SECTION 5
GEOTECHNICAL ENGINEERING

Table of Contents

Page No.

Index of Tables

List of Exhibits

APPENDICES

SECTION 5

GEOTECHNICAL ENGINEERING

5.1 DEFINITIONS

Definitions as provided below supersede definitions located elsewhere within the NJTA document library, for the purposes of this document only. Defined terms, where shown in this Section, will have only the first letter capitalized. Where capitalized terms are noted in the text but not below, the reader is implicitly directed to either the NJTA Procedures Manual, the NJTA Standard Specifications for definition of those terms.

ADDENDA: Written interpretations or revisions of any of the Contract documents made prior to the receipt of bids.

AUTHORITY: The New Jersey Turnpike Authority.

BIDDER: An individual, partnership or corporation, acting directly or through a duly authorized representative, legally submitting a Proposal.

BORING CONTRACTOR: Party of the second part to the Contract, acting directly or through agents or employees, and primarily liable for the acceptable performance of the Project and for the payment of all debts pertaining to the Project. The Boring Contractor services include all field work including Borings, sampling, testing, ground monitoring, permitting, and other work discussed in this Manual.

BORING(S): Unless otherwise noted, the term "Boring(s)" refers to a component of the Geotechnical Exploration which consists of soil drilling (hollow stem augers, mud rotary, sampling, down hole in-situ testing, etc.) with Standard Penetration Testing performed in accordance with ASTM D1586 and rock coring, if required.

BORING CONTRACT: The agreement covering the performance of the Project, hereinafter defined, and payments thereof, including the Invitation for Proposals, executed Proposal, executed Contract Agreement, executed Contract Bond (when required), Specifications, Plans, Addenda if issued, and supplementary agreements which may be entered into, all of which documents are to be treated as one instrument as if set forth at length in the form of Contract Agreement.

CONTRACT DOCUMENTS: Advertisement for Proposal, Proposal Guaranty, Contract Agreement, Contract Bond, Power of Execution, Standard Specifications, Supplemental Specifications, Special Provisions, Plans, Addenda, or other information mailed or otherwise transmitted to the prospective bidders prior to the receipt of bids, Change Orders, Field Orders, and Supplementary Agreements, all of which are to be treated as one instrument whether or not set forth at length in the written Contract Agreement.

CORROSION SPECIALIST: The Corrosion Specialist is an authorized representative of the Engineer who has knowledge and experience in corrosion damage mechanisms, metallurgy, materials selection, and corrosion monitoring techniques. NACE certification is required unless a registered PE with certification or licensing in corrosion control of buried metal pipes and tanks.

DESK STUDY: The Desk Study is part of the Phase A Geotechnical Engineering Report. The Desk Study should include review of available information and previously performed explorations including (but not limited to) published geologic information, aerial photographs, existing Boring information, existing construction information, and site visit(s). For additional information related to desk study, see Section 5.3.1.

ENGINEER: The Chief Engineer of the Authority, or his duly authorized representative acting within the scope of the particular authority vested in him.

ENGINEER OF RECORD (EOR): Professional Engineer licensed to practice in New Jersey, responsible for the preparation of the Contract Documents. All communications with the Authority shall be through the EOR.

GEOLOGIC ENGINEER: The Geologic Engineer is an authorized representative of the Engineer with at least 10 years of geologic engineering experience and at least 5 years working as a geologic engineer in New Jersey. Practical experience with all the exploration, design and construction issues required for the project. The Geologic Engineer shall be a Professional Engineer or Geologist licensed to practice in New Jersey.

GEOTECHNICAL ENGINEER (GE): The Geotechnical Engineer is an authorized representative of the Engineer with at least 10 years of geotechnical engineering experience and at least 5 years working as a geotechnical engineer in New Jersey. Practical experience with all the exploration, design and construction issues required for the project. A Professional Engineer licensed to practice in New Jersey.

GEOTECHNICAL EXPLORATION: The term Geotechnical Exploration includes in-situ and ex-situ testing performed to characterize the geotechnical aspects of the Project site. The term includes a broad range of field and laboratory testing and sampling including but not limited to Borings, CPT soundings, Seismic Testing, Vane Shear Testing, Pressuremeter Testing, Dilatometer soundings, Groundwater Exploration, laboratory testing, geophysical testing, etc.

GEOTECHNICAL EXPLORATION PLAN (GEP): The GEP is part of the Phase A Geotechnical Engineering Report. This plan provides the rationale for any proposed geotechnical exploration plus detailed information such as a Boring Location Plan, Boring quantities, field and laboratory testing, and relevant procedures. The GEP should be referential of the Desk Study and all additional information to be obtained in that document should be accounted for in the GEP.

GEOTECHNICAL FIELD REPRESENTATIVE (GFR): Authorized representative of the GE, assigned to monitor the Subsurface Exploration. The purpose of the GFR is to monitor conformance of the Contractor’s work with Plans and Specifications, and to collect relevant data as directed by the GE.

PLANS: The standard drawings, the official approved drawings specially prepared for the Project, profiles, cross-sections, and any supplemental drawings, or exact reproductions thereof, and that are current on the date the bids are received, and were furnished by the Authority, that indicate the location, character, dimensions, and details of the Work to be done.

PROJECT: The entire work to be performed within the limits and requirements specified for the Contract.

SPECIFICATIONS: The Standard Specifications, the Supplementary Specifications and Addenda, if issued, pertaining to the method or manner of performing the Project and to the qualities of the materials to be furnished for the Project.

SURETY: The corporate body which is bound with and for the Contractor, and which is responsible for the contractor's acceptable performance of the Project and for the payment of all debts pertaining to the Project.

WALL MANUFACTURER: Wall supplier/vendor and shall also include a Professional Engineer licensed in NJ, responsible for the preparation of the Working Drawings and calculations associated with the Retaining Wall.

5.2 PURPOSE & CONTENT

Section 5 of the Authority's Design Manual provides guidance, policies, and standard practice for the Geotechnical Exploration Plan (GEP), geotechnical analysis and design, and construction monitoring. The instructions found within Section 5 constitute the minimum required level of effort on the part of the EOR. The EOR is encouraged to exceed the minimum required level of effort when best practices dictate. The Authority desires the "best value" geotechnical solution, not the "lowest cost" geotechnical solution in cases when these two conditions are not the same.

Section 5 of the manual is intended to work in tandem with the Authority's Procedures Manual. As stated in the Procedures Manual, the Geotechnical Engineering effort will have four phases:

- <u>Phase A - Geotechnical Engineering</u>: Prepare and submit Phase A Geotechnical Engineering Report, Desk Study and GEP.

- <u>Phase B - Geotechnical Engineering</u>: Perform the Geotechnical Exploration, preliminary design recommendations, and preliminary Phase B Geotechnical Engineering Report.

- <u>Phase C - Geotechnical Engineering</u>: Finalize design recommendations, preliminary plans and specifications, and finalized/revised Phase B Geotechnical Engineering Report.

- <u>Phase D - Geotechnical Engineering:</u> Finalize plans and specifications.

All submittals shall include an electronic copy. Refer to Section 5 of the Procedure Manual for more information on submittal requirements.

The most current edition of the AASHTO LRFD Bridge Design Specifications (AASHTO-LRFD-BDS), with interims at the time the design is let, shall be used for geotechnical and foundation design features, unless otherwise requested by the Authority or when the following exceptions apply:

A. Foundations for structural features or protective features adjacent to railroads shall be designed in accordance with the applicable sections in the current AASHTO-LRFD-BDS with consideration given to AREMA Manual for Railway Engineering for vehicle load and load combinations.

B. Foundations for railroad structures shall be designed in accordance with the current edition of the AREMA, local railroad owner design criteria and supplemented by AASHTO Standard Specifications for Highway Bridges.

C. Foundations for sign structures, toll gantries, tower foundations, luminaries, and traffic signals shall be designed in accordance with the current edition of the AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaries, and Traffic Signals, with interims.

D. Foundations for buildings shall be designed in accordance with current International Building Code (IBC) and International Building Code - New Jersey Edition (NJBC), applications include toll plazas, maintenance facilities, and service areas.

E. Infiltration basins shall be designed in accordance with current New Jersey Department of Environmental Protection (NJDEP) New Jersey Stormwater Best Management Practices (BMP) Manual.

F. Temporary works shall be in accordance with the NJTA Construction Manual, AASHTO Guide Design Specifications for Bridge Temporary Works, and AASHTO Construction Handbook for Bridge Temporary Works.

G. Analyzing Soil and Rock Slopes and Embankment Settlement and Global Stability shall be performed in accordance with the AASHTO Standard Specifications for Highway Bridges.

H. Foundations for fenders shall be designed in accordance with AASHTO Guide Specifications and Commentary for Vessel Collision Design of Highway Bridges.

Section 5.6 of this Manual follows the organization of AASHTO-LRFD-BDS, and provides supplementary guidance and commentary when necessary, and remains silent otherwise.

Section 5.7 of this Manual provides additional guidance on geotechnical features and technical issues not addressed in AASHTO-LRFD-BDS.

Section 5 of the Design Manual shall be used with other Design Manual sections (e.g. Structures-Section 2) and shall be supplemented by current versions of the Authority's Procedures Manual, Construction Manual, Standard Specifications, and Standard Supplementary Specifications. Additional local, state, or federal specifications, supplemental text books, and technical articles are referenced throughout this Section, when appropriate or necessary.

Although this Section provides guidance on design and analysis procedures, it does not preclude the need for additional engineering analysis and design procedures to produce a safe, economical and maintainable structure. All calculations shall provide complete and accurate references, and clearly identify the meaning of any abbreviations and symbols. They shall adhere to accepted QA/QC protocol and include initials and date performed and checked. Often special conditions will require engineering judgment to be applied and shall be assessed by the Authority on a project specific basis.

5.3 PREPARATION OF THE PHASE A GEOTECHNICAL REPORT

The Phase A Geotechnical Report shall, at a minimum, consist of a Desk Study and the GEP. The purpose of the Phase A Geotechnical Report is to provide a sound basis for the Phase B work.

5.3.1 DESK STUDY

A Desk Study is an exploration of relevant existing information which has been previously performed and often by others. Results of this study will be helpful to define existing project conditions and assist in determination of appropriate geotechnical features and their respective design and construction procedures and analyses. Minimum requirements for the Desk Study include:

<u>Basic Geologic Understanding</u>: The Desk Study shall provide a basic understanding of the geologic conditions of the site by thorough acquisition and review of available geologic literature relative to the site. “The Geologic Map of New Jersey by Lewis and Kummel” is the standard reference geology map of New Jersey. The New Jersey Geological Survey (NJGS) Surficial Geology Maps of New Jersey and NJGS Bedrock Geology Maps of New Jersey also provide valuable reference information. Additional engineering soils information and soil maps can be obtained from the Rutgers University publications entitled, “Engineering Soil Survey of New Jersey” for each county. The United States Department of Agriculture, Soil Conservation Service also publishes soil maps which are useful when evaluating the upper layers of soil.

<u>Existing Information</u>: Through the EOR the GE shall request from the Authority any existing information relative to the site. The information request shall address existing Borings/soundings made for previous construction nearby, aerial photographs, foundations plans, soil and foundation reports, agricultural soil maps, and Photogeologic Interpretation reports. Sources of information may include the NJDOT Soil Boring Database which is available online. Existing information from the Authority is typically available during the solicitation process.

<u>Site Visit</u>: Upon coordination with the Authority, it is recommended that the GE conduct a field reconnaissance of the proposed project site. Observe and record site features that may warrant additional consideration during the exploration.

<u>Design Understanding</u>: The GE shall establish a comprehensive understanding of the proposed design through meetings and open communication with the EOR's entire design team. It is understood that the complete project design will not be developed at this early project phase. However, it is important to obtain the anticipated approximate locations of major structures and project features that will affect the Geotechnical Exploration. If during initial meetings the overall project design has not been adequately advanced to provide preliminary structure and project feature locations within the Phase A deliverable deadline, the EOR shall contact the Engineer to discuss either accelerating the work, or rescheduling the Phase A deadline. Alternatively, the Geotechnical Exploration may be staged to allow the GE to advance their understanding of the subsurface conditions as the design progresses.

<u>Design Parameters</u>: The GE shall develop an understanding of the design parameters as they relate to the structures and other elements of design that require said parameters and geotechnical analysis. A list of these parameters with the proposed approach to their selection based on the testing and evaluation proposed shall be included as part of Phase A submission. Include an evaluation of the sensitivity of proposed and/or existing structures to these parameters and through the EOR advise the Authority if additional testing could reduce cost and/or accelerate schedule if alternative investigative methods are employed (as applicable).

<u>Geotechnical Hazards</u>: The Phase A Geotechnical Engineering Report shall include a list of potential geologic hazards such as soft compressible soils, urban fills, degradable shale rock, erratic rock conditions, presence of cobbles and boulders, potential for scour, foundation construction near existing structures and utilities, construction in confined space, spread footing on future thick engineered fills, and high and variable groundwater levels that could affect the design and construction of the Project. Additional examples include aggressive corrosivity, sulfate attack, microbiologically influenced corrosion, acid producing soils, and seismic hazards. Where these hazards present an obstacle to traditional subsurface exploration or design, recommend special geotechnical exploration and design to mitigate the identified potential risks to existing facilitates and/or the proposed project facilities. When hazards are identified, the Phase A Geotechnical Engineering Report shall include a hazard matrix which details each hazard, the anticipated risks of the hazard(s), and the recommendations to reduce risk associated with those hazard(s).

<u>Environmental Constraints</u>: Describe site specific environmental constraints known to exist based on discussions with the Project's environmental specialist. This may include the potential for the presence of contaminated soils which impact the proposed subsurface exploration methods in regards to Health and Safety Requirements, procedures, and costs. This information may also skew the foundation costs to be more favorable of one particular foundation type that would otherwise not be economically preferred (i.e. driven displacement piles require no soil disposal opposed to drilled shafts). Environmental subsurface exploration program shall be coordinated with geotechnical subsurface exploration program to minimize

the cost. In addition, this section shall include any available historic maps such as Sanborn maps, which may provide an indication of past land use, which may be useful in identifying abandoned utilities, abandoned buried building foundations, etc.

5.3.2 DEVELOPMENT OF THE PHASE A GEOTECHNICAL EXPLORATION PLAN

The Phase A Geotechnical Engineering Report shall include a detailed GEP developed and submitted during Phase A and executed during Phase B. The plan shall include a boring location plan which indicates the anticipated locations of relevant structures and other project features. This plan should include a justification for the exploration and how the information is related to design and construction needs, design parameters, and how geotechnical hazards will be evaluated, analyzed and presented. The plan should also include a schedule of Borings and other investigative methods, including proposed boring type, termination, and sampling criteria. Also, the GE should provide a schedule of anticipated laboratory and in-situ testing (as required), as well as procedures for all field testing. The procedures shall reference standards such as ASTM, AASHTO and other standard tests, as appropriate. Also, if appropriate, the GE should provide recommendations for a multi-phase exploration approach that may be warranted for large or complex Projects or Projects subject to significant geometric or structure change.

The GE shall execute a ‘right-sized’ Geotechnical Exploration program consisting at a minimum of Borings and laboratory testing of samples retrieved. Other in-situ strength testing, direct-push technologies and geophysical testing may be incorporated into the program to further define soil and rock parameters. The program will serve to delineate changes in strata, provide information to assist with geomorphologic interpretation of the soil and rock encountered, and facilitate the planning, design and construction of the proposed bridges, roadways, embankments, retaining walls, sign structures, noise barriers, culverts, buildings and other facilities shall also be included or design features that will require geotechnical analysis.

Groundwater monitoring installations shall also be incorporated into the Geotechnical Exploration to identify the groundwater conditions for design and as described in Section 5.4.3. In-situ permeability testing to assist in the design of storm water management facilities and is discussed in Section 5.4.3.5. Soil and rock samples shall be tested in accordance with Article 5.4.10, “Laboratory Testing Program”.

The GE shall provide a GEP narrative including the criteria and justification used in developing the program, and the location of all in-situ testing and Borings with their frequency, depths, and sampling intervals.

The Geotechnical Exploration shall be completed in and the findings published with the Phase B Geotechnical Engineering Report. Results of the Geotechnical Exploration shall be sufficient to provide all planned geotechnical input to advance the project through Phase C. The Phase B

Geotechnical Engineering Report shall provide for suggested construction execution and sequence of foundation elements (constructability). The constructability of foundation elements is a critical component to the Phase B submission as it directly affects the design and constructability of all foundation elements, is of direct importance to the Maintenance and Protection of Traffic (MPT) and overall project constructability reports. Another purpose of the Phase B submission is to help identify significant risk related to constructability or performance after construction is completed. The GE shall consider availability of large or unusual foundation installation equipment and its potential to impact facilities that remain in service during construction. Additional consideration for appropriateness and cost shall be given to foundation designs that utilize techniques or foundation elements which are outside the expertise of the regional contractor community.

Phase A Geotechnical Engineering Report – Desk Study and Geotechnical Exploration Plan (GEP) – The minimum requirements for the Phase A GEP shall include the following. Deviations from this outline must be authorized by the Authority.

- Rationale for exploration plan
- Boring Location Plan
- Boring spacing, depth and termination criteria
- Exploration testing quantities
- Access constraints
- MPT impacts and needs
- Design parameter evaluation
- Procedures
- Inspection
- Schedule and phasing

5.3.2.1 Justification for the Geotechnical Exploration Plan

The GEP shall also present a detailed justification for the exploration, including the reasoning for the selected boring locations, spacing, termination criteria, anticipated depths, and proposed in situ and laboratory testing. The GEP shall include a discussion of the design features that will be addressed. This shall include a discussion of the requirements established by the design team including structure and facility locations, sizes, preliminary loads and deformation tolerances, other key design and construction inputs; and how the proposed exploration will provide information to support a design that meet those requirements. The GEP shall also discuss how the site conditions (geologic understanding, existing Borings, etc.) are anticipated to affect the proposed exploration. The GEP should also include a discussion on how the results will help minimize construction related risks associated with differing site conditions and the design and construction of the facilities.

5.3.2.2 Proposed Boring Location Plan

The Engineer shall submit a boring/structure location plan as a part of the GEP. The plan should identify all project features that will require geotechnical study. The approved boring location plan shall be prepared for the use of the Boring Contractor and the GE for field stake out purposes. The final construction boring location plan shall be accompanied by a list or schedule of the Borings, showing the NJ State Plane Northing and Easting coordinates, North American Vertical Datum ground elevation, baseline, station and offset, boring type, and planned depth.

The boring location plans shall include the following information and the location of all features of the project requiring geotechnical assessment, but not limited to:

A. Project baseline stationing (if available) at maximum 100’ intervals.
B. Property lines and Right of Way limits.
C. Block and Lot Numbers of private properties.
D. Aerial Photographs – if available.
E. Existing utilities known at the time of development of the GEP
F. Existing structures, roadways, embankments and retaining walls.
G. Anticipated facility locations.
H. Proposed boring locations with boring numbers as per Section 5.4.7.

If base mapping is not available at this stage of the project, the Engineer shall utilize photogrammetric images obtained from the NJ GIS database and the NJTA’s Enterprise GIS to locate the exploration points in an effort to expedite the program.

5.3.2.3 Boring Spacing and Depth

The GE shall refer to the indicated publications below and Tables 5.3-1 and 5.3-2 to present and justify soil boring spacing and depths for:

A. Shallow and deep foundations and retaining walls shall be in accordance with Article 10.4.2 of AASHTO-LRFD-BDS, current edition, with interims.

B. Roadways, cuts, embankments, and culverts shall be in accordance with Article 2.5.2 of the FHWA document NHI-01-031, “Subsurface Investigations – Site Characterization.

C. Stormwater management facilities shall be in accordance with the “New Jersey Stormwater Best Management Practices (BMP) Manual”, current edition.

D. Buildings and Toll Plazas shall be in accordance with the "International Building Code-New Jersey Edition".

The design of the exploration program, including but not limited to spacing and depth, is the responsibility of the GE. The GE may decrease spacing and increase depths to further explore areas with highly variable subsurface conditions as deemed appropriate. Through the EOR the GE shall obtain the concurrence of the Authority to modify exploration spacing and depths in the event that subsurface conditions are uniform over large areas. Through the EOR the GE shall obtain concurrence of the Authority to use exploration information previously performed from another exploration in lieu of and to supplement the proposed Subsurface Exploration Program and reduce the number of proposed Borings. The decision to rely upon existing data and the potential consequence are the sole responsibility of the GE.

The GE shall be responsible for performing field reconnaissance to determine accessibility of the proposed boring locations. Exploration contractors shall be present during the field reconnaissance to determine necessary equipment, access points, and staging areas for the proposed work. The GE shall be responsible for locating and staking out all soil boring and in-situ testing locations using appropriate methods suited for the project needs. The Boring Contractor may offset the boring only at the direction of the GE.

After having gained a preliminary knowledge of the geology and surface soils in the area, the GE shall prepare a boring layout plan that will provide the maximum amount of information at the lowest practical cost and shortest practical schedule. The size and complexity of the Project and the anticipated foundation type(s) along with the subsurface conditions expected will dictate the size of the GEP. The GE is encouraged to err on the side of additional and/or deeper Borings when project complexity or uncertainty regarding the subsurface conditions warrants.

THE LOCATION, SPACING, AND DEPTH OF BORINGS FOR STRUCTURAL FOUNDATIONS SHALL ADHERE TO THE MINIMUM MODIFIED AASHTO GUIDELINES PRESENTED IN

Table 5.3-1. Similar minimum guidance for facilities not addressed in AASHTO are presented in Table 5.3-2. However, the GE must use judgment in determining the proposed location and spacing of the Borings for each Project. In general, if the subsurface profile is expected to be uniform and consistent, the spacing between Borings may be greater. On the other hand, if variable or non-uniform conditions are expected, the spacing should be reduced in an attempt to delineate variable conditions.

TABLE 5.3-1: MODIFIED AASHTO CRITERIA FOR BORING SPACING AND DEPTH

Application	Minimum Number of Exploration Points and Location of Exploration Points	Minimum Depth of Exploration
Retaining Walls	A minimum of one exploration point for each retaining wall. For retaining walls more than 100 ft. in length, exploration points spaced every 100 to 200 ft. with locations alternating from in front of the wall to back limit of the wall. For anchored walls, additional exploration points in the anchorage zone spaced at 100 to 200 ft. For soil-nail walls, additional exploration points at a distance of 1.0 to 1.5 times the height of the wall behind the wall spaced at 100 to 200 ft.	Investigate to a depth below bottom of wall at least to a depth where stress increase due to estimated foundation load is less than ten percent of the existing effective overburden stress at that depth and between one and two times the wall height. Exploration should be deep enough to fully penetrate soft highly compressible soils, e.g., peat, organic silt, or soft fine grained soils, into competent material of suitable bearing resistance, e.g., stiff to hard cohesive soil, compact dense cohesionless soil, or bedrock.
Shallow Foundations	For substructure, e.g., piers or abutments, widths less than or equal to 100 ft., a minimum of one exploration point per substructure is required however two is preferred. For substructure widths greater than 100 ft., a minimum of two exploration points per substructure is required. Additional exploration points should be provided if erratic subsurface conditions are encountered.	Investigate to a depth below the bottom of footing deep enough to fully penetrate unsuitable foundation soils, e.g., peat, organic silt, or soft fine grained soils, into competent material of suitable bearing resistance, e.g., stiff to hard cohesive soil, or compact to dense cohesionless soil or bedrock. The Borings shall extend at least to a depth where stress increase due to estimated foundation load is less than ten percent of the existing effective overburden stress at that depth; and if bedrock is encountered before the depth required by the second criterion above is achieved, exploration depth should be great enough to penetrate a minimum of 10 ft. into the bedrock, but rock exploration should be sufficient to characterize compressibility of infill material of near-horizontal to horizontal discontinuities. Note that for highly variable bedrock conditions, or in areas where very large boulders are likely, more than 10 ft. of rock core may be required to verify that adequate quality bedrock is present.

Application	Minimum Number of Exploration Points and Location of Exploration Points	Minimum Depth of Exploration
Deep Foundations	For substructure, e.g., bridge piers or abutments, widths less than or equal to 100 ft., a minimum of one exploration point per substructure is required however two points is preferred. For substructure widths greater than 100 ft., a minimum of two exploration points per substructure is required. Additional exploration points should be provided if erratic subsurface conditions are encountered, especially for the case of drilled shafts socketed into bedrock. To reduce design and construction risk due to subsurface condition variability and the potential for construction claims, at least one exploration per shaft should be considered for large diameter shafts (e.g., greater than 5 ft. in diameter), especially when shafts are socketed into bedrock.	In soil, depth of exploration should extend below the anticipated pile or shaft tip elevation a minimum of 20 ft., or a minimum of two times the minimum anticipated pile group dimension, whichever is greater. All Borings should extend through unsuitable strata such as unconsolidated fill, peat, highly organic materials, soft fine-grained soils, and loose coarse-grained soils to reach hard or dense materials. For piles bearing on rock, a minimum of 10 ft. of rock core shall be obtained at each exploration point location to verify that the boring has not terminated on a boulder. For shafts supported on or extending into rock, a minimum of 10 ft. of rock core, or a length of rock core equal to at least three times the anticipated shaft diameter for isolated shafts or two times the minimum shaft group dimension, whichever is greater, shall be extended below the anticipated shaft tip elevation to determine the physical characteristics of rock within the zone of foundation influence. Based on the geologic conditions, bedrock is relatively competent along the Turnpike and Parkway and the rock core depth specified above may be excessive. Note that for highly variable bedrock conditions, or in areas where very large boulders are likely, more than 10 ft. of rock core may be required to verify that adequate quality bedrock is present.

1. From AASHTO-LRFD-BDS
2. Consider taking at least one boring per project geologic setting to a minimum of 100 feet or to bedrock, to determine seismic site class.

TABLE 5.3-2: CRITERIA FOR BORING SPACING AND DEPTH NOT SPECIFIED IN AASHTO

Application	Minimum Number of Exploration Points and Location of Exploration Points	Minimum Depth of Exploration
Roadways	A minimum of one boring every 100 to 400 feet along the center of the roadway per lane direction, additional borings should be provided for poor foundation conditions such as deep layers of soft clays or complex conditions where lenses of soft clay are found in layers of sand. Very wide areas, such as approaches to toll booths may require 2 to 3 boring per cross section. In marshes or swamps, continuous sampled Borings shall be specified at closer spacing to delineate the bottom of the organic deposits.	Embankments - For proposed embankments over 10 ft. in height, a minimum depth below the base elevation of the embankment of twice the embankment height or until the SPT result is 20 for two consecutive sample intervals, whichever is greater. For embankments less than 10 ft., the minimum depth is 15 ft. below the base elevation of the embankment Extend Borings 15 ft. below zones of peat, highly organic soils, soft fine-grained soils, or loose coarse-grained soils. Cuts - For roadway cut areas, the Borings shall extend a minimum of 15 ft. below the anticipated profile grade.

Application	Minimum Number of Exploration Points and Location of Exploration Points	Minimum Depth of Exploration
Sign Structures, E-Z Pass Tolling, VMS, High Mast Lighting, Tower Structures	A minimum of one boring for each foundation substructure.	Extend borings to minimum of 40 ft. in soil or 10 ft. into rock, whichever occurs first.
Stormwater Facilities	A minimum of one boring for each stormwater facility with less than 4000 square feet, and additional boring for large facilities. Additional borings are required where subsurface conditions are variable.	Extend borings a minimum of 10 ft. below the deepest anticipated grade of the stormwater facility.
Buildings	A minimum of one boring every 2,500 square feet of building plan footprint.	Follow the guidance as for shallow and deep foundations, respectively
Sound barriers, privacy walls, or solid type fencing	Follow the guidance as for retaining walls.	Follow the guidance as for retaining walls.

5.3.2.4 Exploration Quantities

The GEP shall summarize all proposed exploration quantities. The summary shall be in tabular form. The proposed quantities shall, at a minimum, include:

- Boring – by number and depth
- Sampling - by type
- In-situ testing (if utilized) - by type
- Laboratory testing - by type
- Groundwater exploration- by type

5.3.2.5 Design Parameter Evaluation

The GEP shall clearly state how the proposed borings, in situ testing and laboratory testing will be used to provide the required parameters for design and construction.

5.3.2.6 Subsurface Exploration Procedures

The GEP shall present relevant procedures for each element of the proposed exploration, and cite applicable ASTM and AASHTO standards when appropriate. If proprietary procedures are proposed, include them in the appendix. Also present the quality control procedures to be used for each method (see 5.3.2.7 below). When borings are to be taken on or adjacent to existing Authority roadways, applicable procedures must be followed regarding lane or shoulder closings, signing, etc.

5.3.2.7 Quality Assurance

Field oversight of all subsurface exploration work shall be under the supervision of a GE. It is the GE's responsibility to provide oversight and documentation of the boring contract.

The GE shall provide trained and experienced Geotechnical Field Representatives (GFR's) to monitor all boring operations and other field operations, and prepare boring logs. The GFRs will be under the supervision of the GE who will make periodic visits to the Project site during the course of the boring work. The GFR shall, at a minimum:

- Ensure that the locations for investigative methods presented in the GEP have been staked out per the boring contract plans. Confirm that both horizontal control and vertical ground elevations are recorded to the nearest one-tenth of a foot.
- Provide full-time monitoring (one assigned inspector per rig) of each boring rig on site and prepare a thorough record of the entire operation of that rig.
- Make and record measurements of the downhole tools used for the boring, including drill rods, samplers, other testing equipment and casing. The GFR shall measure accurately the depth to the top of each sample, record the penetration resistance of the sampler or the pressure used to push thin-walled samplers into the ground, and also record any unusual observations (driller's notes). Hammer type shall be automatic or safety.
- The GFR is responsible for QA monitoring of all field activities. The GFR shall have copies of all relevant procedures to the work. The GFR shall be knowledgeable about the procedures and with the GE identify any missing procedures, criteria, or issues related to the quality of the work. This shall be followed with a pre-work meeting with the Boring Contractor to review all quality procedures. It should be recognized that ASTM and other similar procedures are generally acceptable in their content; however, it is the responsibility of the GE to identify and supplement these procedures when they may be missing detailed information relative to the work.
- The GFR shall keep a Daily Field Report per Appendix C.
- The GFR shall identify the samples recovered with the split spoon sampler and shall be responsible for collecting representative samples and sufficient material in a labeled moisture resistant jar at the time the sample is retrieved. The jars shall be kept in a safe location and away from open sunlight or freezing temperatures.
- The GFR shall verify that groundwater levels are recorded at each boring in accordance with 5.4.3. Any water loss during drilling shall also be noted.
- The GFR shall verify that all boring holes are either backfilled or grouted in accordance with N.J.A.C. 7:9D-3.4 "Specific requirements for the decommissioning of Category 5 Wells-Geotechnical Borings" at the completion of the borings with the appropriate surface treatment and site restoration applied.
- The soil identification system to be used by the GFR is the

Modified Burmister Soil Identification System which is noted in Appendix B. Undisturbed samples shall be properly sealed in accordance with the Boring Contract, by the driller in the presence of the GFR, and placed in the required sample container. The GFR shall prevent damage from sunlight, impact or vibration, temperatures below the point of freezing, and carefully transport them to the laboratory.

