for additional considerations.

Drilled shafts constructed using temporary or permanent casing must consider the reduced side shear resistance.

All resistance must be strain compatible.

*Commentary: Peak side shear in rock will normally occur well before peak side shear in soil. Consider the difference in the deformation required to mobilize skin friction in soil and rock versus end bearing. See Reference **8-38** and the AASHTO Code.*

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

8.2.3.3 Design Procedure for Miscellaneous Structures

Design drilled shafts for miscellaneous structures considering both axial and lateral loads, however the design for lateral loads will normally govern. When designing, consider the groundwater level at the ground surface. When drilled shafts for miscellaneous structures will be founded in limestone, the guidelines in ***Appendix B*** for rock may be used. An example lateral load analysis using Broms' Method for a cable barrier end terminal is presented in ***Appendix G.***

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

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

Design ACP considering both axial and lateral loads. Lateral loads typically govern when ACP are used for noise wall foundations. See ***SDG*** for restrictions on the use of ACP for bridges and other structures.

8.2.4.1 Design Procedure

Design ACP for bridges (when allowed) using the same design procedures as for side shear resistance of drilled shafts. For side shear resistance of rock or cohesive IGM materials, use the design procedures outlined in ***Appendix A.*** Unit side shear values must be strain compatible; in particular for auger cast pile bridge foundations. For design of rock or IGM socketed auger cast piles supporting bridges, neglect the side shear resistance from the overburden soil unless strain compatible values are determined by site specific load tests.

Generic designs for noise barrier wall foundations on level ground are presented in the ***Standard Plans***. When walls are founded through compacted select fill berms, 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.

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, design ACP in accordance with the procedure outlined in ***Appendix B.*** Consult the DGE for local guidelines regarding ACP.

8.2.4.2 Considerations

Considerations for ACP supporting precast noise walls are presented in the ***SPI*** for Index 534-200.

8.2.5 Micropiles

In special cases such as low overhead, vibration sensitive sites, zones with utility conflicts or retrofit, micropiles may be the preferred foundation system. See ***SDG*** for restrictions on the use of micropiles for bridges and other structures. When used for bridge support in rock-socket applications, ***Appendix A*** must be used for design.

8.2.5.1 Design Procedure

Designs must comply with Section 10.9 of the *AASHTO Code* for soil and ***Appendix A*** for rock and IGM. However, disregard all side shear resistance in the casing plunge length. Static Load Tests are required to verify the design.

*Note: Reference **8-6** and the AASHTO Code are recommended for background information.*

8.2.6 GRS Abutments

GRS abutments are part of FHWA's Every Day Counts initiative to reduce bridge construction time and cost. (Reference ***8-2***). GRS abutments are permitted for simply supported spans as described in the ***Structures Manual***. For additional background, see References ***8-2*** and ***8-3***.

8.2.6.1 Design Procedure

Designs must comply with Appendix C of Reference ***8-2***, except as otherwise indicated in sections 3.12.7 and 3.13.4 of ***SDG***.

Present GRS abutments in the Plans. The Plans may or may not utilize ***Developmental Standard Plans*** Index D549-025, however, the same information needs to be presented. GRS abutments shall be constructed using ***Developmental Specification*** 549. Request the Developmental Specification from the District Specifications Office to incorporate into the specifications package.

8.2.6.2 Considerations

Limitations and considerations are presented in Sections 3.12.7 and 3.13.4 of ***SDG*** and in the Instructions for ***Developmental Standard Plans*** Index D549-025.

8.3 Foundation Analysis

Along with an axial analysis outlined in the previous section, address the following factors in the Geotechnical Report for deep foundations:

8.3.1 Punching Shear

For shallow foundations and the end bearing component of deep foundations supported on layered profiles where limestone or IGM bearing materials are underlain by weaker materials – such as those depicted in ***Figure 8-3*** – ensure the bearing layer thickness below the bearing elevation is sufficient to prevent punching failure into a weaker stratum below the bearing stratum. Perform this check as part of the bearing analysis for the strength limit state. For spread footings, use a trapezoidal pressure distribution.

Because the Rock Mass Rating (RMR) and Geological Strength Index (GSI) methods in the *AASHTO Code* are unreliable for nearly all Florida limestone and IGM materials, estimate the shear resistance within the limestone and IGM lenses using the method outlined in ***Appendix A*** for determining ultimate side shear resistance, f_{su}. Limit the sample set to the borings closest to each foundation to best estimate the bearing conditions.

*Commentary: The McVay method applied in **Appendix A** is based on the shaft socket interface being sufficiently rough that the failure surface is entirely within the rock or IGM in which the shaft is socketed. Therefore. f_{su} is the rock shear strength. For details see Reference **8-35**. See Reference **8-36** for a discussion of the applicability of RMR and GSI to Florida limestone.*

When the limestone/IGM material has not been cored and tested, the shear resistance of the material below the tip elevation may be estimated using SPT N-values using the

following equation:

$$q_{frac} = 0.1 \times N_{60}\,(tsf) \leq 4\,tsf$$

The resistance factor, ϕ, for this check is taken from Table 3.6.4-1 of ***SDG*** as appropriate for redundant or nonredundant drilled shafts. For piles, use the resistance factor for redundant shafts including end bearing from Table 3.6.4-1 of ***SDG***. For spread footings, use the resistance factors in the *AASHTO Code* (Table 10.5.5.2.2-1).

8.3.2 Lateral Loads

Provide soil parameters to the Structural Engineer for lateral load analyses for deep foundations on all retaining structures and all bridges permitting navigation. Check the final lateral load analysis for correct soil property application. The associated minimum tip elevations requirement (as outlined is ***SDG*** 3.5.9 and 3.6.5) must be reviewed. Designs may need to be changed if lateral deflection is excessive. See Reference ***8-41***.

8.3.3 Soil and Rock Moduli

When laboratory testing of rock specimens is used to determine Young's modulus, the results must be corrected to reflect the anticipated response in the field (i.e., from intact to mass modulus; see Reference ***8-32***). For cast-in-place bridge foundations in rock and IGM, SPT-N correlations may be used for preliminary assessments only; q_u, and q_t must be obtained from laboratory testing.

Direct measurements may be used when sufficient testing is performed, in coordination with the DGE. The correlations below provide an upper bound when using SPT N-values or other index properties for soil and rock as applicable.

8.3.3.1 Young's Modulus, E

For clean cohesionless soils (After Cosentino, 2025):

$$E = 50 \times N_{60}\,(ksf)$$

For sands with fines, refer to the correlations in the *AASHTO Code*.

For cohesive soils (BSI):

$$E = \beta \times s_u\,(psf)$$

Where:

β = range of beta as shown in the table below (from FB-MultiPier Help Manual)

s_u = undrained shear strength (psf)

Table 8-1 Range of Beta for Clay (from FB-MultiPier's Help Manual)

Plasticity Index	β				
	OCR = 1	OCR = 2	OCR = 3	OCR = 4	OCR = 5
< 30	1500 – 600	1380 – 500	1200 – 580	920 – 380	730 – 300
30 to 50	600 – 300	550 – 270	580 – 220	380 – 180	300 – 150
> 50	300 – 150	270 - 120	220 – 100	180 – 90	150 – 75

8.3.3.2 Mass Modulus, E_m

For rock and Cohesive IGM:

$$E_m = 115 \times q_u$$

Where:

$$q_u = 0.8 \times N_{60}\ (ksf)^*$$

**Modified from McMahan and McVay*

8.3.3.3 Shear Modulus, G

For all materials:

$$G = \frac{E}{2(1+v)}$$

Where:

v = Poisson's ratio (unitless)

Note: The upper limit for the shear modulus, G, must not exceed 10 ksi unless site specific data for the layer analyzed warrants a higher value.

8.3.3.4 Subgrade Modulus, k

For cohesionless soils:

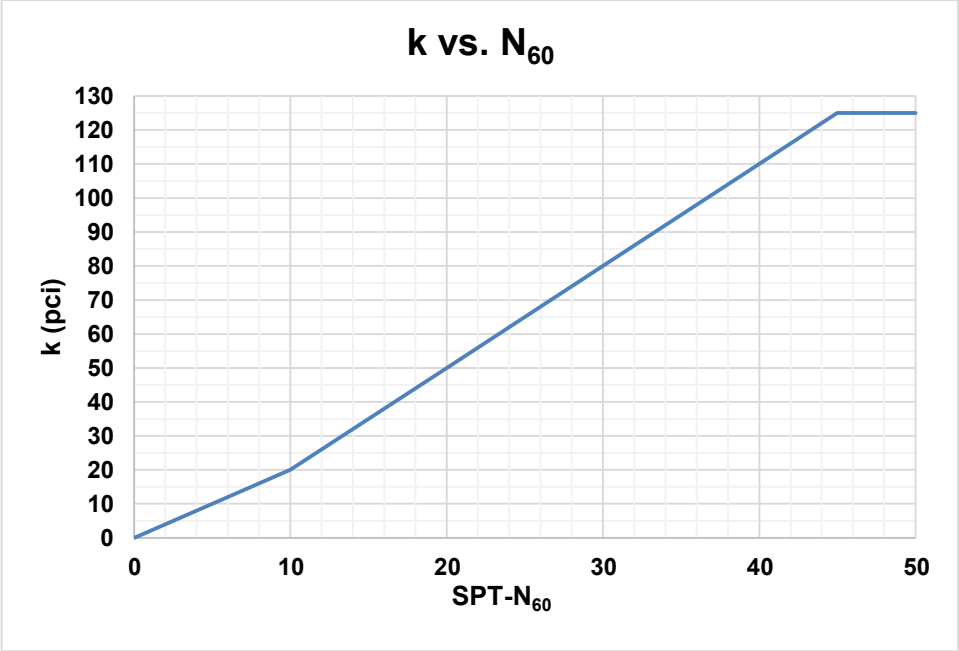


Figure 8-1 Subgrade Modulus for Cohesionless Soils

For cohesive soils:

Table 8-2 Subgrade Modulus, k, for Cohesive Soils based on s_u (after COM624P, 1993)

	Average Undrained Shear Strength, s_u (tsf)*		
	0.5 – 1	1 – 2	2 – 4
k_{static} (pci)	500	1000	2000
$k_{dynamic}$ (pci)	200	400	800

*Computed from the shear strength of the soil to a depth of 5 piles diameters

For cohesive soils, design values for s_u must not exceed 1 tsf unless laboratory stress-strain measurements indicate otherwise.

8.3.4 Scour

For structures over water where scour susceptibility controls the design, coordinate with the Hydraulics Engineer and the Structures Engineer as described in ***SDG***.

8.3.5 Downdrag and Drag Load

Refer to ***1.1*** for the definition of downdrag and drag load. The potential detrimental effect of drag loads can be either excessive settlement or overstressing if the foundation element is tipped or socketed in unyielding competent material.

- Consider drag load at the *geotechnical service limit state* for driven piles embedded and tipped in granular cohesionless soil and cohesionless IGM.

Compute drag load in accordance with ***Appendix C*** steps 1 through 5. Perform settlement calculations under a service limit state load combination. Coordinate with the structural designer to ensure differential settlement between interior piers/bents and end-bents is acceptable.

- Consider drag load at the *geotechnical strength limit state* for all other cases. Follow the procedure outlined in ***Appendix C***.
- The structural strength limit state is evaluated by the structural designer in accordance with the *AASHTO Code*.

Provide the anticipated drag load(s) in the geotechnical report and include recommended methods to reduce the effect of drag forces. See ***Appendix C*** or Reference ***8-10*** for a step-by-step design procedure for the analysis of drag loads.

To minimize drag loads, consider: (a) placing the embankment fill and allowing the compressible soil(s) to consolidate prior to foundation installation, (b) perform set-checks after embankment construction to induce downward displacement of the pile and reduce or eliminate drag loads, provided pile resistance is maintained or improved, (c) implement a ground improvement program to minimize embankment settlement.

8.3.6 Construction Requirements

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

8.3.7 Cofferdams and 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 within a distance of 5D to temporary cofferdams or sheet piles (References ***8-15*** and ***8-16***). Extraction of the cofferdam or sheet pile (particularly vibratory extraction) may 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. If seepage is not an issue, use braced sheets with short penetrations.
2. Use preformed pile holes to reduce resistance above the tip elevation of the cofferdam or sheet pile and ensure Nominal Bearing Resistance (NBR) is achieved from only the soil and/or rock below the tip elevation of the cofferdam or sheet pile.
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.
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 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.
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.
8. Consider seepage flow and hydrostatic pressure in determining the seal thickness and sheet pile penetration depth.

8.3.7.1 Correction Factors for Sheet Pile Proximity in Cohesionless Soil

Apply the following correction factors to the Nominal Bearing Resistance (NBR) for driven piles embedded and tipped in cohesionless soil. Larger correction factors may be applicable where required by the prevailing subsurface conditions, construction sequence, or anticipated sheet pile installation methods.

Table 8-3 Correction Factor for Sheet Pile Proximity

Correction Factor for Sheet Pile Proximity (CFSP)*					
	Distance From Edge of Pile to Edge of Sheet Pile				
Embedment Ratio**	1D	2D	3D	4D	5D
0.25	1.20	1.15	1.10	1.10	1.05
0.50	1.40	1.15	1.10	1.10	1.05
0.75	1.60	1.36	1.25	1.25	1.20
1.00	1.90	1.60	1.50	1.47	1.45

*Applied to NBR
**Ratio of sheet pile to pile embedment

Add 0.05 to the values specified in the table when all the elements in a group are affected by sheet pile extraction on more than one side, as is the case with four-pile groups or pile bents that will be surrounded by cofferdam sheets. See ***Appendix H***.

For all other cases (e.g., cast-in-place foundations and piles embedded through or into cohesive or rock layers) evaluate the loss in frictional resistance due of sheet pile extraction using static methods of analysis (e.g., FB-Deep).

Provide the anticipated loss in frictional resistance due to sheet pile proximity in the Geotechnical report and the following Plan note:

“Notify the Engineer if temporary sheet pile walls are tipped at deeper elevations or at

shorter distances to the foundation element(s) than shown in the Plans."

Modification for Non-Conventional Projects:
Delete the Plan note above and use the following: "Notify the GFDEOR and the EOR if temporary sheet pile walls are tipped at deeper elevations or at shorter distances to the foundation element(s) than shown in the Released for Construction (RFC) Plans"

Commentary: The correction factors address the additional resistance required during installation to account for the anticipated vibration-induced loss of capacity between the soil and the pile due to sheet pile extraction. The CFSPs are not to be interpreted to mean that existing foundations at distances greater than 5D will not be affected by the installation or extraction of steel sheet pile. Perform a separate settlement-impact evaluation and include recommendations in the Geotechnical Report.

8.3.8 Preforming and Jetting

Consider the effects of preforming and jetting on axial, uplift, lateral, and torsional resistance of deep foundation elements. Coordinate the design details with the DGE.

8.3.8.1 Axial Resistance

When modeling preformed zones in FB-Deep, the SPT N_{60} values used in the software must not exceed the N_{60} value for loose sand (refer to ***Chapter 6, Table 6-1***).

When modeling jetted zones in FB-Deep the SPT N_{60} values used in the software must reflect very loose sands or very soft clay, whichever applies (refer to ***Chapter 6, Table 6-1***).

Coordinate the minimum number of set-checks and redrives with the DGE.

8.3.8.2 Lateral Resistance

When modeling preformed zones in FB-MultiPier, provide soil parameters based on SPT $N_{1\text{-}60}$ values that do not exceed 50% of the recorded values in the field.

When modeling jetted zones in FB-MultiPier provide soil parameters that reflect very loose sands or very soft clay, whichever applies, to a depth of at least three (3) feet below the anticipated jet tip elevation or top of competent (non-erodible) rock, whichever is higher (refer to ***Chapter 6, Table 6-1***).

8.3.9 Evaluation of Vibration Induced Settlement due to Pile Driving

In projects where pile driving operations through granular soil layers are required, perform an evaluation of the effects of pile driving on nearby structures using Reference ***8-5.***

8.4 Embankment Settlement and Stability

The completed embankment must provide sufficient support for value added pavement (see ***Standard Specifications*** Section 338 and 355). Address embankment settlement and global stability concurrently.

8.4.1 Settlement

Compute embankment settlement using the methods included in the *AASHTO Code*.

8.4.1.1 Design Procedure

Follow the procedure in the *AASHTO Code* Chapter 10 and References ***8-14*** and ***8-20*** for determining settlement.

8.4.1.2 Considerations

Calculate time-dependent settlement based on the results of consolidation tests performed on high-quality samples. Plot the results of consolidation calculations on a time-settlement curve and include in the Geotechnical Report. For compressible clay and organic materials, base total settlement estimates on primary consolidation, and secondary compression (creep) settlements over the design life of the roadway. In these cases, creep estimates must be based on coefficients of secondary compression values obtained from laboratory consolidation test results. Include time rate of settlement estimates; basing these estimates on laboratory or field tests.

For high organic content materials (organic content greater than 50%), base total settlement estimates on primary consolidation, secondary, and tertiary compression (creep) settlements over the design life of the roadway. In these cases, creep estimates must be based on coefficients of secondary and tertiary compression values obtained from laboratory consolidation test results.

If excessive settlement due to compressible clays or organic materials is predicted over a lengthy period of a time (criteria can vary), propose a method of addressing the issue. Design and monitor a field instrumentation program if needed.

If surcharge is required, maintain surcharge load until at least 90% of the total expected settlement due to the fill with surcharge has occurred. Design the surcharge loading such that 100% of primary consolidation plus at least half of the creep consolidation for non-surcharged embankment has completed before the surcharge is removed.

Design lightweight fill embankments to provide a factor of safety ≥ 1.20 against buoyancy, and lateral movement due to the 500-year storm. Provide details for a PVC or HDPE liner to protect lightweight fills such as EPS geofoam which may be damaged by accidental exposure to chemical or petroleum spills.

8.4.1.3 Possible Solutions

Possible solutions to address settlement include but are not limited to:

1. Reduction of fill height.
2. Providing a waiting period to allow for the majority of consolidation to occur.
3. Increase of the surcharge height.
4. Use of a lightweight fill.
5. Installation of wick drains within the compressible material to be surcharged.
6. Excavation of soft compressible material and backfilling with granular soil.

7. Implementing ground modification techniques such as stone columns, dynamic compaction, deep soil mixing, etc. See References ***8-12***, ***8-21***, and ***GeoTechTools***.
8. Combinations of the above.

8.4.2 Stability

Conduct stability analyses using data from in situ strength tests and/or laboratory strength tests performed on high quality samples. Consider a range of possible material strengths to allow the evaluation of a spectrum of soil resistance and assess slope stability. Evaluate the overall stability of the retaining wall, retained slope, and foundation soil or rock using limit equilibrium methods of analysis in accordance with the *AASHTO Code*. The analysis shall provide an equivalent Allowable Stress Design (ASD) minimum factor of safety of 1.5.

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

8.4.2.1 Design Procedure

See References ***8-1, 8-7***, ***8-14***, and the *AASHTO Code* for design procedures. Identify any required reinforcement materials as R-2 or R-3 Geosynthetics when required for Embankments Over Soft Soils or Reinforced Soil Slope applications, respectively. Refer to ***FDM*** and the ***Standard Specifications*** for more information on R-2 and R-3 Geosynthetics.

*Commentary: References **8-1**, **8-7**, and **8-14**, are based on Factor of Safety or Service Limit State analyses and require modification for an equivalent LRFD-based design.*

8.4.2.2 Considerations

Calculate soil resistance for all possible slope conditions (i.e., surcharge loading, varying fill heights and/or slopes, varying water tables, etc.) for the service limit state. Include a design method for dealing with potential stability problems and a field monitoring and instrumentation program if needed.

8.4.2.3 Possible Solutions

Possible solutions to address stability issues include but are not limited to:

1. Realignment of the highway.
2. Reduction of the fill height.
3. Flattening of the slope (considering right-of-way requirements)
4. Staged construction to allow soft soil to gain strength through consolidation.
5. Excavation and replacement of soft soils.
6. Design of geotextile or geogrid within the embankment.

7. Placement of a berm at the toe.
8. Use of lightweight fills.
9. Implementing ground modification techniques such as stone columns, dynamic compaction, deep soil mixing, etc. (See References ***8-13***, ***8-21***, and ***GeoTechTools***).
10. Use of obstructions to keep vehicles from parking on or approaching the crest of the slope.
11. Installation of an underdrain system to depress the phreatic surface in the slope.
12. Construction of 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, MSE walls, and soil nail walls; must be designed in accordance with the *AASHTO Code* (except as noted in ***SDG*** and ***FDM***) with adequate soil resistance against bearing, sliding, overturning, eccentricity, and overall stability. A design analysis is still required when standard index walls are used on a project.

8.5.1 Gravity Walls

8.5.1.1 Design Procedure

See Reference ***8-47*** for gravity wall design.

8.5.1.2 Considerations

All gravity walls including those taken from the ***Standard Plans*** must be checked for stability. Refer to ***SDG*** and ***FDM*** for procedures on design of walls.

8.5.2 Counterfort Walls

8.5.2.1 Design Procedure

See the *AASHTO Code and* Reference ***8-47*** for counterfort wall design.

8.5.2.2 Considerations

This type of wall can be used in extremely aggressive environments and in congested areas (due to speed of construction).