- Each and every soil sample shall be photographed in accordance with the sample soil and core box photographs provided in Exhibit 5-1.
- A GFR's Manual is provided in Appendix C. The boring log forms to be used are provided in Exhibit 5-2 through 5-6. Sample boring log templates are provided in Appendix C.

At the conclusion of the boring program, the GE shall prepare a revised list of borings showing the “as-drilled” boring station and offset, northing, easting, ground surface elevation, actual type and depth of each boring. This revised list of “as-drilled” boring information shall be utilized in locating the borings on the final construction contract plans.

Throughout the boring program, the field boring logs shall be reviewed by the GE for consistency and completeness. The GE shall provide a layout of the boring logs applicable to each final construction contract on standard size bordered plan sheets suitable to be incorporated in the construction plans as reference drawings. These drawings shall bear the PE number and signature of the GE.

The GE shall provide storage for the samples obtained during the boring program. All samples shall be carefully stored so that they are readily available until the design phase of the project is completed. Upon completion of the Design Contract, the GE shall select a representative 20 percent of the soil and rock samples collected to be shipped to the Authority's geotechnical storage facility. Before delivery of the samples, the GE shall write the Authority requesting authorization to ship selected samples and dispose of the remaining materials. Until further direction is given from the Authority, the samples shall be stored by the GE at their own facility for at least 7 years.

5.3.2.8 Schedule and Phasing

The GEP and the geotechnical design need to be completed prior to proceeding with other tasks and are often driven by overall project schedule, the GE shall prepare a schedule that shows all major work elements, which at a minimum shall include the following:

- Permitting and rights of entry
- Mobilization
- Field Work
- Laboratory Work

- All reporting documents and all Geotechnical Engineering Reports beyond Phase A.

The schedule shall be prepared using Primavera, Microsoft Project, or comparable software approved by the Authority. If phasing of work is required, this shall be shown in the schedule.

5.3.2.9 Boring Contract

It is required that proposals be solicited for the boring work from a minimum of three (3) qualified Boring Contractors, with the lowest responsible bidding contractor being awarded the work. Deviations from this recommendation may be permitted by the Authority on a case-by-case basis.

5.3.2.10 Submission

Unless the Authority approves otherwise, the Boring Contract for the entire Project is to be completed to ‘ready-to-advertise’ status and submitted no later than the overall Project Phase A submission.

Also submitted with the ‘ready-to-advertise’ Boring Contract for review shall be the Subsurface Exploration Cost Estimate and a list of not less than three qualified Bidders who shall be invited to bid. The Authority reserves the right to add or delete from this list as it deems to be in its best interest.

Shortly after the Phase A submittal, the Authority will notify the EOR of any changes to be made on the boring location plan by the GE. After any changes have been made, the EOR will be notified by the Authority to invite bids for the Boring Contract.

If it appears the Subsurface Exploration Cost Estimate for Boring Contract cost will exceed the authorized boring contract amount, permission shall be obtained from the Authority prior to exceeding the authorized contract amount.

5.3.2.11 Subsurface Exploration Cost Estimate

Each boring program must be accompanied by an Engineer’s cost estimate, stating the various items proposed, the estimated quantities (See 5.3.2.4), the estimated unit cost, and the total Boring Contract amount. In addition to the normal boring items specified in Appendix A, the estimate should include a separate lump sum item to cover the Contractor’s cost for Mobilization and Demobilization. The cost estimate should also include costs associated with site restoration and Maintenance and Protection of Traffic, to cover these costs when appropriate.

5.3.2.12 Laboratory Selection

At a minimum, laboratory qualifications shall include evidence of AASHTO Materials Reference Laboratory Accreditation (AMRL) certification for all applicable test items.

5.3.2.13 Contract Award

The GE will invite bids by sending a complete set of Boring Contract documents to each of the selected Bidders.

Through the EOR the GE will formally notify the Authority of the results of the bidding and recommend the disposition of the Boring Contract. The Authority will review the results of the bidding and the GE's recommendation. The Authority will formally notify the GE of their concurrence and the contract awarded as recommended.

When the Contract Agreement, the Contract Bond (when applicable) and all the required Insurance Certificates have been submitted by the selected Bidder, properly signed and executed, the GE will sign the Boring Contract Agreement and notify the Boring Contractor to proceed with the work. Boring Contractor shall not execute the work until a traffic permit is obtained.

5.4 GEOTECHNICAL EXPLORATION

5.4.1 INTRODUCTION

The Geotechnical Exploration shall be performed during Phase B and comply with the GEP approved by the Authority as part of the Phase A work.

5.4.1.1 Owner Notification (Notice of Entry)

Notice of Entry is discussed in Section 1.4.3 of the "Procedures Manual".

5.4.1.2 Locating and Protecting Utilities

The Boring Contractor shall be responsible for confirming the location of any utilities on the project site before performing work and shall be responsible for protecting the utilities throughout the performance of the proposed work. The Boring Contractor shall contact New Jersey One-Call at 811 or 1-800-272-1000 and any other municipal or private entities necessary to locate and mark out all utilities on the Project site prior to performing any work. In situations where records of known public or private utilities are incomplete, hand digging and/or soft digging shall be performed as directed by the GE to delineate utilities on the project site. A Ground Penetrating Radar (GPR) Survey, in accordance with ASTM D6432, and other geophysical methods may be considered at the discretion of the GE. The Boring Contractor shall also submit a written request to the Authority for information on utilities and other subsurface facilities in the Project area. A private locating service may be needed to locate and mark utilities if there are utilities that do not belong to the one-call system. The GE shall maintain a record of all

One-Call Notification tickets, including their submittal and expiration dates.

5.4.2 SOIL BORINGS AND ROCK CORING

5.4.2.1 Soil Testing and Sampling

The Authority preference for advancing soil borings shall be mud-rotary with casing employed for borehole stability when applicable. However, when deemed warranted by the GE, other methods may be permissible, such as hollow stem augers.

Sampling types and intervals will be based upon the Project performance requirements, proposed loads and foundation systems, as well as the soil and rock type, thickness of the strata sampled, and the scheduled laboratory testing. The GE shall tailor sampling intervals to identify stratigraphic transitions or loose, soft, weak, or compressible soils.

Standard Penetration Testing (SPT) shall be performed within all soil borings in accordance with ASTM D1586. All SPT samplers shall be driven 24 inches if feasible. SPT sampling shall be performed continuous to 12 ft. below grade and at 5 foot intervals thereafter. Extended or truncated depths of continuous sampling may be warranted for certain applications as deemed necessary by the GE.

The Boring Contractor may use a 3-inch OD Split-Spoon sampler within gravelly soils, but only after the 2-inch OD Split-Spoon sampler has failed to retrieve adequate samples. Becker Hammer Penetration Test may also be used in gravelly soils.

Undisturbed soil sampling may be obtained using the thin walled tube sampler, Shelby Tube, Osterberg Piston, other piston samplers or the Dennison sampler (ASTM D1587). Thin-Walled Tube sampling shall be performed within fine-grained soils as required. When undisturbed samples are taken, at depths where SPT testing would otherwise be performed, a split spoon sample shall be taken immediately following the undisturbed sample.

5.4.2.2 Rock Coring

Boring termination (refusal of the SPT sampler) usually marks the beginning of coring. However, coring this zone may produce very low core recoveries. For this reason, the GEP shall describe in detail how refusal will be determined and how this transition zone will be investigated so as to maximize information. Acoustic Televiewer (ATV) or Optical Televiewer (OTV) logging may also be used to better define this zone.

The Rock Core samples shall be obtained using a five-foot long NX or NQ size, Double Tube Core Barrel (ASTM D2113). The use of wire line or triple barrel drilling equipment shall be permitted when deemed appropriate by the GE. Rock Core sampling shall begin upon encountering bedrock which is defined in Appendix C.

All attempts shall be made to increase core recovery. When core pieces must be broken to fit into the core box they shall clearly identified as such. The GFR shall make a careful record of the advance of the coring (advance rate, down pressure, water losses) and correlate this to the recovered core to the extent possible. The rock coring log forms to be used are provided in Exhibit 5-4.

5.4.2.3 Supplemental In-Situ Testing

A. Cone Penetration Tests (CPT): Cone Penetration Tests with pore water pressure sensors (CPTu), and Seismic Cone Penetration Tests (SCPTu) shall be performed in accordance with ASTM D3441 (mechanical systems) or ASTM D5778 (electric and electronic systems). The CPT may be performed in conjunction with the soil borings to better define soil and other subsurface parameters. This test method is relatively inexpensive and can provide continuous data. Pore water dissipation testing may be performed and is useful for consolidation settlement analysis. Seismic CPT may be useful in characterizing the shear wave velocity and maximum (low strain) shear modulus.

B. Pressuremeter Testing (PMT): Where requested by the GE through the EOR and approved by the Authority, PMT shall be performed within the borehole in accordance with ASTM D4719. Pressuremeter tests may be used in conjunction with the soil borings to obtain geotechnical parameters including but not limited to undrained shear strength, effective angle of internal friction, elastic modulus, shear modulus, and at-rest earth pressure. The test results can also be utilized to develop site specific p-y curves used in lateral load response of deep foundation elements.

C. Vane Shear Testing (VST): Where requested by the GE through the EOR and approved by the Authority, VST shall be performed within the borehole in accordance with ASTM D2573. The Vane Shear Test may be used in conjunction with the soil borings to obtain the undisturbed undrained shear strength of fine-grained soils. Remolded undrained shear strength is also measured after a predetermined period of time has passed from the initial shearing.

D. Flat Plate Dilatometer Test (DMT): Where requested by the GE through the EOR and approved by the Authority, DMT shall be performed within the borehole in accordance with ASTM D6635. The DMT may be performed in conjunction with the soil borings to interpret soil type and other geotechnical parameters including at-rest lateral earth pressure, elastic modulus, effective angle of internal friction, and shear strengths of sand, silts, and clays.

5.4.2.4 Geophysical Testing

Geophysical Testing offers nondestructive and/or non-invasive methods that can be used for stratigraphic profiling and delineation of subsurface geometries. Certain geophysical tests shall either be required or recommended for different situations. A comprehensive reference on this subject is provided in the Federal Highway Administration publication FHWA-IF-04-021 entitled “Application of Geophysical Methods to Highway Related Problems”. A list of common methods is described below:

A. Crosshole Seismic Testing (CST): Where requested by the GE through the EOR and approved by the Authority, CST shall be performed in accordance with ASTM D4428. Crosshole Seismic Testing shall be performed to obtain soil shear wave velocities and is the preferred method for determination of this parameter. CST may be performed for site specific seismic design or liquefaction evaluation.

B. Downhole Seismic Testing (DST): Where requested by the GE through the EOR and approved by the Authority, DST shall be performed in accordance with ASTM D7400 as a substitute to the CST. Similar to the CST, it provides soil shear wave velocities, however only one cased borehole is required to perform the test. The DST may be replaced by suspension PS logging system.

C. Multichannel Analysis of Surface Waves (MASW): Where requested by the GE through the EOR and approved by the Authority, MASW may be performed to delineate construction debris within fills to assess the potential for obstructions, estimate removal volumes and costs or to identify shallow bedrock surfaces to estimate volumes and costs of rock excavation.

D. Acoustic Televiewer (ATV) and Optical Televiewer (OTV): Where requested by the GE through the EOR and approved by the Authority, ATV or OTV logging shall be performed in accordance with ASTM D5753. ATV and OTV logging may be performed within boreholes to log bedrock conditions including fracture location, orientation, size, strike, dip, and infill material. ATV and OTV logging may also be used to investigate the soil/rock interface, particularly if soil boring or rock coring may not provide adequate definition of conditions. ATV and OTV should be considered where rock socketed foundations are anticipated.

E. Ground Penetrating Radar (GPR): Where requested by the GE through the EOR and approved by the Authority, shall be performed in accordance with ASTM D6432. GPR is performed from the ground surface and is often limited in its depth of survey depending on subsurface conditions. It is often used to detect near surface utilities or obstructions.

5.4.2.5 Sample Identification and Documentation

Soil samples shall be identified in accordance with the Modified Burmister Soil Identification System which is described in Appendix B. Rock Core samples shall be identified in accordance with the Rock Identification System provided in Appendix B. Soil samples for stormwater facilities shall be identified in accordance with the United States Department of Agriculture System provided in Appendix B and the Modified Burmister Soil Identification System. The rock samples shall be assigned a Recovery and Rock Quality Designation (RQD) in accordance with ASTM D6032.

5.4.2.6 Sample Photographs

A digital photo of every sample shall be taken and incorporated into the Phase B Geotechnical Engineering Report. The soil photographs shall meet the following criteria:

- Image quality is adequate to discern soil type, grain size and color
- The photo taken perpendicular to the sample from above with sample in full view.
- A folding rule placed along the sampler for scale.
- The sampling jar cap with complete markings set aside sampler and within the photo for identification.
- The drilling fluid should be scraped from the sample to reveal the natural material’s color and grain size distribution.

The rock core photographs shall meet the following criteria:

- Image quality is adequate to identify changes in rock type, weathered zones, and joints including joint filling.
- The rock sample and core box inside lid shall be included in the photograph
- Depths shall be clearly labeled on with core box, including wooden spacers.
- It may be desirable to wet the core to enhance the features.
- All rock cores shall be photographed in the field as soon as feasible after recovery.

Sample photos of soil and rock core documentation are shown in Exhibit 5-1.

5.4.2.7 Sample Labeling, and Handling

Care shall be taken during the labeling, handling, storage, and transportation of soil and rock samples to prevent disturbance. Securing of soil samples shall be in accordance with ASTM D4220, “Standard Practices for Preserving and Transporting Soil Samples.” Group “B” provisions of ASTM D4220 shall apply to SPT samples, with the samples placed in moisture-proof jars, and “Group D” provisions for Thin-Walled Tube samples. The soil and rock samples shall be transported to a geotechnical laboratory for further review and select laboratory testing in accordance with Section 5.4.10. See Exhibit 5-7 for examples of soil sample labeling.

A. <u>Sample Jar Lids</u>: Each sample jar lid shall contain the following information:
   1. NJTA Project Contract Number
   2. Boring Number
   3. Sample Number; denoted as S-1, S-2, S-3
   4. Sample depth
   5. SPT Blow Counts for each 6-inches of penetration
   6. Sample recovery length
   7. Date sample was taken

B. <u>Sample Jar Boxes</u>: Sample jar boxes shall contain the following information on the top and on one of each of the long and short sides of the box:
   1. Geotechnical Engineering Firm
   2. NJTA Project Name
   3. NJTA Project Contract Number
   4. Boring and Sample Numbers; denoted BR-1 (S-1 to S-10), BR-2 (S-1 to S-16), etc.
   5. Dates samples were taken
   6. Initials of the GFR

C. <u>Thin Wall Samples</u>: Each Thin-Walled Tube Sample shall contain the following information, on the tube and on each of the end caps:
   1. Geotechnical Engineering Firm
   2. NJTA Project Contract Number
   3. Boring Number
   4. Sample Number should be designated by “U” and numbered according to occurrence in the boring sequence; i.e. S-1, S-2, U-1, S-3, S-4, S-5, U-2
   5. Sample depth
   6. Sample recovery length
   7. Label the top and bottom of the tube
   8. Date sample was taken
   9. Initials of the GFR

D. <u>Rock Core Boxes-Outside</u>: The outside top and two side (one long and one short) of a Rock Core box shall contain the following information:
   1. Geotechnical Engineering Firm
   2. NJTA Project Contract Number
   3. NJTA Project Name
   4. Boring Number

5. Core run and depth; i.e. C-1 45’-50’, C-2 50’-55’, etc.
6. Core run recovery; length and percent of total 60 inches run
7. Core run RQD in length and percent of total 60 inches run
8. Date core was taken
9. Initials of the GFR

E. Rock Core Boxes-Inside: The inside of the Rock Core box lid shall be divided into four compartments by drawing three lines to mimic the compartments of the core box and contain the following information:
   1. Boring Number; i.e. BR-1
   2. Core run and depth; i.e. C-1 45’-50’
   3. Core run recovery in percent of total 60 inches run.
   4. Core run RQD in percent of total 60 inches run.
   5. Label the top and bottom of the core run starting in the left corner of the upper compartment and work to the right. When compartment is filled, move down to the left corner of the next compartment and work to the right.
   6. Draw break line in between cores where wooden block separates core runs in the compartments
   7. Indication of natural and mechanical fractures (as shown in Exhibit 5-8)

   See Exhibit 5-8 for examples of rock core box labeling.

F. Rock Core Preservation: When desirable to maintain sample moisture, wrap the core in plastic to prevent drying. In the GEP, specify whether core will require wrapping (i.e. shale).

5.4.2.8 Test Pits

Test Pits may be advanced in conjunction with soil borings to perform infiltration testing and further delineate stratigraphy. Test Pits may be advanced to delineate limits of construction debris in fill material, locate the bedrock surface, identify soil mottling and perched groundwater, or to determine the thickness of surficial soil strata. Test pits shall be benched or sloped back in accordance with OSHA regulations. Bag and bulk soil samples shall be obtained for further review and select laboratory testing. The test pit log forms to be used are provided in Exhibit 5-5.

5.4.3 GROUNDWATER CONSIDERATIONS

Groundwater considerations are often critical for a variety of Project aspects. Specific issues often include the depth to groundwater, fluctuations in ground water depth, and ground permeability as it affects groundwater inflow. The monitoring well log forms to be used are provided in Exhibit 5-6.

5.4.3.1 Bore Holes

The groundwater level shall be measured upon completion of each soil boring and after 24 hours, if feasible. Boreholes may be left open for up to 48-hours after the completion of a soil boring if and where directed by the GE. Emphasis shall be placed on observing perched groundwater conditions and soil mottling which is an indication of the seasonal high water level. However, it is also understood that a borehole may not provide a reliable indication of groundwater levels due to drilling fluids.

5.4.3.2 Temporary Observation Wells

At selected locations a temporary observation well, shall be inserted into the borehole to keep it open, and the water level measured each day by the GFR until it stabilizes. The method of installing the temporary observation wells shall be described in the GEP and the Boring Contract.

5.4.3.3 Monitoring Wells

Monitoring wells shall be installed and monitored to better identify the static groundwater level, artesian conditions and seasonal fluctuations. When authorized by the Authority, the GE may make groundwater level readings over an extended period to better define seasonal fluctuations. Rights of access shall be considered.

5.4.3.4 Piezometers

Piezometric levels of confined water bearing zones (aquifers) may be of importance to the design and construction of certain projects. Piezometers should be sealed into the zone of interest.

5.4.3.5 In-Situ Permeability Testing

In-situ Permeability Testing shall be performed to assist in the design of stormwater management basins and trenchless installations. The Authority prefers Double Ring Infiltrometer tests be performed when the zone of infiltration is readily accessible. This testing can also be accomplished utilizing Tube Permeameters in accordance with the BMP Manual, and also to as laboratory confirmation where the relative density of the in-situ material is known and can be established in the lab. Permeability rates shall be determined, when specified, using the following in-situ testing methods:

A. <u>Double-Ring Infiltrometer Test:</u> shall be performed in accordance with ASTM D3385. One test shall be performed at every soil boring location within a test pit excavated adjacent to the soil boring. The level at which the test is conducted shall be in accordance with Section 2 of Appendix E of the BMP Manual.

B. <u>Tube Permeameters:</u> shall be taken and tested in accordance with Appendix E of the BMP Manual.

C. <u>Basin Flooding Test:</u> shall be performed in accordance with Appendix E of the BMP Manual, when permeability testing is required in Bedrock is deemed necessary by the GE and approved by the Authority.

D. Well Pump Test: shall be performed in accordance with ASTM D4050. Well Pump Tests may be performed in areas where cuts or excavations extend below the static groundwater level and construction dewatering plans will be required.

E. Borehole Percolation Test: shall be performed in accordance with Appendix E of the BMP Manual, when permeability testing is required in proposed cut depth to bottom of basin is deep enough that test pits cannot be installed.

5.4.4 ACID PRODUCING SOILS

Areas containing acid producing soils shall be identified and delineated as part of the subsurface exploration program, so that mitigation measures can be incorporated into the Contract Documents. Refer to Section 7 of the "NJDEP Draft Flood Hazard Area Technical Manual, NJAC 7:13" and the Standards for Soil Erosion and Sediment Control In New Jersey for general locations of acid-producing soil deposits, minimum boring frequencies and depths, chemical testing procedures, and mitigation and disposal standards. This reference shall be supplemented by the following:

A. In proposed cut sections, select test jar samples obtained from soil borings from existing ground elevation to three feet below bottom of proposed grade shall be tested for acid producing soils.

B. The GE shall interpolate the limits of acid producing soils, from each boring within this zone and provide the delineated anticipated volume of "Excavation, Acid Producing Soils" in the Contract Documents. The EOR shall prepare a supplemental specification for Section 202, which requires, "Acid Producing Soils as delineated by the Contract Documents, or otherwise approved by the EOR, shall be removed and replaced with non-acid producing soils or shall be treated to neutralize the acidity with lime for the top three feet. The above treatment shall take place for soils with pH values of 4.0 or less within the construction limits only."

C. A complete list of all pH tests, listed by Station and Offset, shall be included in the Phase B Geotechnical Engineering Report.

D. Tests for acid producing soils shall be performed in accordance with Section 7.4 of the NJDEP Draft Flood Hazard Area Technical Manual.

E. The current Standards for Soil Erosion and Sediment Control in New Jersey require that "soils having a pH of 4 or less or containing iron sulfide shall be covered with a minimum of 12 inches of soil having a pH of 5 or more before seedbed preparation."

5.4.5 PAVEMENT SUBGRADE TESTING

The Authority has developed standard pavement sections and details, which are used for Turnpike and Parkway mainlines and ramps.

Applications where pavement design may be required include Authority Projects and pavement for other agencies including but not limited to the NJDOT, South Jersey Transportation Authority (SJTA), county, and municipality roadways. The following in-situ tests may be used to aid the pavement design, if deemed necessary by the EOR and approved by the Authority.

A. Pavement Cores shall be taken every 500 feet
B. California Bearing Ratio (CBR) shall be performed in accordance with ASTM D4429.
C. Dynamic Cone Penetrometer (DCP) shall be performed in accordance with ASTM D6951.
D. Falling Weight Deflectometer (FWD) shall be performed in accordance with ASTM D4694.

Pavement subgrade testing for other agencies roadways shall be performed in accordance with their standards.

5.4.6 BOREHOLE AND WELL ABANDONMENT AND SITE RESTORATION

All monitoring well casings shall be cut off a minimum of 2-ft. below the ground surface or removed completely. Monitoring wells shall be abandoned in accordance with NJAC 7:9D-3.1. Boreholes shall be abandoned in accordance with NJAC 7:9D-3.4. Borings greater than 25 feet should be grouted using a tremie grout method. The top surface shall receive the same treatment type and thickness as the existing condition. Boreholes in pavements shall be grouted using a tremie grout method and the upper portion backfilled with concrete to the same thickness as the existing pavement.

Test pits shall be backfilled in maximum 12-inch thick lifts and compacted at a minimum by repeatedly striking the soil with the excavator bucket. If directed, the GE may require additional compaction provisions based upon the sensitivity of the area to settlement.

All drilling mud and cuttings shall be hosed off or disposed of beyond developed areas, wherever feasible, and in a legal and environmentally approved manner. Test Pit locations shall be re-graded to match the existing conditions and grassed areas shall be seeded.

5.4.7 AS-DRILLED BORING LOCATION PLANS

For projects that require multiple plans, the first sheet of the Boring Location Plan set shall be a Key Plan with the entire project alignment at a maximum scale of 1:2400. The Key Plan shall be boxed out along the mainline alignment depicting the match line limits of each sheet of the Boring Location Plan at a scale of 1:600. Additionally, the Key Plan shall provide a Boring Schedule containing:

A. Boring numbers shall follow the following convention:
Facility Feature - Consecutive Number. Boring type, groundwater monitoring installations and in situ testing designations shall be included within parenthesis after the boring number. All borings are assumed to include SPT and as such need not be denoted with "SPT". See Section 5.4.7G for examples.

B. Facility Feature Designation:
- BD – Buildings
- BR – Bridges
- CV – Culverts
- DB – Detention, Infiltration, Stormwater Management Facilities
- NB – Sound Barriers
- RD – Roadways
- RR – Railroads
- RW – Retaining Walls
- SS – Sign Structures
- TT – Trenchless Technologies

C. Boring Type Designation:
- SPT – Standard Penetration Test Boring
- CPT – Cone Penetration Test Boring
- SCPT – Seismic Cone Penetration Test Boring
- TP – Test Pit
- PC – Pavement Core

D. Groundwater Monitoring Designation:
- MW – Monitoring Wells
- OW – Observation Wells

E. In situ Test Designation:
- ATV – Acoustic Televiewer
- CST – Crosshole Seismic Test
- DMT – Flat Plate Dilatometer Test
- DST – Downhole Seismic Test
- OTV – Optical Televiewer
- PMT – Pressuremeter Test
- PSL – PS Logging
- PT – Permeability Test
- VST – Vane Shear

F. Sample Designation
- D – Denison Sample
- G – Grabbed Sample/Bulk Sample
- P – Piston Sample
- S – Split Spoon Sample
- U – Shelby Tube Sample

G. Examples for Borings, Groundwater Monitoring, and In situ Tests are as follows:
   - Boring Numbers – BR-08, CV-11 (CPT), RW-02 (SCPT), RD-04 (SCPTu), PC-14, TP-05
   - Groundwater Monitoring – BR-12(OW), RD-06(MW)
   - Field Tests – BR-25 (CST), DB-12 (PT), TP-21 (PT)

The legend shall also contain the following information for each Boring in tabular form:

- Boring Designation
- NAVD Ground Surface or Mudline Elevation at each Boring
- NJ State Plane Northing and Easting Coordinates for each Boring
- Station and Offset and baseline for each Boring
- Estimated Boring Depth
- Estimated Rock Coring Depth

In addition, notes shall be provided, stating the sampling Intervals, geophysical testing requirements, in-situ permeability testing requirements, and any other pertinent remarks.

Refer to Exhibit 5-9 to view a sample Boring Location Plan.

5.4.8 BORING LOGS

The GFR shall record all information associated with soil borings. Subsurface Exploration logs (boring logs) shall comply with the examples presented in Appendix C, and in the approved GEP. Boring logs shall be prepared using gINT or another commercially available software package as approved by the Authority.

5.4.9 SUBSURFACE PROFILE

Using the subsurface information obtained during the Subsurface Exploration, the GE shall construct multiple subsurface profiles or cross sections as required.

The subsurface profiles shall contain:

- Existing and proposed grade lines
- Generalized descriptions of soil strata with boundaries
- Substructure locations
- Stationing or offsets
- Water elevation (e.g., mean high, mean low etc)
- SPT N Value data corrected for hammer energy
- Undrained shear strength if laboratory testing is available
- CPT, VST, DMT, or PMT locations
- Groundwater levels
- Rock type
- Rock core recovery and RQD

- Rock Strength if laboratory testing is available

The minimum vertical and horizontal scales shall be 1”=10’ and 1”=100’, respectively. The subsurface profile shall be included in the Phase B Geotechnical Engineering Report. Refer to Exhibit 5-10 to view a sample Subsurface Profile. The subsurface profiles or cross sections shall be constructed at the following locations:

A. Longitudinally along the mainline roadways, ramps, and bridges

B. Transversely across each abutment, pier, and substructure

C. Longitudinally along a retaining wall, sound barrier wall, culvert centerline, railroad alignments, or trenchless alignment if the conditions vary considerably from those along the mainline.

D. Longitudinally and transversely along foundation grids for buildings

5.4.10 LABORATORY TESTING PROGRAM

A laboratory testing program shall be tailored to meet the specific design needs of the Project. As discussed previously, the GE shall present in the GEP the justification for the testing program and how the results will be used to provide the necessary information to the design team.

An abbreviated list and description of conventionally specified laboratory tests, including analysis and their application is presented below. A list of laboratory methods is also presented in Appendix D.

A. Moisture content (ASTM D2216) and specific gravity (ASTM D854) may be performed on disturbed or undisturbed soil samples.

B. Mechanical grain size analysis (ASTM D422 and D6913) and Atterberg limits (ASTM D4318) may be performed on disturbed or undisturbed soil samples, as appropriate. Testing laboratories shall use appropriate sieve sizes to perform Burmister classification. Hydrometer testing may be utilized when infiltration or permeability is of interest to better characterize the silt and clay contents. Test for amount of fines content (materials finer than No. 200 sieve) in accordance with ASTM D1140 may be utilized when only interested in fines content.

C. Resistivity (AASHTO T288), pH (AASHTO T299), chloride content (AASHTO T291), sulfate content (AASHTO T290), and organic content (AASHTO T267) shall be performed on disturbed or undisturbed soil samples to assess the aggressivity of the soil as related to corrosion. Specifically, the above battery of tests should be performed at different depths and in consideration location of the groundwater with respect to the testing and the facilities shall be considered in developing the testing program. For more accurate test results, these tests shall be expedited upon sampling to minimize the elapsed time between samples collected and samples tested.

D. One-dimensional consolidation testing (ASTM D2435) shall be performed on select undisturbed soil samples to provide compression properties of the soil which will be used to calculate primary, secondary settlement, and time-rate of settlement. Laboratories shall allow sufficient time for each load increment to obtain the secondary consolidation index.

E. Unconsolidated-undrained (UU) (ASTM D2850), consolidated-undrained (CU) (ASTM D4767 with and without pore pressure measurements), and consolidated-drained (CD) (ASTM D7181) triaxial compression tests shall be performed on select undisturbed soil samples to determine strength parameters of organic and inorganic silts and clays. The test sample total and effective stress conditions will affect the shear strength and must be selected depending on several conditions of the soil including mode of deposition, stress history, overburden stress, pre-consolidation stress, and whether the soil strength in the short term or long-term condition is a desired test output. The GE shall ensure the testing laboratory include images of the soil sample before and after the testing.

F. Consolidated-drained (ASTM D3080) and consolidated-undrained (ASTM D6528) direct shear tests and drained torsional shear tests (ASTM D6467, D7608) shall be performed on disturbed or undisturbed soil samples.

G. Unit weight determination shall be performed in accordance with ASTM D2937.

H. Several compressive strength tests (ASTM D7012) shall be performed on rock core samples. Point Load Index Tests may be performed on irregular or broken rock specimens, but shall not be used solely to estimate rock unconfined compressive strength. When Point Load Index Tests are conducted, a site specific correlation between Unconfined Compressive Strength Test results and Point Load Index Test results shall be developed. Splitting Tensile (Brazilian) Tests may be performed normal to the transverse axis of a trimmed rock core specimen to model horizontal discontinuities or foliations in the upper portion of the bedrock surface, if deemed necessary by the GE and approved by the Authority. The Authority’s preferred test is the Uniaxial Compressive Test with Elastic Modulus and stress strain curve. The test is performed normal to the longitudinal axis on a trimmed rock core specimen to model intact rock strength. The testing laboratory shall include images of the rock sample before and after the testing in the test report.