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

8.5.3 MSE Walls

8.5.3.1 Design Procedure

See Reference ***8-7*** and the *AASHTO Code* for MSE wall design.

8.5.3.2 Considerations

Refer to ***SDG***, Section 3.13.2 for water elevation design requirements.

All MSE wall systems must be listed in the ***Approved Products List (APL).***

For all proprietary MSE wall systems, perform external stability analysis and ensure that the design is compatible with the actual subsurface conditions. The system proprietor is responsible for internal stability. Refer to ***SDG*** and ***FDM*** for additional requirements.

8.5.3.3 Widening Existing Walls

When existing MSE walls are to be widened, evaluate the remaining service life (internal stability) of the existing MSE wall based on the minimum density, friction angle, creep reduction factors, 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 DGE, perform tests for:

a. Modified Proctor Density (***FM 1-T 180***)

b. Direct Shear (***FM 3-D3080***)

c. Environmental Corrosion Tests (see Section ***4.9***)

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.

See Reference ***8-7*** and the *AASHTO Code*.

8.5.4 Sheet Pile Walls

8.5.4.1 Design Procedure

See the *AASHTO Code* Chapter 11, Reference ***8-47***, and the requirements outlined in ***SDG***.

Provide all the necessary geotechnical design parameters in the Geotechnical Report for permanent, temporary critical, and temporary non-critical walls that are detailed in the Plans.

For critical and non-critical temporary steel sheet pile walls, refer to the ***Standard Plans*** Index 455-500 and the requirements outlined in the associated SPI. Ensure the design assumptions are satisfied prior to use. Loading or subsurface profile conditions that differ from the assumed parameters used to develop the Standard will require a project specific re-design.

8.5.4.2 Considerations

When coatings are 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.

Alert the contractor of any soil or rock layers that will make sheet pile installation difficult by providing appropriate information in the Plans. Consider preforming and other installation effects on the resulting friction between the wall panel and the soil or rock.

A reduction in resistance of the foundation must be determined to account for the extraction of temporary sheet pile walls near the foundation. Refer to ***8.3.7*** for evaluating the effects of sheet pile proximity.

8.5.5 Soil Nail Walls

8.5.5.1 Design Procedure

See the *AASHTO Code*, Chapter 11 and References ***8-31*** and ***8-47*** for soil nail wall design.

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

For granular soils, determine the passive earth resistance, K_p with or without wall friction and neglect the soil resistance to a depth equal to 1B below the bottom of excavation. When wall friction is considered, the interface angle, δ, must not exceed the value given in ***Figure 8-4*** for the applicable soil and soldier pile materials. See Reference ***8-47*** and the *AASHTO Code* Chapter 11 for soldier pile/panel walls.

Commentary: The analysis and design of soldier piles is different from sheet pile walls because the failure of individual pile elements is different from continuous walls. The failure mechanism of the individual pile is analogous to a bearing failure in front of the pile; the total resistance force assumes the pile has an effective width of 3B (or three times the width of the pile) for all types of soil. The bearing resistance pressures for cohesive soils are considered to be uniform with a magnitude of 2c (two times the cohesion) neglecting the soil resistance of 1.5 times the pile width (B) from the bottom of excavation.

8.5.6.2 Considerations

For permanent applications in Florida, concrete pile and panel lagging is preferrable. Consider soldier pile/panel walls in locations where sheet pile walls are needed, however, sheet pile installation difficulties are expected. Refer to ***SDG*** and ***FDM*** for additional requirements.

8.5.7 GRS Walls

8.5.7.1 Design Procedure

GRS walls are similar to GRS abutments. Designs must comply with Appendix C of Reference ***8-2***, except as otherwise indicated in Sections 3.12.7 and 3.13.4 of ***SDG***.

Present GRS walls in the Plans using ***Developmental Standard Plans*** D549-025. Incorporate ***Developmental Specification*** 549 into the specifications package.

8.5.7.2 Considerations

Limitations and considerations are presented in Sections 3.12.7 and 3.13.4 of ***SDG*** and in the ***Developmental SPI*** D549-025.

8.6 Steepened Slopes

Design all steepened slopes for external stability including all failure possibilities such as sliding, deep-seated overall instability, local bearing capacity failure at the toe (lateral squeeze), and excessive settlement from both short and long-term conditions. Design reinforcement requirements to adequately account for the internal stability of the slope.

8.6.1 Design Procedure

See Reference ***8-7***. Identify Reinforced Slope reinforcement materials in the Plans or TSPs as R-3 Geosynthetics.

8.6.2 Considerations

Coordinate the use of steepened slopes with the District Maintenance Office. As with all slopes steeper than 1V:3H, steepened slopes require maintenance berms for mowing equipment – see ***FDM***.

8.7 Computer Programs used in FDOT

See the listing of ***Geotechnical Computer Programs*** approved for use in FDOT projects.

Table 8-4 Geotechnical Engineering Analysis Required in Reference 8-22

TABLE 3

REQUIRED GEOTECHNICAL ENGINEERING ANALYSIS

Soil Classification			Embankment and Cut Slopes		Structure Foundations (Bridges and Retaining Structures)		Retaining Structures (Conventional, Crib and MSE)	
Unified	AASHTO[1]	Soil Type	Slope Stability[2] Analysis	Settlement Analysis	Bearing Capacity Analysis	Settlement Analysis	Lateral Earth Pressure	Stability Analysis
GW GP GM GC SW SP SM SC	A-1-a A-1-a A-1-b A-2-6 A-2-7 A-1-b A-3 A-2-4 A-2-5 A-2-6 A-2-7	GRAVEL Well-graded GRAVEL Poorly-graded GRAVEL Silty GRAVEL Clayey SAND Well-graded SAND Poorly-graded SAND Silty SAND Clayey	Generally not required if cut or fill slope is 1.5H to 1V or flatter, and underdrains are used to draw down the water table in a cut slope. Erosion of slopes may be a problem for SW or SM soils.	Generally not required except possibly for SC soils.	Required for spread footings, pile or drilled shaft foundations. Spread footings generally adequate except possibly for SC soils	Generally not needed except for SC soils or for large, heavy structures. Empirical correlations with SPT values usually used to estimate settlement	GW, SP, SW & SP soils generally suitable for backfill behind or in retaining or reinforced soil walls. GM, GC, SM & SC soils generally suitable if have less than 15% fines. Lateral earth pressure analysis required using soil angle of internal friction.	All walls should be designed to provide minimum F.S. = 2 against overturning & F.S. = 1.5 against sliding along base. External slope stability considerations same as previously given for cut slopes & embankments.
ML	A-4	SILT Inorganic silt Sandy	Required unless non-plastic. Erosion of slopes may be a problem.	Required unless non-plastic.	Required. Spread footing generally adequate.	Required. Can use SPT values if non-plastic.	These soils are not recommended for use directly behind or in retaining or reinforced soil walls.	
CL	A-6	CLAY Inorganic Lean Clay	Required	Required				
OL	A-4	SILT Organic	Required	Required				

For FDOT projects, analysis is required for all slopes steeper than 2.5H to 1V. (see 8.4.2)

[1] This is an approximate correlation to Unified (Unified Soil Classification system is preferred for geotechnical engineering usage, AASHTO system was developed for rating pavement subgrades).

[2] These are general guidelines, detailed slope stability analysis may not be required where past experience in area is similar or rock gives required slope angles.

Table 8-4 for use with Reference 8-22 for Embankments, Cut Slopes, Structure Foundations, and Retaining Walls

TABLE 3 (Continued)

Soil Classification			Embankment and Cut Slopes		Structure Foundations (Bridges and Retaining Structures)		Retaining Structures (Conventional, Crib and MSE)	
Unified	AASHTO[1]	Soil Type	Slope Stability[2] Analysis	Settlement Analysis	Bearing Capacity Analysis	Settlement Analysis	Lateral Earth Pressure	Stability Analysis
MH	A-5	SILT Inorganic	Required. Erosion of slopes may be a problem.	Required.	Required. Deep foundation generally required unless soil has been preloaded.	Required. Consolidation test data needed to estimate settlement amount and time.	These soils are not recommended for use directly behind or in retaining walls.	All walls should be designed to provide minimum F.S. = 2 against overturning & F.S. = 1.5 against sliding along base. External slope stability considerations same as previously given for cut slopes & embankments
CH	A-7	CLAY Inorganic Fat Clay	Required.	Required.				
OH	A-7	CLAY Organic	Required.	Required.				
PT	----	PEAT Muck	Required.	Required. Long term settlement can be significant	Deep foundation required unless peat excavated and replaced.	Highly compressible and not suitable for foundation support		
Rock			Fills – not required for slopes 1.5H to 1V or flatter. Cuts – required but depends on spacing, orientation and strength of discontinuities and durability of rock		Required for spread footings or drilled shafts. Empirically related to RQD[3]	Required where rock is badly weathered or closely fractured (low RQD). May require in situ test such as pressuremeter.	Required. Use rock backfill angle of internal friction.	

REMARKS:
Soils – temporary ground water control may be needed for foundation excavations in GW through SM soils.
Backfill specifications for reinforced soil walls using metal reinforcements should meet the following requirements in insure use of non-corrosive backfill:
pH range = 5 to 10; Resistivity > 3000 ohm-cm; Chlorides < 100 ppm; Sulfates < 200 ppm; Organic content 1% maximum

Rock – Durability of shales (siltstone, claystone, mudstone, etc.) to be used in fills should be checked. Non-durable shales should be embanked as soils, i.e., placed in maximum 0.3 m (1 ft) loose lifts and compacted with heavy sheepsfoot or grid rollers.

[1] This is an approximate correlation to Unified (Unified Soil Classification system is preferred for geotechnical engineering usage, AASHTO system was developed for rating pavement subgrades).

[2] These are general guidelines, detailed slope stability analysis may not be required where past experience in area is similar or rock gives required slope angles.

[3] RQD (Rock Quality Designation) = sum of pieces of rock core 4" or greater in length divided by the total length of core run.

Table 8-4 continued

Table 8-5 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			

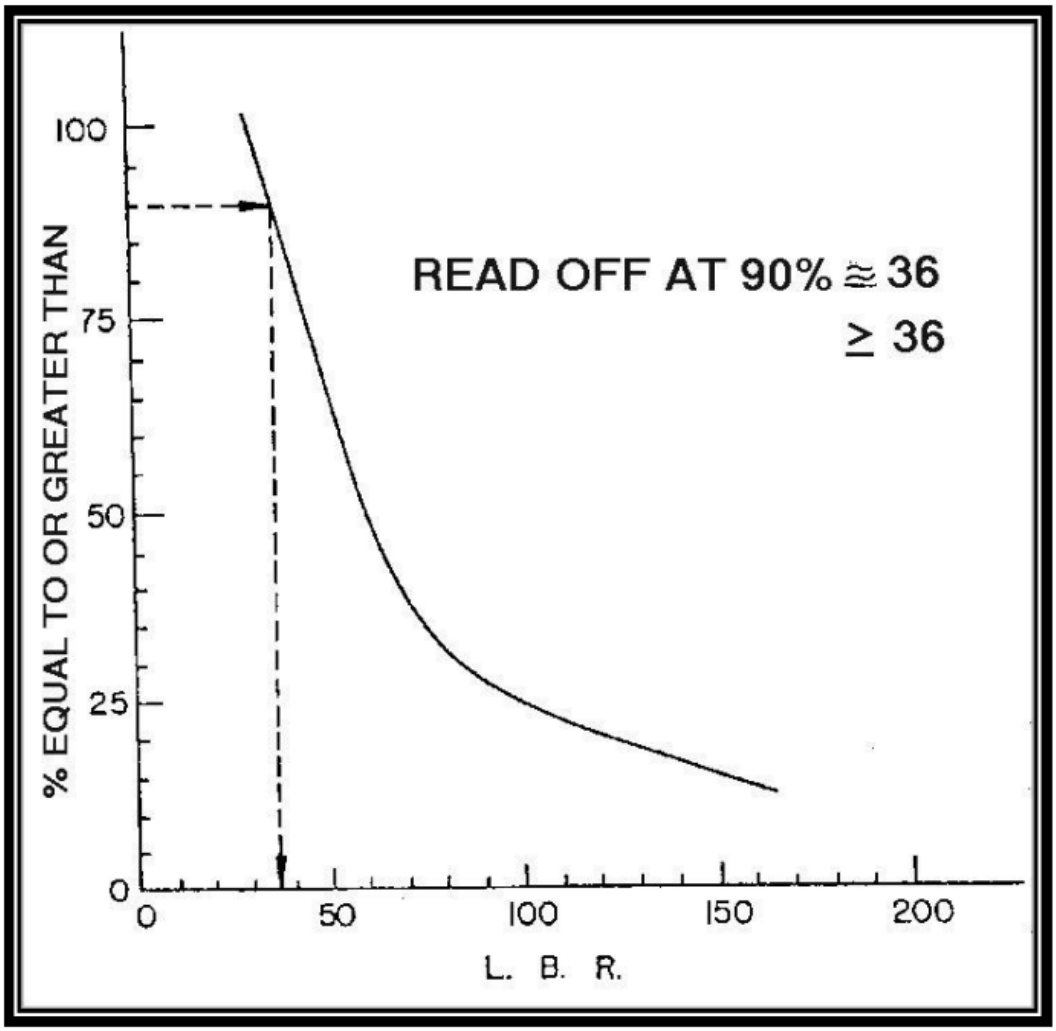


Figure 8-2 Design Example 1 (LBR Design Methods) 90% Method

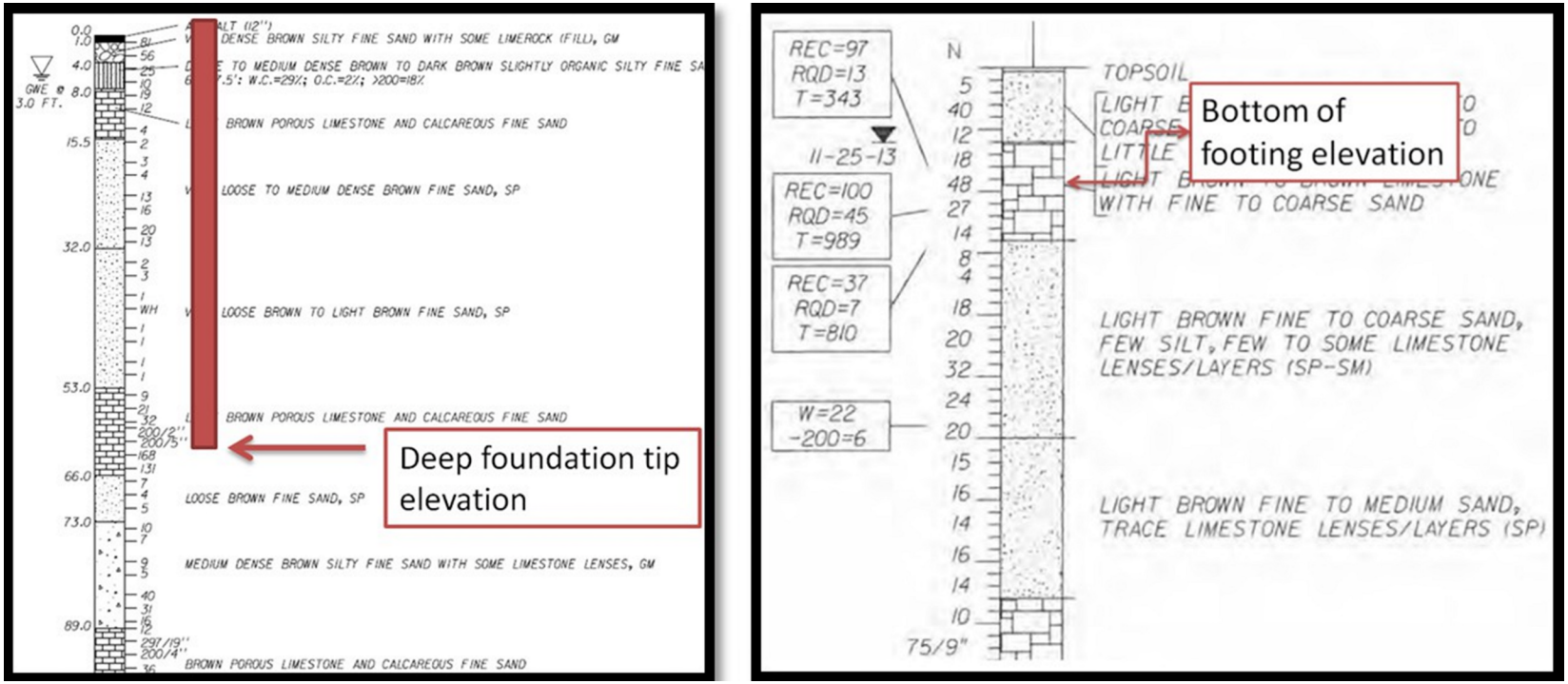


Figure 8-3 Example of Rock Lens Punching Condition

Ultimate Friction Factors and Adhesion for Dissimilar Materials

Interface Materials	Friction factor, tan δ	Friction angle, δ degrees
Mass concrete on the following foundation materials:		
Clean sound rock	0.70	35
Clean gravel, gravel-sand mixtures, coarse sand	0.55 to 0.60	29 to 31
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	0.45 to 0.55	24 to 29
Clean fine sand, silty or clayey fine to medium sand	0.35 to 0.45	19 to 24
Fine sandy silt, nonplastic silt	0.30 to 0.35	17 to 19
Very stiff and hard residual or preconsolidated clay	0.40 to 0.50	22 to 26
Medium stiff and stiff clay and silty clay	0.30 to 0.35	17 to 19
(Masonry on foundation materials has same friction factors.)		
Steel sheet piles against the following soils:		
Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	0.40	22
Clean sand, silty sand-gravel mixture, single size hard rock fill	0.30	17
Silty sand, gravel or sand mixed with silt or clay	0.25	14
Fine sandy silt, nonplastic silt	0.20	11
Formed concrete or concrete sheet piling against the following soils:		
Clean gravel, gravel-sand mixture, well-graded rock fill with spalls	0.40 to 0.50	22 to 26
Clean sand, silty sand-gravel mixture, single size hard rock fill	0.30 to 0.40	17 to 22
Silty sand, gravel or sand mixed with silt or clay	0.30	17
Fine sandy silt, nonplastic silt	0.25	14
Various structural materials:		
Masonry on masonry, igneous and metamorphic rocks:		
Dressed soft rock on dressed soft rock	0.70	35
Dressed hard rock on dressed soft rock	0.65	33
Dressed hard rock on dressed hard rock	0.55	29
Masonry on wood (cross grain)	0.50	26
Steel on steel at sheet pile interlocks	0.30	17

Interface Materials (Cohesion)	Adhesion C_a (psf)
Very soft cohesive soil (0 - 250 psf)	0 - 250
Soft cohesive soil (250 - 500 psf)	250 - 500
Medium stiff cohesive soil (500 - 1000 psf)	500 - 750
Stiff cohesive soil (1000 - 2000 psf)	750 - 950
Very stiff cohesive soil (2000 - 4000 psf)	950 - 1,300

Figure 8-4 Limits for the Interface Angle, δ (NAVFAC DM-7.2, 1971)

8.8 References

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8.9 Specifications and Standards

Subject	ASTM	AASHTO	FM	STD. INDEX
Temporary Steel Sheet Pile Wall	-	-	-	455-500
GRS-IBS	-	-	-	D549-025
Embankment Utilization	-	-	-	120-001
Subsoil Excavation	-	-	-	120-002
Standard Test Method for Determining the Resilient Modulus of Soils and Aggregate Materials	-	T 307	-	-

Chapter 9 Geotechnical Report

Compile the information obtained in a report format that is clear and easy to follow. This report will serve as the permanent record of all geotechnical data known during design of the project and will be referenced throughout the design, construction, and service life of the project.

The Geotechnical Report must present the data collected in a clear manner, draw conclusions from the data, and make recommendations for the geotechnical related portions of the project.

This chapter outlines the format for presentation of geotechnical data for each type of project, finalization and distribution of the Geotechnical Report, and the incorporation of its recommendations into the design.

9.1 Roadway Soil Survey

Present in the Geotechnical Report for a roadway soil survey conclusions and recommendations concerning the suitability of in situ materials for use as embankment materials. Include special problems affecting roadway design, such as slope stability or excessive settlement as applicable. The following is a general outline of the topics to be included.

9.1.1 General Information

1. List of information provided for the geotechnical investigation (alignment, foundation layout, 30% Plans, scour estimate, etc.)
2. Description of the project, including location, type, and any design assumptions.
3. Description of significant geologic and topographic features of the site.
4. Description of width, composition, and condition of existing roadway.
5. Description of all methods used during subsurface exploration, in situ testing, and laboratory testing; along with the raw data from these tests.
6. The make and model of the GPS unit used to determine the Latitude and Longitude coordinates of borings, bulk samples, muck probe areas, etc.

9.1.2 Conclusion and Recommendations

1. Provide an explanation of stratification of in situ materials including observed groundwater level and estimated seasonal high/low groundwater levels.
2. 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.
3. 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.

4. Provide recommendations for cut or fill sections when seepage, stability, or settlements are significant.
5. Provide recommendations and considerations for any proposed walls.
6. Provide recommendations and considerations for any proposed storm water retention ponds.
7. Include the permeability and infiltration rate of the stormwater pond subsurface soils.
8. Include aquifer parameters such as soil permeability, infiltration rate, and aquifer bottom elevation.
9. 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

Present material descriptions and results of classification and corrosivity tests for each stratum. Provide recommendations for material utilization 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. Include the number of laboratory tests performed for each stratum for corrosion tests results as well as for 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 SMO. Include the Report of Tests sheet in the Geotechnical Report and in the Plans. See ***Figure 9-1*** for an example of a typical Report of Tests sheet.

9.1.4 Roadway (and Pond, etc.) Cross Sections

Include plots of stratified boring logs on the cross section sheets of the Plans. Each material stratum is numbered corresponding to the strata on the test results sheet. See ***Figure 9-2*** for an example of a typical cross sections sheet and ***Figure 9-3*** for an example of a typical generalized soil profile. If cross sections sheets are to be prepared by others, provide the appropriate subsurface information. Verify that the data has been correctly incorporated.

Indicate on the cross sections the anticipated horizontal and vertical limits for removal of unsuitable materials.

9.1.5 Appendix

Include an appendix for all roadway soil survey reports containing the following information:

1. Soil conservation (Natural Resources Conservation Service [NRCS]/U.S

Department of Agriculture [USDA]) and USGS maps depicting the project location.

2. Boring location plan and plots of boring logs and/or cone soundings.
3. Results of roadway soil survey borings performed. For all SPT borings, include the information required in ***9.2.3***.
4. Any other pertinent information.
5. Analysis of the geotechnical information including correlations and calculations.

9.1.6 Geotechnical Data for NexGen Plans

The Final Geotechnical Data deliverable for a project is an XML file generated from the FDOT Geotechnical Data Manager application. This application is delivered with the FDOT Connect software.

Name the Geotechnical Data file as follows: *XXXXXXXXXXX- modifier.xml* where XXXXXXXXXX is the project number, the modifier is optional but can be used to differentiate data files if more than one is delivered with the project.

*Commentary: The Geotechnical Data Manager application provides the ability to manually create a deliverable XML file from scratch. If a database system is used for storing Geotechnical data, the data can be exported out in Excel format that the Geotechnical Data Manager software can convert to the required XML format. Excel formatted templates are provided to show how the exported data must be formatted for conversion by Geotechnical Data Manager. These spreadsheets are provided in the project's Geotechnical folder. Do not modify the existing spreadsheet format in any way. Once the database data is exported, import the data into the Geotechnical Data Manager and export to the final deliverable XML data file. Training for this process can be found **here**.*

9.2 Structures Investigation

9.2.1 Introduction

Present in the Geotechnical Report for a structure the conclusions and recommendations for the most suitable foundation types and information required for incorporating such foundations into the design of the structure. Include recommendations for related work, such as approach embankments and retaining walls.

Note any special construction considerations. Do not repeat or copy items stated in ***Standard Specification*** Section 455 into the report. Recommend only the site-specific items for TSPs. The following is a general guide to the contents of a typical structure foundation report.

9.2.2 Scope of Investigation

1. Description of type of project, location of project, local geology and any assumptions related to the project.
2. Vicinity map, including potentiometric map, USGS and soil survey maps

(NRCS/USDA) depicting project location.

3. Summary of general content of report.

9.2.3 Interpretation of Subsurface Conditions

1. Description of the methods used in the field investigation, including the types and frequencies of all in situ tests.
2. Description of the laboratory-testing phase, including any special test methods employed.
3. Boring location plan and plots of boring logs and cone soundings. See ***Figure 9-4*** and ***Figure 9-5*** for examples of Report of Core Borings and Report of Cone Soundings sheets. Provide the longitude and latitude of each boring or sounding below the station, offset and elevation, and the depth of temporary casing used to perform the boring on the Report of Core Borings and Report of Cone Soundings sheets. Use the standard soil type symbols shown in ***Figure 9-6*** as described in ***Table 9-2*** when plotting boring logs. Note the size of rock core sampled. Provide the make and model of the GPS unit used to determine the Latitude and Longitude coordinates of borings, bulk samples, muck probe areas, etc.

   These sheets are included in the final Plans; see the **Core Borings** section of the Structures Manual, Volume II, ***Structures Detailing Manual (SDM)*** for additional requirements for these sheets.
4. Estimated depths of scour, if applicable.
5. Environmental class for both substructure and superstructure based on results of corrosivity tests. This information is also reported on the Report of Core Borings sheet. For extremely aggressive classification note the parameter that that caused this classification.
6. Summary table of soil parameters determined from field and laboratory testing.
7. Table of soil parameters to use with computer modeling (such as FB-MultiPier program). These parameters can be broken up into zones across the bridge length. Include all correlations used to develop the table.
8. Recommendations and considerations for any proposed walls. Include MSE or cast-in-place wall recommendations.
9. Discussion of undesirable conditions observed in the borings and undesirable conditions present in the geologic formation(s) encountered at the site, together with any impact on proposed construction.

Modification for Non-Conventional Projects:
Add 10. Discussion of anticipated procedures for mitigating undesirable conditions observed in the borings or expected due to the geologic formation(s) encountered at the site.

9.2.4 Existing Structures Survey and Evaluation

Existing structures to be protected may include sensitive sites, such as those listed in ***FDM*** Chapter 117. Determination of sensitive sites in proximity to the project will be made by the Roadway Engineer with the Department making the final determination whether revised thresholds of settlement and vibration are warranted.

When requested by the EOR:

1. 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 and settlements more stringent than those specified in ***Standard Specification*** Section 108 should be required for these structures.
3. Recommend and discuss with the Department the potential need to specify different movement thresholds.
4. Prepare an MSP to specify the revised thresholds of vibration and settlement identifying the sensitive sites where these thresholds apply.

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

Ensure settlement monitoring of existing foundations including Department-owned structures is specified in the Plans when the structure is potentially vulnerable to vibrations, pile driving, and excavations as discussed in ***8.2***.

9.2.5 Structure Foundation Analysis and Recommendations

Provide alternate foundation recommendations for all structures including recommendations for GRS abutments, spread footings, driven piles, auger cast piles and drilled shafts. Include an explanation for any of these alternatives judged not to be feasible. Include a summary of the types of analyses performed.

Modification for Non-Conventional Projects:
Delete the previous paragraph and replace with: Provide a summary of the analysis and recommendations for the preferred foundation.

9.2.5.1 GRS Abutments

1. Summarize the evaluation including reason(s) for selection and/or exclusion.
2. Design soil pressure based on settlement and bearing capacity.
3. Estimated short- and long-term settlements assuming GRS abutments are constructed in accordance with ***Developmental Specification*** 549.
4. Soil improvement method(s).
5. Estimate the reduction in settlement anticipated resulting from these improvement methods.
6. Sinkhole potential.
7. Provide the information required in the ***Developmental SPI, Index D549-025.***

9.2.5.2 Spread Footings

1. Summarize evaluation including reason(s) for selection and/or exclusion.
2. Elevation of bottom of footing or depth to competent bearing material.
3. Design soil pressure based on settlement and bearing capacity.
4. Estimated short- and long-term settlements assuming spread footings are constructed in accordance with ***Standard Specifications*** Section 455.
5. Soil improvement method(s).
6. If soil material needs to be over excavated and replaced, recommend plan notes specifying the depth of excavation. Provide recommendations for TSPs for footing construction, including compaction requirements and the need for particular construction methods such as dewatering or proof rolling in addition to the requirements in ***Standard Specifications*** Sections 125 and 455. Estimate the reduction in settlements anticipated resulting from these special requirements.
7. Sinkhole potential.

9.2.5.3 Driven Piles

1. Suitable pile types and reasons for design selections and exclusions.
2. Plotted design curves of soil resistance for selected pile size alternates. Plotted curves must present the Davisson capacity, ultimate skin friction, and mobilized end bearing versus pile tip elevation for the existing soil 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 Quality Control method used to verify uplift resistance, see ***SDG*** Table 3.5.7-1.)
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 3.5.13-1 of ***SDG***, determine the preforming or jetting elevations required to reduce the driving resistance to an acceptable magnitude. Provide additional capacity curves required by SDG on separate pages.
9. Recommended limitations on predrilling/preforming operations to prevent impacts from observed or expected artesian conditions.
10. 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 two times the factored design load (FDL) or the design nominal resistance).
12. Recommendations for special notes and possible MSP to avoid impacts of potentially damaging installation procedures on existing foundations including Department-owned structures as discussed in Section ***8.2.***
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 ***0***.
14. Recommendations and special techniques to address the effects of temporary cofferdams or sheet piles on the pile capacity; see ***8.3.7.***
15. Recommendations for information to be placed in the Pile Data Table shown in the ***Standard Plans*** SPI, Index 455-001.
16. Present soil parameters to be used for lateral analysis accounting for installation techniques and scour. Check the final lateral load analyses for correct soil property application.
17. On small projects with reasonably predictable bearing layers, provide the

production pile lengths in the Pile Data Table for each bent rather than test pile lengths.

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

19. Include a list of pay items (if applicable) and their associated quantities in the Geotechnical Report and ensure they are implemented in the plans. Refer to the ***Standard Specifications*** Section 455 for driven pile pay items and the ***Basis of Estimates (BOE)*** for details.

9.2.5.4 Drilled Shafts

1. Include plots of resistance versus tip elevation for selected alternate shaft sizes. Develop plots for both factored (Q_r) and nominal (Q_n) resistance and show end bearing, skin friction, and total resistance (end bearing shall not be discounted). Include the depths of scour analyzed.

2. Unless otherwise specified, perform separate shaft analyses for the recommended shaft sizes 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, show the corresponding resistance versus tip elevation curves 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. Indicate the minimum socket length, 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. Estimated drilled shaft settlement and shaft group settlement.

7. Recommended test types, locations, and depths. For static, Statnamic, or Osterberg load testing, the ultimate load the test is to be taken to must be shown in the Plans (the greater of two times the FDL or the nominal resistance).

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

9. Evaluate the need for TSPs for shaft installation (special needs or restrictions). Special construction techniques may be needed to minimize the effects of foundation installation discussed in ***0***.

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

11. Include the potentiometric Surface Map information.

12. Present soil/rock parameters to be used for lateral analysis accounting for installation techniques and scour.
13. Check the final lateral load analysis for correct soil/rock property application.
14. Check for sinkhole potential and its implications for drilled shaft construction and performance.

9.2.5.5 Auger Cast Piles

1. Suitable pile sizes.
2. Plotted design curves of soil resistance for selected pile size alternates. The ultimate skin friction capacity is equivalent to the LRFD nominal resistance (R_n). Design rock-socketed ACP for structures, other than miscellaneous, in accordance with ***Appendix A.***

   Provide separate pile analyses for each recommended pile size for each SPT boring and/or CPT sounding. Provide a corresponding pile capacity curve for each analysis. When more than one boring is taken at a pile group or when it is appropriate to otherwise generalize the soil strata, show the corresponding pile capacity curves together on the same plot and establish the lower bound for that pile group.
3. Estimated elevation of consistent bearing layer suitable for providing the required nominal resistance.
4. Recommendations for providing the nominal uplift resistance (see ***SDG*** Table 3.5.20-2).
5. Estimated pile settlement and pile group settlement for the recommended tip elevation.
6. Effects of scour, downdrag, and lateral squeeze, if applicable.
7. Recommended locations of demonstration piles and load test piles.
8. Include the ultimate load for the load test in the Plans (the greater of two times the FDL or the design nominal resistance).
9. Recommendations and special techniques to address the effects of temporary cofferdams or sheet piles on the pile capacity; see ***8.3.7***.
10. Present recommendations for information to be placed in the Drilled Shaft Data Table shown in the ***SDM*** (Change the title of the table to “Auger Cast Pile Data Table”).
11. Present soil parameters to be used for lateral analysis accounting for installation techniques and scour. Check 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 Code* 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. Verification of 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 TSPs regarding embankment construction.

9.2.7 Retaining Walls and Seawalls

1. Settlement potential
2. Recommended lateral earth pressure parameters.
3. Recommended wall type according to ***FDM***.
4. Factored soil resistance or alternate foundation recommendations.
5. Factored soil resistance and loads with respect to sliding and overturning (including standard index wall designs).
6. Overall stability of walls.
7. The design water elevation.
8. Recommendations for TSP for fill material (except MSE walls) and drainage.
9. Special considerations for tiebacks, geotextiles, reinforcing materials, etc., if applicable.
10. MSE reinforcement lengths required for external stability, if applicable. See ***SDG*** and ***FDM*** for details.

9.2.8 Steepened Slopes

1. Estimated resistance factor against internal and external stability failure based on LRFD.

2. Spacing and lengths of reinforcement to provide a stable slope.
3. Design parameters for reinforcement (design strength, durability criteria, and soil-reinforcement interaction).
4. Fill material properties.
5. Special drainage considerations (subsurface and surface water runoff control).
6. Verification of stability for fully saturated conditions.

9.2.9 Technical Special Provisions (TSPs)

Use TSP's to change the ***Standard Specifications*** for a project only when extraordinary, project-specific conditions exist.

The Department has a number of TSPs available for various items of work tailored to previous projects. These TSPs can be obtained from the DGE or from the following website: ***http://www.fdot.gov/geotechnical/publications.shtm***.

Update and revise the TSP's obtained from the Department to reflect the needs of your specific project.

9.2.10 Appendix

Include an appendix for all structure investigation reports with the following information:

1. Report of Core Borings (or Report of SPT Borings) Sheet (see ***Figure 9-4***). (Note the FDOT Geotechnical CADD Standard menu is available.)
2. Color photographs of rock cores indicating boring and core elevation.
3. Report of Cone Sounding Sheet (see ***Figure 9-5***).
4. Data logs or reports from specialized field tests.
5. Laboratory test data sheets. The following are examples of what should be provided:
    a. Rock Cores: Location, elevation, maximum load, core length, core diameter, moist density, dry density, splitting tensile strength, unconfined compressive strength, strain at 50% of unconfined compressive strength, strain at failure, and corrected tangent modulus (adjust the origin to eliminate seating stresses; use the adjusted origin and the slope of the linear portion of the stress vs. strain curve).
    b. Rock core data reduction and statistical analyses obtaining design side resistance for drilled shaft socket in rock, if applicable, according to ***Appendix A*** of this manual.
    c. Gradations: Location, elevation, test results.
    d. Corrosion Tests: Location, elevation, test results.
    e. Consolidation Tests: plots of e vs. log p' and displacement vs. time (both square root time and log time), and index properties of tested materials.

6. Engineering analyses (bearing resistance, lateral stability, group effects, settlement, global stability, punching shear, downdrag, scour, and other applicable analyses). Include correlations and calculations.
7. Recommended Plan notes.
8. FHWA checklist.
9. ***Protection of Existing Structures Checklist***.
10. Copies of field boring logs with all drillers' notes and handwritten refinements, if any (not typed logs).
11. Any other pertinent information.

9.3 Final or Supplementary Report

Maintain clear lines of communication with the EOR throughout the duration of the project. Prepare a final or supplementary report to revise any assumptions and recommendations, if necessary, in accordance with the final design Plans. Include the following topics 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 FDLs.
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 must be signed and sealed by the Professional Engineer in responsible charge in accordance with Florida Statutes and the Rules of the State Board of Professional Engineers. The following documents are included:

Table 9-1 Signing and Sealing Placement

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

For supplemental specifications and special provisions, which cover other topics in addition to Geotechnical Engineering, the engineer in responsible charge of the geotechnical portions must indicate the applicable pages. See Chapter 130 of ***FDM***.

9.5 Distribution

Provide the following offices and personnel copies of Geotechnical Reports, as applicable:

1. Project Manager.
2. District Geotechnical Engineer.
3. District Drainage Engineer.
4. District Structural Design Section.
5. Roadway Design Section.
6. State Geotechnical Engineer (for Category II structures).

Modification for Non-Conventional Projects:
Delete the above distribution list and see the RFP for requirements.

9.6 Plan and Specification Review

In addition to writing the report, review all phases of the Plans and specifications to ensure that the geotechnical recommendations have been correctly incorporated.

Submit a marked up set of prints from the Quality Control Review, signed by the geotechnical reviewer, with each phase submittal. The Professional Engineer responsible for performing the Quality Control review must provide a signed statement certifying the review was conducted.

Department Standard, Supplemental, and Developmental Specifications must not be changed except in rare cases with the approval of the DGE. The Specifications Office requires an MSP for all project-specific changes to the Standard and Supplemental Specifications.

Modification for Non-Conventional Projects:
Delete the last paragraph and insert the following: Department Standard, Supplemental, and Developmental Specifications must not be changed except in rare cases with the approval of the Engineer.

9.7 Electronic Files

Submit an electronic copy of:

1. The final approved signed and sealed Geotechnical Report in PDF format. Include the boring log sheets in DGN format and include the input files used in the analysis programs (FB-Deep, FB-MultiPier, etc.).
2. The completed Excel spreadsheet with soil boring and location information for the FDOT ***GIS Soil Boring Database*** together with the boring profiles in PDF

format.

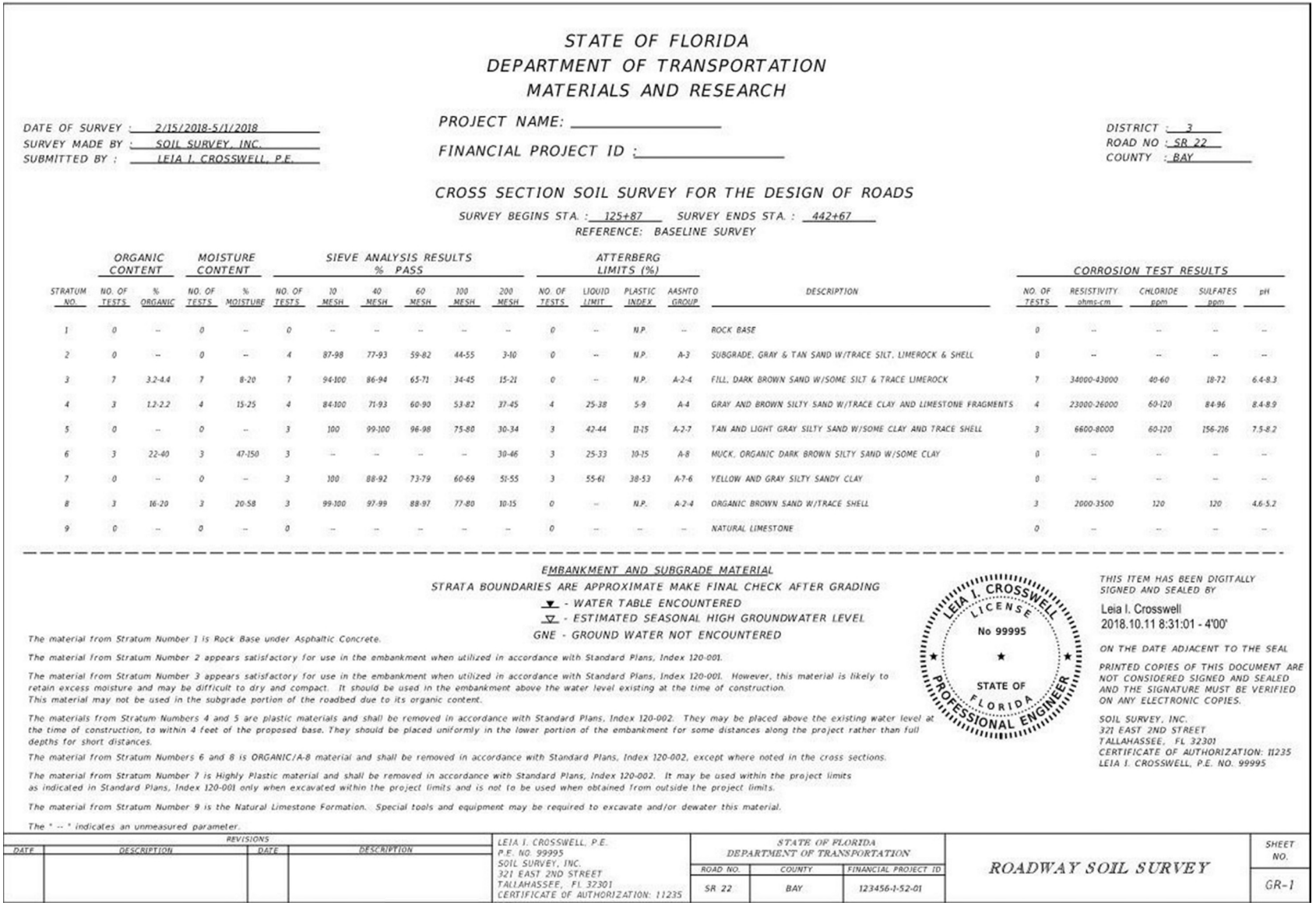
If a computer program is used in the design process that is not listed for use in this manual, the following additional items must be included in the report submittal:

1. Example hand calculations verifying the results of the computer programs included in the calculations package.
2. An electronic copy of the program and the computer input data files.

9.8 Exclusions

Do not include the following in the Geotechnical Report:

1. Summaries or retyping of standard test methods or Department specifications. Reference specifications and test methods by number.
2. Changes to the ***Standard Specifications*** without valid justification. (e.g., change the MSE wall backfill gradation). Base designs on the requirements in the Standard Specifications.