I. Slake Durability Test (ASTM D4644) may be performed on rock core samples of shales and similar weaker rocks to determine the Slake Durability Index (SDI). Los Angeles Abrasion Test (ASTM C131) and CERCHAR Test (ASTM D7625) may also be performed on similar rock types. The SDI and Abrasion Loss may be correlated to predict scour, erosion, degradation and deterioration of rock for design and during construction of shallow and deep foundations or earth retaining structures to account for residual strengths or excessive settlements from relaxation.

J. Chemical tests (Sulfate and pH) for acid producing soils shall be performed in accordance with Section 7 of the NJDEP Draft Flood Hazard Area Technical Manual.

K. Permeability tests shall be performed in accordance with ASTM D2434 and D5084.

5.5 COMPUTER PROGRAMS

Most Projects will require the GE to use commercially available computer software as part of the analysis and design. Computer programs are explicitly noted as tools to be used by the GE. Use of these tools does not relieve the GE of responsibility for correctness and accuracy of geotechnical designs. Sole responsibility for correct use of software and verification/validation of all computer output rests with the GE.

The Phase B Geotechnical Engineering Report should identify the software name, developer and version used. For each software tool, provide a narrative which explains the input values and how they were determined, scope of the analysis and design, an interpretation and recommended use of the results. Tabular and graphical results should be provided. As appropriate, limitations of the software related to the limit state and performance requirement being studied should also be addressed. For example, limitations may include search routines for slope stability analysis, analytical models, convergence tolerance and accuracy and precision of results.

Commercially available spreadsheets or spreadsheets developed in house shall be validated. Formulas or other types of algorithm should be verified for correctness and erroneous calculations due to data remaining from previous calculations must be avoided. The templates should also be suitably checked for accuracy and reliability. These spreadsheets shall be dated and initialed by the developer with a Quality Control check on completion by the GE.

5.6 GEOTECHNICAL ENGINEERING ANALYSIS AND DESIGN – SUPPLEMENT TO AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

5.6.1 STRUCTURAL FOUNDATIONS

5.6.1.1 Soil and Rock Properties

Soil and rock properties shall be obtained from the in-situ tests, laboratory tests, and published correlations developed for similar type of materials. Use of published correlations should be used with caution. Consideration to the source documents, basis of studies and sample population and study setting employed to establish such correlations needs to be carefully considered. Judgment shall be applied based on the relative importance and reliability of the methods. Published AASHTO-LRFD-BDS Section 10.4, FHWA Geotechnical Engineering Circular (GEC) No. 5, FHWA Soils and Foundation Reference Manual, FAVFAC Design Manuals 7.01 &

7.02, and EPRI Manual on Estimating Soil Properties for Foundation Design shall be utilized. Parameters provided in technical manuals of computer programs may be used.

5.6.1.2 Limit States and Resistance Factors

Design and analysis shall be performed for all appropriate Strength, Service, and Extreme Limit States using the load and resistance factors provided in the AASHTO-LRFD-BDS. The Phase B Geotechnical Engineering Report should document the assumptions, loads, limit state conditions, and the date and source of this information, and performance requirements for all geotechnical features.

5.6.1.3 Selection of Structural Foundation Type

The GE shall investigate practical (constructible) and technically feasible foundation types to establish an economical design that safely conforms to prescribed structural criteria and properly accounts for the intended function of the structure. Essential to this work is a rational method of design, whereby various foundation types are systematically evaluated and the optimum alternative selected. The following foundation design approach is recommended.

A. Determine the direction, type and magnitude of foundation loads to be supported, tolerable temporary and permanent deformations and special constraints such as:
   1. Vertical under-clearance, terrain, access, and MPT requirements that limit foundation installation, and access of construction equipment.
   2. Structure type and span length that limit allowable deformations and angular distortions.
   3. Time constraints on construction.
   4. Extreme event loading and construction load requirements.
   5. The presence of cobbles/boulders, rock, or obstructions that may render particular foundation types infeasible.
   6. Scour, settlement, or liquefaction which may preclude the use of a shallow foundation
   7. Construction of proposed foundations in close proximity to existing foundations, such that construction may impact the performance of the existing structures which must remain in service.
   8. Presence of contaminated soils which may skew the cost towards one particular foundation system.
   9. The need for excavation support and dewatering.

B. Evaluate the subsurface exploration and laboratory testing data with regard to reliability and completeness. The chosen design methods shall be commensurate with the quality and quantity of available geotechnical data.

C. Consider alternative foundation types as applicable and conduct a cost and a schedule evaluation. Whether a shallow or deep foundation is feasible the cost evaluation shall be determined as a foundation support to cost assessment. The foundation system support cost should be expressed in terms of dollars per ton load that will be supported. For an equitable comparison (or total cost using the same loading assumptions for each alternative), the total foundation cost should include all costs and schedule impacts associated with a given foundation system such as the need for excavation and retention systems, environmental restrictions on construction activities, e.g. vibrations, noise, disposal of contaminated spoils, pile caps and cap size.

D. The Authority prefers all bridge foundations and directly adjacent bridge approach structures (such as wing wall and retaining walls) to be supported on deep foundations such as piles or drilled shafts. However, where subsurface conditions are appropriate to support spread footing foundations the GE shall ensure the structure can accommodate bearing and settlement criteria as per AASHTO and the constraints provided by the structural engineer. The use of spread footings with deep foundations on the same structure shall not be permitted without the written consent of the Authority.

E. The use of ground improvement methods shall also be considered as discussed in Section 5.7.5.

5.6.1.4 Spread Footings

A. Placement of spread footing foundations shall be based on the following conditions:

1. Spread footings shall be placed on competent soil and rock materials.
2. Bottom of footing shall be placed below a frost depth of 4 feet.
3. Bottom of footing shall be placed a minimum of 1 foot below the depth of total scour for the 500-year storm event.
4. Footings on slopes shall be minimum 4 feet from edge of slope.
5. Consideration of seasonal high groundwater shall be taken into account as it relates to design and constructability.
6. Consideration of seismic hazards specified in Section 5.8.

B. Nominal bearing resistance shall be obtained in accordance with AASHTO-LRFD-BDS Section 10 and verified to be adequate to resist factored bearing pressure under Strength and Extreme Event Limit States.

C. Overburden within the depth of total scour (for the 100 and 500-year storm events) shall be ignored in nominal bearing resistance determination.

D. Sliding and eccentricity limit states shall be checked in accordance with AASHTO-LRFD-BDS for strength limit state and extreme event.

E. Footings bearing on shallow rock shall be keyed or tied into rock to avoid exceeding the sliding or eccentricity limit states. The Authority prefers to not have an individual footing partially bearing on both soil and rock. The GE may choose to over-excavate the rock and place the foundation on compacted stone or over-excavate the soil to the rock surface and backfill with Class C concrete to the bottom of footing.

F. Settlement, lateral deformation and overall global stability shall be computed for service limit state. The global stability of footings in flood plains shall be checked for sudden drawdown conditions. Tolerable total and differential settlement shall be established in consultation with the structural engineer. Potential instability caused by extreme events shall be investigated.

G. Effective footing dimensions and factored and nominal resistances shall be based on meeting all Project specific performance requirements and limit states.

H. The following information shall be included in the Phase B Geotechnical Engineering Report and in Contract Documents:
- Nominal bearing resistance for strength limit state
- Factored bearing resistance for strength limit state
- Nominal bearing resistance for extreme event
- Coefficient of sliding friction (tan δ)
- Footing dimensions (base, length, thickness)
- Bottom of footing elevation
- Groundwater considerations, if necessary, to protect the subgrade from loosening by groundwater infiltration upon dewatering.

I. Vibration and displacement monitoring shall be performed in accordance with Section 5.10.2.6.

5.6.1.5 Driven Piles

A. In addition to the methods and recommendations outlined in the AASHTO-LRFD-BDS, the procedures in the Federal Highway Design and Construction of Driven Pile Foundations Reference Manual Volumes I & II, FHWA-NHI-05-042 & FHWA-NHI-05-043 shall be followed. Driven piles utilized for the support of fenders shall be designed with the requirements specified in this Section and Section 5.7.8.

B. Steel H-piles, cast-in-place (aka reinforced concrete filled steel pipe piles), open ended steel pipe piles, steel tapered piles, timber piles, and prestressed concrete piles shall be considered as driven piles.

C. Axial and lateral nominal resistance within the depth of total scour for the 100 and 500-year storm events shall be ignored as specified in the AASHTO and FHWA references.

D. Axial and lateral resistance shall be reduced in soils susceptible to liquefaction. See Section 5.8.4.

E. External loads from lateral squeeze due to underlying soft compressible soils, and lateral spread and lateral flow conditions due to soil liquefaction specified in Section 5.8.4 shall be considered in the design.

F. Strength limit state loads shall be utilized in determining nominal axial compression resistance and behavior of the pile under combined loads and moments. Service limit state loads shall be utilized to estimate the pile vertical and lateral deformations and overall stability. Extreme Event loads shall be utilized to verify the deformation, and structural resistance of piles. When analyzing pile groups, group efficiencies for axial load and lateral load reduction factors (aka P Multipliers) shall be obtained in accordance with AASHTO-LRFD-BDS. Lateral and group analysis should be performed for piles with pile lengths corresponding to an assumed estimated and minimum pile tip elevation. Vertical and lateral deformation analyses should also address tolerable post construction deformations of the driven pile group at the top of the pier or abutment seat.

G. Steel pile sections shall be reduced for corrosion as specified in FHWA-NHI-05-042. Additional section loss shall be considered where aggressive conditions exist. Refer to AASHTO and the FHWA driven pile reference for guidance on deterioration mechanisms and mitigation measures for piles. If the soils are of an aggressive nature, a site specific corrosion assessment shall be considered and if deemed necessary shall be performed by an underground Corrosion Specialist. Specific corrosion rates and protection methods shall be included in the Phase B Geotechnical Engineering Report submittal.

H. The design of driven piles shall consider the following requirements:

   1. Nominal Axial Compression Resistance: Nominal Axial Compression Resistance is defined as the nominal resistance obtained from suitable soil or rock bearing materials (ignoring resistance from scourable soils,

liquefiable soils, and unsuitable materials) using the design methods provided in AASHTO-LRFD-BDS. This resistance shall be multiplied by resistance factors provided in AASHTO-LRFD-BDS to determine the factored axial compression resistance.

2. Nominal Driving Axial Compression Resistance: Nominal Driving Axial Compression Resistance shall be obtained by including the nominal side resistance from scourable soils, soils susceptible to downdrag, and other unsuitable materials not utilized as part of the nominal axial compression resistance determination. In addition to nominal side resistance from soils susceptible to downdrag, the factored downdrag load shall be added to determine nominal driving axial compression resistance. This resistance should account for changes in soil resistance which will occur during the driving process.

3. Maximum Driving Resistance: If hard/very dense materials or obstructions are expected in the intermediate layers and a higher resistance than the nominal driving axial compression resistance is anticipated to drive through such materials then it shall be reported as Maximum Driving Resistance. In this circumstance, an independent pile drivability analysis shall be performed to model this situation and assess any special guidance which should be included in the Contract Documents.

4. Minimum Pile Tip Elevation: Minimum Pile Tip Elevation is defined as the elevation corresponding to the minimum required depth of penetration. Not all projects will require a minimum pile tip elevation (i.e. structures with axial loads only). Pile length for driven pile foundations supporting bridge substructures should be determined to satisfy the second point of inflection on the deflection diagram. Minimum Pile Tip Elevation shall also be set deep enough to satisfy the nominal uplift resistance, and penetrate scourable and unsuitable materials, which may be present above the bearing stratum. A schematic for point of inflection is provided in Exhibit 5-11.

5. Estimated Pile Tip Elevation: Elevation corresponding to the depth where nominal axial compression resistance was estimated by static analysis methods shall be reported as Estimated Pile Tip Elevation.

I. The following downdrag reduction methods shall be considered where it's economical and feasible:
   1. Isolating the piles from soils susceptible to downdrag by driving piles through preinstalled isolation casings.

2. Commercially available spray-on friction reducing coating (durability and survivability must be considered).
3. Wrapping the piles with Tyvek or similar barrier though embankment zones.
4. Driving piles after the settlement sufficient to develop downdrag has occurred (monitoring should be considered).

J. Test piles shall be installed to verify the estimated pile lengths, establish final production pile lengths and to establish a pile driving criteria for production pile driving. Production piles can be utilized as test piles or test piles may be located outside the footing area or, nearby, where the subsurface conditions are similar. The overburden materials above the bottom of the footing elevation shall be removed prior to driving test and production piles or the resistance associated with these materials carefully assessed and discounted from the resistance available. The use of a cleaned out shell or casing through the overburden is not a desired procedure and shall only be used under exceptional circumstances, and then only with prior approval of the Authority. The decision to require static load test piles shall be based upon the recommendation of the GE and the approval of the Authority. The GE should consider issues such as redundancy, pile type, load demand, cost benefits and site constraints in making this recommendation. The requirement to test two piles dynamically per substructure unit is the Authority's minimum standard (see 5.6.1.5L).

K. Based on review of the test pile results, estimated production pile lengths and recommended pile driving criteria shall be established by the GE and through the EOR submitted to the Authority. The pile driving criteria should specify the desired hammer blow count and corresponding stroke similar to that observed for the test piles. In addition, the GE should provide guidance for pile acceptance during original driving or restrike if the required nominal driving axial compression resistance is not achieved during initial driving. Finally, a refusal blow-count shall be provided in the event the piles are unable to advance below minimum tip elevation recommended.

L. Pile Dynamic Analyzer (PDA) tests with Case Pile Wave Analysis Program (CAPWAP) shall be performed on driven piles used for bridge substructures. A minimum of two piles or 5 percent of the piles, whichever is greatest, per substructure shall be subjected to PDA testing. If pile resistance is anticipated to increase due to pile setup, then the PDA test also shall be performed during restrike. The GE shall provide the appropriate waiting period for restrike and tip elevation or PDA resistance to halt the initial drive in the Contract Documents. If the Contractor elects to not perform restrikes when recommended where pile set is anticipated and drive the piles deeper to achieve the necessary capacity, the additional depth will be at no additional cost to the Authority. The restrike waiting periods shall be determined based on project specific conditions and recommended in the Phase B Geotechnical Engineering Report and reported on the Contract Plans. The CAPWAP shall be performed at the end of initial drive and at the beginning of restrike. Pile restrikes should be performed with an adequately sized hammer and appropriate stroke as to mobilize sufficient end bearing when side shear values may increase to the point where end bearing appears to diminish due to insufficient energy making it to the tip. Restrike testing shall also be performed on piles driven in dense to very dense granular materials or weathered shale to confirm there is no reduction in pile resistance due to relaxation.

M. Static axial Load tests shall be performed if required by the GE and through the EOR approved by the Authority in accordance with ASTM D1143, "Standard Test Methods for Deep Foundations Under Static Axial Compressive Load". Other methods such as "Standard Test Methods for Axial Compressive Force Pulse (rapid) Testing" (ASTM D7383) shall be specified and used only with approval by the Authority. When performing such tests on production piles, care shall be taken not to overstress the pile. Specific Project guidance should be included in the Contract Documents.

N. Lateral and uplift testing shall be performed if required by the GE, and approved by the Authority. Lateral load testing shall be performed in accordance with ASTM D3966 "Standard Test Method for Deep Foundations under Lateral Load". The Authority prefers that if lateral load tests are required by the GE, they be performed in a pre-bid phase. Given the cost of installing a test pile and performing a lateral load test pre-bid, this is generally not considered cost efficient to the Authority. Uplift testing shall be performed in accordance with ASTM D 3689M "Standard Test Method for Deep Foundation under Static Axial Tensile Load"

O. The following information shall be determined and included in the Phase B Geotechnical Engineering Report and in Contract Documents:
   - Minimum Pile Tip Elevation
   - Estimated Pile Tip Elevation
   - Nominal Axial Compression Resistance
   - Factored Axial Compression Resistance
   - Nominal Driving Axial Compression Resistance
   - Nominal and Factored Uplift Resistance
   - Maximum Driving Resistance
   - Bottom of Footing Elevation
   - Pile type, size, and material
   - Load testing requirement (PDA, Static Load Test)

- Cap dimensions (Length, Width, Thickness)
- Reinforcing cage details and compressive strength of concrete (if applicable)
- Overhead clearance, access, or MPT restrictions
- Environmental constraints
- Sequence of construction
- Pile Layout Plan

P. Vibration and displacement monitoring shall be performed in accordance with Section 5.10.2.6.

5.6.1.6 Drilled Shafts

A. In addition to the methods outlined in the AASHTO-LRFD-BDS Specifications, the procedures in the Federal Highway GEC No. 10 (FHWA-NHI-10-016) shall be followed. Drilled shafts utilized for the support of fenders shall be designed with the requirements specified in this section and Section 5.7.8.

B. The minimum diameter of a drilled shaft shall be 30 inches. The minimum center to center spacing of any two drilled shafts shall be 3.0 diameters. However, if the center to center spacing is less than 4.0 diameters, the group reduction factors presented in Table 10.8.3.6.3-1 (Group Reduction Factors for Bearing Resistance of Shafts in Sand) of AASHTO-LRFD-BDS shall be applied, and the sequence of construction should be specified in the contract document.

C. Foundation redundancy shall be defined as when a single foundation unit contains 3 or more drilled shafts. Where a single foundation contains less than 3 drilled shafts, a 20 percent reduction shall be applied to the resistance factors presented in Table 10.5.5.2.4-1 "Resistance Factors for Geotechnical Resistance of Drilled Shafts" of AASHTO-LRFD-BDS.

D. Axial and lateral nominal resistance within the depth of total scour, for the 100 and 500-year storm events, shall be ignored as specified in the AASHTO and FHWA references.

E. Lateral resistance shall be reduced in soils susceptible to liquefaction when socketed into rock. Both axial and lateral resistance shall be reduced in soils susceptible to liquefaction when founded in soils. See Section 5.8.4.

F. External loads and overburden pressure which result in lateral squeeze due to underlying soft compressible soils, and lateral spread and lateral flow conditions due to soil liquefaction specified in Section 5.8.4, shall be considered in the design.

G. Strength limit state loads shall be utilized in sizing drilled shafts to have sufficient nominal axial compression resistance and behavior of shafts under combined loads and moments. Service limit state loads shall be utilized to estimate the shaft deformations and compared with tolerable limits. Extreme Event and Service shall be utilized to verify the deformation, and structural resistance. When analyzing shaft groups, group efficiencies for axial loads and lateral load reduction factors shall be obtained in accordance with AASHTO-LRFD-BDS. Lateral and group analysis should be performed for shafts with lengths corresponding to the estimated or minimum shaft tip elevation.

H. Permanent Steel Casing sections shall be reduced for corrosion as specified in FHWA-NHI-05-042. Additional section loss shall be considered where aggressive conditions exist. Refer to AASHTO and the FHWA driven pile reference for guidance on deterioration mechanisms and mitigation measures for piles If the soils are of an aggressive nature, a site specific corrosion assessment shall be considered and if deemed necessary shall be performed by an underground Corrosion Specialist. . Specific corrosion rates and protection methods shall be included in the Phase B Geotechnical Engineering Report submittal.

I. Factored downdrag load shall be included in design.

J. Rock socket length of drilled shafts shall be at least 1.5 times the shaft diameter.

K. Shaft length for drilled shaft foundations supporting bridge substructures should be determined to satisfy the second point of inflection on the deflection diagram, nominal axial compression resistance, and nominal uplift resistance. A schematic for point of inflection is provided in Exhibit 5-11. Lateral analysis should also address tolerable post construction lateral deformations of the drilled shaft group at the top of pier or bridge abutment seat.

L. Casings shall be defined as one of the following types:

Temporary – This casing is not specified in the contract drawings. It is used and selected as required by the Contractor to facilitate shaft construction upon approval of the GE and is removed during shaft construction.

Interim – The use of this casing is specified in the Contract Document; however, is not incorporated into the structural design of the shaft. It is used to facilitate construction and remains in place. The casing utilized is selected by the contractor to withstand installation forces.

Permanent – This casing is specified by the GE in the contract drawings and may be incorporated into the structural resistance of the shaft. Where an outer permanent casing is necessary to satisfy the bending or deflection requirements, proper structural design shall be performed for composite behavior of shafts if so desired. Shear rings and shear studs shall be considered inside and outside the casing. Consideration of corrosion loss or protection should be considered given the installed setting. Positive mechanical shear attachment shall be included to consider true composite action. However, the stiffness of the casing should still be considered additive to that of the shaft if no positive mechanical attachment is to be used. Composite or non-composite casings shall not be considered effective for end moment restraint at rock sockets or top of shaft to substructure element connection points.

M. One verification boring shall be performed by the Contractor prior to shaft construction at each drilled shaft location with a rock socket for drilled shaft diameter 6 feet or larger. The number of construction phase borings for smaller diameter shafts shall be estimated based on the site variability. All verification borings shall extend to twice the shaft diameter below the design tip elevation, unless otherwise approved by the Authority.

N. Shaft Inspection Device (SID) inspection of the drilled shaft excavation bottom shall be performed for all shaft designs which include any contribution of end bearing to the nominal axial compressive resistance. The GE shall evaluate use of the SID based on the shaft design, and make specific recommendation for its use in the Contract Documents.

O. Crosshole Sonic Logging (CSL) tests shall be performed in all drilled shafts to confirm their integrity in accordance with ASTM D6760. Additional investigation such as tomography, concrete cores, etc. shall be performed when CSL results indicate discontinuities are present which may affect the structural or geotechnical nominal resistance of the drilled shaft.

P. Thermal Integrity Profile (TIP) testing in accordance with ASTM D7949 shall be required by the Authority for all shafts with diameters of 6 feet or greater.

Q. The decision to require load testing and/or demonstration shafts shall be based upon the recommendation of the GE at the approval of the Authority. The GE should consider issues such as redundancy, shaft diameter, load demand, and site constraints in making this recommendation. At a minimum, load tests shall be performed on demonstration or production shafts (as recommended) to verify the geotechnical resistance or establish the final shaft tip elevation during construction for bridge substructures. Osterberg Load Cells, AASHTO TP 100 “Standard Method of Test for Deep Foundation Elements under Bidirectional Static Axial Compressive Load”, and or ASTM

D1143, "Standard Test Methods for Deep Foundations Under Static Axial Compressive Load" are acceptable test methods. Other methods such as Statnamic load tests or the "Standard Test Methods for Axial Compressive Force Pulse (rapid) Testing" (ASTM D7383) shall be specified and used only with approval by the Authority. When performing such tests on production shafts, care shall be taken not to fail the shafts and specific project guidance should be included in the Contract Documents.

R. Lateral and uplift testing shall be performed if required by the GE, and through the EOR approved by the Authority. Lateral load testing shall be performed in accordance with ASTM D3966 "Standard Test Method for Deep Foundations under Lateral Load". Uplift testing shall be performed in accordance with ASTM D3689M "Standard Test Method for Deep Foundation under Static Axial Tensile Load". The Authority prefers that if lateral load tests are required by the GE, they be performed in a pre-bid phase. Given the cost of installing a test shaft and performing a lateral load test pre-bid, this is generally not considered cost efficient to the Authority.

S. The following information shall be obtained from the design and included in the Phase B Geotechnical Engineering Report and in Contract Documents:
   - Nominal Axial Compression Resistance
   - Factored Axial Compression Resistance
   - Nominal Uplift Resistance
   - Factored Uplift Resistance
   - Top of Drilled Shaft Elevation
   - Estimated Top of Rock Socket Elevation (where appropriate)
   - Estimated Tip of Drilled Shaft Elevation
   - Shaft diameter
   - Rock socket diameter (where appropriate)
   - Reinforcing size, type, grade and layout
   - Spiral bar No and Pitch or Hoop Bar No and Spacing
   - Concrete compressive strength
   - Casing outside diameter (thickness, and grade if permanent)
   - Casing type (interim or permanent)
   - Demonstration and load testing requirement (O-Cell or Static Load Test)
   - Boring requirements
   - Thermal Integrity Testing and Shaft Inspection Device Requirements
   - Bottom of cap elevation (if applicable)
   - Cap dimensions (if applicable)
   - Length of Casing Seated into Rock
   - Overhead clearance, access, or MPT restrictions
   - Environmental constraints

- Rock laboratory test results and boring logs
- Sequence of construction (i.e. communication, downdrag)
- Drilled Shaft Layout Plan

Q. Vibration and displacement monitoring shall be performed in accordance with Section 5.10.2.6.

5.6.1.7 Micropiles

In addition to the methods outlined in the AASHTO-LRFD-BDS, the procedures in the Federal Highway Micropile Design and Construction FHWA-NHI-05-039 shall be followed.

A. The minimum center to center spacing between micropiles shall be not less than 30 in. or 3.0 diameters center to center whichever is greater. Axial and lateral nominal resistance within the depth of total scour, for the 100 and 500 Year storm events, shall be ignored as specified in the AASHTO and FHWA references.

B. Axial and lateral resistance shall be reduced in soils susceptible to liquefaction. See Section 5.8.4.

C. External loads and overburden pressure which result in lateral squeeze due to underlying soft compressible soils, and lateral spread and lateral flow conditions due to soil liquefaction or other phenomenon shall be considered in the design.

D. Factored downdrag load shall be included in design.

E. Micropiles shall be specified and paid for on a per micropile basis to allow the Contractor to optimize installation methods as they are directly related to resistance.

F. Strength limit state loads shall be utilized in determining nominal axial compression and uplift resistances and behavior of piles under combined loads and moments. Service limit state loads shall be utilized to estimate the pile deformations and overall stability and compared with tolerable limits. Extreme event loads shall be utilized to verify the deformation, and demand to capacity ratio to be within the tolerable limits. When analyzing micropile groups, the group efficiencies for axial loads and lateral load reduction factors shall be obtained in accordance with the AASHTO-LRFD-BDS. Lateral and group analysis should be performed for micropiles with lengths corresponding to minimum tip elevation or estimated tip where there is no lateral or uplift demand on the foundation.

G. Steel casing sections shall be reduced for corrosion as specified in FHWA-NHI-05-042. Additional section loss shall be considered where aggressive conditions exist. Refer to AASHTO and the FHWA driven pile reference for guidance on deterioration mechanisms and mitigation measures for piles. If the soils are of an aggressive nature, a site specific corrosion assessment shall be considered and if deemed necessary shall be performed by an underground Corrosion Specialist. Specific corrosion rates and protection methods shall be included in the Phase B Geotechnical Engineering Report submittal.

H. Micropile casing thickness shall be reduced by 50% to account for casing joint thread reduction in accordance with Section 5.18.3 of FHWA Micropile Reference Manual, FHWA-NHI-05-039.

I. Combined axial compression and bending stresses shall be evaluated in accordance with Section 6.12 AASHTO-LRFD-BDS.

J. End bearing resistance shall be ignored in soils and soft rock. In hard rock micropiles may be designed for end bearing, although this is usually negligible.

K. Downhole Close Circuit Television (CCTV) inspection shall be performed for all rock socketed micropiles. The depth of the camera shall be shown continuously on the screen as the camera is moving along the socket during the inspection. Video copy of all downhole televised inspections shall be submitted to GE for review.

L. Load tests shall be defined as verification and proof tests.

M. Verification tests shall be performed on sacrificial micropiles to confirm design assumptions and establish nominal unit grout-to-ground bond resistances. The GE shall specify the verification test(s) to maximize the micropile resistance by potentially shortening the bond zone and possibly increasing the bar size to accommodate a large enough load to fail the micropile grout-to-ground bond resistance.

N. One verification test micropile shall be performed for each geologic unit and each different micropile size and type. Verification test piles shall be loaded to failure, or two times the nominal resistance. These piles are thus sacrificed and shall not be included as part of the structure.

O. Load tests shall be performed in accordance with ASTM D1143, "Standard Test Methods for Deep Foundations Under Static Axial Compressive Load". A tension test is an acceptable test method to establish bond resistance in lieu of a compression test for piles not deriving capacity on hard rock which may include end bearing. Generally, uplift (tension) testing is preferred by the Contractor as it generally is less expensive. Uplift testing shall be performed in accordance with ASTM D3689M "Standard Test Method for Deep Foundations under Static Axial Tensile Load".

P. Lateral testing shall be performed if required by the GE, and approved by the Authority. Lateral load testing shall be performed in accordance with ASTM D3966 "Standard Test Method for Deep Foundations under Lateral Load".

Q. Proof load tests on production piles shall be performed in accordance with ASTM D3689 “Standard Test Method for Deep Foundations under Static Axial Tensile Load”. Production piles shall be tested at a frequency of one pile per substructure or five (5) percent of the total piles, whichever is greater. Production piles are not tested to failure. Production piles shall be tested to 0.7 times the nominal resistance shown on the plans for the substructure where the proof load test is being performed.

R. The following information shall be obtained from the design and included in the Phase B Geotechnical Engineering Report and in Contract Documents:
- Nominal Axial Compression Resistance
- Factored Axial Compression Resistance
- Nominal Uplift Resistance
- Factored Uplift Resistance
- Top of micropile elevation (Micropile Cutoff Elevation)
- Minimum tip of casing elevation
- Estimated Bond Zone Length
- Minimum Bond Zone Length (may be required)
- Micropile casing outside diameter, grade and wall thickness
- Micropile type (typically A in Rock, B in Soil)
- Number and location of Verification Load Tests
- Modified structural properties for Verification Load Test (i.e. bigger bar, if required)
- Location of proof load test (only if larger bar is required for structural tensile resistance, otherwise, select suspect micropiles during construction based on installation observations)
- Reinforcing size, type, grade and layout
- Grout compressive strength
- Cap dimensions (length, width, depth)
- Micropile Layout Plan

S. Vibration and displacement monitoring shall be performed in accordance with Section 5.10.2.6.

5.6.2 WALLS AND ABUTMENTS

In addition to Section 11 of AASHTO-LRFD-BDS, Section 2.3 of this Design Manual and FHWA Earth Retaining Structures Reference Manual (FHWA-NHI-07-071) the guidance below shall be followed.

5.6.2.1 Wall Type Selection

Wall type selection should be based on a systematic evaluation process. The objective of wall selection is to determine the most appropriate wall type that is cost effective, practical to construct, stable, and aesthetically and environmentally consistent with its surroundings. Consideration of existing and proposed utilities shall be considered as they relate to constructability of the proposed wall systems. The GE shall consider both the drained and undrained conditions where appropriate in areas where cohesive soils are retained. All wall systems shall be designed to support all earth, hydrostatic, and surcharge loads as required in Section 3.11 of AASHTO-LRFD-BDS. Additionally, all wall systems including walls not subjected to surcharge loading during service shall be designed to support a minimum live load surcharge of 250 psf or greater as deemed appropriate by the GE for the anticipated construction equipment traffic above the wall. Where it is anticipated that construction staging will require large cranes or other specialized equipment, the contract documents shall include a maximum surcharge load for an assumed construction approach as well as require the Contractor to perform independent assessment such as surcharge, strip, line, or point loads and be taken into account in the crane mat design. When the foundation recommendations indicate the use of wall types other than Cast in Place type walls based on cost, a cost comparison of the alternates assessed shall be provided so the Authority may determine the final wall type.