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

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR 22	BAY	123456-1-52-01

ROADWAY SOIL SURVEY

SHEET NO.
GR-1

Figure 9-1 Typical Report of Tests Sheet

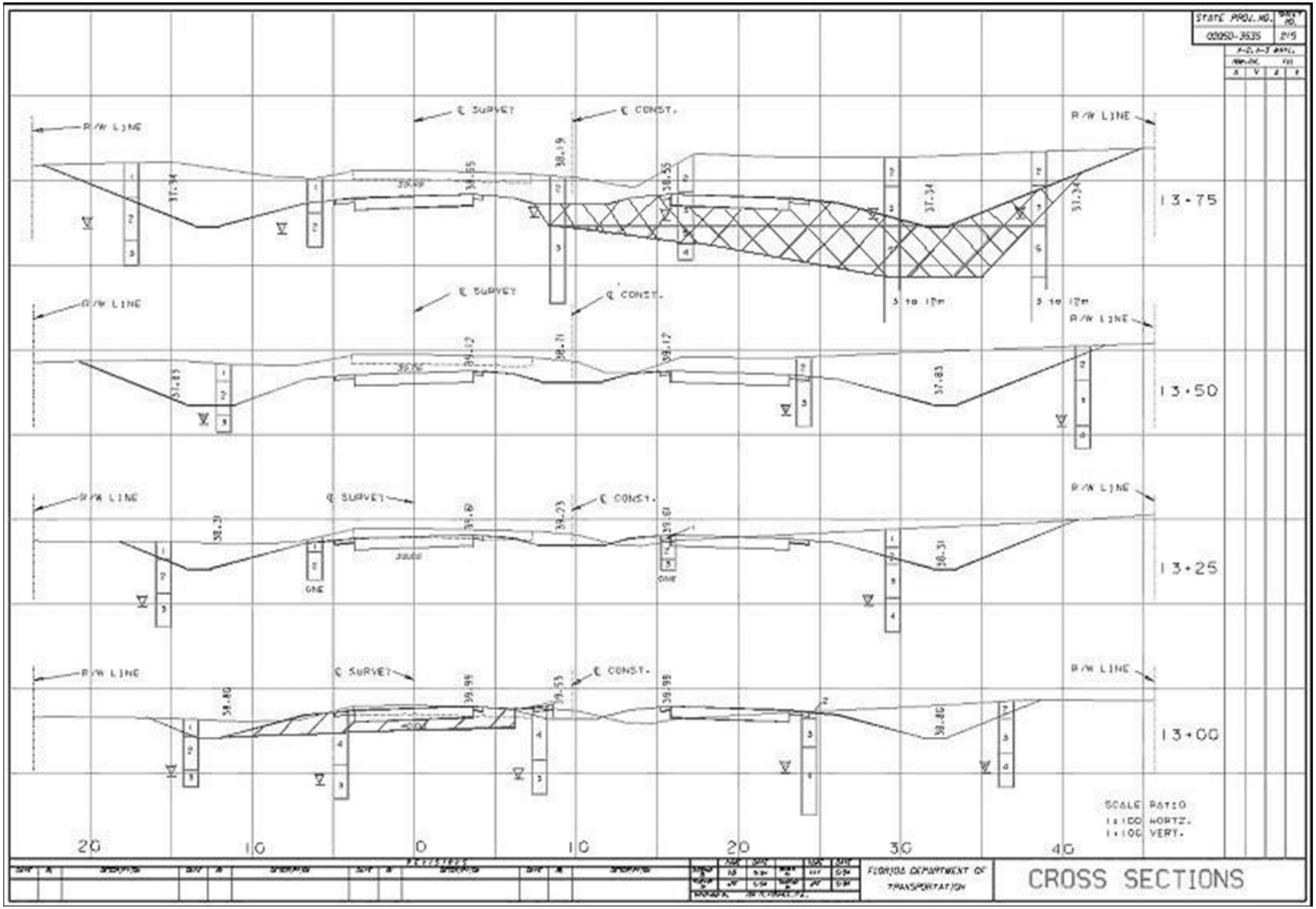


Figure 9-2 Typical Roadway Cross-Section Sheet

Note: The required border may differ

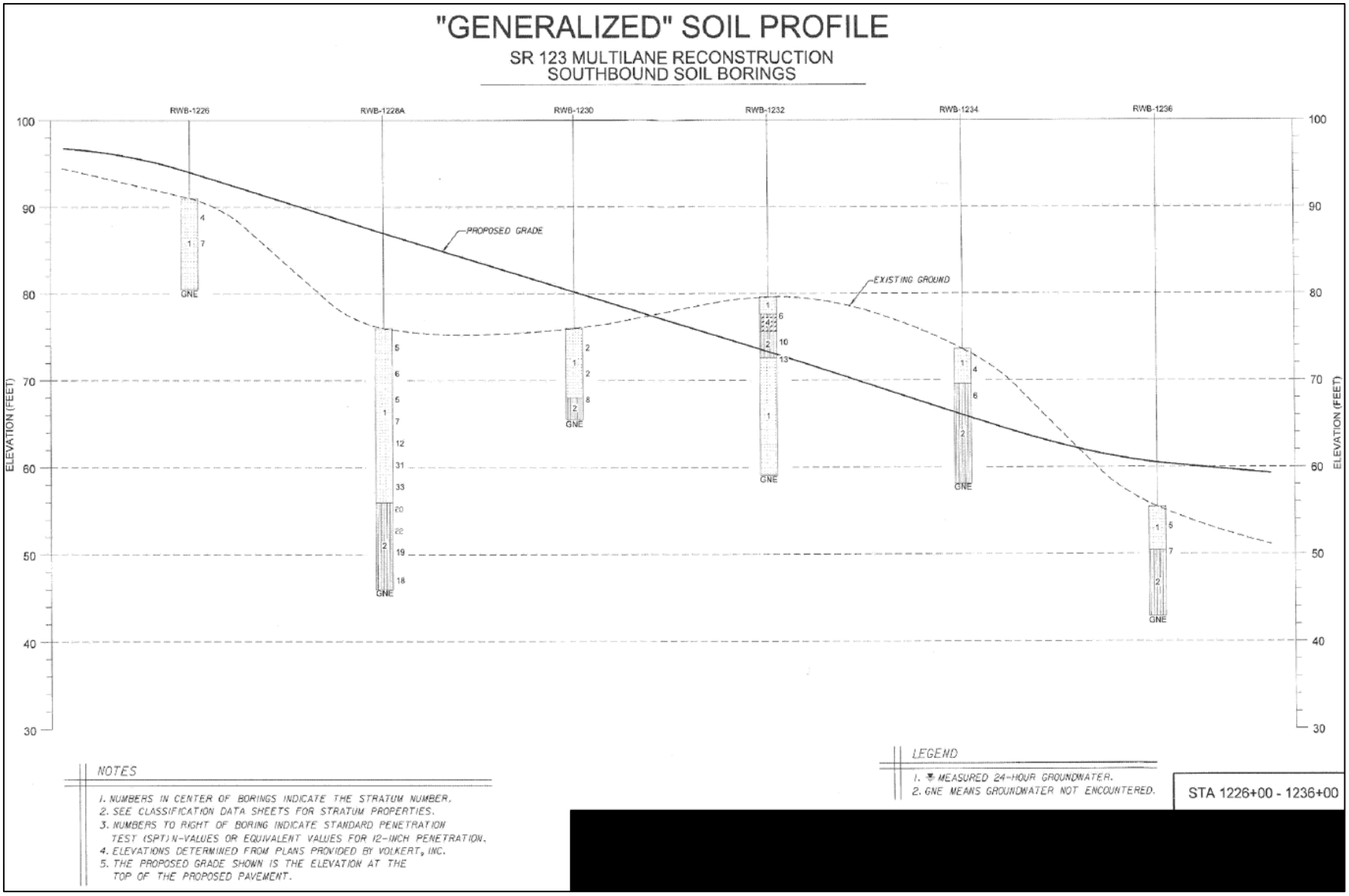


Figure 9-3 Example of Generalized Soil Profile

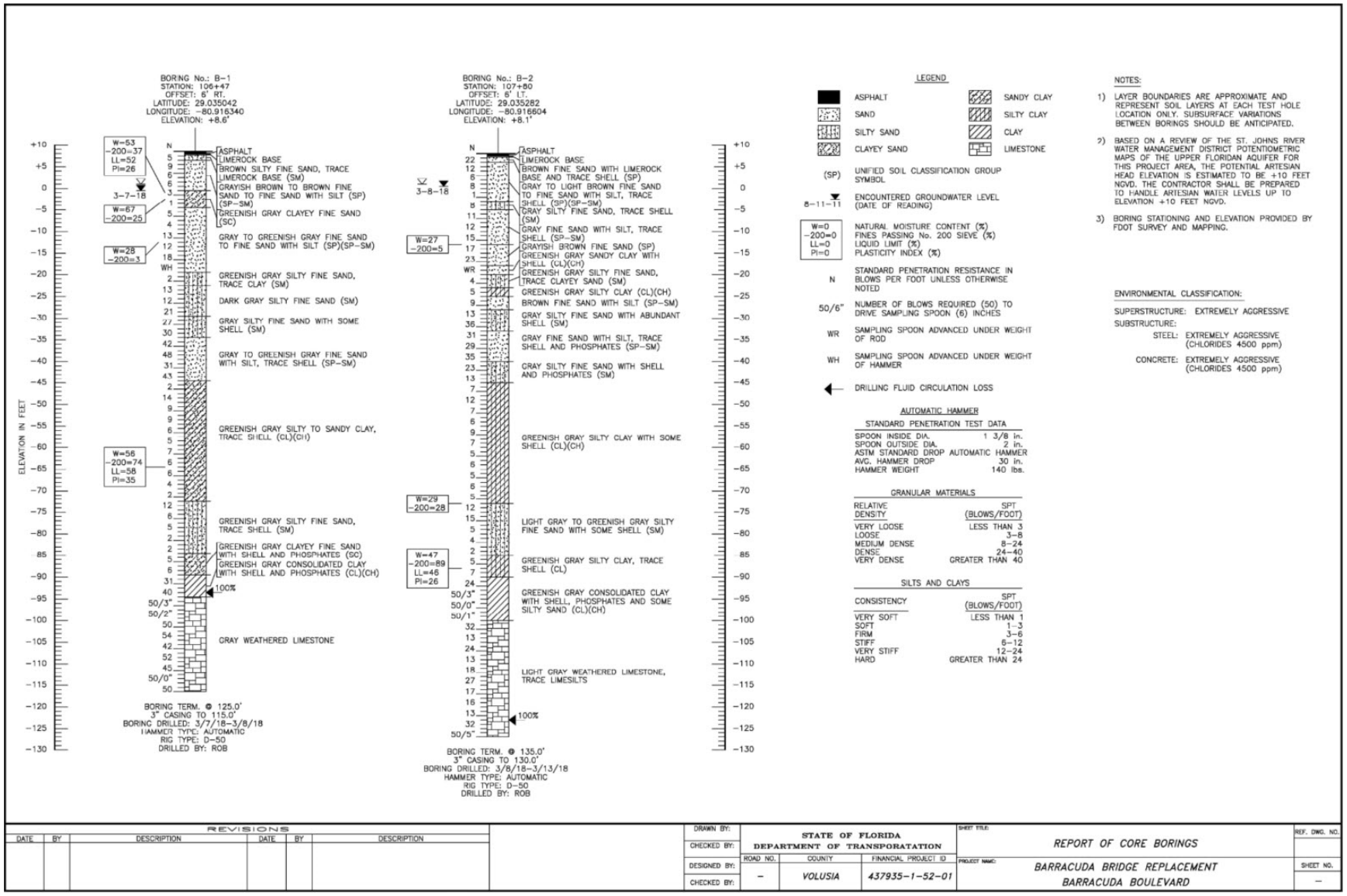


Figure 9-4 Typical Report of Core Borings Sheet

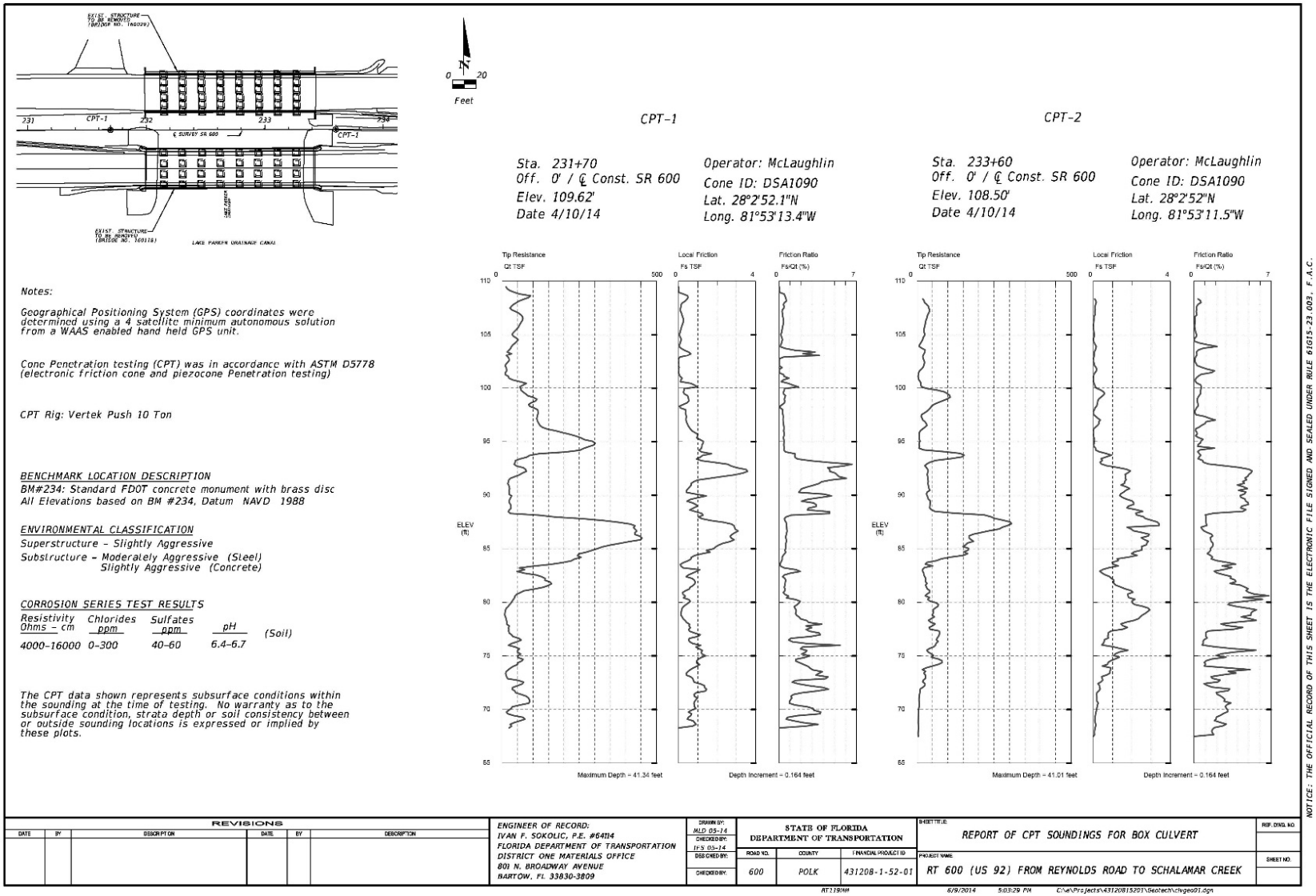


Figure 9-5 Typical Report of Cone Soundings Sheet

Note: Required border may differ

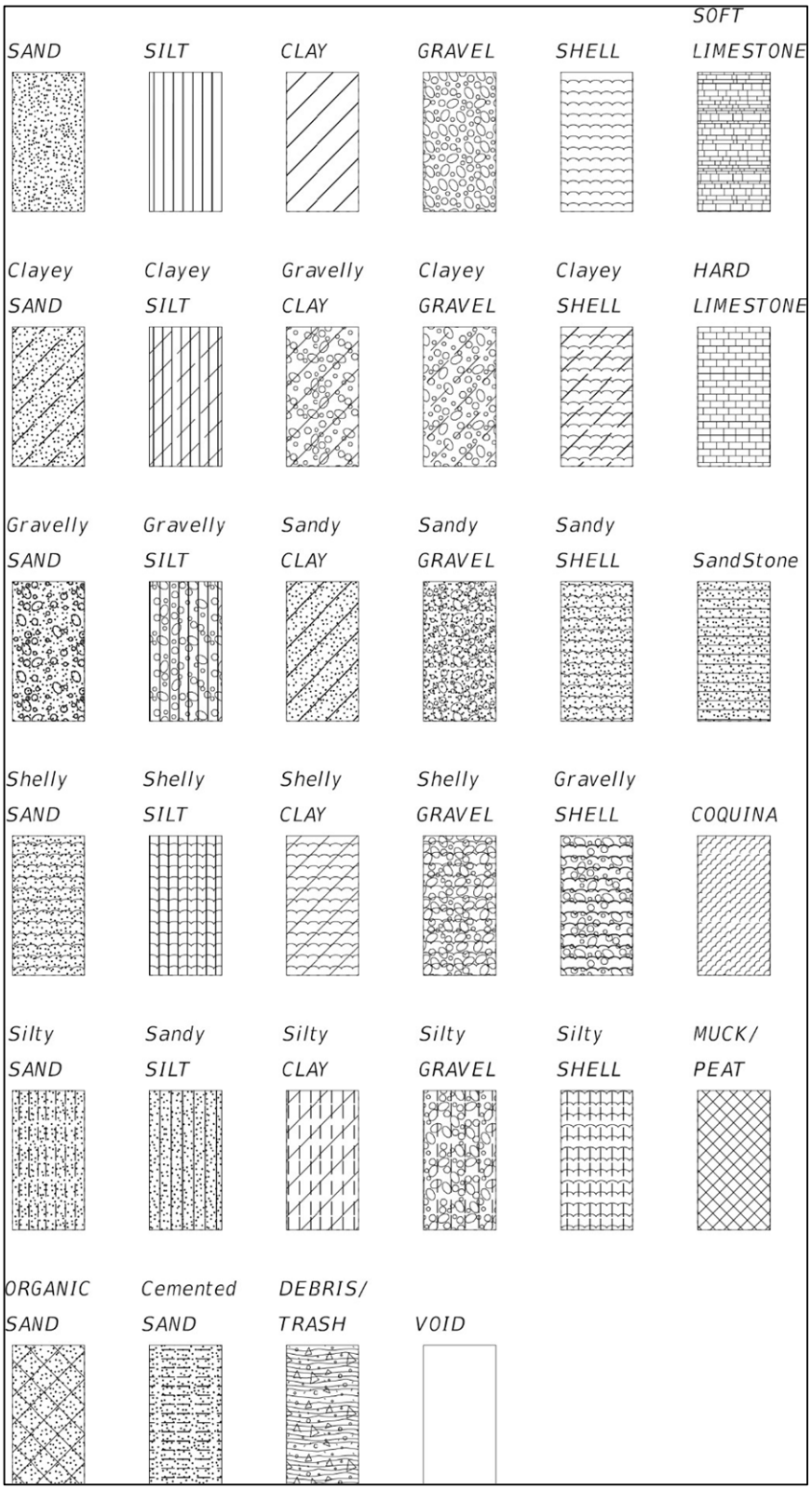


Figure 9-6 Standard Soil Type Symbols

Table 9-2 Applicability of Standard Soil Type Symbols

Symbol	Soil Type
SAND	Sand with ≤ 12% fines
Clayey SAND	Sand with 12% to 50% Clay
Gravelly SAND	Sand with > 30% Gravel
Shelly SAND	Sand with > 30% Shell
Silty SAND	Sand with 12% to 50% Silt
SILT	Silt with LL< 50
Clayey SILT	Elastic Silt
Gravelly SILT	Silt with > 30% Gravel
Sandy SILT	Sand/Silt mixture with > 50% Silt
Shelly SILT	Silt with > 30% Shell
CLAY	Fat Clay
Gravelly CLAY	Clay with > 30% Gravel
Sandy CLAY	Clay with > 30% Sand
Shelly CLAY	Clay with >30% Shell
Silty CLAY	Clay with > 30% Silt
GRAVEL	Gravel with ≤ 12% fines
Clayey GRAVEL	Gravel with 12% to 50% Clay
Sandy GRAVEL	Gravel with > 30% Sand
Shelly GRAVEL	Gravel with > 30% Shell
Silty GRAVEL	Gravel with 12% to 50% Silt
SHELL	Shell with ≤ 12% fines
Silty SHELL	Shell with 12% to 50% Silt
COQUINA	Cemented Coquina
SANDSTONE	Sandstone
MUCK/PEAT	Highly Organic Soils with Organic Content > 20%
Organic SAND	Sand with Organic Content = 5% to 20%
Soft LIMESTONE	Limestone with N ≤ 50
Hard LIMESTONE	Limestone with N > 50
VOID	Apparent Cavity or Void

Chapter 10 Load and Integrity Testing

During construction, inspect in situ materials and construction methods to ensure compliance with the design assumptions and the project specifications. 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. Ensure that sufficient geotechnical information is provided to the qualified inspector and review the procedures and the inspection records as needed.

Where existing structures are sensitive to vibrations or movement, pre-construction and post-construction surveys of the structures are required. Recommend any necessary mitigating action to reduce the impact in the Geotechnical Report. Monitor construction-induced vibrations, groundwater level changes, and/or settlement or heave of the structures as required in the Contract Documents.

10.1 Dynamic Monitoring of Pile Driving

Measurements of the dynamic pile response can be obtained during driving using the Pile Driving Analyzer (PDA), Goble Pile Check (GPC), or Embedded Data Collectors (EDCs). Refer to ***Appendix F*** and ASTM D4945 (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.

Refer to ***Standard Plans***, Index 455-003.

10.2 Load Tests

Involvement in the load testing and interpretation of the resultant data is required. Modify designs if the load tests fail to verify and fully mobilize the design values.

Do not use design values with a load testing resistance factor based on extrapolated trends from underloaded tests.

10.2.1 Static Load Tests

Perform static load tests in accordance with ASTM D1143 for axial compression tests, ASTM D3689 for axial tension tests, and ASTM D3966 for lateral load tests (see ***Figure 10-1***).

10.2.2 Statnamic Load Tests

See ***Figure 10-2*** and ***Figure 10-3***. Instrument drilled shafts tested by the Statnamic method with electronic resistance strain gauges at various elevations to measure load

transfer characteristics. Perform tests in accordance with ASTM D7383, Procedure A.

10.2.3 Other Rapid Load Tests

Refer to ASTM D7383 Procedure B for Alternative Axial Compressive Force Pulse (Rapid) Load Tests. However, these alternative test methods have not been calibrated to static load test results to determine an appropriate resistance factor for Department projects and can only be considered for proof testing, as long as the data collection and analysis involves accepted engineering methods and redundancy (e.g., the use of Unloading Point and signal matching to confirm results). DGE approval is required for their use in Category 1 structures and SGE approval for Category 2 structures.

10.2.4 Bi-Directional Static Axial Compressive Load Tests

See ***Figure 10-4*** for an example. Perform tests in accordance with ASTM D8169.

10.3 Pile/Drilled Shaft Damage Assessment

Assess the quality of deep foundation units by performing quality assurance tests. Ensure these tests are performed by qualified personnel and results are analyzed and interpreted by an experienced engineer.

10.3.1 Pile Integrity Testing

For low strain impact non-destructive testing (pulse-echo, etc.) perform tests in accordance with ASTM D5882. These tests can be used to determine cracks or breaks in driven piles, severe necking or large voids in drilled shafts, and to determine the length of piles for existing structures. See ***Figure 10-5*** for an example.

10.3.2 Crosshole Sonic Logging (CSL)

When performing CSL tests, the design of the reinforcement cage must take the total number and location of the access tubes into account. Concrete mixtures producing large amounts of bleed-water may cause CSL tests to indicate apparent zones of poor-quality concrete. This test method is limited to detecting defects between the access tubes (within the reinforcement cage) used during each test. Perform in accordance with ASTM D6760 and following the guidelines of the ***Standard Specifications*** Section 455.

10.3.3 Gamma-Gamma Density Logging

This test method can be used to detect anomalies outside the reinforcement cage. The use of this method requires approval from the SGE.

10.3.4 Thermal Integrity Testing of Drilled Shafts and ACP

Perform the test in accordance with ASTM D7949 and following the guidelines of the ***Standard Specifications*** Section 455 and ***Developmental Specification*** 455ACP.

10.4 Drilled Shaft Construction

For construction of drilled shafts using the wet method the following control tests must be performed on the slurry to ensure it meets the requirements to perform properly: density

(***FM 3-D4380***), viscosity (***FM 8-RP13B-2***), sand content (***FM 8-RP13B-3***), and pH (***FM 8-RP13B-4***).

Take rock core samples to a minimum depth of five (5) feet and up to 20 feet below the bottom of the drilled shaft excavation of redundant drilled shafts or three shaft diameters below the bottom of the drilled shaft excavation for nonredundant shafts to evaluate the quality of the rock directly below the shaft excavation. Perform coring in accordance with ***2.4.1.4*** using a double wall or triple wall core barrel that is able to provide core samples four (4) inches in diameter or larger and allow the cored material to be removed in an undisturbed state. Refer to ASTM D2113 and ASTM D5079. See Reference ***10-7***.

10.5 Field Instrumentation Monitoring

Refer to ***Chapter 7*** for descriptions of some of the common types of field instrumentation. Post-construction services may be required for sensitive projects that require close monitoring.

Supervise all field instrumentation and ensure instrumentation is installed and readings are taken by qualified personnel. Interpret all data and recommend any necessary action. Communicate with the EOR when required waiting periods determined from actual measurements differ from predicted periods so the project schedule can be properly adjusted.

10.6 Troubleshooting

Investigate unforeseen issues that occur during construction and recommend changes in the design or construction method if necessary to minimize construction downtime. If maintenance problems are determined that have a geotechnical basis, recommend remedial actions that will eliminate or reduce the problems.

10.7 Records

Keep complete records of the geotechnical aspects of the construction and maintenance phases of a project. Note any specialized construction procedures or design changes. Describe construction and maintenance problems and their solutions in detail. Provide this information to the DGE and the SGE in Tallahassee.

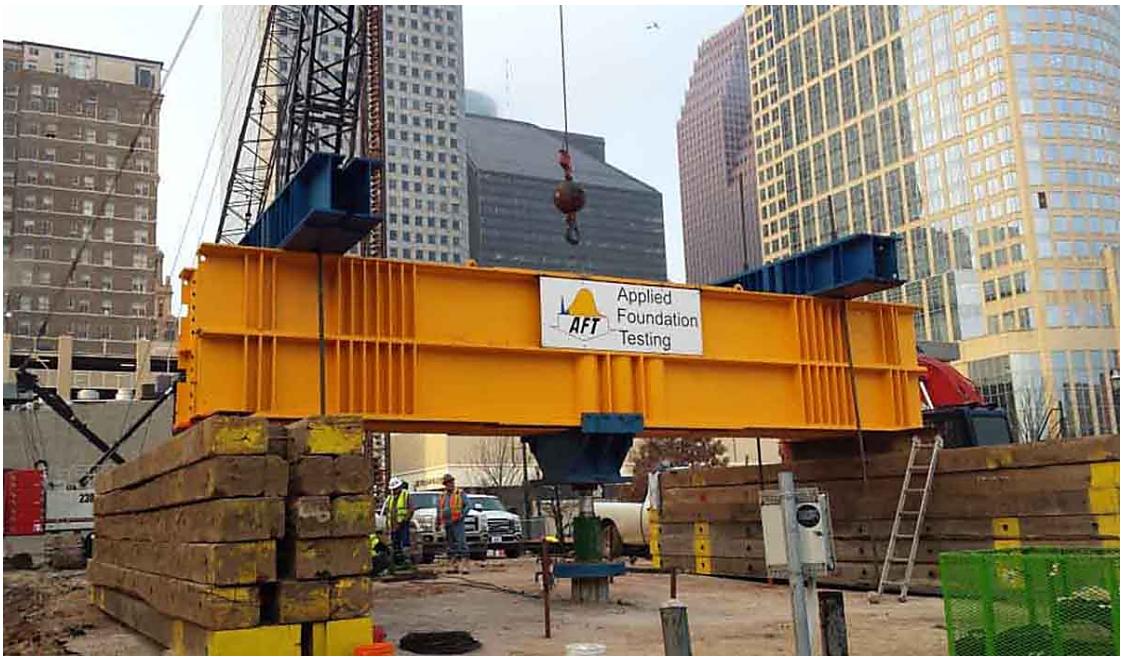


Figure 10-1 Static Load Test (Source: Applied Foundation Testing)



Figure 10-2 Axial Statnamic Load Test (Source: Allnamics)



Figure 10-3 Lateral Statnamic Load Test



Figure 10-4 Bi-directional Load Cell (Source: Fugro)



Figure 10-5 Pile Integrity Tester (After PDI, 1993)



Figure 10-6 Shaft Inspection Device

10.8 References

10-1. Butler, H.D., Hoy, H. E., ***The Texas Quick-Load Method for Foundation Load Testing - User's Manual***, FHWA-IP-77-8, 1976.

10-2. Dunnicliff, J., Geotechnical Instrumentation for Monitoring Field Performance, NCHRP Synthesis 89, Transportation Research Board, 1993.

10-3. Dunnicliff, J., ***Geotechnical Instrumentation***, FHWA-HI-98-034, 1998.

10-4. Goble, G.G. & Rausche, F., GRLWEAP, Wave Equation Analysis of Pile Foundations, GRL & Associates, Inc., 1991.

10-5. Hannigan, P.J., Rausche, F., Likins, G.E., Robinson, B.R. Becker, M.L., Design and Construction of Driven Pile Foundations ***Volume I*** and ***Volume II***, FHWA-NHI-16-009, 2016.

10-6. Kyfor, Z. G., Schmore, A. R., Carlo, T. A., Baily, P. F., ***Static Testing of Deep Foundations***, FHWA-SA-91-042, 1992.

10-7. Mullins, G., Gunaratne M., Sarsour A., Mobley S., ***Load and Resistance Factors Design (LRFD) Resistance Factors for Tip Grouted Drilled Shafts,*** BDV25-977-37, 2019.

10-8. Osterberg, J.O.; ***The Osterberg CELL for Load Testing Drilled Shafts and Driven Piles***, FHWA-SA-94-035, 1995.

10-9. Pile Dynamics, Inc, Pile Driving Analyzer Manual, PAK, Cleveland, Ohio, 1997.

10-10. Paikowsky, S. G., Tolosko, T. A., ***Extrapolation of Pile Capacity from Non-Failed Load Tests***, FHWA-RD-99-170, 1999.

10-11. Wang, S., and Reese, L. C., ***COM624P – Laterally Loaded Pile Analysis Program for the Microcomputer Version 2.0***, FHWA-SA-91-048, 1993.

10.9 Specifications and Standards

Subject	ASTM	AASHTO	FM
Florda Method of Test for Viscosity of Slurry	-	-	8-RP13B-2
Florda Method of Test for pH of Slurry	-	-	8-RP13B-4
Standard Test Method for Piles Under Static Axial Compressive Load	D1143	-	-
Standard Test Method for Individual Piles Under Static Axial Tensile Load	D3689	-	-
Standard Test Method for Piles Under Lateral Loads	D3966	-	-
Florda Method of Test for Density of Slurry	D4380	-	3-D4380
Florda Method of Test for Sand Content of Slurry	D4381	-	8-RP13B-3
Standard Test Method for High-Strain Dynamic Testing of Piles	D4945	T 298	-

Subject	ASTM	AASHTO	FM
Standard Practices for Preserving and Transporting Rock Core Samples	D5079	-	-
Standard Test Method for Low Strain Integrity Testing of Piles	D5882	-	-
Standard Test Method for Integrity Testing of Concrete Deep Foundations by Ultrasonic Crosshole Testing	D6760	-	-
Standard Test Methods for Axial Compressive Force Pulse (Rapid) Testing of Deep Foundation	D7383	-	-
Standard Test Methods for Thermal Integrity Profiling of Concrete Deep Foundations	D7949	-	-
Standard Test Methods for Deep Foundation Elements Under Bi-Directional Static Axial Compressive Load	D8169	-	-

Chapter 11 Non-Conventional Projects

A subsurface exploration that satisfies the requirements of this manual must be completed to provide the Design-Build team with an understanding of the geotechnical conditions of the project. A more extensive geotechnical investigation may be performed to assist the project teams in offering the most cost-effective solution for the project. The Design-build team may still need to perform additional exploration to finalize their project-specific design and is responsible for the data collected and the analysis performed.

11.1 Planning and Development Phase

The Department's geotechnical engineer performs a geotechnical investigation to support the Concept Plans. Perform as complete a geotechnical field and laboratory investigation as access permits and provide the data to the Design-Build teams for use in preparing preliminary designs. Upon completion of the preliminary subsurface investigation, sign and seal the information obtained and compile in a format for presentation of the data collected to the various Design-Build teams. In the evaluation of the subsurface data, establish the limits of areas of relative uniformity for load testing. For preliminary Geotechnical Reports prepared by the Department for use by Design-Build teams, do not include analysis of the geotechnical information or any suggestions for handling any potential problems.

11.2 Technical Proposals and Bidding Phase

11.2.1 Department's Geotechnical Engineer Responsibilities

The Department's geotechnical engineer is responsible for answering questions from the Design-Build team through the Project Manager, reviewing proposals, and providing recommendations to other technical reviewers regarding the completeness and appropriateness of proposed supplemental field testing, ground modification, and load testing programs, etc.

11.2.2 Design-Build Team Responsibilities

Short listed Design-Build teams perform analyses of the geotechnical data and any additional data they gather independently. The teams determine the appropriate design and construction methods based on their approach and equipment, the requirements provided in this manual, and the Contract Documents for the project and submit 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.

Appendix A Design Side Shear Resistance from Rock Core Test Data for Drilled Shafts, Auger Cast Piles, and Micropiles Socketed in Florida Limestone and Cohesive IGM

A.1. Introduction

Characterization of the design strength of Florida limestone and well-cemented Marl that classifies as cohesive IGM, requires a substantial amount of laboratory testing to develop the required statistics for each layer. Thick layers may need to be subdivided to account for variability in the vertical direction and avoid averaging errors. In addition, the project site may also need to be subdivided to account for horizontal variability, and a separate set of statistics developed for each zone. The resistance factors associated with bridge foundation design of drilled in-place deep foundations embedded in rock or competent cohesive IGM are associated to the procedure outlined in this appendix and require a statistically significant number of unconfined compression (ASTM D7012 [Method D]) and splitting tensile (ASTM D3967 [with t/D ≥ 1.0]) tests.

A.2. Design Method

Based on the study performed by the University of Florida, the following method proposed by Professor McVay is appropriate for design of drilled foundations in Florida limestone and cohesive IGM. The ultimate side shear resistance for the portion socketed in the rock is expressed as:

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

Where,

f_{su} is the ultimate side shear resistance,

q_u is the unconfined compression strength of rock core, and

q_t is the splitting tensile strength (McVay, 1992 Reference ***A-1***).

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

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

A.3. Rock Sampling and Laboratory Testing

The rock core sample quality is dependent upon the sampling techniques as well as the size and type of the core barrel used. Due to the porous nature of Florida limestone, the larger diameter samplers are favorable as they will provide more representative specimens. The minimum core barrel size allowed is 61 mm (2.4-inch) interior diameter (I.D.). A 101.6 mm (4-inch) I.D. core barrel is recommended. Furthermore, a triple or double barrel must be used to have accurate percentages of recovery and rock quality designation (RQD). After obtaining quality core samples, the engineer can select representative specimens for laboratory unconfined compression and splitting tensile tests.

A.4. Variability

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

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

This equation is useful to gauge the number of core specimens needed for a particular design confidence level. Generating a frequency distribution of the rock/IGM core test results (both the q_u and q_t results individually) may prove useful to the designer when evaluating the need for additional sampling and testing, and identifying the type of distribution of the data (e.g., normal, log-normal, etc.) GeoStat is an available tool for assessing the variability of the data.

A.5. Check the Big Picture

Determine whether the available data is reasonably consistent, or if generating vertical and horizontal layers is necessary (Paikowsky, 2004, Reference ***A-3***). A histogram of the rock core test results can help identify secondary peaks in the data which suggest the need for subdividing the site.

When the strength characteristics are so variable that trends cannot be reasonably discerned, evaluate the need for more data or a different foundation type.

A.6. Data Reduction Method

The data reduction method outlined herein is designed to yield $\mathbf{q_u}$, $\mathbf{q_t}$, and **REC**, which are used for estimating side shear resistance in rock formations and IGM. This method involves a sequence of analytical steps applied to each area or sub-site within the project boundaries.

1. Find the mean and standard deviations of both the $\mathbf{q_u}$ and $\mathbf{q_t}$ strength test data sets.
2. Establish the upper and lower limits of each type of strength test data set by using the mean values, plus and minus one standard deviation, respectively.
3. Discount all the data in the data sets that are larger or smaller than the established upper and lower limits, respectively.
4. Recalculate the mean and standard deviation of the data within the boundaries of each modified strength test data set computed above.
5. Establish the upper and lower bounds of $\mathbf{q_u}$ and $\mathbf{q_t}$ by setting the calculated mean value as the design upper bound value and the mean minus one standard

deviation as the design lower bound value.

6. Use the q_u and q_t obtained from steps 4 and 5 to calculate the respective upper and lower bounds of the ultimate side shear resistance, f_{su}.

7. Multiply the upper and lower ultimate side shear resistances, f_{su} by the mean **REC** (in decimal) to account for the spatial variability. This step is performed to obtain a global design value. Check against the local recovery at each pier/bent.

8. Consider these two design boundaries the global side shear resistance values, $f_{su(DESIGN)}$, for the area or site within the project. Select the corresponding resistance factor in accordance with the Structures Manual, Volume I – ***SDG***, Table 3.5.20-2, Table 3.6.4-1 or Table 3.6.4-2, as applicable.

9. Generate a chart to index SPT N-values to the global side shear resistance boundaries determined in Step 8. Use the site-specific correlation to reduce the design resistance at elevation intervals where the blow count falls below anticipated values. These correlations are site-specific. Refer to ***3.2*** for requirements related to SPT borings and rock coring for structures.

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

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

Example*: Design a 48-inch diameter shaft in a group of four shafts. Each shaft will support a factored design load of 2,500 kips. Assuming there will be no load tests for the project, a resistance factor of 0.55 will be used for the design.*