5.6.2.2 Soil and Rock Properties

Soil and rock properties shall be obtained from the in-situ tests, laboratory tests, and published correlations developed for similar type of materials. Judgment shall be applied based on the relative importance and reliability of the methods. AASHTO-LRFD-BDS Section 10.4, FHWA GEC No. 5, and FHWA Soils and Foundation Reference Manual shall be utilized. Backfill placed behind walls should be specified, and compactive effort near the wall should be limited to avoid excess earth pressures.

Reinforced soils, retained soils, foundation soils, and fills shall be tested for pH, resistivity, chloride content, and sulfate content in accordance with Section 5.4.10 to assess if the soils are of an aggressive nature.

5.6.2.3 Drainage

In addition to AASHTO-LRFD-BDS Section 11.6.6, FHWA-NHI-07-071, “Earth Retaining Structures”, shall be followed. The drainage system shall be designed to completely drain the entire retained and reinforced soil volumes. The details of the drainage system should be prepared by the Project Drainage Engineer. Long term plugging of the drainage system can cause drainage system failure and create much higher hydrostatic loads on the wall than considered in the design. Weep holes or geocomposite panel drains at the wall face do not assure fully drained conditions and in some cases blanket slope drains, foundation base drains and drilled horizontal drains may be used to achieve a complete drained condition behind and immediately beneath the wall.

For cantilever walls, gravity, semi-gravity, and MSE walls (see definitions in Section 2 of the Design Manual); the drainage system shall include a perforated collector drain pipe. The drainage system shall run along the heel of cantilever, semi-gravity or gravity retaining walls; at the bottom of the rear face of the reinforced soil mass in MSE walls; or at the bottom of the back of the wall facing elements and solid outlet pipes. Section 2 of the Design Manual provides standard drainage details for cast in place and MSE walls. The outlet pipes shall convey the water through weep holes in the wall face or to a storm water basin or drainage system. The size of the pipes shall be determined on a Project specific basis based on analyzing flow nets and estimating seepage volumes. Seepage volumes shall be determined utilizing USACOE – Seepage Analysis and Control for Dams EM1110-2-1901. Appropriately designed geocomposite drains may be substituted for aggregate drain details for all wall types.

The collector drain pipe shall be either perforated corrugated HDPE drainage pipe that conforms to AASHTO M 294 and is Type S (smooth interior with annular corrugations) with gasket water-tight joints or perforated PVC drainage pipe that conforms to ASTM D2729. The outlet pipes shall be solid plastic drainage pipe conforming to NJTA Standard Specifications. All drainage details shall include clean out pipes to maintain the drainage system.

A layer of washed gravel, in accordance with NJTA Standard Specifications, surrounding the perforated collector pipe and wrapped in a geotextile with an apparent opening size equivalent to the #30 sieve and in accordance with NJTA Standard Specifications or geotextile filters shall be designed in accordance with AASHTO M 288 and FHWA Geosynthetic Design and Construction Guidelines (FHWA-HI-95-038).

For temporary sheet pile walls drainage weep holes should be created in the sheets prior to driving below the elevation of static groundwater in order to provide equilibrium in groundwater on both sides of the sheet face. In addition, weep holes should be installed on the exposed face using commercially available prefabricated drains at the toe of wall.

The potential should be assessed for ponding of overland flow at the top of the wall, which could create greater hydrostatic forces behind the wall. As stated previously, drainage behind all walls is a critical issue.

5.6.2.4 Lateral Earth Pressures

Active, at-rest, and passive lateral earth pressure coefficients shall be calculated in accordance with Section 3.11 of AASHTO-LRFD-BDS and Section 10 of FHWA Soils and Foundations-Volume II. Earth pressures due to external loads (uniform surcharge, point load, line load, and strip load) shall be calculated in accordance with Section 3.11.6 of AASHTO-LRFD-BDS. Live Load Surcharge shall be obtained from Section 3.11.6.4 of AASHTO-LRFD-BDS. Load factors shall be obtained from Section 3 of AASHTO-LRFD-BDS. Seismic earth pressures shall be calculated in accordance with Section 11 of AASHTO LRFD and Section 5.8 of this manual. All permanent retaining walls shall be designed for drained and undrained conditions.

5.6.2.5 Limit States and Resistance Factors

Design for all walls and abutments shall be performed for Strength, Service, and Extreme Limit States using the resistance factors provided in AASHTO-LRFD-BDS Section 11.5.

5.6.2.6 Abutments and Conventional Retaining Walls

Rigid gravity and semi-gravity retaining walls may be used for bridge and other substructures or grade separation applications and are generally permanent. Rigid gravity and semi-gravity walls shall not be used without deep foundations support where bearing soil/rock is prone to excessive total or differential deformations. Abutments supported on spread footings on MSE walls are not permitted for use on NJTA facilities.

5.6.2.7 Non-Gravity Cantilevered Walls

Non-gravity cantilevered walls may be considered for either temporary or permanent support. These walls are all used in cut (top down construction) and require much less ROW behind the wall when compared to a conventional retaining wall. The feasibility of using a non-gravity wall at a particular location shall be based on the suitability of the soil and rock conditions within the depth of the vertical element embedment to support the wall. Non-gravity cantilevered walls including discrete and continuous vertical elements shall be considered. Ground anchors should be considered based upon tolerable deformations, ROW limits, and cost.

Tolerable lateral and vertical deformations at the top of wall shall be determined by the EOR. The GE shall consider protection of structures behind the wall and potential damage to the wall. The justification for tolerable deformation shall be presented in the Phase B Geotechnical Engineering Report.

The selection of cantilever wall systems such as sheet pile or soldier pile and lagging systems should include consideration of excessive vibration and deformations resulting from the installation process into some formations as well as the potential that obstructions could prevent or alter the installation of soldier piles. The retained soil should be studied carefully as it is the required support at the base of the wall to support the cantilever structure.

The resistance from upper 2 feet in front of the wall shall be discounted to accommodate the likelihood of future excavation in front of the wall. Global stability analysis shall be performed for permanent walls.

Care should be taken as many computer software applications used to design these types of walls were not developed for LRFD methods. The following steps shall be followed for design:

- Perform force and moment equilibrium analysis with load and resistance factors provided in AASHTO-LRFD-BDS to determine the minimum embedment length.
- Design minimum sheet pile section or soldier pile section with or without drilled shaft.
- Perform lateral analysis using computer software with the minimum section properties and embedment length obtained from force and moment equilibrium analysis.
- Increase section properties and embedment length (if needed) to limit lateral deflection.

Drainage requirements shall be in accordance with Section 5.6.2.3.

Corrosion rates published in Section 8.8 of FHWA-NHI-05-042 shall be used as a guideline for predicting corrosion rates in marine and non-marine applications; however, if the soils are of an aggressive nature, a site specific corrosion assessment shall be performed by an underground Corrosion Specialist. Corrosion will occur on all exposed surfaces and therefore may advance from both sides of steel sheet piles, and piles. The corrosion rate may differ between sides based on the level of exposure and this phenomenon shall be accounted for when designing corrosion protection measures.

The steel sheet pile and pile sections shall be designed to account for corrosion while maintaining the factored structural and geotechnical resistances under Strength, Service, and Extreme Event Limit States throughout the structure's design life. Providing sacrificial steel thickness to the steel sheet pile section shall be the primary mode of protection from corrosion. Secondary measures to protect from corrosion shall be galvanizing or coal-tar epoxy coating. Coal-tar epoxy coatings shall be in accordance with the NJTA

Standard Specifications and the NJTA Supplementary Specifications.

A. Steel Sheet Pile Walls

1. In addition to FHWA-NHI-07-071, NAVFAC DM7.02 – Foundations & Earth Structures, and the US Steel Sheet Piling Design Manual shall be followed.

2. Ground anchors shall be installed to limit lateral wall deflection and bending moments if increasing the sheet piles section modulus is not feasible. If and where required, ground anchors shall be designed in accordance with Section 5.6.2.8.

3. Temporary sheeting design is the responsibility of the Contractor; however, all relevant geotechnical soil properties, groundwater level, and minimum loading and surcharge criteria shall be provided on the Contract Plans. Permanent steel sheeting design is the responsibility of the EOR, including complete detailing of the wall. The EOR shall take care to execute designs that either do not conflict or appropriately modify the provisions of the NJTA Standard Specifications.

4. Vibration and displacement monitoring shall be performed in accordance with Section 5.10.2.6.

5. The following minimum information shall be provided in the Contract Documents for permanent sheet pile walls:
   - Assumed Sheet Pile Section and Type
   - Minimum Plastic Section Modulus and Moment of Inertia
   - Minimum Sheet Pile Tip Elevation
   - Coating

6. The following minimum information shall be provided in the Contract Documents for temporary sheet pile walls:
   - Soil properties (friction angle, unit weight, undrained shear strength, drained shear strength)
   - Groundwater level
   - Limits of sheeting
   - Surcharge loads

B. Solider Pile and Lagging Walls

Soldier pile and lagging wall systems (also known as post and panel or post and plank walls) are commonly used for temporary excavation support in dense or stiff soil and are preferred over sheet pile. They are also frequently used for permanent earth retaining structures. Soldier pile and lagging walls consist of soldier piles usually spaced at 5 to 10 ft. on center and lagging which spans the distance between the soldier piles. The soldier piles may be driven or placed in drilled shafts. The lagging is used to retain the soil face from sloughing and to transmit the lateral earth pressure to the soldier piles. Non-Gravity Cantilevered Walls shall be designed in accordance with AASHTO-LRFD-BDS Section 11.8.

The following information shall be provided in Contract Documents:

- Soldier Pile: type, size, length, grade and top of pile elevation.
- Drilled shaft diameter, top of shaft elevation, tip of shaft elevation, estimated top of rock elevation, rock socket length, and center to center shaft spacing.
- Lagging type, size and material requirements

C. Tangent/Secant Pile Walls

Tangent/Secant Piles are continuous vertical elements that can be a viable alternative to steel sheet piles when a stiffer wall system with higher resistance to bending moments is required. Additionally, Tangent/Secant Pile Walls may also be installed in close proximity to vibration sensitive structures, in soils containing cobbles and boulders where driving steel sheeting or piles may not be feasible, or also in environmentally critical locations due to their low permeability.

1. In addition to AASHTO-LRFD-BDS Section 11.8, FHWA GEC No. 2 FHWA-SA-96-038 shall be followed.

2. The diameter of the individual piles within the wall shall be no greater than 42 inches. The length of the individual piles shall be limited to 100 feet. Should these restrictions prevent the use of a cantilevered wall, ground anchors shall be installed and the wall shall be considered an anchored wall.

3. The installation of the individual piles shall be phased such that the primary piles are installed first at every other pile location along the wall alignment and the secondary piles are installed between the primary piles. The duration between the primary and secondary pile installation shall be staged such that the concrete/grout in the primary piles can be partially drilled and the primary and secondary piles can be overlapped.

4. Tangent/Secant Piles Walls comprised of continuous flight augers (CFA) piles shall not be permitted in soil profiles with very soft silt and clay layers or loose granular soils with a shallow groundwater table. Drilled Displacement (DD) piles shall be installed in these conditions to preclude the occurrence of necking, bulges, structural defects, and soil mining in the pile.

5. Earth pressures shall be calculated in accordance with AASHTO-LRFD-BDS Section 3.11. Drainage requirements shall be as addressed in Section 5.6.2.3. The individual piles shall then be modeled as a simple concrete beam in bending to size pile diameters and determine the amount of reinforcing steel required. Steel reinforcement in the form of a rebar cage or a steel beam shall be placed within the secondary piles.

6. The following information shall be provided in the Contract Documents:
   - Pile diameter
   - Pile reinforcement and required concrete strength
   - Top and bottom pile elevation
   - Pile spacing
   - Field QA/QC Requirements

5.6.2.8 Anchored Walls

Anchor walls (also referred to as tieback walls), whose elements may be proprietary, employ grouted anchor elements, vertical elements and facing. Any of the above Non-Gravity Cantilever Walls could be augmented with ground anchors to increase stability or decrease ground movements. The feasibility of using an anchored wall at a particular location should be based on the suitability of subsurface soil and rock conditions within the bonded anchored stressing zone. The availability of permanent ROW for the ground anchors must also be considered.

AASHTO-LRFD-BDS Section 11.9 and FHWA GEC No. 4 "Ground Anchors and Anchor Systems" FHWA-IF-03-017 shall be followed. Corrosion protection for anchored wall systems shall be in accordance with AASHTO-LRFD-BDS Section 11.9.7 and GEC No. 4, Chapter 7 Corrosion Consideration in Design" Class I or Class II corrosion protection is required for permanent applications. Anchored Wall drainage systems shall be in accordance with Section 5.6.2.3.

The following ground anchor information shall be provided on the contract documents in addition to the wall information previously discussed:

- Type (i.e. A tremie, B pressure), location, and spacing of ground anchors
- Strand/Bar size, minimum bond length, minimum unbonded length, and proposed inclination, Bar or Strand material
- Anchor loads, test loads, lock off loads and test requirements
- Corrosion protection as required
- Wall facing/ground anchor details

- Grout compressive strength
- Drill hole diameter
- Hollow bars not allowed for permanent applications
- Depth to rock (if applicable)

5.6.2.9 Mechanically Stabilized Earth Walls (MSE)

Mechanically Stabilized Earth (MSE) walls may be considered where conventional gravity, cantilever or counterfort retaining walls and prefabricated modular walls are considered, and particularly when substantial total and differential settlements are anticipated. MSE systems are defined as:

- Walls whose elements may be proprietary, and
- Employ either metallic or geosynthetic tensile reinforcing elements in the soil mass, and
- Include a facing system which is vertical or near vertical.

The design shall address issues relating to backfill quality, reinforcement conflicts with drainage and electrical facilities, vehicular impact resistance, and traffic barrier moment slab construction. The GE shall conduct a cost analysis to verify cost effectiveness of MSE over other wall types. The design shall also allow only permissible MSE systems shown as permissible in the NJTA Specifications. MSE Wall Systems shall not be used in zones of potential scour without permission of the Authority.

Due to past experience with construction of MSE systems in NJTA projects, the Authority prefers alternate wall types under the following circumstances:

- Walls along single lane roadways and ramps where staging is a concern
- Wall supporting large skewed bridges
- Utility and drainage structure crossings where post construction repairs are problematic.

AASHTO-LRFD-BDS and FHWA GEC No. 11 FHWA-NHI-10-024 and 025 shall be supplemented with the following:

A. Wall systems specified in NJTA Specifications shall be used. Other type of MSE wall types shall be pre-approved by the Authority.

B. NJTA Standard and Supplementary Specifications.

C. The GE is responsible for the external stability of MSE walls including bearing resistance, eccentricity, sliding and global stability. Slope stability analysis shall be performed to investigate global stability and compound stability.

   - Global stability analysis shall be conducted such that the failure surfaces are forced outside of the reinforced zone by modeling the reinforced portion of the wall as a rigid body.

- Compound stability analyses shall be performed for MSE walls to investigate potential compound failure surfaces by allowing failure planes to pass behind or under and through a portion of the reinforced soil zone. When compound stability becomes as a concern, the GE shall provide minimum reinforcement requirements in the Contract Plans (Ex: minimum 3 layers of reinforcement in bottom 5 feet). See Section 11.10.4.3 of AASHTO-LRFD-BDS and Section 4.4.10 of Federal Highway GEC No. 11, FHWA-NHI-10-024 for more details.

D. Design guidelines for geometrically complex MSE wall systems such as tiered walls, back to back walls, or walls which have trapezoidal sections shall be in accordance with the procedures in the Federal Highway GEC No. 11, FHWA-NHI-10-024.

E. Drainage details shall follow the guidance in Section 5.6.2.3.

F. The top of the leveling pad shall be situated a minimum 3 feet below the proposed ground surface in front of the wall. Embedment provisions for sloping ground, erosion, scour, or future excavation shall be in accordance with AASHTO-LRFD-BDS Article 11.10.2.2.

G. The tolerance for MSE wall to settlements shall be determined by the wall supplier and the GE. If estimated total and/or differential settlements are greater than the tolerance for the wall or AASHTO-LRFD-BDS guidelines, ground improvement techniques maybe required or a two-stage wall considered. However, technical and cost evaluation of ground improvement techniques in lieu of surcharge or other wall types shall be performed.

H. Guidelines for corrosion/degradation of steel or geosynthetic reinforcements shall be in accordance with FHWA-NHI-10-024 and 025 and shall be supplemented with NJTA Standard and Supplementary Specifications.

The following information shall be included in Contract Documents:

- Nominal Bearing Resistance for Strength Limit State
- Factored Bearing Resistance for Strength Limit State
- Nominal Bearing Resistance for Extreme Event
- Tolerable vertical and lateral deformation criteria
- All drainage requirements, flood elevations.
- Coefficient of sliding friction at the base of the MSE reinforced soil volume.
- Confirmation that external stability of the MSE Wall system has been checked and determined adequate for static and dynamic conditions by the EOR.

- Minimum reinforcement requirements to satisfy compound stability.

The following additional information shall be reported in contract documents which include MSE "wrap-around" abutments supported on deep foundations. These loads are associated with forces on the stub abutment backwall restrained by reinforcing elements attached to the backwall:

- Nominal lateral load from abutment for strength limit state
- Nominal lateral load from abutment for seismic event

Geosynthetically Reinforced walls shall not be used to wrap or support bridge abutments or utilized in retaining walls in excess of 20 feet tall, unless approved by the Authority.

For wrap-around abutments or MSE walls directly adjacent to cast in place abutments, post construction settlement must be less than 1 inch, otherwise ground improvement techniques shall be implemented as described in the Design Manual.

Refer to Section 2 of the Design Manual for additional considerations for MSE type retaining walls.

5.6.2.10 Prefabricated Modular Walls

Prefabricated Modular Wall systems (PMW) may also be considered where conventional gravity, cantilever or counterfort concrete walls are technically feasible. Prefabricated modular wall systems, whose elements may be proprietary, generally employ interlocking soil filled reinforced concrete or steel modules or bins, rock filled baskets, precast concrete units, or dry cast segmental masonry concrete units (without soil reinforcements) which resist lateral earth pressures by acting as gravity retaining walls. AASHTO-LRFD-BDS should be supplemented with the following:

A. Wall systems specified in NJTA Supplemental Specification shall be used. Other type of prefabricated wall types may be pre-approved by the Authority on a case by case basis.

B. Leveling pads and footings shall be placed a minimum of 3 feet below the proposed ground surface.

C. The GE shall check the external stability of the PMW. The GE is responsible for the external stability of PMW walls including bearing resistance, eccentricity, sliding, global stability, and compound stability.
   - Global stability analysis shall be conducted such that the failure surfaces are forced outside of the wall by modeling the wall as a rigid body.
   - Compound stability analyses shall be performed to investigate potential compound failure surfaces by

allowing failure planes to pass behind or under and through a portion of the wall. When compound stability becomes as a concern, the GE shall provide minimum wall dimensions in the Contract Plans (Ex: minimum width of 20’ in bottom 5 feet).

D. The GE shall check the anticipated post construction vertical and lateral deformations and compared to tolerable limits for the wall system and the Project performance requirements. If estimated deformations are greater than the tolerance for the wall or the serviceability requirements, ground improvement techniques maybe required. However, technical and cost evaluation of ground improvement techniques in lieu of surcharge or other wall types shall be performed.

E. In addition to AASHTO-LRFD-BDS Article 11.11.8, Prefabricated Modular Wall drainage systems shall be in accordance with Section 5.6.2.3.

The following information shall be included in Contract Documents:

- Nominal Bearing Resistance for Strength Limit State
- Factored Bearing Resistance for Strength Limit State
- Nominal Bearing Resistance for Extreme Event
- Tolerable vertical and lateral deformation criteria
- All drainage requirements
- Coefficient of sliding friction at the base of the modular wall and the soil volume
- Confirmation that external stability and compound stability of the modular wall system has been checked and determined adequate for static and dynamic conditions by the EOR.

5.6.2.11 Soil Nail Walls

Soil nail walls shall be used in applications where ground anchor walls are considered but where construction access, and subsurface conditions indicate a soil nailed structure may be more cost effective or technically feasible. Soil nail walls may require additional right-of-way. Soil nail walls are constructed to support temporary excavations, reinforce existing soil slopes, and permanent cut walls. Soil nails are typically closely spaced (4 to 6 ft.) and unlike ground anchors are not prestressed. Soil nail walls should be designed and constructed following the recommendations and procedures outlined in FHWA GEC No. 7 “Soil Nail Walls” (FHWA-NHI-14-007). Only solid bars are allowed by the Authority for permanent soil nail applications. Hollow bars are allowed for temporary applications. The AASHTO-LRFD-BDS do not currently (2014) address the design and construction of soil nail retaining walls.

The following information shall be included in Contract Documents:

- Nominal Tensile Resistance for Strength Limit State
- Factored Tensile Resistance for Strength Limit State

- Nominal Tensile Resistance for Extreme Event
- Tolerable vertical and lateral deformation criteria
- All drainage requirements
- Confirmation that external stability of the soil nail wall has been checked and determined adequate for static and dynamic conditions by the GE.
- Soil nail bar size, length and grade
- Grout compressive strength
- Type of soil nail (Type A or Type B)
- Corrosion protection requirements
- Soil nail spacing and inclination (typical section and layout plan)
- Soil nail testing requirements
- NJTA does not have a standard or supplemental specification for soil nails and a supplemental specification need to be prepared by the GE.

5.6.2.12 Temporary Wall Designs

The following guidance applies to all walls types for temporary applications. Temporary walls shall be defined as walls with a design life of three years or less and their design shall be the responsibility of the Contractor. It is common practice to not design temporary walls for seismic limit states; however, seismic design shall be performed if and where directed by the Authority. Contract documents shall explicitly direct the Contractor to refer to this section of the Design Manual where, in addition to all other provisions of this section, the following shall apply:

The Contractor shall submit to the Authority for approval prior to the construction of the wall:

A. The temporary wall design calculations and construction plans signed and sealed by a Professional Engineer licensed in the State of New Jersey with a minimum of 10 years of geotechnical engineering experience.

B. The temporary wall design engineer shall have completed a minimum of 10 projects with temporary and/or permanent wall systems, shoring, and cofferdams.

C. Design for temporary works shall be performed in allowable stress design in accordance with AASHTO Guide Design Specifications for Bridge Temporary Works and AASHTO Construction Handbook for Bridge Temporary Works.

5.6.3 BURIED STRUCTURES

The design of all buried structures shall follow the AASHTO-LRFD-BDS, Section 12. Buried structures include:

- Metal pipe
- Structural plate pipe

- Long-span structural pipe
- Structural plate box
- Reinforced concrete pipe
- Reinforced concrete cast-in-place and precast arch
- Box and elliptical structures
- Thermoplastic pipe
- Fiberglass pipe
- Buried structures may include, but are not limited to, culverts or tunnels.

5.6.3.1 Soil and Rock Properties

Subsurface exploration shall be carried out to determine the presence and influence of geologic conditions that may affect the performance of buried structures. For buried structures supported on footings and for pipe arches and large diameter pipes, a foundation exploration should be conducted to evaluate the capacity of foundation materials to resist the applied loads and to satisfy movement requirements of the structure. Soil and rock properties shall be obtained from the in-situ tests, laboratory tests, and published correlations developed for similar type of materials. Judgment shall be applied based on the relative importance and reliability of the methods. AASHTO-LRFD-BDS Section 10.4, FHWA GEC No. 5, and FHWA Soils and Foundation Reference Manual shall be utilized.

5.6.3.2 Limit States and Resistance Factors

Buried structures and their foundations shall be designed by appropriate methods specified in the AASHTO-LRFD-BDS, Sections 12.7 through 12.15 to resist the load combinations specified in Sections 12.5. The factored resistance shall be calculated for each applicable limit state.

Buried structures shall be investigated at Service Load Combination I, as specified in AASHTO-LRFD-BDS Section 12.5.2.

Buried structures and tunnel liners shall be investigated for construction loads and at Strength Load Combinations I and II, as specified in AASHTO-LRFD-BDS Section 12.5.3.

Resistance factors for buried Structures shall be taken from AASHTO-LRFD-BDS Section 12.5.5.

5.6.3.3 Design Considerations

The following variables shall be considered during the design:

- Strength and compressibility of foundation materials;
- Chemical characteristics of soil and surface waters, e.g. pH, resistivity, sulfate and chloride content of soil and pH, resistivity, and sulfate and chloride content of surface water;
- Stream hydrology, e.g. flow rate and velocity, maximum width, allowable headwater depth, and scour potential; and

- Performance and condition survey of culverts in the vicinity.

The following information shall be included in Contract Documents:
- Nominal Bearing Resistance for Strength Limit State
- Factored Bearing Resistance for Strength Limit State
- Nominal Bearing Resistance for Extreme Event
- Tolerable vertical and lateral deformation criteria

5.6.4 SOUND BARRIERS

Sound Barriers shall be designed in accordance with AASHTO-LRFD-BDS Section 15 and shall be supplemented with the following.

Sound barriers shall be designed to satisfy the following loads and performance requirements:
- Lateral Wind Load
- Earth Load
- Vehicle Collision Load
- Extreme (Seismic) Load
- Lateral deformation under the service limit state factored lateral load and moment at the top of the sound barrier shall be 1.5 inch or less.

Spread Footings shall follow Section 5.6.1.4
Driven Pile Foundations shall follow Section 5.6.1.5
Drilled Shafts shall follow Section 5.6.1.6 (3.) a. through d.

5.7 GEOTECHNICAL ENGINEERING ANALYSIS AND DESIGN – GENERAL DESIGN TOPICS AND TOPICS NOT EXPLICITLY ADDRESSED IN AASHTO-LRFD-BDS OR FHWA GUIDANCE

5.7.1 SCOUR

The following should be considered in addition to the guidance provided in the AASHTO-LRFD-BDS Sections 2.6.4.4.2 and 3.7.5:

A. Scour analysis and scour countermeasures shall be in accordance with AASHTO-LRFD-BDS 2.6.4.4, Hydraulic Engineering Circulars HEC 18 (FHWA-HIF-12-003), HEC 20 (FHWA-HIF-12-004) and HEC 23 (FHWA-NHI-09-111 & 112). Scour in cohesive soils are specified in FHWA-HRT-15-033 and scour in erodible rocks are specified in NCHRP Report 717. Changes in the bed level of a stream affect highway structures and may be described by three types of actions: (1) general scour (contraction scour), (2) local scour and (3) degradation or aggradation of the stream channel. Scour and degradation are discussed in this section. Scour evaluations of new and existing bridges should be evaluated by an interdisciplinary team of hydraulic, geotechnical and structural engineers. Hydraulic studies shall include estimates of scour at bridge piers and evaluation of abutment stability. Bridge foundations shall be designed to withstand the effects of scour for the worst conditions resulting from flood events specified in HEC 18.

B. In general, foundations shall be designed to be stable without relying on scour countermeasures. The only exception to this is when designing for local scour at abutments. Because the local scour equations tend to overestimate the magnitude of scour at abutments, they are generally used only to gain insight into the scour potential at an abutment. In most cases, a scour countermeasure, properly designed and installed in accordance with the procedures outlined in HEC 23, is provided to resist the local scour at abutments. Both the abutment foundation and the scour countermeasure must be designed to be stable after the effects of the estimated long-term degradation and contraction scour. Ensure that the top of the footing is below the sum of the long-term degradation, contraction scour, and lateral migration; stub abutments are an exception to this requirement, but the slopes in front of them should be adequately protected and/or sheeting should be provided to prevent undermining of the abutment and loss of fill. Riprap shall be designed and used to protect abutments from erosion for maintenance purposes, even if it is not required to resist the effects of local scour.

5.7.2 SIGN STRUCTURES, LUMINARIES, TOLL GANTRIES AND TOWERS

The GE shall refer to Section 2.5 (Sign Supports) of the Design Manual and the NJTA standard sign structure foundation drawings (SI-22 to SI-22B for Turnpike structures, SI-39 to SI-41 for Parkway structures, VM-08 & 09 for VMS structures) and the details shown in the CM, E, ITS, SI, and VM drawings. The EOR shall reference AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaries, and Traffic Signal, current edition and in accordance with Section 2 of the NJTA Standards.

A. Spread Footings: Procedures shall follow AASHTO-LRFD-BDS Section 10.6.

1. Bottom of footing shall be placed below the frost depth
2. Footing thickness shall be a minimum 2 feet

B. Driven Pile Foundations: Procedures shall follow AASHTO-LRFD-BDS Section 10.7 and be supplemented as follows:

1. Pile head deflections shall be limited to 0.75 inches unless a deflection criteria provided by the Structural Engineer.
2. Minimum pile tip elevation, estimated pile tip elevation, ultimate axial compression capacity, and allowable axial compression capacity shall be provided in Contract Documents.

C. Drilled Shafts: Procedures shall follow AASHTO-LRFD-BDS Section 10.8 and be supplemented as follows:

1. Minimum size of drilled shaft shall be 30 inches
2. Minimum center-to-center spacing shall be 2.5 diameter of drilled shaft

3. Deflection at top of shaft shall be limited to 0.75 inches unless a deflection criteria provided by the Structural Engineer.
4. Short pile behavior shall be avoided. Deflection at tip of the shaft shall be limited to 0.1 inch. See Exhibit 5-11.
5. Overhead sign structures supported on two drilled shafts per each side shall be designed to satisfy the following factored loads:
   a. Axial compression load
   b. Uplift load
   c. Lateral wind load
6. Overhead sign structures and toll gantries supported on one drilled shaft per each side or butterfly sign structure shall be designed to satisfy the following factored loads:
   a. Axial compression load
   b. Lateral wind load
7. Cantilever sign structures supported on one drilled shaft shall be designed to satisfy the following factored loads:
   a. Axial compression load
   b. Lateral wind load
   c. Torsional moment
8. The following minimum resistance factors shall be achieved by design:
   a. For axial compression load - AASHTO-LRFD-BDS Table 10.5.5.2.3-1
   b. For axial uplift load – AASHTO-LRFD-BDS Table 10.5.5.2.3-1
   c. 0.65 for torsional moment

5.7.3 SOIL AND ROCK SLOPES

This section addresses geotechnical design issues associated with cuts and fill in soil and rock.

5.7.3.1 Soil and Rock Properties

Soil and rock properties shall be obtained from the in-situ tests, laboratory tests, back analysis and published correlations developed for similar type of materials. Judgment shall be applied based on the relative importance and reliability of the methods. AASHTO-LRFD-BDS Section 10.4, FHWA GEC No. 5, and FHWA Soils and Foundation Reference Manual shall be utilized.

5.7.3.2 Slope Stability

Stability of embankments and soil and rock cut slopes shall be evaluated using computer program in accordance with Section 5.5. Design procedures should follow FHWA-NHI-01-026 “Soil Slope and Embankment Design” and FHWA-TS-89-045 “Rock Slopes: Design, Excavation, Stabilization”, and FHWA’s Rockfall Catchment Area Design Guide.

The design analyses shall consider both short and long-term stability (drained and undrained) and shall address translational, rotational and irregular shape failure modes of surficial and deep seated failure surfaces. Slope stabilization measures to address stability shall consider long-term performance, constructability and initial and long-term cost. A performance requirement of a minimum safety factor of 1.3 shall be provided for all new slopes and rehabilitated soil and rock slopes.