- **Steps 1 through 5:** Rock test data reduction

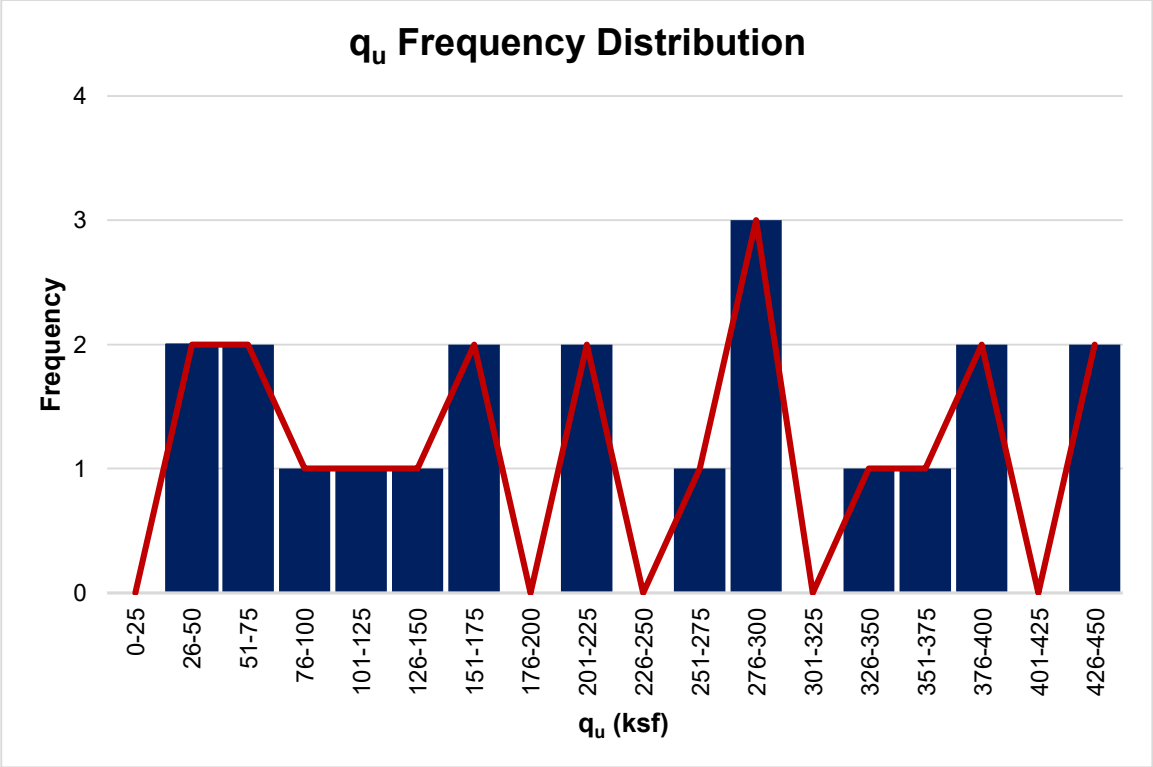


Figure A-1 q_u Frequency

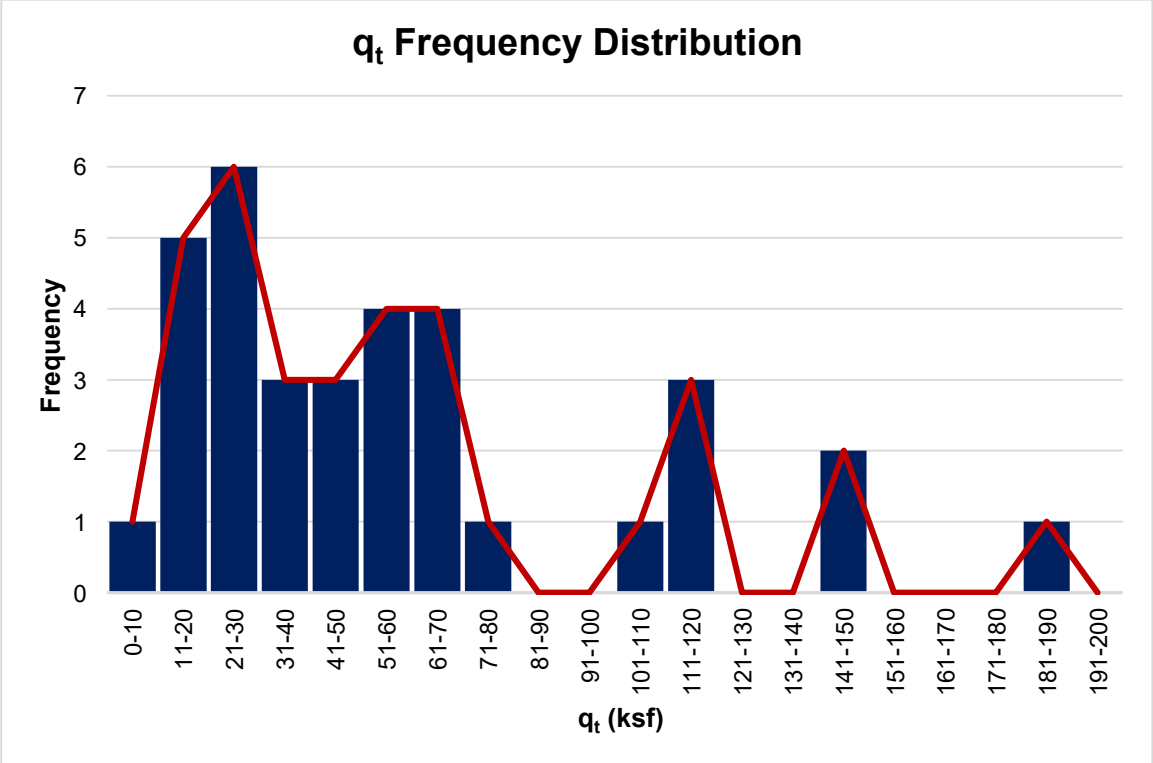


Figure A-2 q_t Frequency

Table A-1 Example Data Set

Boring No.	Core Sample Elevations (ft)		% REC	q_u (ksf)	q_t (ksf)
	Top	Bottom			
B-1	-62.24	-65.42	30		32.2
B-1	-72.42	-75.42	67	97.2	27.4
B-1	-82.42	-87.42	13		114.2
B-2	-36.58	-41.58	18	169.1	
B-9	-74.42	-82.42	5		26.5
B-9	-89.42	-94.40	43		24.7
B-9	-89.4	-94.40	43		32.9
S-12	-30.00	-35.00	60	211.2	68.4
S-12	-35.00	-40.00	48	117.0	19.4
S-12	-50.00	-55.00	48		19.6
B-7	-44.40	-52.40	18		43.5
B-7	-92.90	-97.40	98		26.3
B-7	-97.40	-102.40	66		117.5
B-7	-134.40	-142.40	35	140.6	64.7
B-11	-34.20	-39.20	38		144.0
B-11	-34.20	-39.20	38	379.5	189.1
B-11	-34.20	-39.20	38		112.6
B-11	-76.40	-81.40	33		26.3
B-11	-90.40	-95.40	60		68.7
N-14	-40.00	-43.00	63	389.4	
B-10	-33.40	-41.40	46	283.5	148.8
B-10	-33.40	-41.40	46		52.7
B-10	-46.40	-51.40	69	444.4	49.9
B-10	-46.40	-54.40	69	212.9	60.5
B-10	-46.40	-51.40	69		65.8
B-8	-48.90	-57.90	48	158.7	55.5
B-8	-48.90	-57.90	48	272.8	54.0
B-8	-48.90	-57.90	48		76.8
B-8	-59.90	-67.90	50	285.1	40.4
B-8	-99.90	-107.90	17		14.1
N-17	-58.10	-63.00	33	432.0	45.3
S-15	-48.50	-53.50	55	51.4	
S-15	-48.50	-53.50	55		17.5
S-15	-65.00	-70.00	61	38.4	7.7
B-6	-64.10	-72.10	51	58.2	12.4

Boring No.	Core Sample Elevations (ft)		% REC	q_u (ksf)	q_t (ksf)
	Top	Bottom			
B-6	-74.00	-82	57	365.4	101.4
B-6	-114.00	-122	45		21.0
N-25	-58.80	-63.3	85	26.6	
N-25	-68.80	-73.3	80	281.3	
N-25	-73.30	-78.3	47	331.5	
SUM			1941	4745.7	1981.1
MEAN ($\bar{x}$)			**48.5**	**226.0**	**58.3**
STANDARD DEVIATION (σ)			**-**	**134.3**	**44.3**
UPPER BOUND ($\bar{x} + \sigma$)			**-**	**360.2**	**102.6**
LOWER BOUND ($\bar{x} - \sigma$)			**-**	**91.7**	**14.0**

Use the upper and lower bounds of q_u and q_t as guides to limit the data set so that no data are higher than the upper bound value and no data are lower than the lower bound value. The modified data set is presented in the following table:

Table A-2 Example of Modified Data Set

Boring No.	Core Sample Elevations (ft)		% REC	q_u (ksf)	q_t (ksf)
	Top	Bottom			
B-1	-62.24	-65.42	30		32.2
B-1	-72.42	-75.42	67	97.2	27.4
B-1	-82.42	-87.42	13		~~114.2~~
B-2	-36.58	-41.58	18	169.1	
B-9	-74.42	-82.42	5		26.5
B-9	-89.42	-94.40	43		24.7
B-9	-89.40	-94.40	43		32.9
S-12	-30.00	-35.00	60	211.2	68.4
S-12	-35.00	-40.00	48	117.0	19.4
S-12	-50.00	-55.00	48		19.6
B-7	-44.40	-52.40	18		43.5
B-7	-92.90	-97.40	98		26.3
B-7	-97.40	-102.40	66		~~117.5~~
B-7	-134.40	-142.40	35	140.6	64.7
B-11	-34.20	-39.20	38		~~144.0~~
B-11	-34.20	-39.20	38	~~379.5~~	~~189.1~~
B-11	-34.20	-39.20	38		~~112.6~~
B-11	-76.40	-81.40	33		26.3

Boring No.	Core Sample Elevations (ft)		% REC	q_u (ksf)	q_t (ksf)
	Top	Bottom			
B-11	-90.40	-95.40	60		68.7
N-14	-40.00	-43.00	63	~~389.4~~	
B-10	-33.40	-41.40	46	283.5	~~148.8~~
B-10	-33.40	-41.40	46		52.7
B-10	-46.40	-51.40	69	~~444.4~~	49.9
B-10	-46.40	-54.40	69	212.9	60.5
B-10	-46.40	-51.40	69		65.8
B-8	-48.90	-57.90	48	158.7	55.5
B-8	-48.90	-57.90	48	272.8	54.0
B-8	-48.90	-57.90	48		76.8
B-8	-59.90	-67.90	50	285.1	40.4
B-8	-99.90	-107.90	17		14.1
N-17	-58.10	-63.00	33	~~432.0~~	45.3
S-15	-48.50	-53.50	55	~~51.4~~	
S-15	-48.50	-53.50	55		17.5
S-15	-65.00	-70.00	61	~~38.4~~	~~7.7~~
B-6	-64.10	-72.10	51	~~58.2~~	~~12.4~~
B-6	-74.00	-82.00	57	~~365.4~~	101.4
B-6	-114.00	-122.00	45		21.0
N-25	-58.80	-63.30	85	~~26.6~~	
N-25	-68.80	-73.30	80	281.3	
N-25	-73.30	-78.30	47	331.5	
SUM			**1941**	**2560.7**	**1134.9**
MEAN ($\bar{x}$)			**48.5**	**213.4**	**43.6**
STANDARD DEVIATION (σ)			**-**	**77.0**	**22.2**
LOWER BOUND ($\bar{x}$ - σ)			**-**	**136.4**	**21.5**

- **Step 6:** Determination of the ultimate side shear resistance, f_{su}

  By using the above q_u and q_t values, the following f_{su} values are calculated:

  Ultimate Upper Boundary: $f_{su} = \frac{1}{2} \times \sqrt{213.4} \times \sqrt{43.6} = 48.3\ ksf$

  Ultimate Lower Boundary: $f_{su} = \frac{1}{2} \times \sqrt{136.4} \times \sqrt{21.5} = 27.1\ ksf$

- **Steps 7 & 8:** Rock recovery consideration and establishing the design ultimate side shear boundaries

  The global design ultimate side shear resistance should also account for the spatial

variability of the site by multiplying the mean **REC** value (in decimal) by the above ultimate values.

Commentary: Based on FDOT experience, this is more representative of the high and low side resistance values.

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

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

Select the corresponding resistance factor in accordance with the Structures Manual, Volume I – ***SDG***, Table 3.6.4-1.

- **Step 9:** Side shear resistance chart generation

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

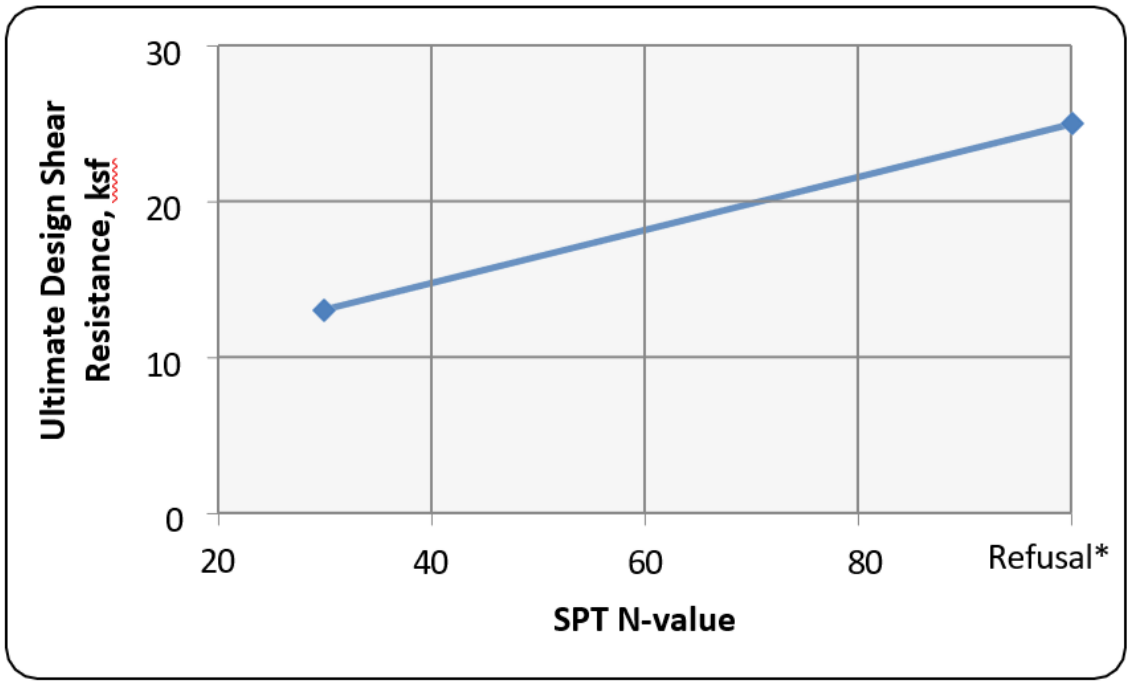
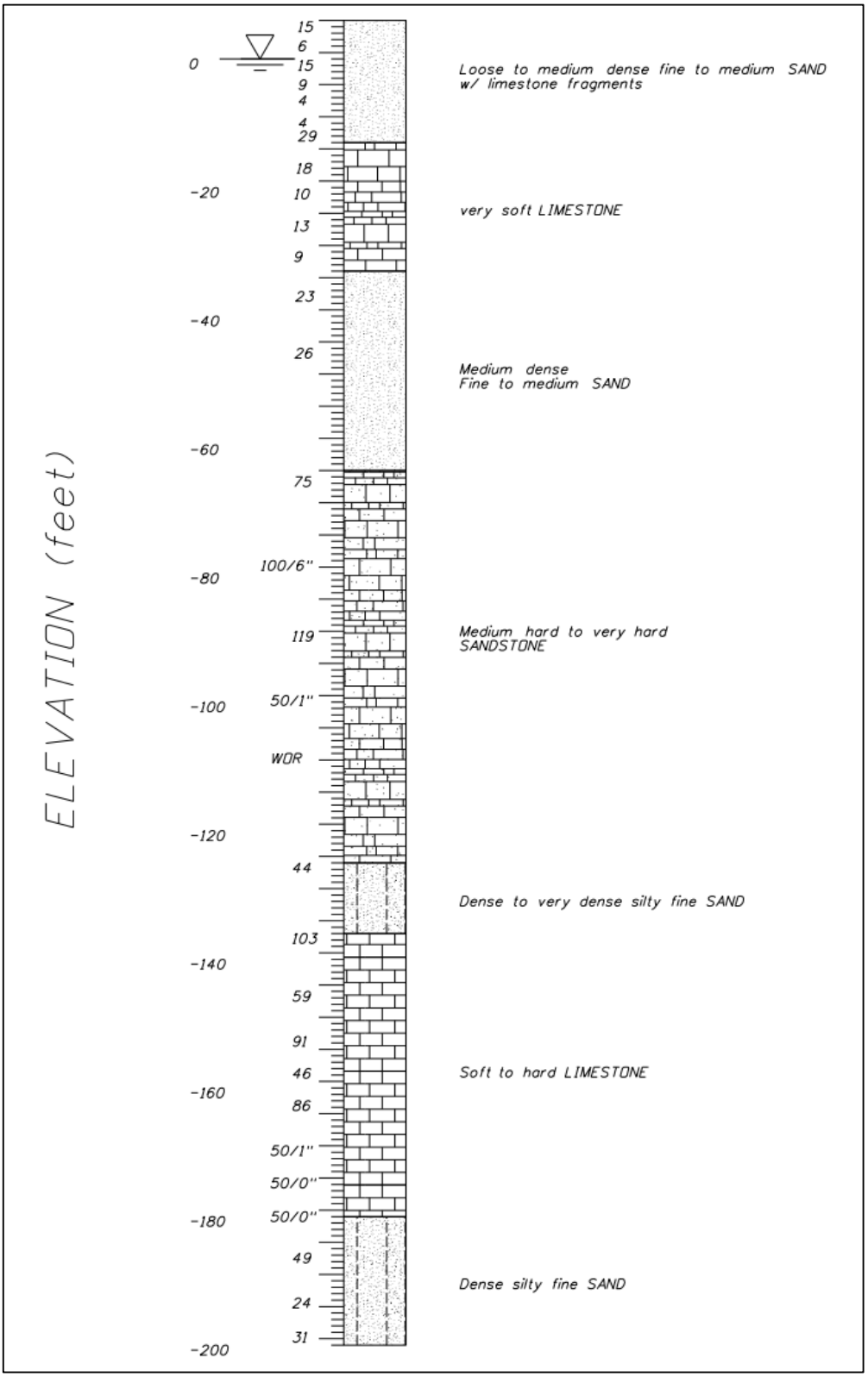


Figure A-3 Ultimate Design Shear Resistance vs. SPT N-value

*Refusal as defined in ASTM D1586.

- **Step 10:** Design of the cast-in-place foundation using local subsurface information – boring log/rock core at the pier location



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

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

Notes:

*Neglected because of high ground water table and possible use of casing.

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

Diameter of shaft, D (ft)	D = 48 inches = 4 feet
Perimeter, P, (ft) $P = \pi \times D$	$P = \pi \times 4\ ft = 12.57\ ft$
Side resistance, R_s, per foot of rock socket (kips/ft) $R_s = P \times f_{su}$	$R_s = 12.57\ ft \times 23.4\ ksf = 294.4\ kips/ft$
Nominal Bearing Resistance, NBR (kips) $NBR = \frac{Factored\ Design\ Load}{\varphi}$	$NBR = \frac{2500\ kips}{0.55} = 4545.5\ kips$
Total required socket length, L (ft)	$L = \frac{4545.5\ kips}{294.4\ kips/ft} = 15.4\ ft$

A rock socket embedment of 15.4 feet into the limestone with a tip elevation of -79.4 feet is required for a foundation designed to provide resistance in friction only.

A.7. References

A-1. McVay, M. C., Townsend, F.C., and Williams, R.C., Design of Socketed Drilled Shafts in Limestone, ASCE Journal of Geotechnical Engineering, Vol. 118, No. 10, 1992.

A-2. O'Neill, M. W. and Reese, L. C., ***Drilled Shafts: Construction Procedures and Design Methods***, FHWA-IF-99-025, 1999.

A-3. Paikowsky, S. G., ***Load and Resistance Factor Design (LRFD) for Deep Foundations***, NCHRP Report 507, 2004.

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

B.1. General

The typical diameter for ACP for noise walls is 30 inches to accommodate the post supports and reinforcement with the required cover. If the design indicates a 30-inch diameter foundation will need to be longer than 30 feet, consider a larger diameter foundation. Refer also to ***8.2.4.1***.

B.2. Noise Barrier Foundations

Refer to ***8.2.4.1***.

B.3. Lateral Load Resistance

Use a Load Factor in accordance with the *AASHTO Code*.

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

B.3.1. k values in Sands

For structures subject to lateral loads due to a storm event, k values input into FB-MultiPier, LPILE, or COM624 shall not exceed values in ***Figure 8-1*** without lateral load tests. Since submerged conditions are likely to exist when the design load condition occurs, make no distinction between dry and submerged conditions.

B.3.2. k values in Clay

Refer to ***Table 8-2*** to determine k value for clay and limit the properties of clay to stiff clay or weaker (design values for undrained shear strength not exceeding 1 tsf and ε_{50} of 0.007), unless laboratory stress-strain measurements indicate otherwise.

B.4. Friction Angles in Sand

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

$$\phi = \frac{N_{1-60}}{4} + 28$$

As an alternative, use the procedure described in ***6.1.1.5***. Limit the maximum ϕ value to 35 degrees for silty sand (A-2-4) and 38 degrees for clean sand (A-3), unless higher friction angles are statistically supported by laboratory shear strength test results.

B.5. Rock

Less conservative designs require more sampling and testing to demonstrate the design values are appropriate in all locations. In the absence of a comprehensive sampling and testing program, the design based on SPT borings shall be as follows:

Model rock material with N-values less than 10 as sand. Model rock material with N-values between 10 and 20 as sandy gravel using the following correlation for ϕ:

$$\phi = \frac{N_{60}}{4} + 33$$

Limit the maximum friction angle value to 40 degrees, unless higher friction angles are statistically supported by laboratory shear strength test results.

Rock material with N-values of 20 or more:

- Use the FB-Multipier, 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 or more and reasonable conservatism in the selection of k and undrained shear strength are adopted.

Note: Axial Load Resistance does not normally control the design of noise barrier foundations.

B.6. Side Resistance in Sands

Compute side resistance in cohesionless soils by the FHWA Method (Beta Method) specified in the Publication FHWA-IF-99-025 (Reese and O'Neil, 1999) for drilled shafts as follows:

$$f_s = \sigma'_v \times \beta_c$$

Where,

f_s = Ultimate unit side resistance

σ'_v = Effective vertical stress

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

$\beta = 1.5 - 0.135(z^{0.5})$; z is depth in ft and 1.2 ≥ β ≥ 0.25

*N is uncorrected.

The maximum value of $\mathbf{f_s}$ shall be limited to 2.1 tsf, unless load test results indicate otherwise.

B.7. Side Resistance in Rock

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

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

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

  $f_s = 0.1N_{60}\ (tsf)$ where $f_s \leq 4\ tsf$

B.8. Side Resistance in Clay

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

The shear strength of clay estimated from SPT-N values or CPT results shall not exceed

2000 psf, unless laboratory stress-strain measurements indicate otherwise.

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

$$f_s = \alpha \times s_u$$

Where,

s_u = Design undrained shear strength of clay (psf)

α = A dimensionless correlation coefficient as defined below:

α = 0 between 0 to 5 feet depth and a distance of B (the pile diameter) above the base

$\alpha = 0.55$ for $1.5 \geq \frac{S_u}{p_a}$

$\alpha = 0.55 - 0.1\left(\frac{S_u}{p_a} - 1.5\right)$ for $2.5 \geq \frac{S_u}{p_a} \geq 1.5$

α = ***Figure B-1*** for $\frac{S_u}{p_a} > 2.5$

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

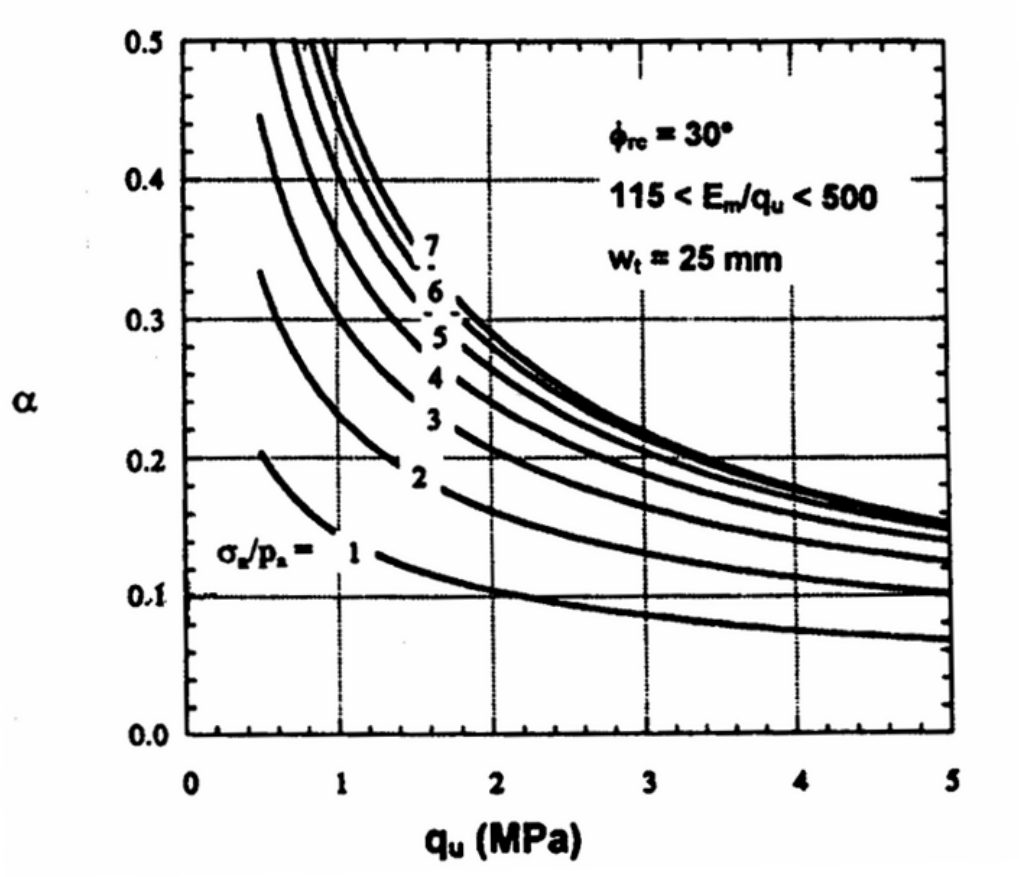


Figure B-1 Factor α for IGMs (O'Neill et al., 1996)

B.9. Organic Soils

Neglect any side resistance in soils with an organic content greater than 5.0% per AASHTO T 267 (ASTM D2974).

B.10. End Bearing Resistance

Neglect any end bearing resistance.

B.11. Factors of Safety and Resistance Factors

For LRFD design, use Load Factors in accordance with the *AASHTO Code* and a Resistance Factor of 0.6.

To compute an allowable axial load, use a minimum factor of safety of 2.0. The service axial load must not exceed this allowable load.

B.12. Design Water Table

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

For load conditions not associated with storm related wind loads, use the seasonal high water table estimated for the location for computation of axial resistance and lateral load

analysis. If no information is available to determine the seasonal high water table, assume the water table is at the ground surface. Include a justification for the selected design water level in the foundation analysis.

B.13. SPT Energy Corrections

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

B.14. Use of Static Cone Penetrometer Tests

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

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

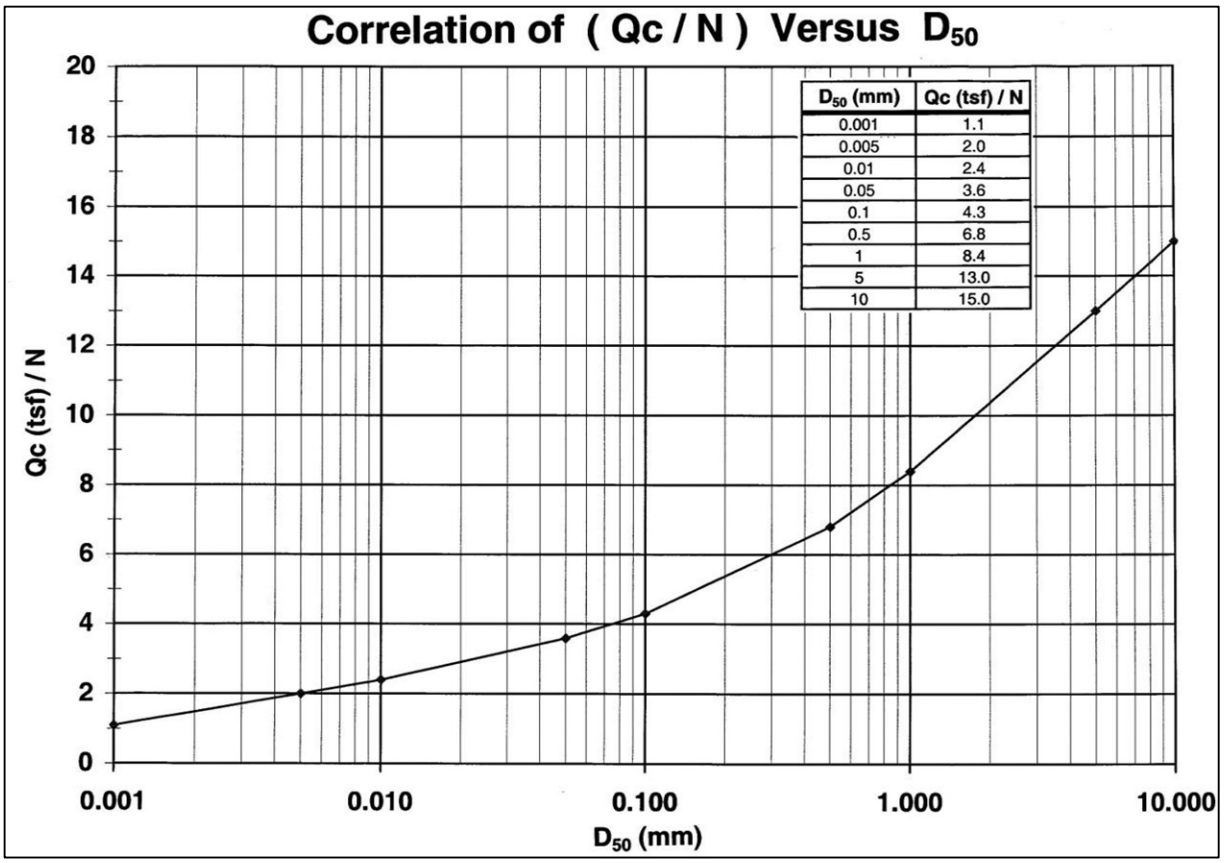


Figure B-2 Correlation of (Q_c/N) vs. D_{50}

B.15. Required Computations for Geotechnical Review

Reports, Shop Drawings, CSI submittals, and Design-Build submittals, must include calculations and numerical program outputs of all the cases and loadings considered in the design. Include copies of structural calculations indicating wind loads computations and structural deflections at the top of the wall (due to pole and panel bending) in the geotechnical package of computations.

Appendix C Step by Step Design Procedure for the Analysis of Drag Load

C.1. Negative Shaft Resistance or Drag Load

Perform settlement analysis to determine the amount of relative displacement between the foundation element and the surrounding soil. When the settling soil around the foundation element equals or exceeds one-half inch (½ inch) of relative displacement, drag loads are anticipated to take full effect, and the maximum value of negative shaft resistance is equal to the soil-pile adhesion.

STEP-BY-STEP DESIGN PROCEDURE FOR ANALYSIS OF DOWNDRAG LOADING

1) Establish the soil profile and soil properties for computing settlement.
2) Determine the overburden pressure increase, **Δp**, versus depth due to the approach embankment fill. An acceptable hand method is included at the end of this appendix.
3) Perform settlement computations for the soil layers along the embedded length.
   a) Determine the consolidation parameters for cohesive layers from Shelby tube samples that have been tested in the laboratory and the elastic modulus for granular soil layers.
   b) Compute the settlement of each soil layer.
   c) Compute the total settlement over the embedded pile length, i.e. the sum of the settlement from each soil layer and partial soil layers. Do not include soil settlement below the pile toe.
4) Determine the pile/shaft length that will experience negative shaft resistance (drag load), that is, the length of pile/shaft that will experience downward movement of the surrounding soil equal to or in excess of ½-inch of the anticipated pile/shaft settlement.
5) Determine the magnitude of negative shaft resistance (drag load), **DR**. The method used to calculate the ultimate negative shaft resistance over the pile length determined in Step 4 is the same method used to calculate the ultimate positive shaft resistance, except that it will act in the opposite direction.
6) Calculate the available positive resistance, $\mathbf{R_p}$, provided by the sum of the positive shaft and the toe resistances. Positive shaft and toe resistances will develop below the depth where the relative pile/shaft-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 section.
7) Calculate the net ultimate pile/shaft resistance, $\mathbf{R_{net}}$ available to resist imposed loads.

$$R_{net} = R_p - DR$$

8) For driven piles, calculate the drag load for inclusion in the Pile Data Table as follows:

$$Drag\ Load = DR + Driving\ Resistance\ of\ Soil\ Contributing\ to\ DR$$

$$R_n = \frac{FDL + Net\ Scour + Drag\ Load}{\varphi}$$

Where: φ is the resistance factor taken from Table 3.5.7-1 of ***SDG***. During initial drive, the driving resistance of the soil contributing to **DR** equals to 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 **DR** typically equals 1.0 times the ultimate skin friction because the excess pore pressures that built-up during initial drive will have dissipated.

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

Commentary: It is 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. Drag loads can also develop whenever the effective overburden pressure is increased on a compressible layer through which a deep foundation element is installed due to lowering of the ground water table, for example.

C.2. Overburden Pressure Determination

Determine the overburden pressure increase, **Δp**, versus depth due to the approach embankment fill.

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

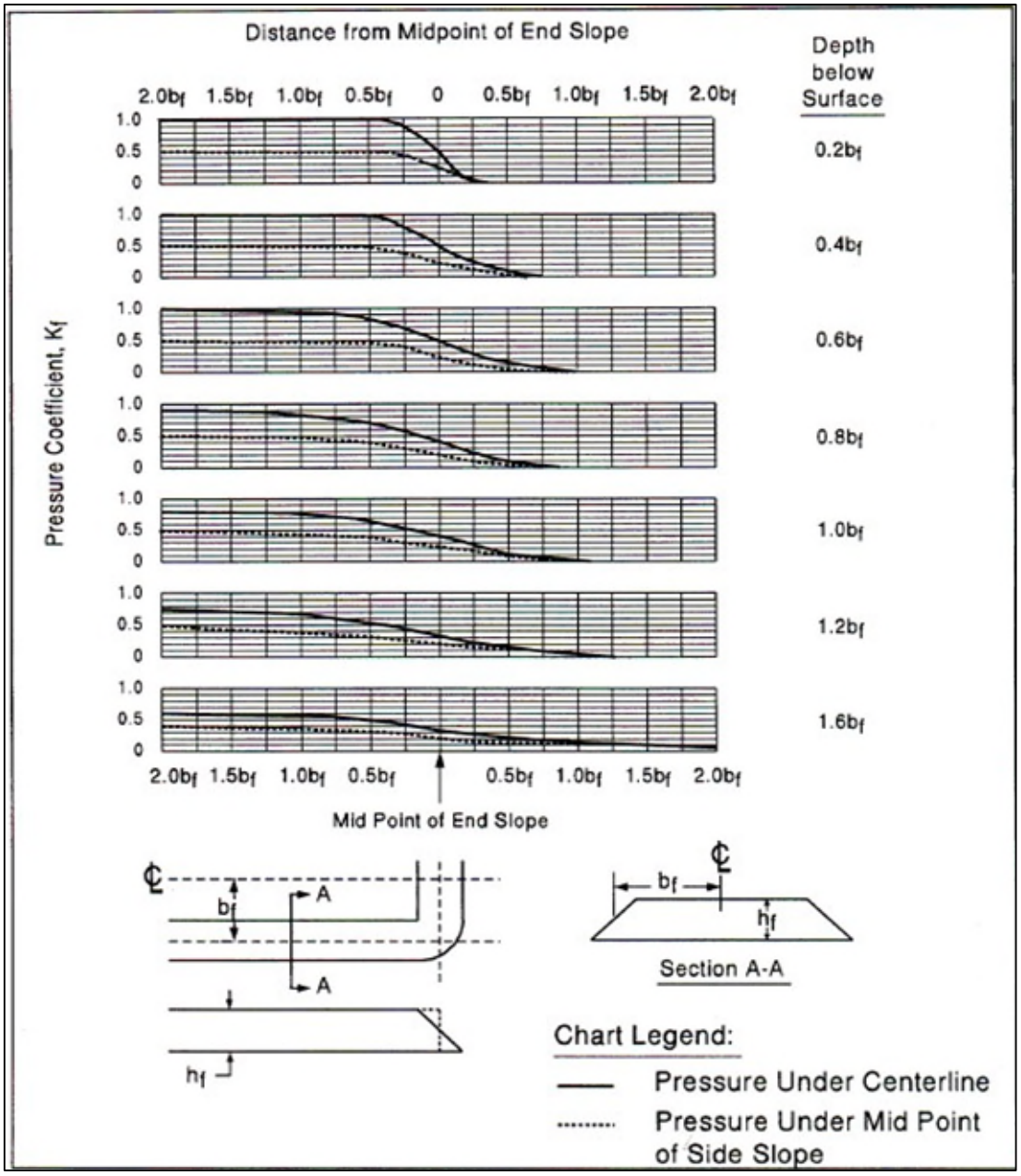


Figure C-1 Pressure Distribution Chart Beneath the End of a Fill (After Cheney and Chassie, 1993)

Appendix D Design Method for Drilled Shaft with Pressure Grouted Tip

D.1. Procedure 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 the 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 **$F_{s\text{-}uplift}$**:

$$F_{s-uplift} = F_s \times Uplift\ Reduction\ Multiplier^{\dagger}$$

†O'Neill cited uplift resistance of shafts to be 0.75 of compression/downward loading.

3) Calculate the ungrouted nominal unit tip resistance of the drilled shaft **q_{tip}*** for 5% diameter tip settlement as per the *AASHTO Code* 10.8.2.2.2.

*The 5% settlement is the default value used in FB-Deep for drilled shafts founded in cohesionless soils. The FB-Deep formula (q_{tip} = 0.6 x N_{60}, in tsf) can be used, where 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, **$F_{s\text{-}uplift}$**, by the cross-sectional area of the shaft, **A_{cross}**:

$$GP_{max} = \frac{F_{s-uplift}}{A_{cross}}$$

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):

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

6) Determine the Tip Capacity Multiplier (TCM) using the following equation:

$$TCM = 0.713(GPI) + 0.3$$

7) Estimate the grouted unit tip resistance, **$q_{grouted}$**, as the product of the Tip Capacity Multiplier (Step 6) and the ungrouted unit tip resistance (q_{tip}), (Step3).

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

8) Compute the nominal tip resistance

$$R_{n-tip} = (q_{grouted})(A_{tip})^{\dagger\dagger}$$

††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**:

$$R_n = R_{n,side\ shear} + R_{n,tip}$$

10) Compute the factored resistance **R_R**:

$$R_R = \varphi(R_{n,side\ shear} + R_{n,tip})$$

Note that the side shear is assumed to develop with very little displacement, thus allowing for the use of this ultimate value. Care must 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% diameter tip settlement.

D.2. Design Example

Given a 3 ft diameter drilled shaft tipped in sand (SPT $N_{60,tip}$ = 30 and F_s = 300 tons).

- Calculate the nominal uplift side shear resistance:

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

- Calculate the nominal end bearing at 5% diameter tip settlement:

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

- Calculate the maximum anticipated grout pressure:

$$GP_{max} = \frac{225\ tons}{\frac{\pi(3\ ft)^2}{4}} = 31.8\ tsf$$

- Calculate the grout pressure index (GPI):

$$GPI = \frac{31.8\ tsf}{18\ tsf} = 1.77$$

- Calculate the Tip Capacity Multiplier (TCM):

$$TCM = (0.713 \times 1.77) + 0.3 = 1.56$$

- Calculate grouted unit end bearing capacity*:*

$$q_{grouted} = (TCM)(q_{tip}) = (1.56)(18\ tsf) = 28.1\ tsf$$

- Nominal Side and Tip Resistances after grouting:

$$R_{n\ side\ shear} = 300\ tons$$

$$R_{n,tip} = \left(q_{grouted}\right)\left(A_{tip}\right) = (28{\cdot}1\ tsf)\left(\frac{\pi}{4}\right)(3ft)^2 = 199\ tons$$

$$R_n = 300\ tons + 199\ tons = 499\ tons$$

$$\boldsymbol{R_R = 0.6(300\ tons + 199\ tons) = 299\ tons}$$

D.3. References

D-1. O'Neill, M. W., Side Resistance in Piles and Drilled Shafts, The Thirty-Fourth Karl Terzaghi Lecture, ASCE Journal of Geotechnical and Geoenvironmental Engineering, 127:3-6, 2001.

Appendix E Reinforced Embankment Design Method

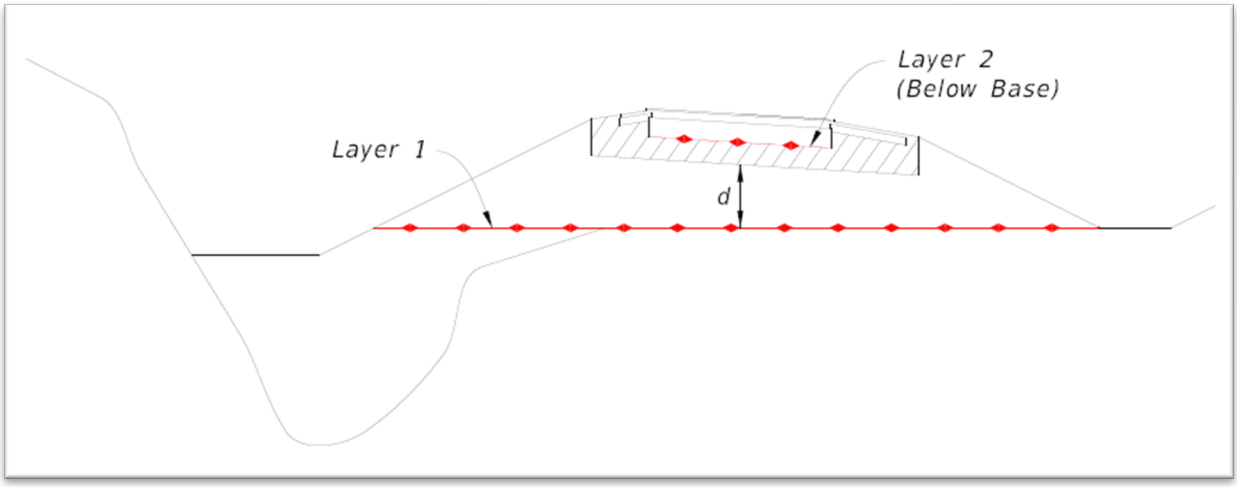


Figure E-1 Reinforced Embankment Design method

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

Step by step instructions are as follows:

1) Loads
   a) Determine construction loads.
   b) Normal highway wheel loads are assumed for this design method.
   c) If wheel loads will exceed legal highway wheel loads, contact a proprietary designer.
2) Strength
   a) Estimate the strength of in situ soils using Vane Shear, CPT, DMT, PMT, etc.
   b) For cohesive layers obtain Shelby tubes and perform laboratory testing to determine s_u.
   c) If s_u design < 250 psf, STOP and use Reinforced Foundation over Soft Soils in ***FDM*** Chapter 263.
3) Determine the **minimum** depth below stabilized subgrade to Layer 1 from ***Table E-1***.
4) Determine the required geosynthetic allowable tensile strength, T_R, from ***Table E-1***.
5) Determine surcharge requirements.
   a) Use 5 feet minimum surcharge height.

b) The anticipated surcharge time must be based on the project-specific calculated time-rate settlement.

6) Verify global stability.
   a) Increase **T_R** and/or surcharge requirements as required.
7) Design the flexible pavement.
   a) Contact the District Materials Office for guidance in selecting the design **M_R** value for the reinforced structural fill layer.
8) Detail the Plans with the required location and **T_R** of the R-4 geogrid or geotextile. The Contractor will choose an R-4 material from the APL.

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

where,

T_{ult} = Ultimate tensile strength

T_R = Required tensile strength

RF_{ID} = Reduction factor for installation damage

RF_D = Reduction Factor for Durability

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

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

Appendix F Determination of Bearing Acceptance Criteria for Driven Piles and Determining the Capacity of a Pile from an Instrumented Set-Check

F.1. Determination of Bearing Acceptance Criteria for Driven Piles

Piles must be installed to the Nominal Bearing Resistance (NBR) in the Plans. For details on the computation of NBR refer to ***SDG***, Chapter 3.

Evaluate the potential effect of nearby construction activities on pile capacity using acceptable theoretical methods and engineering judgment. Consider the influence of jetting concrete sheet pile or vibratory installation/removal of steel sheet pile in the vicinity of foundation piles 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 3.5.7-1 for an exhaustive list). Unless approved by the Director of Design, all driven pile bridge foundations require 100% dynamic testing.