If potential problems of stability or excessive settlement exist for a proposed embankment then methods to overcome these problems shall be investigated, evaluated and addressed.

Rock slope stability evaluation shall require geologic mapping of dip, dip direction, and condition of discontinuities. Subsequently, stereographic analysis shall be performed to identify failure mechanisms kinematically feasible. If any failure mechanisms (planar sliding, wedge, or toppling) are kinematically feasible, kinematic analysis of those failure mechanisms shall be performed to determine the factor of safety. If the factor of safety is less than tolerable, reinforcement such as, but not limited to untensioned fully grouted dowels, tensioned rock bolts, or anchored mesh systems shall be evaluated, which will result in an acceptable factor of safety.

In addition, when required, rockfall simulation modeling shall be performed for catchment ditch and rockfall barrier design. Catchment design shall be in accordance with the FHWA's Rockfall Catchment Area Design Guide, using 95 percent rockfall retention and with 4H:1V catchment area slope, if possible, or otherwise as approved by the Authority.

All rock fall hazard mitigation or rock slope stabilization projects shall include rock drains. All untensioned fully grouted dowels or tensioned rock bolts shall be galvanized or epoxy coated, with a minimum service life of 75 years. All mesh products shall be PVC coated with a minimum service life of 75 years. All Shotcrete shall be fiber reinforced. Cable lashing will not be accepted by the Authority for stabilizing rock blocks. A specification for rock fall hazard mitigation and rock slope stabilization construction methods is not available in the NJTA Standard Specifications and NJTA Supplemental Specification, and would need to be developed by the GE on a project specific basis.

5.7.3.3 Vertical and Lateral Deformations

Vertical and lateral deformations of soil below engineered earthwork features (e.g. embankments) shall be estimated using the methods specified in AASHTO-LRFD-BDS Section 10.4, FHWA GEC No. 5, and FHWA Soils and Foundation Reference Manual. Laboratory test results, in-situ test results, and published correlations shall be utilized in determining accurate soil parameters. Federal Highway GEC No. 5 FHWA-IF-02-034 shall be followed.

A. Deformation estimates should include:

1. Immediate or elastic vertical and lateral deformation
2. Consolidation vertical and lateral deformations of compressible soils
3. Secondary consolidation settlement of soft cohesive soils and organic soils.

B. Differential deformations between adjacent transportation features shall be within tolerable limits.

C. If deformations are excessive, ground improvement techniques shall be considered. In evaluating alternative ground improvement treatments, consideration should be given to relative stability, expected post construction settlement and treatment cost.

5.7.3.4 Rehabilitation of Existing Slopes

The assessment and selection of the stabilization methods shall be predicated upon minimizing risk, minimizing cost and schedule, working within site constraints, and material availability. The GE shall submit a list of considered stabilization methods to the Authority with associated advantages and disadvantages and preliminary unit cost estimates prior to design in the Phase B submission.

A. Existing soil and rock slopes may deteriorate, and become unstable. These slope conditions may require an exploration to determine the cause of distress and assess stabilization treatments for rehabilitation. A preliminary field inspection will be authorized by the Authority and shall include:

   A visual assessment of:
   1. Soil slope movements at the slope toe, slope crest, slope face, adjacent roadways, railroads, structures, embankments, and utilities including scarps, bulges and aerial subsidence.
   2. Rock slope movements at the slope toe, crest, and face, all lithologies, structural discontinuities, wedges, blocks and areas of instability. Measure and record typical dip, dip direction, and condition of discontinuities. Assess the presence of potential triggering mechanism such as seepage, root heave, ice buildup, etc.
   3. Photo documentation of all observations made. All areas with observed slope movement shall be mapped on an oblique photomosaic.
   4. Planning of a subsurface exploration to further assess the existing condition and potential stabilization treatment.
   5. Determine whether a simple, solution exists or a comprehensive investigation is required.

B. Slope Instability Exploration: If deemed necessary given the observed conditions and mechanisms which exist, the GE shall propose a right sized exploration and monitoring program to be submitted to the Authority for authorization. The slope instability exploration may consist of some of the components noted below:

1. Soil Slopes: A geotechnical exploration including soil borings with SPT's, CPT's, installation of monitoring wells or piezometers, and laboratory and in-situ testing of soil shall be performed to an elevation below the anticipated failure surface. Additionally, inclinometers shall be installed to monitor ongoing movements of the slope during and after the exploration and stabilization. The exploration objective is to determine residual soil strength values as well as potentially locate the failure surface. Using the information obtained from the geotechnical exploration, the GE shall perform slope stability analysis to determine acceptable slope geometries, soil and groundwater conditions, and loading conditions to determine an appropriate stabilization method. Appropriate stabilization methods include but are not limited to:

   a. Reducing the driving force of the slope by partial removal and replacement with lightweight fill material.
   b. Improvement of internal and external drainage of the slope.
   c. Implementation of a ground improvement program to compact or reinforce the foundation soils.
   d. Installation of a toe cut-off wall to intercept the failure surface and eliminate continuing movement.
   e. Installation of a ground anchor system or other restraining system through the slope face.

2. Rock Slopes: A geotechnical exploration including geologic mapping for joints, bedding, and other weak planes in the slope, soil borings with rock cores, installation of monitoring wells, and laboratory and in-situ testing or rock shall be performed to an elevation below the anticipated failure surface. An important issue is the alignment of weak planes (dip and dip direction) with respect to the slope. Because laboratory testing of the strength of these weak planes is difficult, an experienced geologic engineer shall be employed. The exploration objective is to determine in-situ rock strength values, determine groundwater conditions as well as to locate the failure surface. Using the information obtained from the geotechnical exploration, the GE shall perform stability analysis to model slope geometries, rock groundwater conditions, and loading conditions and to determine an appropriate stabilization method.

5.7.4 PAVEMENT DESIGN

The GE shall verify subgrade suitability and provide adequate subgrade stabilization methods to improve the subgrade for soils exhibiting inadequate strength as specified in Section 5.4.5.

5.7.4.1 General Guidelines

The following general guidelines shall be applicable to Turnpike and Parkway pavements:

A. NJTA shall participate in any decision to diverge from published NJTA pavement standards.

B. When computing quantities for asphaltic concrete items, the following conversion factors are to be used for preliminary estimates and are to be verified for each project prior to completion of the final quantities:

Surface Course	156.0 ± pcf
Intermediate Course	157.5 ± pcf
Base Course	159.0 ± pcf

Conversion factors are to be verified for each project prior to completion of final quantities.

C. Tack coat shall be applied to all existing (milled) pavement surfaces just prior to asphalt resurfacing. Tack coat shall also be applied to all exposed cut surfaces of an existing asphalt pavement section which is stepped to interface with a proposed pavement section. Tack coat will not be required between subsequent asphalt layers of proposed pavement unless:
   1. The underlying layer has been contaminated.
   2. The underlying layer has been exposed to prolonged traffic use.
   3. It is otherwise required on the drawings or in special provisions.

D. In locations where existing pavement is widened, Grade A material is to be deeper, if necessary, to match template grade of existing pavement. Template grade (top of subgrade below Grade A embankment) shall slope transversely a minimum of 2% or match cross slope of roadway. Template grade shall be constructed transversely under the full section, without breaks in cross slope, on each individual roadway and in such a manner as to provide positive drainage (daylight section or underdrains).

E. At interfaces between Turnpike or Parkway pavement and the pavement of outside agencies, the higher-class pavement shall be constructed first, with offset and steps per course as shown. Account for offset and stepping quantity computations.

F. Hot mixed asphalt pavements shall be constructed in accordance with the Standard Specifications, as amended by the Supplemental Specifications. Surface and Intermediate courses shall each be placed in a single lift. The base course for Turnpike Pavement shall be placed in two lifts. Pavement course lifts shall conform to the following:
   1. The minimum lift thickness shall be three times the nominal maximum aggregate size of the specified pavement mix type.
   2. The maximum lift thickness shall be five times the nominal maximum aggregate size of the specified pavement mix type.

   The above lift requirements shall apply to U-Turn, Z-Turn, and Car Parking pavement sections, as well as any variations of Turnpike Pavement used in resurfacing / re-grading projects.

5.7.4.2 Turnpike Pavements

The following pavement sections for Turnpike mainlines, ramps, U-Turns, Z-Turns, and Parking Lots and guidelines shall be followed:

A. The mainline pavement section shall be constructed as shown on Exhibit 5-12. Current pavement mix types to be used for each of the courses shown in the pavement sections shall be as directed by the Authority.

B. The pavement section for Z-Turns shall be Turnpike Pavement as shown on Exhibit 5-12.

C. For parking lots and driveways at toll plaza buildings and other locations within the Turnpike right of way, the pavement section shall be as shown on Exhibit 5-13.

D. The pavement section for grade separated U-Turns shall be as shown on Exhibit 5-14.

E. The various pavement interface and stepping details shown on Exhibit 5-15 through Exhibit 5-19 are for Turnpike pavement. Adjust steps accordingly to match other pavement sections. Account for stepping quantity computations. With curb, courses terminate at curb face as shown, any stepping shall be from back of curb.

F. In the pavement interface details shown on Exhibit 5-18, the existing pavement is from the 1985-90 widening construction. Each area shall be reviewed and adjusted to conform with existing construction. Where proposed widening includes resurfacing the adjacent existing pavement, omit the 6-inch removal of the top course and place the new surface course pavement joint at least 2 feet from the existing edge of pavement.

G. Embankment, Grade A, shall be a minimum of 18 inches deep under travel lanes.

H. In areas where existing and currently designed resurfacing depth approaches 12 inches, or more, at the existing pavement / shoulder interface, investigations shall be made as to the feasibility of leaving the existing shoulder in place as a portion of the proposed pavement section.

5.7.4.3 Parkway Pavements

The following pavement sections for Parkway mainlines, ramps, U-Turns, Z-Turns, and Parking Lots and guidelines shall be followed:

A. The mainline pavement section shall be constructed as shown on Exhibit 5-20. Current pavement mix types to be used for each of the courses shown in the pavement sections shall be as directed by the Authority.

B. The pavement section for Z-Turns shall be Turnpike Pavement as shown on Exhibit 5-20.

C. For parking lots and driveways at toll plaza buildings and other locations within the Turnpike right of way, the pavement section shall be as shown on Exhibit 5-21.

D. The pavement section for grade separated U-Turns shall be as shown on Exhibit 5-20.

E. The various pavement interface and stepping details shown on Exhibit 5-22 through Exhibit 5-25 are for Parkway pavement. Adjust steps accordingly to match other pavement sections.

F. Account for stepping (see Exhibit 5-22) in quantity calculations. With curb, courses terminate at the curb face. Any stepping shall be from back of curb.

G. When required, Embankment, Grade A, to be a minimum of 8 inches. Inclusion of Embankment Grade A to be determined on a contract-by-contract basis following an existing substrata investigation.

H. In areas where existing and currently designed resurfacing depth approaches 9 inches, or more, at the existing pavement / shoulder interface, investigations shall be made as to the feasibility of leaving the existing shoulder in place as a portion of the proposed pavement section.

5.7.4.4 Other Pavements

Where local roads are being replaced, the intent of the Authority with respect to any work under the jurisdiction of the state, county, municipality, or any other agency is "replacement in kind", according to present standards of that agency. All such work is subject to the approval of the Authority and must be previously agreed to in writing by the concerned agency, as noted elsewhere in this manual and the Procedures Manual.

Similarly, all detouring and / or closing of local roads during construction must be approved by the appropriate agencies in accordance with the Procedures Manual.

Applications where pavement design may be required include Authority Projects which include pavement of other agencies roadways, follow those agency standards.

5.7.5 GROUND IMPROVEMENT METHODS

Significant sections of the existing Authority roadways are underlain by soft, weak, compressible soils which include peats, organic silty clays and varved clays. These areas have required special foundation treatment to maintain a stable embankment and minimize roadway settlements. In some areas the Authority's roadways are in cuts that extend into clayey soils which have required underdrains and/or undercutting to maintain stability and a smooth pavement surface. These potential problems should be investigated and evaluated as part of the preliminary exploration of embankment foundation and cut areas.

Ground improvement techniques shall be considered to strengthen loose granular soils or compressible organic and inorganic silts and clays, to provide adequate foundation or embankment support or to reduce deformations and accelerate the time rate of consolidation. Guidance for analysis and design of ground improvement techniques shall be found in FHWA-NHI-06-019 and 020, "Ground Improvement Methods Reference Manual", ASCE Geotechnical Special Publications No. 104, 112, 119, 120, 124, 136, 168, 172, 187, 188, 207, 228, 238, and SHARP 2 Geotech Tools Website, however many ground improvement methods are performance based and/or proprietary and may require design input by others. Following methods shall be considered:

A. Remove and Replace
   Removing the unsuitable materials and backfilling with regular weight or lightweight backfill material shall be considered with the following conditions:
   - Removal and disposal of the unsuitable materials
   - Regulated waste materials
   - Support of excavation

- Groundwater

Lightweight materials such as granular lightweight fill, fly ash, expanded shale, geofoam, controlled low strength material, or foamed concrete shall be utilized to reduce the excess pressure from standard fills. The following issues shall be considered:

- Buoyant weight of lightweight material and stability under flooding conditions
- Drainage
- Availability of the materials
- Durability
- Environmental concerns
- Durability with existing ground conditions

B. Preload and surcharge with or without prefabricated vertical drains

Preloading and surcharging until the completion of settlement is a suitable option for loose sands and soft clays. The following issues shall be considered:

- Effect of preloading on existing structures, roadways embankments, and utilities
- Staged construction
- Time taken for settlement to complete

Prefabricated vertical drains may help accelerate the time rate of settlement in fully saturated low permeability soft clays. The GEP should identify the suitable subsurface exploration methods for proper assessment. Site constrains regarding the following should be examined

- Steep slopes
- Contamination migration
- Installation through obstructions, dense to very dense sand or gravelly materials, and stiff to hard clays
- Overhead or subsurface utility interference
- Collection and disposal of contaminated water
- Sand blankets

C. Stone columns or continuous modulus columns

Stone columns are generally constructed by downhole vibratory methods known as vibro-replacement or vibro-displacement techniques and are best suited for improving clays, silts, and loose sands. In addition, vibro-concrete columns (VCC) and geotextile encased columns (GEC) are some other types of applications which may be considered. A feasibility study of stabilizing soft ground with stone columns shall be conducted addressing the following concerns:

- Lateral support from subsurface materials to prevent large deflections and bulging
- Presence of dense overburden, boulders, cobbles, and obstructions which may require predrilling
- Limitations on using wet methods
- Environmental constrains

- Effect of vibrations on existing structures and utilities
- Interference with utilities
- Overhead clearance
- Load transfer platform

D. Vibro-compaction

Vibro-compaction is a technique suitable for densifying granular cohesionless soils using a vibrator. The following concerns shall be addressed:

- Effect of vibrations on existing structures and utilities
- Vibration induced liquefaction and effects
- Interference with utilities
- Overhead clearance
- Obstructions

E. Dynamic Compaction

Dynamic compaction technique is suitable for densifying loose granular soils by dropping large weights. Consideration shall be given to the following:

- Due to the significant vibration generated by the impact of the weight, this method is not recommended within a minimum of 100 feet of existing structures and utilities
- Locations where high groundwater exists
- Overhead clearance
- Ground vibration

F. Soil Mixing

The following shall be considered for deep mixing of soil with lime or cement grout to improve or stabilize the ground:

- Time taken for stabilization
- Presence of dense overburden, boulders, cobbles, and obstructions which requires predrilling
- Overhead clearance
- Disposal of spoils
- Chemical and mineralogical properties of the soil and reaction with stabilizing agent
- Artesian conditions or high hydraulic gradient

G. Grouting

Compaction grouting, permeation grouting, chemical grouting, soil fracture grouting, jet grouting, bulk void filling, and mud/slab jacking techniques shall be utilized based on subsurface conditions, and subsurface material characteristics. Consideration shall be given to the following

- Ground settlement or heave and lateral ground movement
- Effect on existing structures and utilities
- Temporary support system or underpinning to prevent structural damages
- Groundwater, artesian conditions or high hydraulic gradient

- Subsurface material characteristics

H. Column Supported Embankments
Geosynthetically reinforced column supported embankments or load transfer platforms generally consist of vertical columns of stone or deep foundation elements and shall be utilized with the following consideration:
- Lateral stability
- Utility relocation
- Environmental constrains
- Effect of vibrations on existing structures and utilities

The selection of technically feasible and cost effective ground improvement methods will vary based on project requirements and schedule, subsurface conditions including subsurface improvement depth and material and the treatment area size. Subsequent to certain performance based ground improvement the GE may order the execution of either soil borings with SPT's, CPT's, PMT's, DMT's, VST's, other field measures, or laboratory tests to verify the improvement in the subsurface materials. The proposed subsurface exploration program, suitable ground improvement methods, assumptions, limitations, and a cost comparison shall be included in the GEP and the Geotechnical Engineering Report. Project specific performance based special provisions shall be developed which address specific acceptance criteria and the basis of ground improvement acceptance.

The following information shall be included in Contract Documents:
- Project specific performance based special provisions
- Ground improvement plans and details
- Instrumentation locations and details
- Monitoring of vibrations, settlements, and cracks on existing structures and utilities

5.7.6 BUILDINGS

Foundations for toll plaza buildings, NJSP facilities, maintenance facilities, and service areas shall be designed in accordance with current International Building Code (IBC) and International Building Code - New Jersey Edition (NJBC) and supplemented by AASHTO Standard Specifications for Highway Bridges.

5.7.6.1 Spread Footings

A. Requirements specified in Section 5.6.1.4 shall be followed.
B. The following information shall be included in the Contract Documents:
   - Ultimate bearing capacity
   - Allowable bearing capacity for static conditions
   - Allowable bearing capacity for seismic conditions
   - Other parameters specified in Section 5.6.1.4 (H).

5.7.6.2 Deep Foundations

A. Requirements specified in Section 5.6.1.5 shall be followed for driven piles.
B. Requirements specified in Section 5.6.1.6 shall be followed for drilled shafts.
C. Requirements specified in Section 5.6.1.7 shall be followed for micropiles.
D. The following information shall be included in the Contract Documents:
   - Ultimate Capacity in Compression
   - Allowable Capacity in Compression
   - Ultimate Driving Capacity for driven piles
   - Ultimate Uplift Capacity
   - Allowable Uplift Capacity
   - Maximum Driving Capacity for driven piles
   - Other parameters specified in Section 5.6.1.5 (O) for driven piles.
   - Other parameters specified in Section 5.6.1.6 (R) for drilled shafts.
   - Other parameters specified in Section 5.6.1.7 (O) for micropiles.

5.7.7 REUSE OF FOUNDATIONS

Important technical issues must be addressed to ensure that foundation reuse is undertaken appropriately. Foundation reuse is not explicitly addressed in current foundation design standards and may not comply with current standards, particularly for materials and construction quality control. The amount of investigation of a reused foundation system may need to be balanced against perceived risks as well as the amount and veracity of information. The design of foundations that are reused or incorporate reused elements may require explicit assessment to address the uncertainty inherent in current understanding of foundation behavior.

A. As a minimum, the following Existing Foundation Information shall be obtained from contract plans, as-built drawings, and reports:
   1. Bottom of footing elevation
   2. Foundation type
   3. Foundation bearing materials
   4. Foundation loads
   5. As-built tip of deep foundation elevation
   6. Design standards used
   7. Material strength (concrete, reinforcements, shells, timber, etc.)

B. Investigations for existing foundations shall be prepared including the following:
   1. Review existing foundation information
   2. Review existing subsurface explorations

3. Planning additional subsurface explorations to determine subsurface conditions, subsurface material properties, and unknown foundation depth. See Section 5.4.
4. Planning investigations on existing foundations to determine foundation integrity, deterioration of foundation elements, chemical and physical attacks on foundation elements, and service life remaining on foundation elements.

C. Risk Management
1. A Risk Management Plan shall be prepared describing how the risk will be managed on the project.
2. A Risk Register listing the risks that may affect the project and the actions that are proposed to deal with the risks shall be prepared.
3. Qualitative or semi-quantitative risk assessment by assessing the importance of risks without quantification shall be prepared during early stages of the project.
4. Quantitative risk assessment, quantifying the risks and assessing the possible range of outcomes for the project in terms of cost and time shall be prepared at the beginning of the design. A sample risk assessment worksheet is shown as Exhibit 5-26.

D. Foundation design shall be performed to confirm load bearing and deformation criteria are met.
1. Structural foundations shall follow Section 5.6.1.
2. Walls and Abutments shall follow Section 5.6.2.
3. Buried Structures shall follow Section 5.6.3.
4. Sound Barriers shall follow Section 5.6.4.
5. Sign Structures, Luminaries and Toll Gantries shall follow Section 5.7.2.
6. Buildings shall follow Section 5.7.6.

E. Cost analyses shall be performed for partially or complete reuse option and new construction option including the following:
1. Direct costs (investigation cost, material cost, foundation costs)
2. Risk costs (additional foundation cost, delay of construction due to unknown conditions)
3. Benefits (early accomplishments, future redevelopment)
4. Maintenance cost (annual inspection, annual repairs)
5. A Life Cycle Cost Analysis shall be performed.

F. Reuse options for foundations shall be submitted with Phase A and B submissions.
1. Desk Study during Phase A Submission shall include the following:
   - Existing Foundation Information
   - Existing Subsurface Exploration details
   - Proposed Subsurface Exploration details
   - Proposed Investigation for Foundations
   - Qualitative or semi-quantitative risk assessment
2. Phase B Geotechnical Engineering Report shall include the following in addition to the Phase A submission:

- Risk Management
- Foundation Design
- Cost Analysis
- Life Cycle Cost Analysis
- Recommendations

5.7.8 FENDERS

Bridge structures in navigable waterways shall be protected from vessel collision. Bridge structures shall either be designed to withstand collision force or shall be protected against vessel collision forces by fenders, dikes, dolphins, berms, islands, or other sacrifice-able devices. AASHTO-LRFD-BDS and AASHTO Guide Specifications and Commentary for Vessel Collision Design of Highway Bridges shall be followed.

5.7.8.1 Design of Bridge Substructure Foundations for Collision Force

The foundations shall be designed to withstand the impact loads in an elastic manner. Inelastic design shall not be permitted for foundations to prevent collapse. The design shall be in accordance with AASHTO-LRFD-BDS with the foundation design specified in Section 5.5.

5.7.8.2 Design of Protective Systems

The foundations shall be designed to withstand the impact loads in an elastic or inelastic manner and to provide a protective system to reduce the magnitude of the impact loads to less than the strength of the bridge substructure or superstructure components or to independently protect those components. The design shall be in accordance with AASHTO Guide Specification or Vessel Collision with the foundation design specified in Section 5.5. The following shall be considered:

A. Effect on scour and collection of debris shall be taken into consideration in the design.
B. Section loss due to material degradation such as corrosion, oxidization & reduction due to wet & dry shall be considered.
C. Foundation stiffness shall be developed for deep foundation elements.
D. The design information specified in Section 5.5 shall be included in the Phase B Geotechnical Engineering Report and in Contract Documents.

5.8 SEISMIC ANALYSIS AND DESIGN

5.8.1 BRIDGE CLASSIFICATIONS

Seismic ground shaking hazard information, bridge importance classification, and guidelines to select suitable design events are provided in Section 2.2.6.3.

5.8.2 SEISMIC EFFECTS

5.8.2.1 Seismic Site Class

A site shall be classified, based on the stiffness of the subsurface material, as A though F in accordance with the Site Class definitions in Table 3.10.3.1-1 of AASHTO-LRFD-BDS and AASHTO Guide Specification for LRFD Seismic Bridge Design. Site classification shall be determined using shear wave velocity, SPT blow counts, and/or undrained shear strength for the surficial 100 feet of subsurface. The methods specified based on SPT blow count or based on correlated shear strength values may not be representative for sites with zero or very low SPT blow counts especially in very soft clayey/organic soils and very loose sands. A Project site may be subdivided into different site classes depending on site variation.

5.8.2.2 Acceleration Coefficients

Peak Ground Acceleration coefficient on rock (PGA), horizontal response spectral acceleration coefficient at 0.2-sec period on rock (S_S), and horizontal response spectral acceleration coefficient at 1.0 sec period on rock (S_1) are provided in Section 3.10 of AASHTO-LRFD-BDS for 1,000 Year return period event (7% probability in 75 years). The acceleration coefficients for 2,500 Year return period events can be obtained from USGS website: http://earthquake.usgs.gov/hazards/.

The following information shall be included in Contract Documents:

- Seismic Site Class
- Peak Ground Acceleration Coefficient on rock (PGA)
- Short period horizontal response spectral acceleration coefficient at 0.2-sec period on rock (S_S)
- Long period horizontal response spectral acceleration coefficient at 1.0-sec period on rock (S_1)

5.8.3 DESIGN RESPONSE SPECTRUM

5.8.3.1 Standard Response Spectrum

- Site factors for zero-period, short period, and long period shall be obtained from Section 3.10.3 of AASHTO-LRFD-BDS.
- Horizontal response spectrum shall be developed in accordance with Section 3.10.4 of AASHTO-LRFD-BDS.

5.8.3.2 Site Specific Response Spectrum

Site specific response spectrum shall be developed for bridges as to satisfy the criteria specified in Section 2.2.6.3. The site specific seismic study includes development of dynamic soil properties, selection of target spectrum, and selection of bedrock acceleration time histories.

- The bedrock response spectrum or the target spectrum shall be obtained from USGS website (http://geohazards.usgs.gov/hazardtool/application.php).
- Processed bedrock acceleration time histories can be obtained from the following strong motion data websites:
  - Center for Engineering Strong Motion Data (CESMD) – http://www.strongmotioncenter.org/
  - California Strong Motion Instrumentation Program (CSMIP) – http://www.conservation.ca.gov/cgs/smip
  - Pacific Earthquake Engineering Research (PEER) Ground Motion Database – http://ngawest2.berkeley.edu/
  - Consortium of Organizations for Strong-Motion Observation Systems (COSMOS) – http://www.cosmos-eq.org/
- The ground motion data should represent the project site in terms of;
  - Similar geological conditions
  - Earthquake magnitude
  - Distance between the nearest active fault and the project site should be similar to the distance between the epicenter of the earthquake and instrumentation location
- The acceleration time histories shall be adjusted to match the target spectrum using computer programs (RSPMATCH, RASCAL).
- Shear and compressional wave velocity profiles shall be developed using field test methods such as crosshole seismic testing, downhole seismic testing, suspension logging testing, seismic cone testing, etc.
- Shear modulus reduction curves and damping reduction curves shall be developed from laboratory tests or from available models.
- Soil and rock material properties such as initial shear modulus, unit weight, gradation, plasticity index, and relative density are important parameters in obtaining an accurate and appropriate site specific response spectrum. Selection of these material properties can be in accordance with FHWA GEC No. 3. In order to accommodate the variation in material properties, sensitivity studies should be conducted. A range of material parameters an appropriate range based upon the standard deviation above and below the average value should be assessed.
- A one dimensional or two dimensional shake analysis shall be performed using a commercial software in accordance with Section 5.5 and the response spectrum shall not be lower than 2/3 of the standard spectrum specified in Section 5.8.3.1.

5.8.3.3 Vertical Response Spectrum

Vertical response spectrum shall be taken as 2/3 of the horizontal design spectrum.

5.8.4 FOUNDATION DESIGN

- Shallow foundations shall not be permitted on liquefiable soils unless ground improvement to prevent liquefaction is proposed.
- Lateral support from liquefiable zone shall be ignored for deep foundations.
- Downdrag due to earthquake induced ground movement shall be considered.
- Horizontal loads due to lateral spread and lateral flow shall be included.
- Soil-foundation-structure interaction should be evaluated.
- Foundations shall be evaluated for seismic loads (Extreme Event).

5.8.4.1 Soil-Foundation-Structure Interaction

Chapters 8, 9, and 10 of FHWA GEC No. 3 and MCEER Seismic Retrofitting Manual for Highway Structures (MCEER-08-SP-02) shall be followed. Variation in subsurface material properties, foundation element properties, casings left in place, pre and post corrosion effects and foundation cap properties shall be considered in developing foundation stiffness. The upper and lower bound stiffness values shall be provided.

E. The upper bound stiffness shall be obtained by modeling the foundations with upper bound subsurface material properties, upper bound concrete and steel properties, ignoring section losses, and including interim casings and permanent casings.

F. Lower bound stiffness values shall be obtained by modeling the foundations with lower bound subsurface material properties, lower bound concrete and steel properties, including section losses, only including the casings which is considered as structural elements after the section loss reduction, and incorporating subsurface material strength reduction for granular soils susceptible to liquefaction and cohesive soils susceptible to cyclic softening.

Foundation stiffness values (lateral, vertical, rotational, & torsional, a six by six matrix) for dynamic loading shall be developed using the following methods:

- Shallow foundations using Chapter 9 of FHWA GEC No. 3.
- Deep foundations using Chapter 10 of FHWA GEC No. 3.
- Deep foundations using pushover analysis.
- Stiffness values for liquefied conditions should also be provided.

5.8.5 RETAINING WALLS

- Lateral seismic earth pressure coefficient shall be obtained from Section 11 of AASHTO-LRFD-BDS.
- External stability for seismic events shall be performed.
- Internal stability of proprietary walls shall be performed by the wall manufacturer.

- Global stability and compound stability of the wall shall be performed. Resistance factors provided in BDS Section 11.5.8 shall be used.

5.8.6 SEISMIC HAZARDS

- The site-adjusted peak ground acceleration, As (Fpga × PGA, as specified in AASHTO-LRFD-BDS Article 3.10.3.2), shall be used for evaluating seismic hazards.
- Earthquake Magnitude of 6.0 shall be used unless directed by the Authority.
- Liquefaction potential shall be evaluated using FHWA GEC No. 3 or EERI Monograph 12.
- Liquefaction induced ground movements including ground settlement, lateral spread, and lateral flow shall be evaluated.

5.8.7 OVERALL STABILITY OF EMBANKMENT AND SLOPES

- Factor of safety against overall stability shall not be less than 1.1 for the seismic loading condition.
- 50 percent of site-adjusted peak ground acceleration shall be used.

5.9 CONSTRUCTION SPECIFICATIONS

Construction Standard and Supplemental Specifications to be utilized are available at the NJTA website. The GE shall assess these standard specifications and modify them as necessary.

5.10 CONTROL OF CONSTRUCTION

5.10.1 GENERAL

Control of normal methods of construction is covered by the New Jersey Turnpike Standard Specifications, the Supplementary Specifications and is discussed in the NJTA Construction Manual.

The methods of construction and construction control shall be clearly stated in the Contract Documents to the Field Engineers and GFRs. The following provides additional guidance for special construction considerations which should be addressed.

A. Embankments
When the GE has determined, from an examination of the subsurface profile and calculations, that the foundation soils are not competent to support the proposed embankment loadings, the GE shall determine the actual embankment quantity to be placed accounting anticipated settlement and the Contractor shall be advised in the Contract Documents of the anticipated settlement and to allow for the settlement and the potential for staged placement in their bid prices.