1. Standard pile driving criteria with dynamic monitoring equipment with Pile Driving Analyzer (PDA) monitored test pile(s) or monitored production pile(s) in projects without test piles, using signal matching software such as CAPWAP, and Wave Equation Analysis. The dynamic monitoring equipment will normally utilize a program, such as the PDA's PDIPlot program described in this appendix, for viewing the results. The discussions on this method below use the terms "PDA", "CAPWAP", and "PDIPlot" for simplicity.
2. Standard pile driving criteria (similar to method 1) with Goble Pile Check (GPC) dynamic monitoring equipment monitored test pile(s) or monitored production pile(s) and N_GAPA signal match analyses.
3. Embedded Data Collector (EDC) monitoring of all Test Piles and all Production Piles (100%), using top and tip gauges with FDOT method of analysis on all piles, or a combination of piles with top and tip gauges and piles with only top gauges, as outlined in ***F.1.3*** 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 ***F.1.4*** of this Appendix.
5. GPC monitoring of all Test Piles and all Production Piles (100%), with manual N_GAPA signal match analyses on a minimum percentage of the piles in each bent/pier required in Section ***F.1.5*** of this Appendix.

F.1.1. Standard Driving Criteria with PDA Test Piles

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

In this method dynamic load tests are initially performed on test piles or indicator production piles and a resistance factor (ϕ) of 0.65 may be used in the computation of the required NBR. Dynamic Load tests are performed in accordance with ***Standard Specifications*** Section 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:

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 the static resistance distribution is reasonable. Compare with boring logs and pile driving records to ensure reasonable assumptions are being implemented. Do not rely on the results of the automatic search feature to provide an accurate resistance distribution.
- **Match Quality Number (MQN):** Make every reasonable attempt to obtain an 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%), and 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 the 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. Include in the letter 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 Project 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 cannot 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 achieving the static resistance) as outlined below.

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.

F.1.2. Standard Driving Criteria with Goble Pile Check (GPC) Test Piles

This method is applicable for Standard Driving Criteria with Goble Pile Check (GPC) Test Piles or monitored indicator production pile(s) in projects without test piles with 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 used in the computation of the required NBR. Dynamic Load Tests are performed in accordance with ***Standard Specifications*** Section 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, ***Standard Specifications*** Section 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:

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) force 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 must 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.

c. 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 the 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 direction on when to accept piles. Include in the letter 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 to 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 Project Administration Manual*** (CPAM) Chapter 10.1, sample letters.

e. Additional Considerations

It is important to note that the driving criteria applies to the soil/rock material encountered at the elevation at which Signal Match analysis was performed. Piles that satisfy the driving criteria within different soil/rock strata need to be evaluated to confirm resistance has been achieved. In addition, driving criteria based on initial drive must 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 such as 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 achieving the static resistance) as outlined below.

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

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

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

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

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. 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.

EDC piles are monitored in the field using Smart Structures' UF Method. All piles are subsequently analyzed by the FDOT method to ensure sufficient resistance of the bearing layer.

b. EDC monitoring of all Test Piles and all Production Piles, using a combination of top and tip gauges and top only gauges.

1. Under this approach, the resistance factor, **ϕ**, for EDC is 0.75.
2. Use top and tip gauges in at least 10% of the piles (minimum one per bent/group) and top only gauges in the remaining piles. All test piles shall contain top and tip gauges. Test piles are included in the 10% minimum. In unique soil conditions such as extreme scour, large uplift loads, or high variability soils, a higher percentage of FDOT Method analyses is required and thus a higher percentage of piles with top and tip gauges.
3. In the field, use the UF Method of analysis during driving and confirm pile resistance with the FDOT Method after driving is complete for the piles instrumented with top and tip gauges. Use the Fixed Jc/Case Method with back computed/selected **Jc** value (as described in the below points) for piles instrumented with top only gauges.
4. For the piles instrumented with top and tip gauges, review the FDOT Method results for at least the first 10 blows in the six inches of the drive qualifying the pile for acceptance and use the Fixed Jc/Max Case Method equation to back compute the damping (**Jc**) value from the FDOT Method capacity for the representative blow.
5. In the event the back computed **Jc** value using FDOT method appears to be out of an acceptable range (<0.1 or greater than 1.0), use the UF method capacity and good engineering judgment to determine **Jc**.
6. When more than one pile in a bent/group must be analyzed, select the highest **Jc** value of the analyzed piles for the bent/group and/or good engineering judgement to determine which production piles will be based on which **Jc** value.
7. When the need for set-checks is anticipated, the **Jc** value for set-check conditions will be higher than for initial driving. Therefore, the above

procedure must be repeated on a set-checked pile at the required set-up periods with top and 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 DGE.

F.1.4. PDA monitoring of all Test Piles and all Production Piles (100%), with CAPWAP analysis of the percentage of the piles in each bent/pier required in the Specification.

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

F.1.5. GPC monitoring of all Test Piles and all Production Piles (100%), with manual N_GAPA analysis of the percentage of the piles in each bent/pier required in the Specification.

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

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

Apply the procedure below where soil set-up or freeze is anticipated.

In accordance with section 455-5.11.4 of the ***Standard Specifications***, 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.

Disregard the last blow, which is typically a low energy blow after hammer was shut down. See examples on next page.

Develop a project-specific approach with concurrence from the DGE when set-checks are performed to evaluate the potential for relaxation.

Examples:

Example 1: Instrumented set-check with minimum blows

Blow #	Capacity, kips
1	450
2	600
3	590
4	585
5	580
6	575
7	570
8	277

a. Highest capacity recorded = 600 kips
b. Average of next 5 blows/0.95

$$\frac{(590+585+580+575+570)/5}{0.95} = \frac{580}{0.95} = 610\ kips$$

Answer = 600 kips

Example 2: Instrumented set-check and advance pile

Blow #	Capacity, kips
1	450
2	600
3	590
4	585
5	580
6	575
7	570
8	572
9	550
10	530
11	528
12	520
13	513
14	509
15	501
16	494
17	478
18	461
19	216

a. Highest capacity recorded = 600 kips
b. Average of next 5 blows/0.95 =

$$\frac{(590+585+580+575+570)/5}{0.95} = \frac{580}{0.95} = 610\ kips$$

c. Lowest capacity of the following blows (excluding the last one) = 461/.90= 512 kips

Answer = 512 kips

Appendix G Cable Barrier Foundation Analysis using Broms' Method

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

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

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

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

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

where,

γ' = Effective unit weight of soil

D = Depth of Pile

K_p = Coefficient of passive earth pressure (3.0 for φ = 30)

b = Width of Pile

P = Ultimate lateral load (Service Load x Factor of Safety)

Solving for D:

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

G.2. Example

Given:

Horizontal Service Load on Foundation, Q_a = 40 kips

Effective Unit Weight of Soil, γ' = 50 pcf

Friction angle, φ = 30 degrees

Cohesion, C = 0

Factor of Safety = 2 (Overturning)

k_h = 7 pci

Service Load Deflection = 1 inch

For the standard soil and default loading:

$$D = \sqrt{\frac{2 \times 80{,}000\ lbs}{50\ pcf \times 3 \times b}}$$

For 48-inch diameter drilled shaft:

$$D = \sqrt{\frac{2 \times 80{,}000\ lbs}{50\ pcf \times 3 \times 4\ ft}} = 16.3\ ft$$

Check service load deflection.

Computing deflection using Broms' method is less straightforward. Terms need to carefully be taken from the applicable e_c/D curve from the following graph (for cohesionless soils). In this example, the Free Head with e_c/D = 0.0 curve is used.

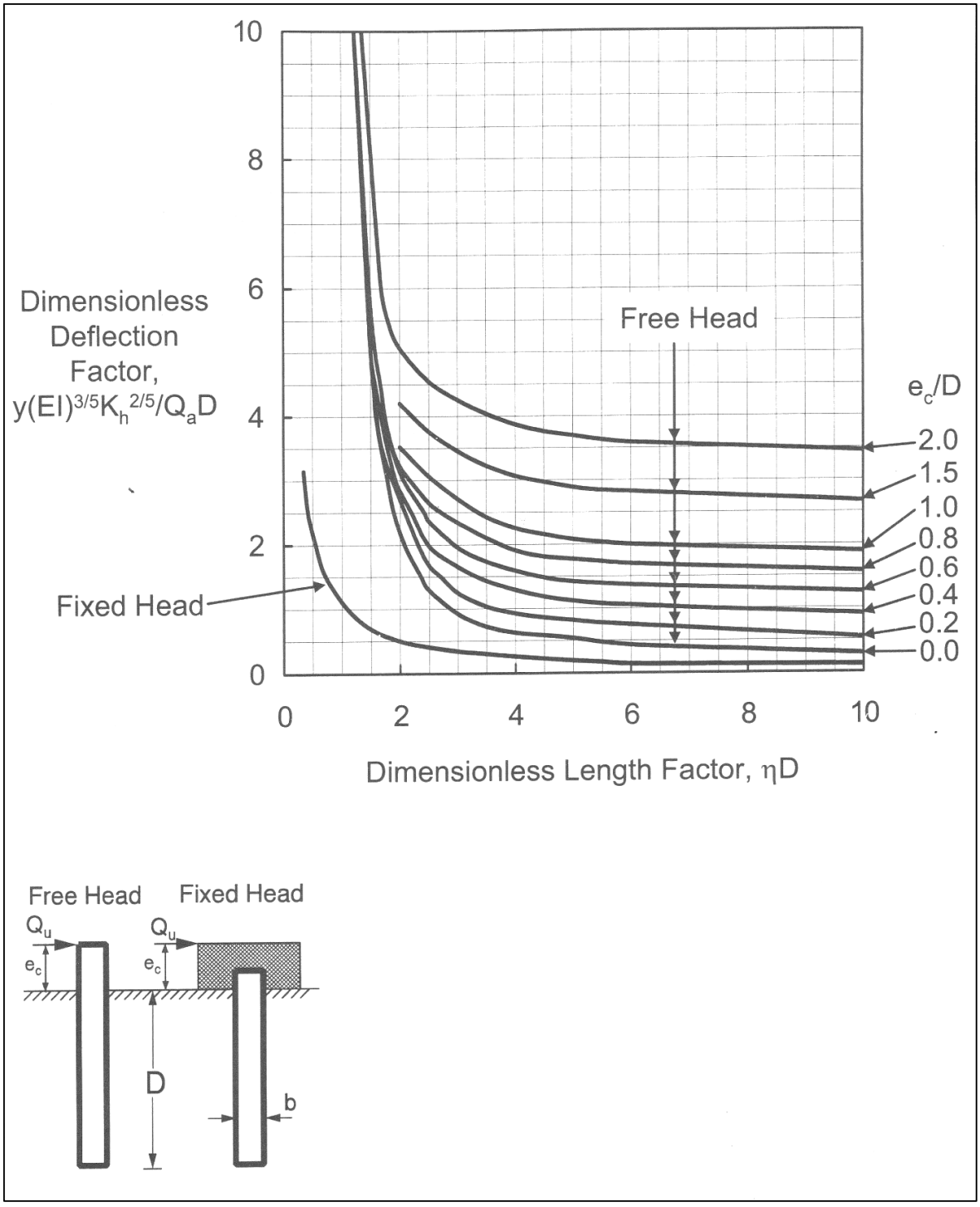


Figure G-1 Broms' Deflection Factor vs. Length Factor (After FHWA-NHI-05-042 [$K_h = k_h$])

The graph is a comparison of two dimensionless terms:

$$\frac{y(EI)^{\frac{3}{5}}(k_h)^{\frac{2}{5}}}{Q_a D} \; vs. \; \eta D$$

Where

$$\eta = \sqrt[5]{\frac{k_h}{EI}}$$

For 48-inch drilled shaft with concrete f_c' = 4000 psi

$$\eta = \sqrt[5]{\frac{7\ pci}{3{,}605{,}000\ psi\ \times 260{,}576\ in^4}} = 0.00595\ in^{-1}$$

$$\eta D = 0.00595 \times 16.3 ft \times \frac{12 in}{1\ ft} = 1.16$$

No good; ηD = 1.16 is not on the graph

Try D where ηD = 1.5

$$D = \frac{1.5}{0.00595 \times 12} = 21.0\ ft$$

From graph (Free Head with e_c/D = 0.0),

$$\frac{y(EI)^{\frac{3}{5}}(k_h)^{\frac{2}{5}}}{Q_a D} = 4.75$$

Solving for y:

$$y = \frac{4.75 \times Q_a D}{(EI)^{\frac{3}{5}}(k_h)^{\frac{2}{5}}} = \frac{4.75 \times 40{,}000 \times 21 \times 12}{(3{,}605{,}000 \times 260{,}576)^{\frac{3}{5}}(7)^{\frac{2}{5}}} = 1.44 > 1\ inch$$

No good.

Try D = 23 ft

$$\eta D = 0.00595 \times 23\ ft \times \frac{12\ in}{1\ ft} = 1.64$$

From graph,

$$\frac{y(EI)^{\frac{3}{5}}(k_h)^{\frac{2}{5}}}{Q_a D} = 3.5$$

Solving for y:

$$y = \frac{3.5 \times Q_a D}{(EI)^{\frac{3}{5}}(k_h)^{\frac{2}{5}}} = \frac{3.5 \times 40{,}000 \times 23 \times 12}{(3{,}605{,}000 \times 260{,}576)^{\frac{3}{5}}(7)^{\frac{2}{5}}} = 1.16 > 1\ inch$$

Try D = 25 ft

$$\eta D = 0.00595 \times 25\ ft \times \frac{12\ in}{1\ ft} = 1.79$$

From graph,

$$\frac{y(EI)^{\frac{3}{5}}(k_h)^{\frac{2}{5}}}{Q_a D} = 2.5$$

Solving for y:

$$y = \frac{2.5 \times Q_a D}{(EI)^{\frac{3}{5}}(k_h)^{\frac{2}{5}}} = \frac{2.5 \times 40{,}000 \times 25 \times 12}{(3{,}605{,}000 \times 260{,}576)^{\frac{3}{5}}(7)^{\frac{2}{5}}} = 0.90 < 1\ inch$$

Okay, y < 1 inch

Use a 4-foot diameter drilled shaft, 25 feet deep.

G.3. References

1. Broms, B.B., Lateral Resistance of Piles in Cohesive Soils, American Society of Civil Engineers, (ASCE), Journal for Soil Mechanics and Foundation Engineering, Vol. 90, SM2, pp. 27-63, 1964.
2. Broms, B.B., Lateral Resistance of Piles in Cohesionless Soils, American Society of Civil Engineers, (ASCE), Journal for Soil Mechanics and Foundation Engineering., Vol. 90, SM3, pp. 123-156, 1964.

Appendix H Design Implementation of Correction Factors for Sheet Pile Proximity (CFSP)

H.1. Pile Group Example 1

Compute the required driving resistance of a four by three pile group composed of 24-inch prestressed concrete piles embedded 100 feet in cohesionless soil, that will be subjected to a Factored Design Load (FDL) of 275 tons per pile. A steel sheet pile cofferdam will be installed to a depth of 45 feet prior to pile driving activities and will be removed after excavation and pile installation. The distance from edge of pile to edge of sheet pile is 6 feet. The site conditions do not require accounting for net scour (NS) or drag loads (DR).

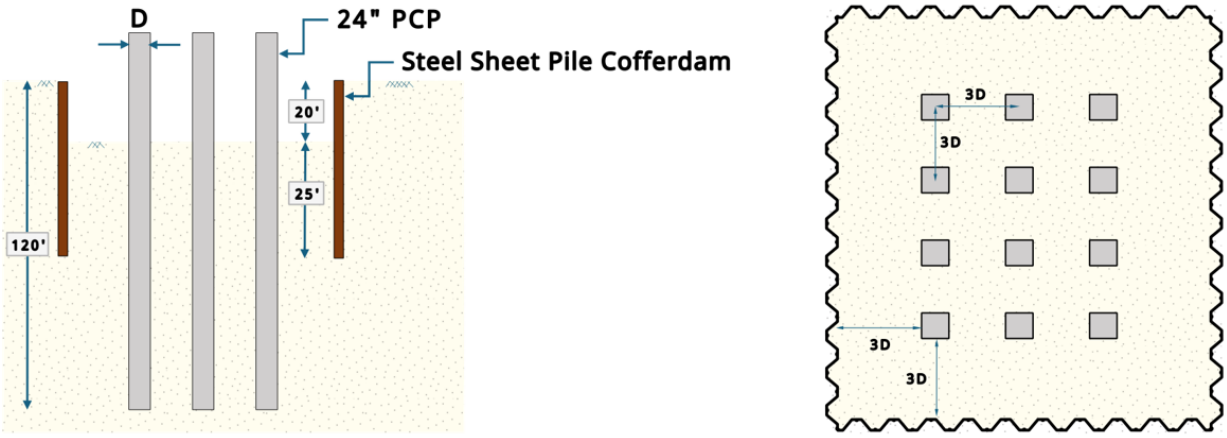


Figure H-1 Elevation (Left) and Plan (Right) View of Pile Group

Resistance factor, φ = 0.75

$$NBR = \frac{(FDL + DR + NS)}{\varphi} = \frac{(275 + 0 + 0)}{0.75} = 367\ tons$$

Embedment Ratio:

$$\frac{25\ ft}{100\ ft} = 0.25$$

Distance from edge of pile to edge of sheet pile: 6 ft = 3D

Using ***Table 8-3:***

Correction Factor for Sheet Pile Proximity (CFSP)*					
	Distance From Edge of Pile to Edge of Sheet Pile				
Embedment Ratio**	1D	2D	3D	4D	5D
0.25	1.20	1.15	1.10	1.10	1.05
0.50	1.40	1.15	1.10	1.10	1.05
0.75	1.60	1.36	1.25	1.25	1.20
1.00	1.90	1.60	1.50	1.47	1.45

*Applied to NBR
**Ratio of sheet pile to pile/drilled shaft embedment

Figure H-2 CFSP Determined from Table 8-3 for Example 1

Corrected NBR:

$$NBR_{corrected} = 367 \times 1.10 = 404\ tons$$

H.2. Small Pile Group Example 2

Compute the required driving resistance of a two-by-two pile group composed of 24-inch prestressed concrete piles embedded 100 feet in cohesionless soil, that will be subjected to a Factored Design Load (FDL) of 275 tons per pile. A steel sheet pile cofferdam will be installed to a depth of 45 feet prior to pile driving activities and will be removed after excavation and pile installation. The distance from edge of pile to edge of sheet pile is 6 feet. The site conditions do not require accounting for net scour (NS) or drag loads (DR).

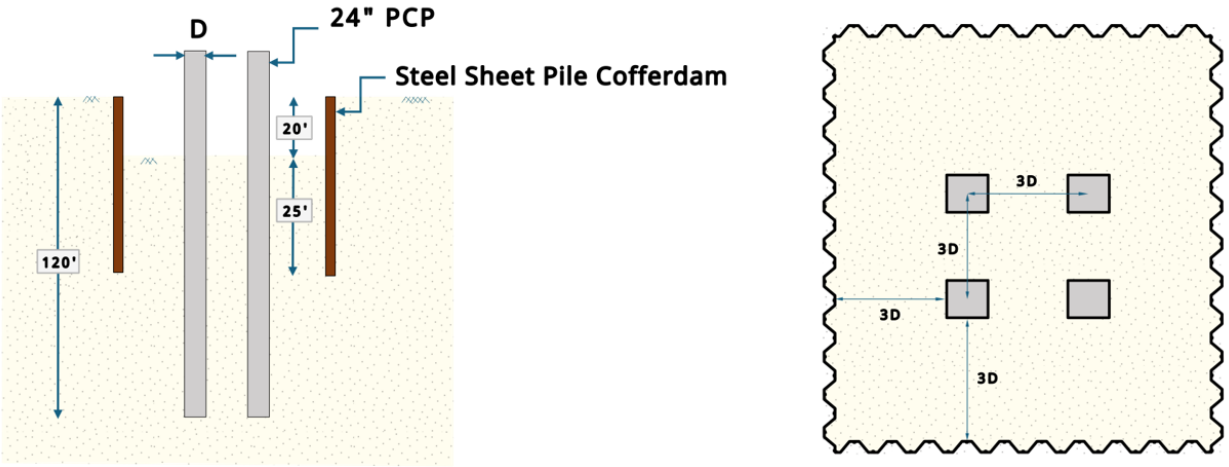


Figure H-3 Elevation (Left) and Plan (Right) View of Small Pile Group

Resistance factor, φ = 0.75

$$NBR = \frac{(FDL + DR + NS)}{\varphi} = \frac{(275 + 0 + 0)}{0.75} = 367\ tons$$

Embedment Ratio:

$$\frac{25\ ft}{100\ ft} = 0.25$$

Distance from edge of pile to edge of sheet pile: 6 ft = 3D

Using ***Table 8-3***:

Correction Factor for Sheet Pile Proximity (CFSP)*					
	Distance From Edge of Pile to Edge of Sheet Pile				
Embedment Ratio**	1D	2D	3D	4D	5D
0.25	1.20	1.15	1.10	1.10	1.05
0.50	1.40	1.15	1.10	1.10	1.05
0.75	1.60	1.36	1.25	1.25	1.20
1.00	1.90	1.60	1.50	1.47	1.45

*Applied to NBR
**Ratio of sheet pile to pile/drilled shaft embedment

Figure H-4 CFSP Determined from Table 8-3 for Example 2

All the members in the pile group are affected by sheet pile extraction on more than one side, therefore add 0.05 to the values in the Table.

Corrected NBR:

$$NBR_{corrected} = 367 \times 1.15 = 422\ tons$$