B. Unsuitable Materials
The GE shall evaluate the extent of soils unsuitable for roadway subgrade based on borings, site inspection, and other information. The

GE should also evaluate the potential for unknown unsuitable soil and present recommendations for proof-rolling or other methods to locate and mitigate these soils. The GE shall prepare specifications that detail the removal of unsuitable soils and replacement with suitable material. The GE shall also establish payment quantities for such removal and replacement, and present detailed methods for measurement.

Where any ground improvement methods are proposed for construction the necessary instrumentation should be included in the Contract Documents. Also, monitoring and instrumentation threshold values should be presented in the Phase B Geotechnical Engineering Report.

C. Soft Clays and Silts
Clays and silts with low shear strength make poor foundation material. However, with proper treatment, their engineering properties can be enhanced, thus improving their performance as a foundation material.

These materials often have low permeability and may require a substantial time period for consolidation to occur. The coefficient of consolidation shall be investigated in both the vertical and horizontal directions, and the soil samples shall be carefully inspected for possible horizontal sand or silt layers.

D. Roadway Cuts
Most problems encountered in roadway cuts involve either, or both, high groundwater conditions and soft subgrades. Two other problems less frequently encountered are unstable cut slopes and sloughing of cut slopes. These latter two problems are usually encountered when clay or silt soils are encountered in highway cuts. The potential for these problems shall be investigated in all proposed cuts.

The evaluation of the potential for these roadways cut problems should be addressed in the Phase B Geotechnical Engineering Report.

5.10.2 FIELD INSTRUMENTATION

Field instrumentation shall be installed to monitor existing and proposed construction as well as adjacent structures. The planning of instrumentation for construction control should consider duplication and location of the instruments because of potential damage during construction operations. The GE shall submit a Field Instrumentation Plan (FIP) included in the Phase B Geotechnical Engineering Report and submitted to the Authority. The FIP shall be developed by an instrumentation specialist providing locations, depths, and installation procedures of all field instrumentation to be installed for approval prior to implementing the plan. The FIP shall be included in the Contract Documents. The GE may be retained by the Authority to read the instruments, analyze the data, and identify and mitigate potential issues during construction; this work may also be performed by others. Potential construction control instrumentation devices are provided below. The GE shall address noise and vibration monitoring requirements and follow guidance provided in the supplemental specifications.

5.10.2.1 Inclinometers

Inclinometers shall be installed to monitor displacements within existing and proposed slopes, embankments, cuts, and adjacent structures before, during, and after construction, when deemed necessary by the GE and approved by the Authority. Inclinometers and inclinometer casings shall be installed in accordance with ASTM D6230.

5.10.2.2 Monitoring Wells

Monitoring wells shall be installed when specified prior to construction to monitor groundwater levels during placement of embankment fills, performance of surcharging programs, excavation of cuts, and performance of construction dewatering. The GE shall use the information to identify potential rising and falling groundwater levels which may induce deformation. Monitoring wells to be installed in accordance with Section 5.4.3.3

5.10.2.3 Vibrating Wire Piezometers

Vibrating wire piezometers when specified shall be used in conjunction with or without monitoring wells or to monitor pore water pressures during construction. Applications of monitoring pore water pressures include:

- Determine safe rates of fill or excavation.
- Locate the piezometric surface through slopes or embankments to aid in slope stability analysis.
- Monitor the effects of dewatering systems used for excavations on adjacent structures to identify drops in water level inducing additional effective stress in the soil and possible settlement.
- Monitoring the effects of ground improvement systems such as surcharging and the use of vertical prefabricated drains. Increases and decreases in the pore water pressure during the surcharging program to determine time for consolidation which must be allotted or when soil consolidation has slowed and the program may be terminated.

5.10.2.4 Settlement Platforms

Settlement platforms shall be installed when specified to monitor the magnitude and rate of settlement experienced by compressible soil layers during foundation loading, embankment loading, or fill placement. The selection of an external or internal reference point system shall be determined on a project specific basis. In general, when an unyielding stratum below the compressible layer is shallow, the GE shall use an internal reference point system, and when it is deep an external reference point system utilizing adjacent benchmarks to measure elevation changes shall be provided. Isolation casing shall be installed for all systems to prevent extraneous sources such as frost heave or down drag to affect the settlement readings. Settlement platforms shall be installed and monitored in accordance with ASTM D6598. In addition to the requirements specified herein, any manufactures requirements for product accessories or installation shall be followed. The GE may also propose the use of remote settlement surface and subsurface instrumentation, as appropriate. The GE shall consider the total cost of such systems (capital cost plus monitoring cost) and the reliability of remote systems.

5.10.2.5 Deep Settlement Points

Deep settlement points (Borros-Type anchors) shall be installed when specified to monitor displacements during foundation or embankment loading, fill placement, and tunnel construction. Guidance on deep Borros anchors, can be found in FHWA-NHI-10-034.

5.10.2.6 Vibration and Displacement Monitoring

Vibration and displacement monitoring shall be considered on a case by case basis given soil/groundwater conditions, proximity to sensitive structures and construction methods anticipated. Where deemed appropriate by the GE a vibration and Displacement Monitoring program shall be developed and performed in accordance with the NJTA Supplementary Specifications and supplemented by the GE on a project specific basis.

5.11 POST DESIGN SERVICES

Geotechnical Engineer (GE) shall be involved in the following tasks during construction:

5.11.1 FOUNDATIONS

5.11.1.1 Driven Piles

A. Review site specific work plan, in particular:
   1. Wave equation and/or driveability analysis & selection of hammer
   2. Load test program
   3. Equipment and access
   4. Refer to Pile Specification for Additional Requirements

B. Observe installation of test piles

C. Observe load testing

D. Review Load test results

E. Provide recommendation for production pile driving criteria:
   1. Penetration at termination (capacity blow count to be achieved over a minimum penetration)
   2. Provisionary criteria if penetration rate is not achieved within a certain number of feet below the production tip.
   3. An order list indicating the production pile length assumed to remain in the completed structure and modified for unanticipated site conditions. The Contractor may increase

the lengths as necessary to accommodate handling per their means and methods, at no additional cost to the Authority.

F. Observe initial period of production pile installation to ensure the construction inspector is aware of the driving criteria and completing the driven pile installation log properly.
G. Reevaluate the foundations for piles out of tolerance and damaged piles.

5.11.1.2 Drilled Shafts

A. Review site specific work plan, in particular:
   1. Method of casing installation
   2. Slurry management
   3. Method of cleaning hole
   4. Method of verifying shaft is cleaned out
   5. Method of casing removal
   6. Method of placing concrete
   7. CSL and TIP testing
   8. Materials manufacturer's data sheets
   9. Load testing, if applicable
   10. Equipment and access
   11. Refer to drilled shaft specification for additional requirements
B. Observe installation of test shafts
C. Observe load testing
D. Review Load test results, if applicable
E. Provide recommendation for production drilled shafts, if load testing is performed or unexpected subsurface conditions are encountered.
F. Observe initial period of production shaft installation to ensure the construction inspector is completing the driven pile installation log properly.
G. Review integrity test reports and provided recommendations for additional testing or repair, if any.
H. Observe the taking of concrete core, if coring of drilled shafts is required.

5.11.1.3 Micropiles

A. Review site specific work plan, in particular:
   1. Method of casing installation
   2. Method of cleaning hole
   3. Method of verifying micropile is cleaned out
   4. Method of casing removal
   5. Method of placing grout
   6. Proof and verification load testing
   7. Materials manufacturer's data sheets
   8. Equipment and access
   9. Refer to micropile specification for additional requirements
B. Observe installation of test micropiles
C. Observe load testing
D. Review load test results
E. Review contractor proposed bond lengths following load testing

F. Observe one day of production pile installation to ensure the construction inspector is aware of the driving criteria and completing the driven pile installation log properly.
G. Review inspection reports and videos.
H. Review proposed repairs for damaged or defective micropile or micropile abandoned.

5.11.2 RETAINING WALLS

5.11.2.1 Proprietary Walls

A. Review shop drawings for internal stability, external stability, and foundation layout
B. Review site preparation plan
C. Review backfill material soil test results

5.11.2.2 Support of Excavation

A. Review shop drawings and calculations

5.11.2.3 Tiebacks

A. Review site specific work plan
B. Review load test program
C. Observe tieback installation and ensure construction inspector is able to complete installation log properly
D. Observe proof and verification load testing
E. Review load test results
F. Review inspection reports

5.11.3 CONSTRUCTION MONITORING

5.11.3.1 Vibration Monitoring

A. Review site specific work plan
B. Review Baseline Reports
C. Review monitoring reports

5.11.3.2 Instrumentation (Vibrating wire piezometers, inclinometers, settlement platforms, etc.)

A. Review site specific work plan
B. Review installation records
C. Review monitoring reports
D. Provide recommendations.

5.11.3.3 Abandoning Existing Monitoring Wells

A. Coordinate with installer to abandon the wells

5.11.4 TRENCHLESS UTILITY INSTALLATIONS, RELOCATIONS AND ADJUSTMENTS

5.11.4.1 Trenchless Installations

A. Review site specific work plan
   1. Materials

2. Equipment
3. Method of construction

B. Review Protection Measures in Place
1. Protection against soil instability
2. Protection against uncontrolled ground water inflow
3. Prevention of soil subsidence/settlement along the alignment
4. Protection against flooding and means for emergency evacuation
5. Protection of falling objects

C. Instrumentation/monitoring procedures

D. Monitoring for hazardous gases

E. Calculations
1. Excavation Support System
2. Determination of loads
3. Adequacy of proposed pipe section and material
4. Scour potential and countermeasures
5. Geotechnical bearing resistance and settlement of pipe
6. Dewatering
7. Ground improvement
8. Preventing pavement box from drilling fluid
9. Impact of vibration to existing foundations or utilities

F. Utility plans

G. Post Installation Survey, Explorations and Remedy

5.12 REFERENCES

1. American Association of State Highway and Transportation Officials (AASHTO), “AASHTO Guide Specifications and Commentary for Vessel Collision Design of Highway Bridges,” 2nd Edition, 2009 with 2010 Interim Revisions.

2. American Association of State Highway and Transportation Officials (AASHTO), “AASHTO Guide Specifications for LRFD Seismic Bridge Design,” 2nd Edition, 2011 with 2015 Interim Revisions.

3. American Association of State Highway and Transportation Officials (AASHTO), “LRFD Bridge Design Specifications,” 7th Edition, 2014.

4. American Association of State Highway and Transportation Officials (AASHTO), “LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals,” 1st Edition, 2015.

5. American Association of State Highway and Transportation Officials (AASHTO), “Standard Specifications for Highway Bridges,” 17th Edition, 2002.

6. American Association of State Highway and Transportation Officials (AASHTO), “Standard Specifications for Transportation Materials and Methods of Sampling and Testing”.

7. American Railway Engineering and Maintenance-of-Way Association, “Manual for Railway Engineering,” Volumes 1-4, 2014.

8. American Society of Civil Engineers (ASCE), Geotechnical Special Publications.

9. Arneson, L.A., Zevenbergen, L.W., Lagasse, P.F., and Clopper, P.E., “Hydraulic Engineering Circular (HEC) 18 – Evaluating Scour at Bridges,” Federal Highway Administration Publication No. FHWA-HIF-12-003, Fifth Edition, April 2012.

10. ASTM International, “Annual Book of ASTM Standards”.

11. Berg, R.R., Christopher, B.R., Samtini, N.C., “Geotechnical Engineering Circular (GEC) No. 11 – Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes – Volumes I & II,” Federal Highway Administration Publication Nos. FHWA NHI-10-024 & FHWA NHI-10-025, November 2009.

12. Brown, D.A., Dapp, S.D., Thompson III, WR., Lazarte, C.A., “Geotechnical Engineering Circular (GEC) No. 8 – Design and Construction of Continuous Flight Auger (CFA) Piles,” Federal Highway Administration Publication No. FHWA-HIF-07-039, April 2007.

13. Brown, D.A., Turner, J.P., Castelli, R.J., “Geotechnical Engineering Circular (GEC) No. 10 - Drilled Shafts: Construction Procedures and LRFD Design Methods,” Federal highway Administration Publication No. FHWA-NHI-10-016, May 2010.

14. Buckle, I.G., Friedland, I., Mander, J., Martin, G., Nutt, R., Power, M., “Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges,” Multidisciplinary Center for Earthquake Engineering Research Publication No. MCEER-06-SP10, December 2006.

15. Burmister, D.M., “Principals and Techniques of Soil Identification,” Proceedings of the Highway Research Board, December, 1949.

16. Carlos A. Lazarte, Ph.D., P.E., Victor Elias, P.E., R. David Espinoza, Ph.D., P.E., Paul J. Sabatini, Ph.D., P.E., “Geotechnical Engineering Circular (GEC) No. 7 – Soil Nail Walls,” Federal Highway Administration Publication No. -. FHWA-NHI-14-007.

17. Collin, J.G. (2004). "Column Supported Embankments - Technical Summary #7. Ground Improvement Methods", FHWA NHI-04-001, October, Federal Highway Administration, Washington, D.C.

18. Collin, J. G., Hung, J. C., Lee, W. S., Munfakh, G., "Soil Slope and Embankment Design Reference Manual," Federal Highway Administration Publication No. FHWA-NHI-01-026, March 2001.

19. Commonwealth of Kentucky Transportation Cabinet (KTC), "Geotechnical Guidance Manual," June 2005.

20. Dalton, R., Monteverde, D., Sugarman, P., and Volkert, R., "Bedrock Geologic Map of New Jersey," New Jersey Department of Environmental Protection, 2014.

21. Elias, V., Welsh, J., Warren, J., Lukas, R., Collin, G., and Berg, R.R., "Ground Improvement Methods Reference Manual – Volumes I & II," Federal Highway Administration Publication No. FHWA-NHI-06-019 & FHWA-NHI-06-020, August 2006..

22. Golder Associates, "Rock Slopes: Design, Excavation, Stabilization," Federal Highway Administration Publication No. FHWA-TS-89-045, September 1989.

23. Han, J. and Akins, K. (2002). "Case Studies of Geogrid-Reinforced and Pile-Supported Earth Structures on Weak Foundation Soils," Proceedings of the International Deep Foundation Congress, Geotechnical Special Publication No. 116 - Deep Foundations 2002, M. W. O'Neill and F. C. Townsend (Eds.), ASCE, Orlando, FL, pp. 668-67

24. Hannigan, P.J., Goble, G.G., Likins, G.E., Rausche, F., "Design and Construction of Driven Pile Foundations – Volumes I & II," Federal Highway Administration Publication Nos. FHWA-NHI-05-042 & FHWA-NHI-05-043, April 2006.

25. Holtz, R.D., Christopher, B.R., and Berg, R.R., "Geosynthetic Design and Construction Guidelines," Federal Highway Administration Publication No. FHWA-NHI-07-092, August 2008.

26. Hung, C.J., Monsees, J., Munfah, N., and Wisniewski, J., "Technical Manual for Design and Construction of Road Tunnels – Civil Elements," Federal Highway Administration Publication No. FHWA-NHI-10-034, December 2009.

27. Idriss, I.M., Boulanger, R.W., "Soil Liquefaction during Earthquakes," Earthquake Engineering Research Institute Monograph 12, 2008.

28. International Code Council Inc., "International Building Code – New Jersey Edition," 2009.

29. Kavazanjian Jr., E, Matasovic, N., Hadj-Hamou, T., Sabatini, P.J., "Geotechnical Engineering Circular (GEC) No. 3 – Design Guidance: Geotechnical Earthquake Engineering for Highways - Volumes I & II," Federal Highway Administration Publication No. FHWA-SA-97-076 & FHWA-SA-97-077, May 1997.

30. Kavazanjian Jr., E., Wang, J.J., Martin, G.R., Shamsabadi, A., Lam, I. (Po), Dickenson, S.E., Hung, J.C., “Geotechnical Engineering Circular (GEC) No. 3 – LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations,” Federal highway Administration Publication No. FHWA-NHI-11-032, August 2011 (Rev. 1).

31. Keaton, J.R., Mishra, S.K., and Clopper, P.E., “Scour at Bridge Foundations on Rock,” Transportation Research Board – National Cooperative Highway Research Program Report No. NCHRP-Report-717, 2012.

32. Kimmerling, R.E. “Geotechnical Engineering Circular (GEC) No. 6 – Shallow Foundations,” Federal Highway Administration Publication No. FHWA-SA-02-054, September 2002.

33. Lagasse, P.F., Clopper, P.E., Pagán-Ortiz, J.E., Zevenbergen, L.W., Arneson, L.A., Schall, J.D., and Girard, L.G., “Hydraulic Engineering Circular (HEC) 23 – Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance – Volumes I & II,” Federal Highway Administration Publication No. FHWA-NHI-09-111 & FHWA-NHI-09-112, Third Edition, September 2009.

34. Lagasse, P.F., Zevenbergen, L.W., Spitz, W.J., and Arneson, L.A., “Hydraulic Engineering Circular (HEC) 20 – Stream Stability at Highway Structures,” Federal Highway Administration Publication No. FHWA-HIF-12-004, Fourth Edition, April 2012.

35. Lewis, J.V., and Kummel, H.B., “Geologic Map of New Jersey,” Geological Survey of New Jersey, 1912.

36. Mayne, P.W., Christopher, B.R., DeJong, J., “Subsurface Investigations – Geotechnical Site Characterization Reference Manual,” Federal Highway Administration Publication No. FHWA-NHI-01-031, May 2002.

37. Munfakh, G., A. Arman, J. G. Collin, J. C.-J. Hung, and R. P. Brouillette. 2001. *Shallow Foundations Reference Manual*, FHWA-NHI-01-023. Federal Highway Administration, U.S. Department of Transportation, Washington, DC.

38. National Cooperative Highway Research Program (NCHRP) Report 408, “Corrosion of Steel Piling in Nonmarine Applications”, 1998.

39. National Resources Conservation Center (NRCS), “Part 633 National Engineering Handbook,” Chapter 26 – Sand and Gravel Filter Design, October 1994.

40. Naval Facilities Engineering Command (NAVFAC), “Soil Mechanics - Design Manual 7.01,” U.S. Navy, 1986.

41. Naval Facilities Engineering Command (NAVFAC), “Foundations & Earth Structures - Design Manual 7.02,” U.S. Navy, 1986.

42. New Jersey Administrative Code, "Title 7 Chapter 9D - Well Construction and Maintenance; Sealing of Abandoned Wells."

43. New Jersey Department of Agriculture, "The Standards for Soil Erosion and Sediment Control in New Jersey," 7th Edition, January 2014.

44. New Jersey Department of Environmental Protection (NJDEP), "New Jersey Stormwater Best Management Practices (BMP) Manual," February 2004.

45. New Jersey Department of Environmental Protection (NJDEP), "Draft Flood Hazard Area Technical Manual," December 2008.

46. New Jersey Department of Transportation (NJDOT), "2007 NJDOT Standard Specifications for Road and Bridge Construction," 2007.

47. New Jersey Turnpike Authority (NJTA) Construction Manual.

48. New Jersey Turnpike Authority (NJTA) Procedures Manual.

49. New Jersey Turnpike Authority (NJTA) Standard Specifications.

50. New Jersey Turnpike Authority (NJTA) Supplementary Specifications.

51. Oregon Department of Transportation (ODOT), "Geotechnical Design Manual," April 2010.

52. Puckett, J.A., Garlich, M.G., Nowak, A., Barker, M., "Development and Calibration of AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals," Transportation Research Board (TRB) National Cooperative Highway Research Program (NCHRP) Report 796, 2014.

53. Rutgers University College of Engineering, "Engineering Soil Survey of New Jersey," Rutgers University College of Engineering Research Bulletins - Nos. 15 through 36.

54. Sabatini, P.J., Bachus, R.C., Mayne, P.W., Schneider, J.A., Zettler, T.E. "Geotechnical Engineering Circular (GEC) No. 5 – Evaluation of Soil and Rock Properties," Federal Highway Administration Publication No. FHWA-IF-02-034, April 2002.

55. Sabatini, P.J., Elias, V., Schmertmann, R. Bonaparte, "Geotechnical Engineering Circular (GEC) No. 2 – Earth Retaining Systems," Federal Highway Administration Publication No. FHWA-SA-96-038, February 1997.

56. Sabatini, P.J., Pass, D.G., Bachus, R.C., "Geotechnical Engineering Circular (GEC) No. 4 – Ground Anchors and Anchored Systems," Federal Highway Administration Publication No. FHWA-IF-99-015, June 1999.

57. Sabatini, P.J., Tanyu, B., Armour, T., Groneck, P., Keeley, J., "Micro Pile Design and Construction Reference Manual," Federal Highway Administration Publication No. FHWA-NHI-05-039, December 2005.

58. Samtani, N.C., Nowatzki, E.A., "Soils and Foundations Reference Manual – Volumes I & II," Federal Highway Administration Publication No. FHWA-NHI-06-088 & FHWA-NHI-06-089, December 2006.

59. Shan, H., Shen, J., Kilgore, R., and Kerenyi, K., "Scour in Cohesive Soils," Federal Highway Administration Publication No. FHWA-HRT-15-033, May 2015.

60. Tanyu, B.F., Sabatini, P.J., and Berg, R.R., "Earth Retaining Structures," Federal Highway Administration Publication No. FHWA-NHI-07-071, June 2008.

61. Terzaghi, K. and Peck, R.B., "Soil Mechanics in Engineering Practice," 2nd Edition.

62. Turner, J. P. 2006. *NCHRP Synthesis 360: Rock-Socketed Shafts for Highway Structure Foundations*. Transportation Research Board, National Research Council, Washington, DC, 148 pp.

63. United States Department of Agriculture (USDA), "Soil Maps".

64. United States Steel (USS), "Steel Sheet Piling Design Manual," Updated and Reprinted by United States Department of Transportation (USDOT)/FHWA with permission, July 1984.

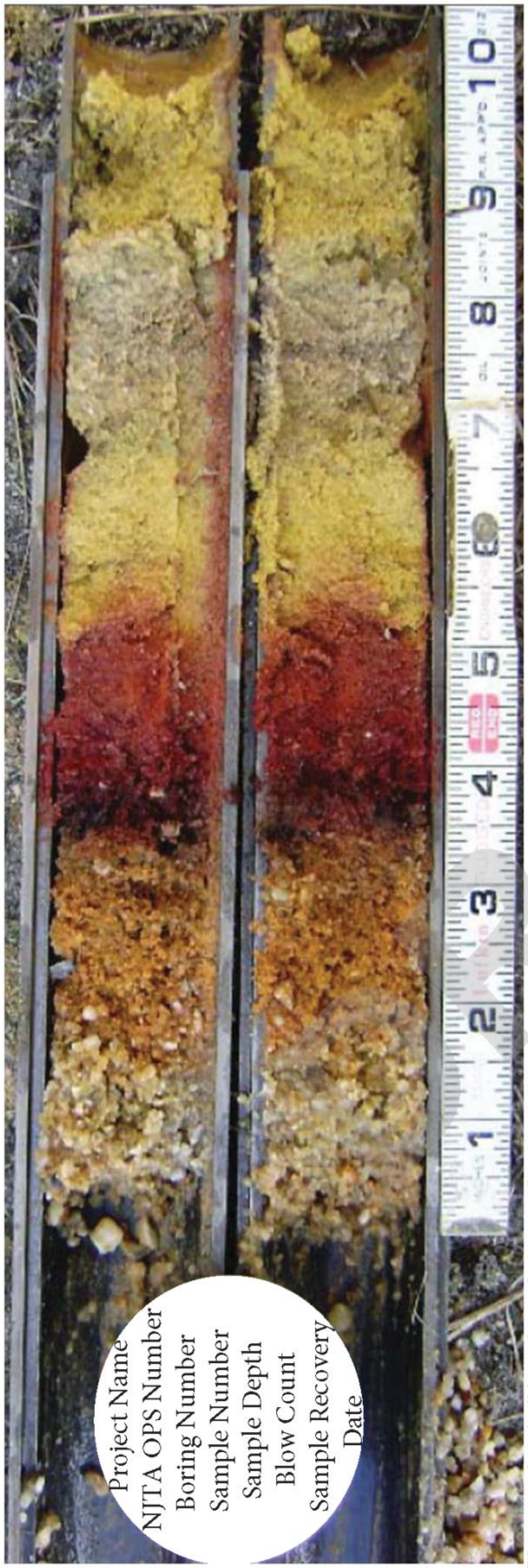
65. U.S. Army Corps of Engineers, "Seepage Analysis and Control of Dams," USACOE Publication No. EM1110-2-1901, April 1993 (change 1).

66. Washington State Department of Transportation (WSDOT), "Geotechnical Design Manual," April 2012.

67. Wightman, W.E., Jalinoos, F., Sirles, P., and Hanna, K., "Application of Geophysical Methods to Highway Related Problems," Federal Highway Administration Publication No. FHWA-IF-04-021, 2003.

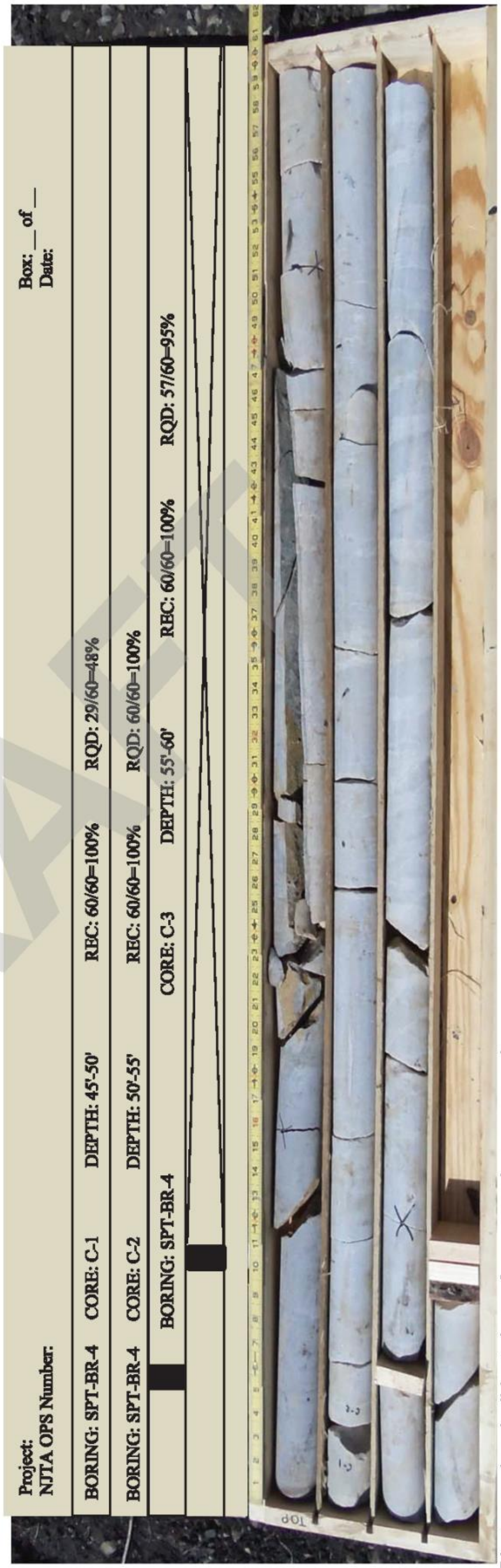
Proper Subsurface Exploration Photo Documentation

Split Spoon Photo Documentation



* Photographs shall be taken of every split spoon sample.

Rock Core Photo Documentation



* Photographs shall be taken of every rock core sample.

EXHIBIT 5 - 1: SAMPLE SOIL AND CORE BOX PHOTOGRAPH

<Consultant Logo>

GEOTECHNICAL BORING LOGS FOR

Boring No. ____________
Sheet No. ____ of ____

New Jersey Turnpike Authority
(Owner)

(Project)

(Contractor)

Contract No. ______________ Purpose ______________________________ Structure No. ________
Location ______________ RDWY. ______________________ STA. __________ OFF. __________

Rig No. ________ Type ____________ Driller ________________ Helper ________________
DATE
TIME STARTED
TIME FINISHED
WEATHER
DEPTH REACHED

GROUND ELEVATION ________________ M.L.W. ELEVATION ________________
ZERO OF BORING LOG ________________ ELEVATION GROUND WATER ________________

PAY QUANTITIES										
LINEAL FEET OF BORING					SAMPLES			LINEAL FEET OF ROCK CORE		
2-½ in	3 in	4 in			ORD. DRY	UNDIST. DRY		1-7/8" ID (NQ)	2-1/8" ID (NX)	
ITEM	ITEM	ITEM	ITEM	ITEM	ITEM	ITEM	ITEM	ITEM	ITEM	ITEM

	Unit Weight		Type	Size	Weight of Hammer	Average Fall	Hammer Type
Drilling Mud	________	Casing:	________	________	________	________	________
Ordinary Dry Samples O.D.	________	I.D.	________		________	________	________

Undisturbed Samples Type ____________ Length __________ O.D. __________ I.D. __________

GROUND WATER READINGS
DATE
TIME
DEPTH

GENERAL REMARKS:

NORTHING: EASTING:

All elevations refer to the NAVD 88 datum. Horizontal locations refer to the NJ State Plane coordinate system as per the NAD 83 datum.

The subsurface information shown hereon was obtained for NJTA design and estimate purposes. It is made available to authorized users only that may have access to the same information available to the NJTA. It is presented in good faith, but is not intended as a substitute for investigations, interpretation, or judgement of such authorized users.

INSPECTOR ______________________ GEOTECHNICAL ENGINEER ______________________

EXHIBIT 5 - 2: BORING LOG FORMS – FIRST PAGE

BORING LOG

Boring No. ____________
Sheet No. ____ of ____

CONTRACT NO. RDWY. STA. OFF.

Elev. (ft)	Blows on Casing	Blows on Spoon For 6-in Penetration		Sample		Log	Material & Remarks
				No.	Depth (ft)		

*indicates that soil description has been verified based on laboratory test results.

EXHIBIT 5 - 2: BORING LOG FORMS – SUBSEQUENT PAGES

<Consultant Logo>

GEOTECHNICAL BORING LOGS FOR

Boring No. ____________
Sheet No. ____ of ____

New Jersey Turnpike Authority
(Owner)

(Project)

(Contractor)

Contract No. ________________ Purpose ______________________________________ Structure No. _________
Location ____________________ RDWY. ___________________________ STA. ___________ OFF. __________

Rig No. _________ Type ______________ Driller ____________________ Helper __________________
DATE
TIME STARTED
TIME FINISHED
WEATHER
DEPTH REACHED

GROUND ELEVATION ______________________ M.L.W. ELEVATION ______________________
ZERO OF BORING LOG ______________________ ELEVATION GROUND WATER______________________

PAY QUANTITIES										
LINEAL FEET OF BORING					SAMPLES			LINEAL FEET OF ROCK CORE		
2-½ in	3 in	4 in			ORD. DRY	UNDIST. DRY		1-7/8" ID (NQ)	2-1/8" ID (NX)	
ITEM	ITEM	ITEM	ITEM	ITEM	ITEM	ITEM	ITEM	ITEM	ITEM	ITEM

Drilling Mud Type _________ Casing: Type _________ Size _________ Weight of Hammer _________ Average Fall _________ Hammer Type _________
Ordinary Dry Samples O.D. _________ I.D. _________ _________ _________ _________
Undisturbed Samples Type ______________ Length ___________ O.D. ___________ I.D. ___________

GROUND WATER READINGS
DATE
TIME
DEPTH

GENERAL REMARKS:

NORTHING: EASTING:

All elevations refer to the NAVD 88 datum. Horizontal locations refer to the NJ State Plane coordinate system as per the NAD 83 datum.

The subsurface information shown hereon was obtained for NJTA design and estimate purposes. It is made available to authorized users only that may have access to the same information available to the NJTA. It is presented in good faith, but is not intended as a substitute for investigations, interpretation, or judgement of such authorized users.

INSPECTOR ______________________________ GEOTECHNICAL ENGINEER ______________________________

EXHIBIT 5 - 3: BORING LOG FORMS FOR STORMWATER FACILITIES – FIRST PAGE

BORING LOG

BORING NO.__________
SHEET NO.____ OF ____

CONTRACT NO. RDWY. STA. OFF.

Elev. (ft)	Blows on Spoon	Sample		Log	Material & Remarks
		No.	Depth (ft)		

* Indicates that soil description has been verified based on laboratory results.

EXHIBIT 5 - 3: BORING LOG FORMS FOR STORMWATER FACILITIES – SUBSEQUENT PAGES

Boring No. ________

Sheet No. ______ of ______

CONTRACT NO. RDWY. STA. OFF.

ROCK CORING LOG

Core Data										Rock Description									Discontinuity Description								Lab Testing			
Run				Water	Rock Core					General				Fabric		Structure			Planes		Joints									
Elev.	Sample no.	Depth (ft)	Core Time (ft/min)	Gain/Loss	No. of Pieces	Recovery		Rock Quality Designation, RQD		Rock Type	Color	Weathering	Strength	Grain Size	Grain Shape	Fracture Spacing	Bedding Spacing	Foliation Angle	Fracture Type	Fracture Aperture	Joint Surface		Joint Infilling	Infilling Strength	Water Content	Joint Roughness Coefficient	Strength (ksi)	Unit Weight (pcf)	Other (Specify)	Other (Specify)
																					Shape	Roughness								
						in	%	in	%	(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)	(j)	(k)	(l)	(m)	(n)	(o)	(p)	(q)	(r)	(s)	(t)	(u)

(a) Amphibolite, Argillite, Basalt, Chert, Claystone, Conglomerate, Diabase, Dolomite, Gneiss, Granite, Hornfers, Limestone, Marble, Metaquartzite, Mudstone, Phyllite, Quartzite, Sandstone, Schist, Serpentinite, Shale, Siltstone, Slate

(b) Bl-Blue, Bk-Black, Br-Brown, Gr-Gray, Grn-Green, Or-Orange, Rd-Red, Tn-Tan, Yl-Yellow, Lt-Light, Dk-Dark

(c) I-Unweathered, II- Slightly Weathered, III-Moderately Weathered, IV-Highly Weathered, V-Completely Weathered, VI- Residual Soil

(d) R0-Extremely Weak, R1-Very Weak, R2-Weak, R3-Medium Strong, R4-Strong, R5-Very Strong, R6- Extremely Strong

(e) Sedimentary: VF- Very Fine, FG-Fine, MG-Medium, CG-Coarse, VC-Very Coarse; Igneous & Metamorfic: P- Pegmatitic, CG-Coarse, MG-Medium, FG-Fine, A- Aphanitic

(f) AG-Angular, SA-Subangular, SR-Subrounded, RD-Rounded, WR-Well Rounded

(g) ECF- Extremely Close, VCF-Very Cloase, CF-Close, MF-Moderate, WF-Wide, VWF-Very Wide

(h) TLB-Thinly Laminated, LB- Laminated, VTNB-Very Thin, TNB-Thin, MB-Medium, TKB-Thick, VKTB-Very Thick (massive)

(j) J-Joint, Ft-Fault, Sh-Shear, Fo-Foliation, V-Vein, B-Bedding

(k) T-Tight, SO-Slightly Open, MO-Moderately Open, O-Open, MW-Moderately Wide, W-Wide

(l) P-Planar, U-Undulating, WU-Wavy, ST-Stepped, IR-Irregular

(m) SL- Slickensided, SM-Smooth, SR-Slightly Rough, RG-Rough, VR-Very Rough

(n) Ca-Calcite, Fe-Iron, Qtz-Quartz, C-Clay, M-Silt, S-Sand, G-Gravel

(o) S1-Very Soft, S2-Soft, S3-Firm, S4-Stiff, S5-Very Stiff, S6-Hard

(p) D-Dry, M-Moist, W-Wet

EXHIBIT 5 - 4: ROCK CORING LOG FORMS

<Consultant Logo>

NEW JERSEY TURNPIKE AUTHORITY
FIELD TEST PIT LOG

TEST PIT NO. ______
SHEET ____ OF ____
DATE: START ______
END ______

Contract No. ______ Purpose ______ Structure No. ______
Location ______ RDWY ______ STA. ______ OFF. ______
NORTHING ______ EASTING ______ GROUND ELEVATION ______
EQUIPMENT USED ______
INSPECTOR ______ DRILLERS NAME/COMPANY ______
PIT DIMENSIONS: LENGTH ______ ; WIDTH ______ ; DEPTH ______
WATER LEVEL DEPTH: ______ NOT ENCOUNTERED ☐
CHECKED BY: ______ ; DATE: ______

DEPTH (FT)	SAMPLE NO. AND TYPE	POCKET PENT/ TORVANE (TSF)	WATER CONTENT	DESCRIPTION	REMARKS
0.0					

NOTE: All elevations refer to the NAVD 88 datum. Horizontal locations refer to the NJ State Plane coordinate system as per the NAD 83 datum.

EXHIBIT 5 - 5: TEST PIT LOG FORMS

Observation Well Instalation Log

WELL NO.: ____________

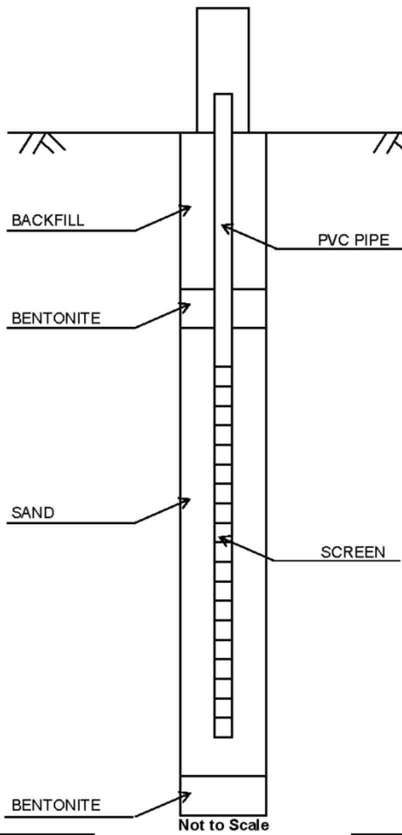
PROJECT NAME: ______________________ PROJECT NO.: ____________

DATE INSTALLED: ____________ INSPECTOR: ____________ HOLE DEPTH: ____________

CONTRACTOR: ____________ DRILLER: ____________ HELPER: ____________

LOCATION: ______________________ GROUND ELEVATION: ____________

POINTS OF INTEREST



________ FT = LENGTH OF COVER FROM THE GROUND SURFACE

____________ BACKFILL MATERIAL

________ FT = BENTONITE SEAL DEPTH

________ FT = TOP OF SAND DEPTH

________ FT = TOP OF SCREEN DEPTH

________ FT = LENGTH OF SCREEN

________ FT = BOTTOM OF SCREEN DEPTH

________ FT = BOTTOM OF SAND DEPTH

________ FT = BOTTOM OF HOLE DEPTH

WELL READINGS

DATE	BY	DEPTH(FT)

Depth recorded is depth below ground surface

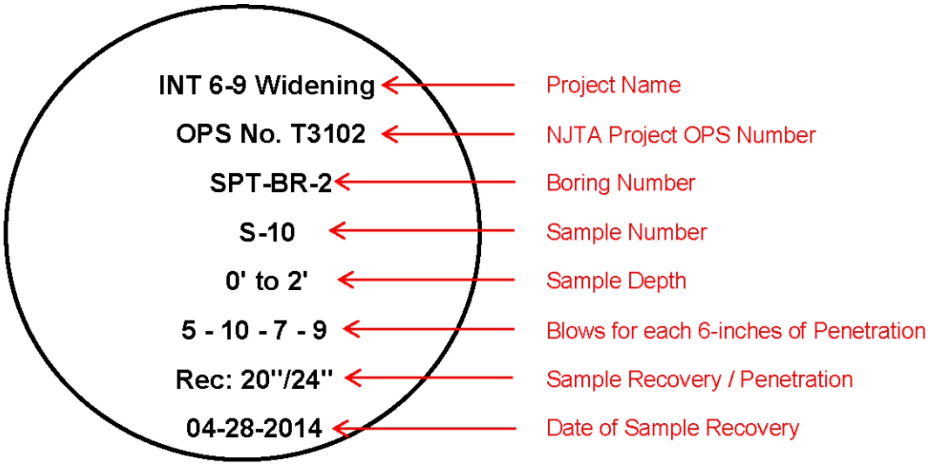
WELL DATA

ITEM	DESCRIPTION
PVC Well Casing Inside Diameter	
Lock Installed	
Standpipe or Flushmount	
Bags of Sand Used	
Bags of Bentonite Used	
Development	

NOTES:

EXHIBIT 5 - 6: MONITORING WELL LOG FORMS

a) Proper Labeling of Soil Sample Jar Tops



b) Proper Labeling of Soil Sample Boxes

Top of Sample Box

Project: INT 6-9 Widening
NJTA OPS Number: T3102

Boring:	SPT-BR-2	SPT-BR-3
Sample:	S-13 to S-19	S-1 to S-5
Depth:	55' to 85'	0' to 17'
Date:	10-23-2007	10-26-2007
Inspector:	MDR	MDR
Box:	2 of 2	1 of 2

Front of Sample Box

Project: INT 6-9 Widening
NJTA OPS Number: T3102

Boring:	SPT-BR-2	SPT-BR-3
Sample:	S-13 to S-19	S-1 to S-5
Depth:	55' to 85'	0' to 17'
Date:	10-23-2007	10-26-2007
Inspector:	MDR	MDR
Box:	2 of 2	1 of 2

Side of Sample Box

Project: INT 6-9 Widening
NJTA OPS Number: T3102

Boring:	SPT-BR-2	SPT-BR-3
Sample:	S-13 to S-19	S-1 to S-5
Depth:	55' to 85'	0' to 17'
Date:	10-23-2007	10-26-2007
Inspector:	MDR	MDR
Box:	2 of 2	1 of 2

Note:
Label both sides of sample box.

c) Proper Labeling of Undisturbed Samples

Side of Sampler

Project: INT 6-9 Widening
NJTA OPS Number: T3102

Boring:	SPT-BR-2
Sample:	U-1
Depth:	55' to 57'
Rec::	20"/24"
Date:	10-23-2007
Inspector:	MDR

Top ↑

Top of Sampler

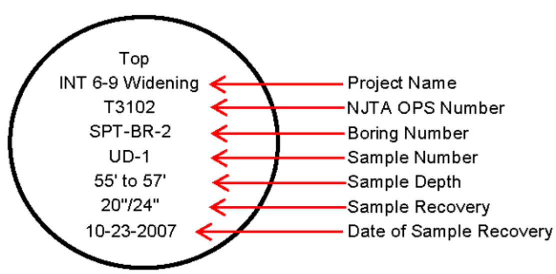


EXHIBIT 5 - 7: SOIL SAMPLE LABELING (JARS, BOXES AND UNDISTURBED SAMPLES)

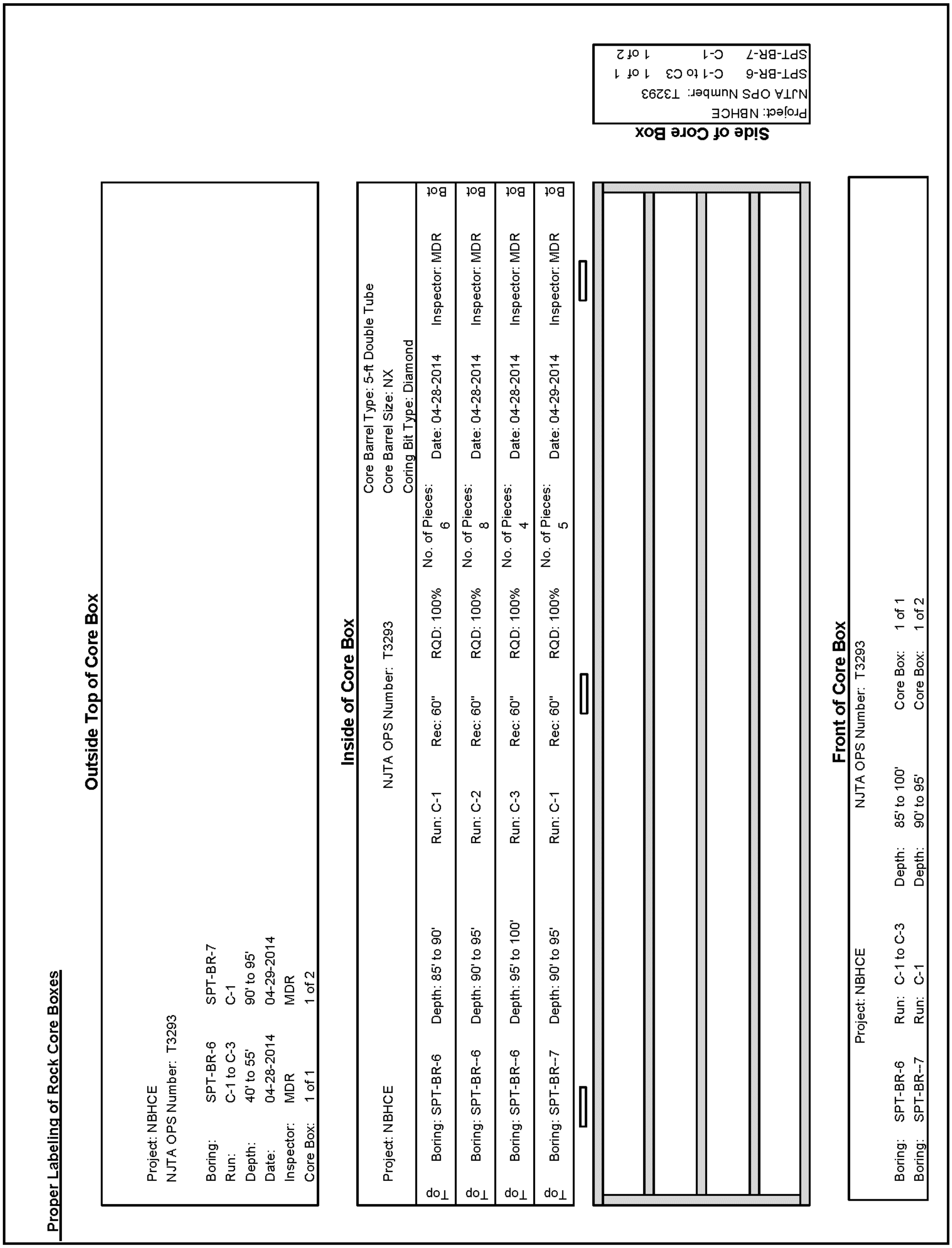


EXHIBIT 5 - 8: ROCK CORE BOX LABELING INSIDE & OUTSIDE

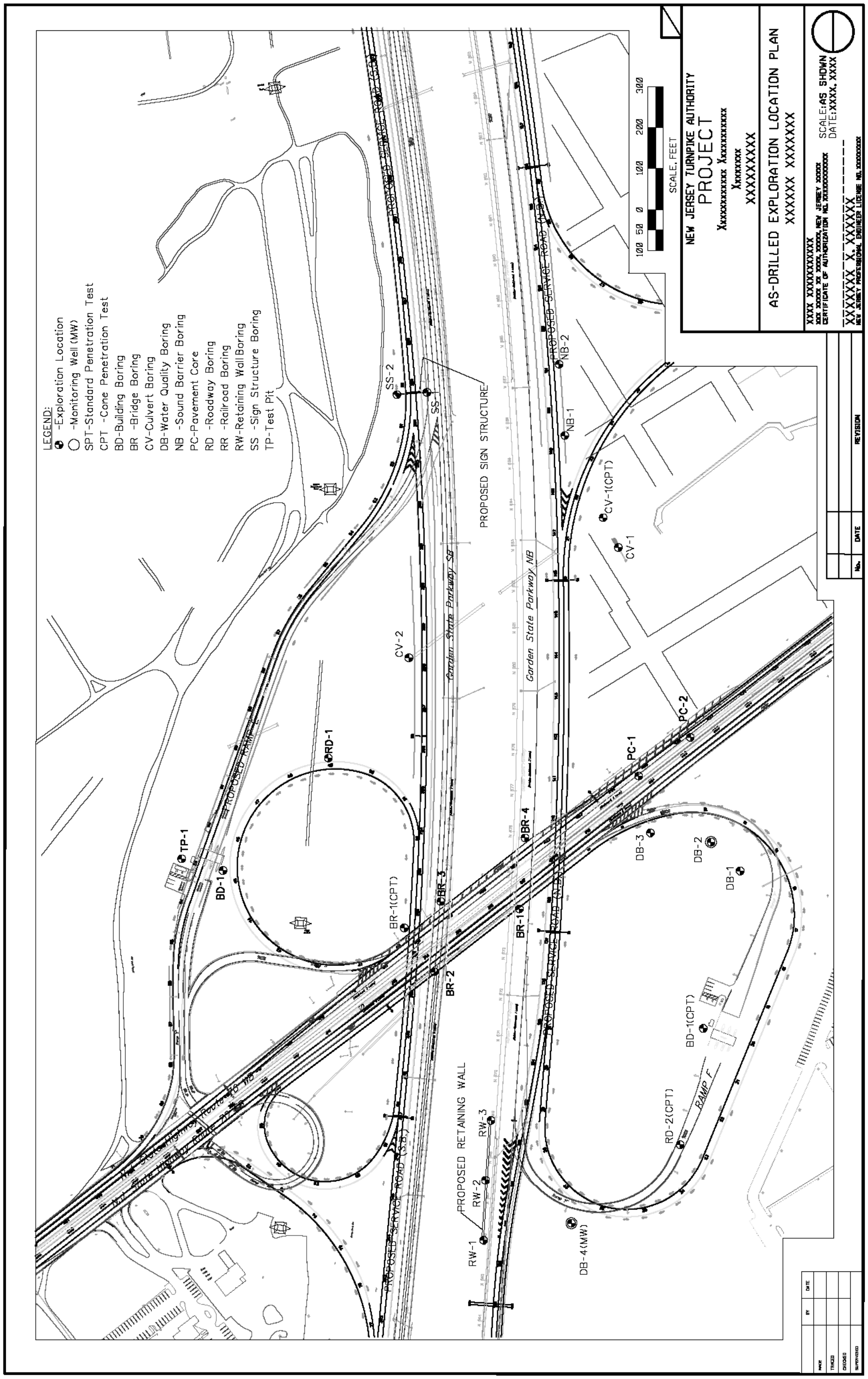


EXHIBIT 5 - 9: SAMPLE BORING LOCATION PLAN

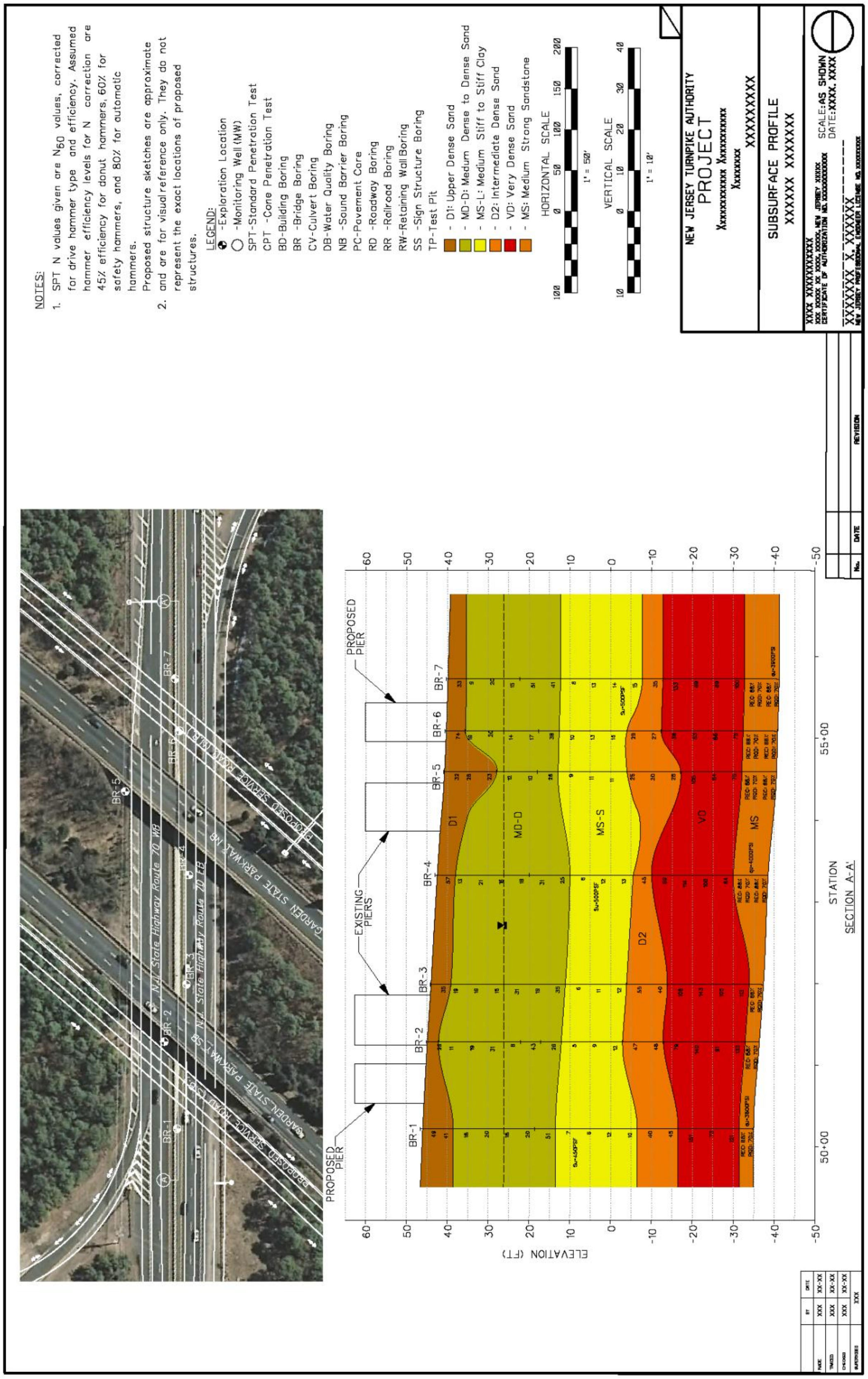
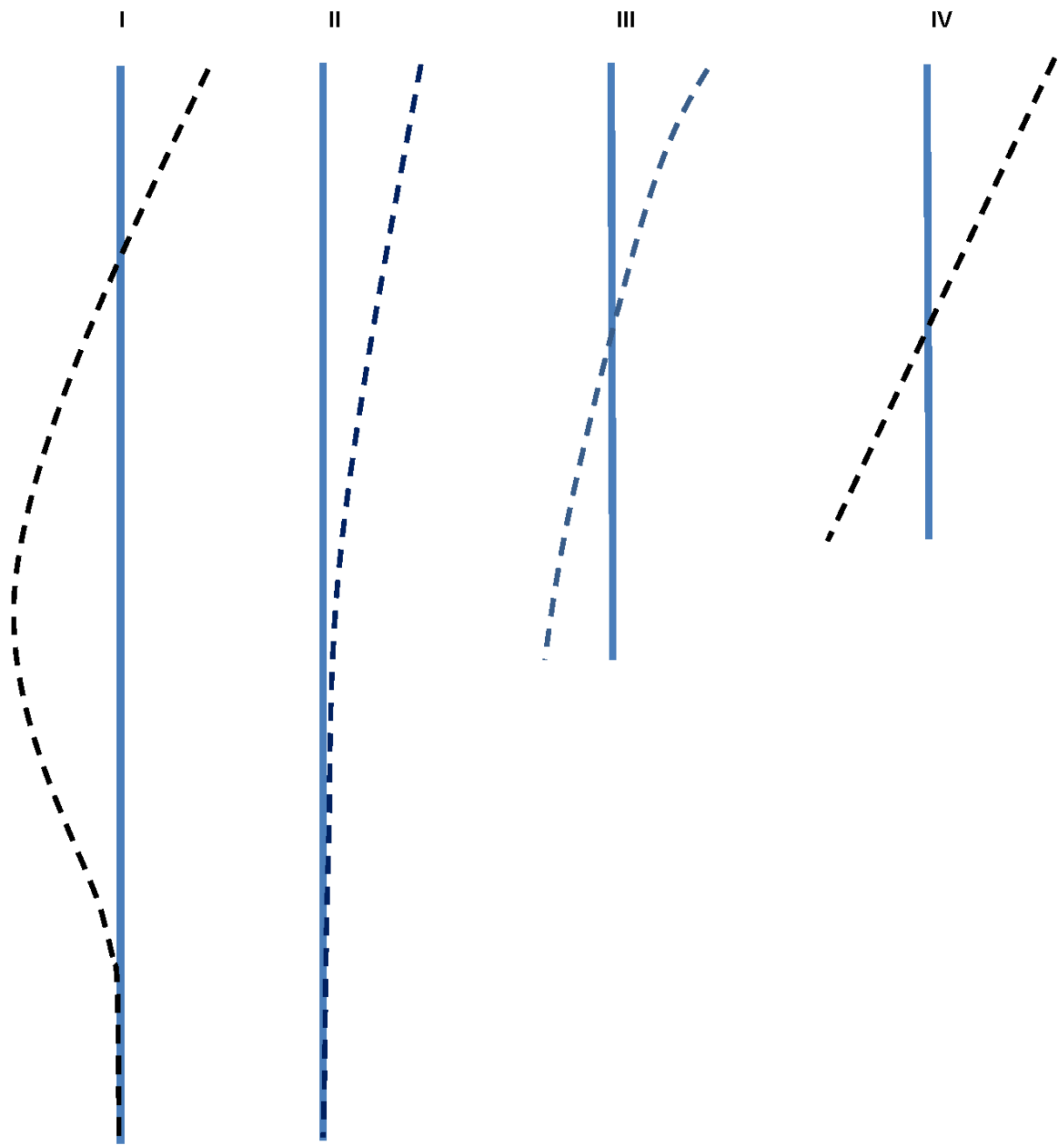


EXHIBIT 5 - 10: SAMPLE SUBSURFACE PROFILE



I - Recommended for deep foundations of Bridge Substructures, Culverts, Buildings, Retaining Walls supporting Structures

II - Same as I.

III - Recommended for deep foundations of Retaining Walls not supporting Structures, Sound Barriers, Sign Structures, High Mast Lighting, Traffic Signals. Lateral tip movement is limitted to 0.1 inch.

IV - Not recommended for any structures. May be considered for temporary works.

EXHIBIT 5 - 11: CRITERIA FOR ESTIMATING MINIMUM LENGTH FOR DEEP FOUNDATIONS BASED ON LATERAL STABILITY

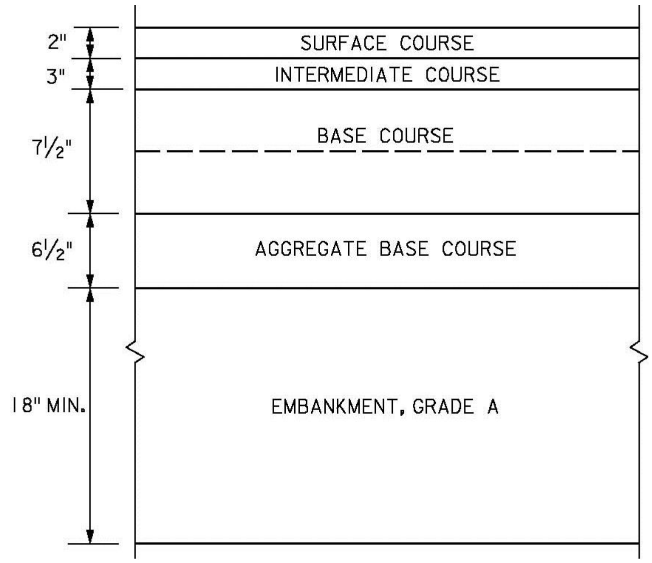
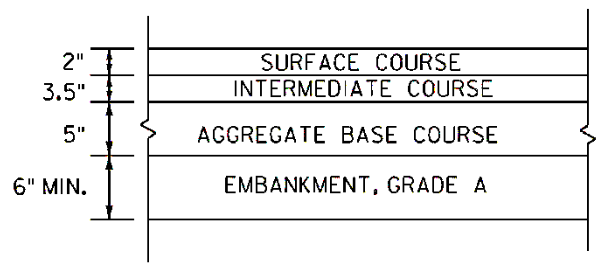


EXHIBIT 5 – 12: TURNPIKE PAVEMENT SECTION (MAINLINE, RAMP & SHOULDERS)



NOTE: TRUCK PARKING AREAS SHALL BE PAVED WITH TURNPIKE PAVEMENT, AS SHOWN IN EXHIBIT 5-12.

EXHIBIT 5 – 13: TURNPIKE CAR PARKING PAVEMENT SECTION

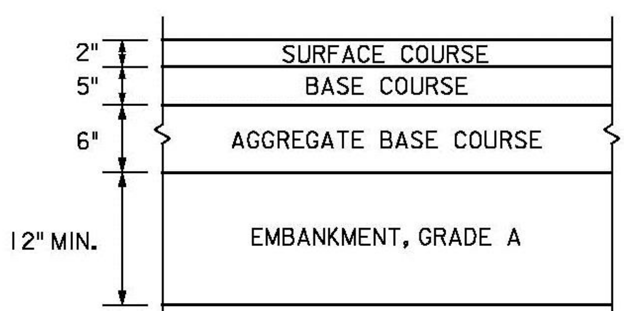
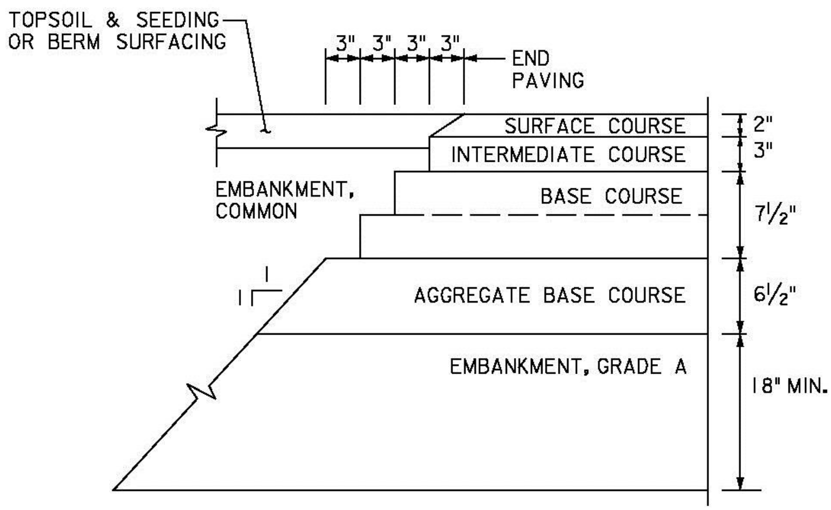
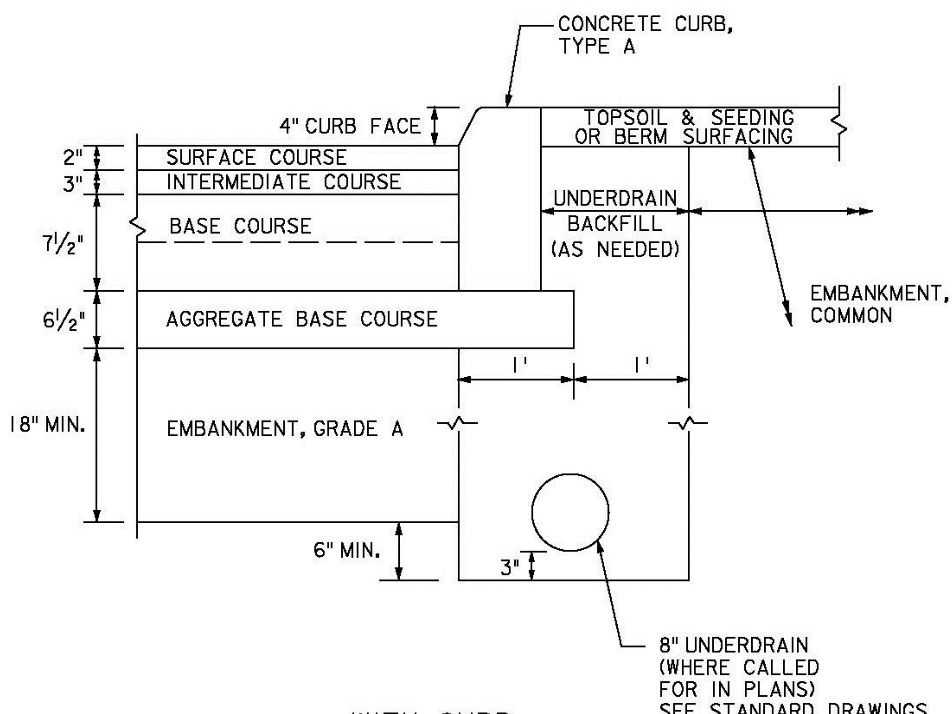


EXHIBIT 5 – 14: TURNPIKE U-TURN PAVEMENT SECTION



WITHOUT CURB



WITH CURB

NOTES:

1. BASE COURSES OVER 4" THICK SHALL BE INSTALLED IN TWO LIFTS.

EXHIBIT 5 – 15: TURNPIKE TRANSVERSE PAVEMENT STEPPING DETAIL

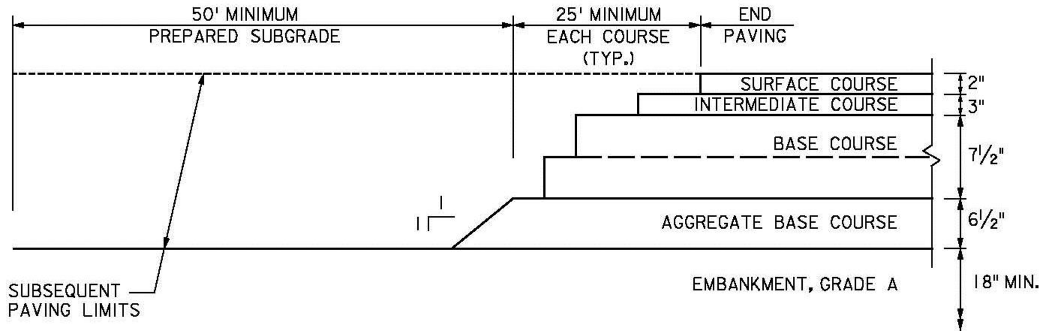


EXHIBIT 5 – 16: TURNPIKE LONGITUDINAL PAVEMENT STEPPING DETAIL

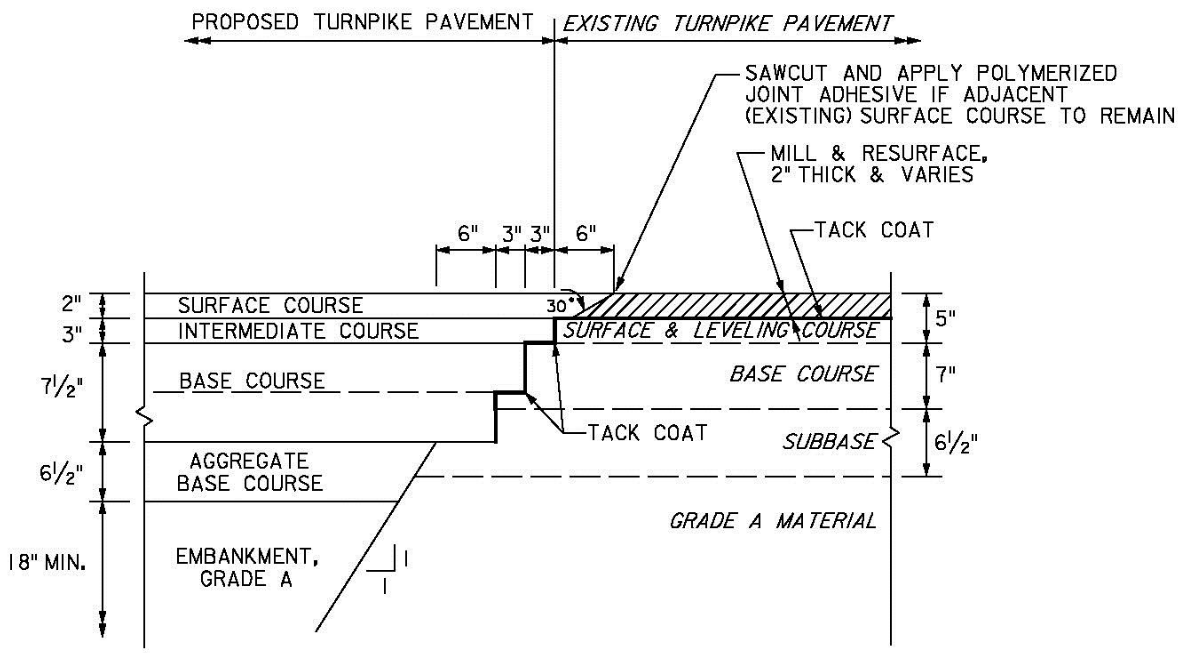


EXHIBIT 5 – 17: TURNPIKE NEW PAVEMENT INTERFACE WITH EXISTING PAVEMENT

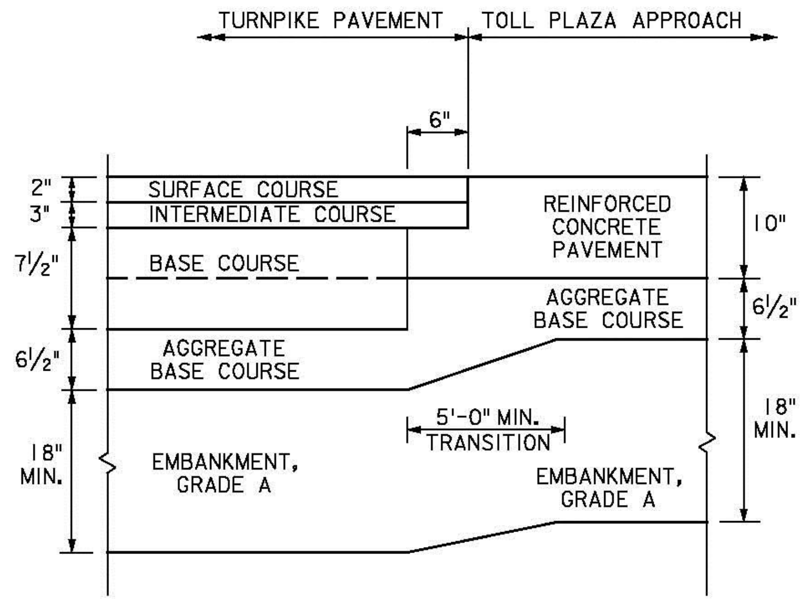


EXHIBIT 5 – 18: TURNPIKE TOLL PLAZA PAVEMENT INTERFACE

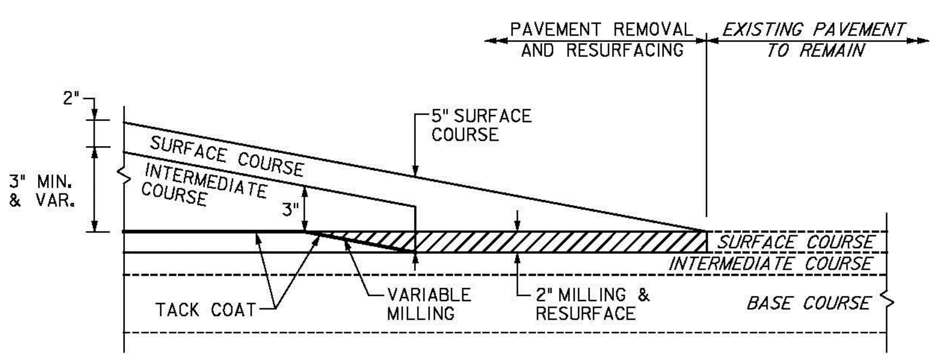


EXHIBIT 5 – 19: TURNPIKE PAVEMENT REMOVAL AND RECONSTRUCTION DETAIL

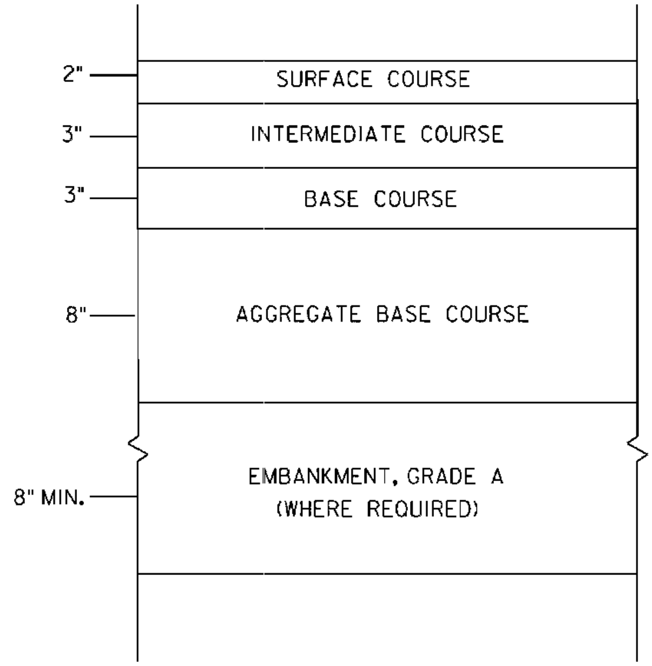
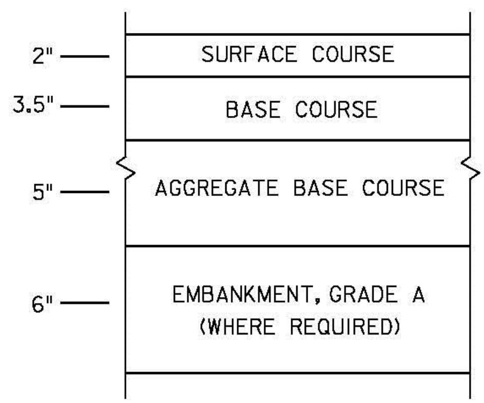


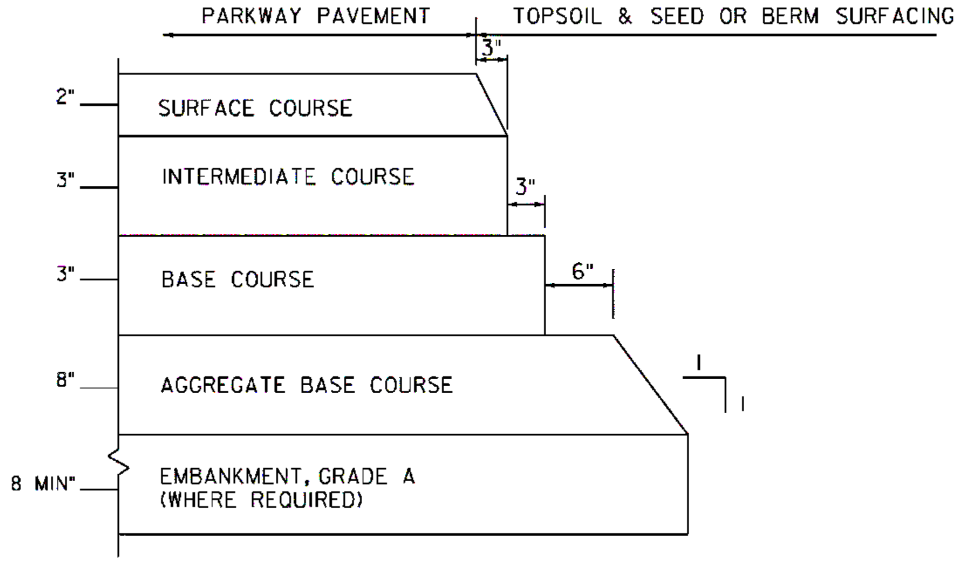
EXHIBIT 5 – 20: PARKWAY PAVEMENT SECTION (MAINLINE, RAMPS & SHOULDERS)



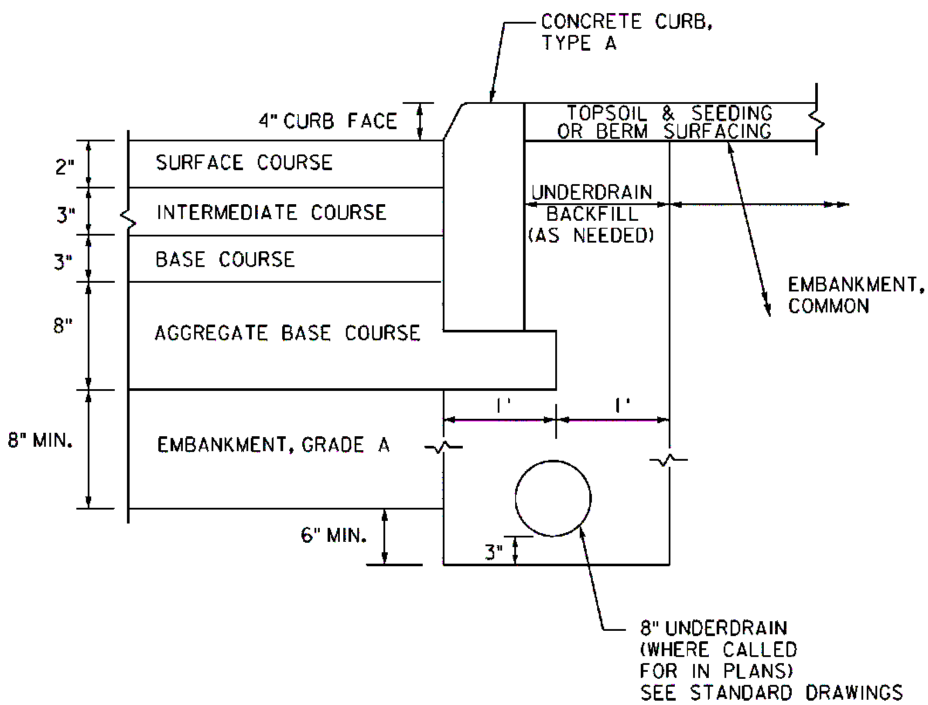
NOTE:

TRUCK PARKING AREAS ARE TO BE PAVED WITH PARKWAY PAVEMENT.

EXHIBIT 5 – 21: PARKWAY CAR PARKING PAVEMENT SECTION



WITHOUT CURB



WITH CURB

EXHIBIT 5 – 22: PARKWAY PAVEMENT STEPPING DETAIL

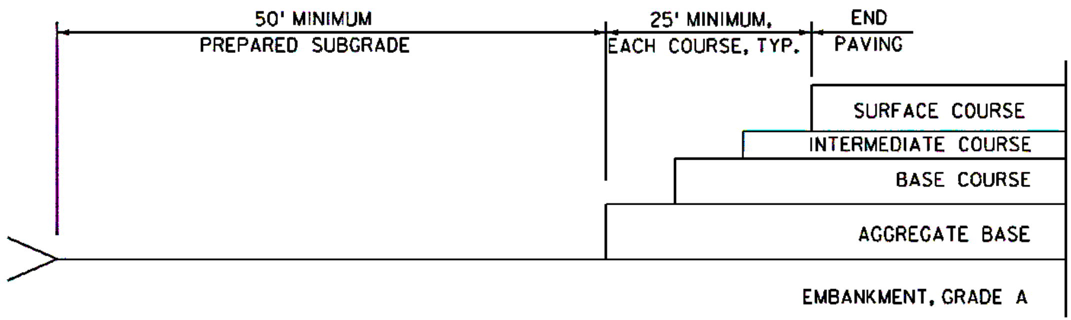


EXHIBIT 5 – 23: PARKWAY LONGITUDINAL PAVING INTERFACE

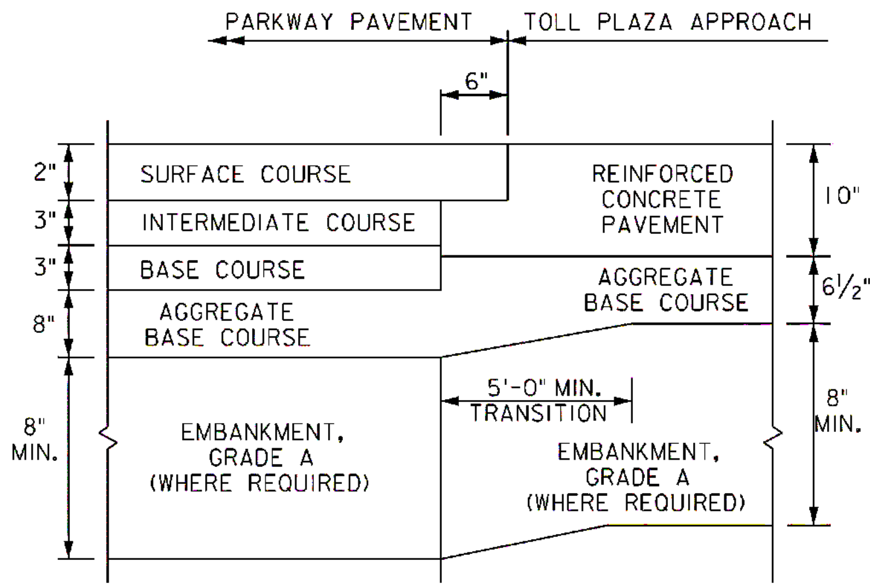


EXHIBIT 5 – 24: PARKWAY TOLL PLAZA-TRANSVERSE / LONGITUDINAL PAVING INTERFACE

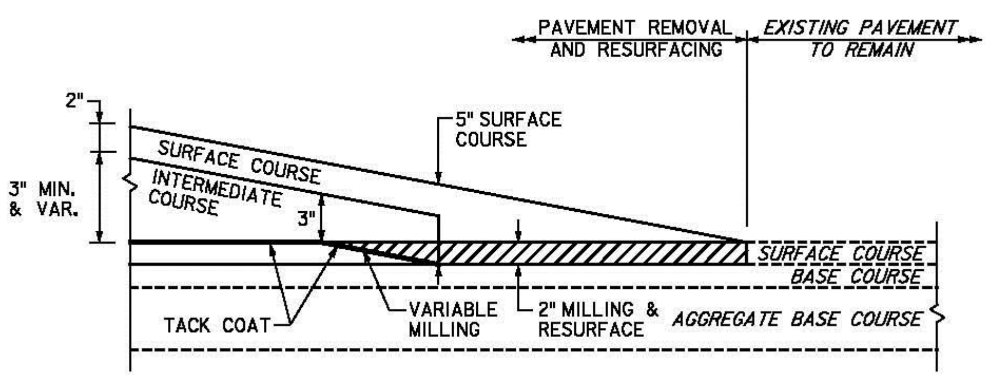


EXHIBIT 5 – 25: PARKWAY PAVEMENT REMOVAL & RECONSTRUCTION DETAIL

RISK ANALYSIS																					
Risk Identification								Quantitative Analysis					Qualitative Display of the Best Guess Impact			Risk Response Plan		Monitoring and Control			Critical Issue
Priority	Status	ID #	Date Ident. ---------- Project Phase	Project Stage	Summary Description Threat and/or Opportunity	Detailed Description of Risk Event (Specific, Measurable, Attributable, Relevant, Timebound) [SMART]	Risk Trigger	Type	Probability (%)		Risk Impact ($M) and (Month)	Expected Impact ($M)	Probability	Impact	Risk Matrix (Probability of Occurrence by Best Guess Impact)	Strategy	Action To Be Taken (response actions including advantages and disadvantages. Specify the time frame)	Risk Owner	Risk Review Dates	Date, Status and Review Comments (to show the history of risk's monitoring do not delete previous comments)	Is Risk on Critical Path?
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	[10a]	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
EXAMPLE	Active		Mar-06 Phase A	Pre-construction	Threat Reuse Option vs New Foundation Threat	Methods identified to reuse existing foundation may be more expensive than designing a new foundation.	A damaged or fatigued component results in loss of communication	Cost Schedule	40%	MIN MAX BEST GUESS MIN MAX BEST GUESS	4.70$M 26.30$M 8.70$M 0.5Mo 2.0Mo 1.0Mo	0.03	High	Low	Probability VH H M L VL x VL L M H VH Impact	Acceptance	Notification alarms will alert responsible team personnel of loss of communication throughout the monitoring system. Replacement components will be available, should they fail. Repairs/replacements will be undertaken within 24 hours of notification.	Geotech	2006-Dec-2 2007-Jan-2		YES
EXAMPLE	Active		Mar-06 Phase B	Pre-construction	Opportunity Traffic Control Threat	Issues during construction may require more traffic control.	Excessive noise or vibrations expected during the pile installation.	Cost Schedule	35%	MIN MAX BEST GUESS MIN MAX BEST GUESS	8.80$M 35.00$M 13.20$M 0.6Mo 2.4Mo 1.0Mo	0.45	Moderate	Very Low	Probability VH H M L VL x VL L M H VH Impact	Avoidance	There is every indication that the impacts associated wth pile installation will be less than initially anticipated.	Traffic	2006-Dec-2 2007-Jan-2		YES
EXAMPLE	Active		Mar-06 Phase B	Pre-construction	Opportunity Remaining Service Life of the Existing Foundation Threat	The anticipated remaining service life of the existing foundation may be less than two years.	Based on lab testing, corrosion potential is very high for the site.	Cost Schedule	40%	MIN MAX BEST GUESS MIN MAX BEST GUESS	7.00$M 19.60$M 14.00$M 0.2Mo 1.2Mo 0.4Mo	1.00	High	Low	Probability VH H M L VL x VL L M H VH Impact	Mitigation	An experienced subcontractor will be utilized with appropriate methods employed to mitigate this potential risk.	Geotech	2006-Dec-2 2007-Jan-2		YES
EXAMPLE	Active		Mar-06 Phase C	Pre-construction	Opportunity Support Current Design Standards Threat	As a result of further analysis, foundation may not be able to support the new design loads	Foundation Loads were very high	Cost Schedule	20%	MIN MAX BEST GUESS MIN MAX BEST GUESS	2.50$M 27.00$M 7.50$M 0.7Mo 8.0Mo 1.1Mo	0.50	Low	Very Low	Probability VH H M L VL x VL L M H VH Impact	Mitigation	Rehabilitate the foundations to satisfy the current standards.	Geotech	2006-Dec-2 2007-Jan-2		YES
EXAMPLE	Active		Mar-06 Phase C	Pre-construction	Opportunity Support Current Design Standards Threat	Identification of major flaws in existing foundation due to poor design, damage during construction or after construction.	Piles exceed deflection tolerance.	Cost Schedule	15%	MIN MAX BEST GUESS MIN MAX BEST GUESS	1.50$M 10.00$M 3.00$M 0.5Mo 1.0Mo 0.8Mo	3.50	Very Low	NO RISK	Probability VH H M L VL VL L M H VH Impact	Mitigation	Strengthen the existing foundations	Structures	2006-Dec-2 2007-Jan-2		YES

Risk Identification (1) – (8): It determines, describes, and characterizes the risks that might affect the project. The risk identification is an iterative process as new risks may come to light as the project progresses.

(1) Priority - provides ranking of importance.

(2) Status - offers three options to choose from:
- **Active -** the risk is actively monitored and controlled.
- **Dormant -** the risk is in low priority but may become high priority in the future.
- **Retired -** the risk is demised and its contribution equals zero.

(3) ID# - assigns a unique number to the risk for tracking purposes.

(4) Date Identified and Project Management Phase - represents the date and project management phase (Phase A, B, C, D & Construction) at which the risk was first identified.

(5) Project Stage - lists project delivery stages: Preconstruction and Construction.

(6) Summary Description (Threat and/or Opportunity) - presents a summary of risk's definitions and outcomes:
- **Threat -** the risk outcome provides negative impact on the project, higher cost and/or longer duration, which should be minimized.
- **Opportunity –** the risk outcome provides positive impact to the project, lower cost and/or shorter duration, which should be maximized.

(7) Detailed Description of Risk Event (SMART) - stands for Specific, Measurable, Attributable (a cause is indicated), Relevant, and Timebound.

(8) Risk Trigger - highlights symptoms and warning signs that point to the likelihood of occurrence. This information is essential to timely implementation of the Risk Response Plan.

Quantitative Analysis (9) - (12): It analyzes numerically the probability of occurrence and consequences of risk impact. Please, note that the "BEST GUESS" value should not be too close to lowest or highest value.

Qualitative Display of the Best Guess Impact (13) - (15): It displays the results of the Risk Quantitative Analysis.

Risk Response Plan (16) – (17): It identifies strategies and actions, developed by The Project Manager and the team, to address the risk. The strategies are:
- **Avoidance -** adopting changes to the project plan aimed at eliminating the risk or protecting the project objectives from risk's impact. This can be achieved by changing scope, adding time or resources thus relaxing the so-called "triple constra".
- **Transference -** transferring the financial impact of the risk by contracting out some work. Its success depends on the contractor's ability to reduce risk.
- **Mitigation -** reducing the probability or consequences of risk's impact to an acceptable threshold. It can be accomplished through some means specific to the project and the risk. However, mitigation is costly and time consuming.
- **Acceptance -** the Project Manager and the team accept the risk.

Monitoring and Control (18) – (20): It identifies and assigns parties to handle each risk response. This process ensures that each risk has an owner. Risk monitoring and control continues throughout the life of the project.
If an unanticipated risk emerges, or a risk's impact appears greater than expected, the planned response strategy and actions may not be adequate. The Project Manager and the team must apply additional response strategies and take actions to stay in control of the risk, for example:
- Choose alternative response strategy,
- Implement a contingency plan,
- Take corrective actions,
- Re-plan the project.

The Task Manager, assigned to the risk, reports periodically to the Project Manager on the effectiveness of the plan, unintended consequences, and makes suggestions on mid-course corrections to further mitigate the risk.

Critical Issue (21): It determines whether the risk is on the Critical Path, because if it is not, its value doesn't influence the total project duration.

EXHIBIT 5 – 26: SAMPLE RISK ANALYSIS WORKSHEET

APPENDIX A
Sample Boring Specifications

Sample Boring Specifications

The contract between the Engineer and the Boring Contractor shall contain all clauses of the contract between the Authority and the Engineer (often referred to as 'flow down clauses'). In addition, the Engineer-Test Boring Contract shall contain the following clauses specific to performing work within the Authority right-of-way.

1.1. Jurisdiction and Authority of the State Police

Traffic on Authority facilities is under the direct supervision and control of the New Jersey State Police who will enforce all statutory laws including the Authority's established "Regulations Relating to the Control of Traffic on the New Jersey Turnpike and Garden State Parkway", as they pertain to the Contractor as well as to the traveling public. A copy of the Regulations will be included with the Contract documents; additional copies will be issued upon request. The Boring Contractor shall familiarize themselves with and adhere strictly to the requirements of these Regulations and to the requirements of the Specifications.

If the State Police should observe any hazardous condition connected with or related to the Boring Contractor's operations, or of any violation of the Authority Regulations they will so notify the Boring Contractor and all work related to such hazardous condition or violation shall immediately be stopped and prompt remedial action shall be taken by the Boring Contractor, to the satisfaction of the NJTA Operations Department, before such work is resumed. All cost incurred as a result of discontinuing the work, and of all remedial action required, shall be borne entirely by the Boring Contractor without recourse against the Authority.

1.2. Traffic Permit

The Boring Contractor shall not commence work under this Contract, which would require occupation of or entry upon any Authority facility until they have been issued a Traffic Permit. Guidelines provided in the latest edition of NJTA Manual for Traffic Control in Work Zones shall be followed and the NJTA Traffic Permit Application (http://www.state.nj.us/turnpike/professional-services.html) completed. NJTA's online Traffic Closure Portal (http://tplc.newjerseyturnpike.com) shall be used for all lane closings, shoulder closings, and slowdowns requests.

At least ten working days prior to the time the Boring Contractor intends to occupy any portion of Authority facilities or intends to start any operations affecting Authority Traffic, and from time to time thereafter as directed by the Engineer, the Boring Contractor shall apply for a traffic permit and submit complete details of the intended methods to employ for the safe restriction to the movement of traffic required for their operations. These methods will be reviewed by the Engineer and when satisfactory, approved. Methods not approved will be returned for revision and shall be resubmitted for final review. Approval by the Engineer will be in the form of a Traffic Permit issued to the Boring Contractor by the NJTA Operations Department through the Engineer. No operations will be performed by the Contractor within 30 feet of a traveled lane until a Traffic Permit has been issued.

The Boring Contractor's methods submitted for approval shall include complete information, the data and/or sketches covering the following:

1. The nature and location of the work.

2. The proposed obstructions or other hazards to traffic, including all operations within 30 feet of a traveled lane.
3. The length of time during which it is anticipated that hazards or obstructions to traffic will exist.
4. The means proposed by the Boring Contractor for the protection of the public and their own personnel and equipment, including layouts and schedules showing the anticipated lane and shoulder closings, truck protection of traffic, and anticipated dates and rates of work.
5. The names and the day and night telephone numbers of the Boring Contractor's Superintendents assigned to the Project.

When work is not progressing in accordance with the Traffic Permit and when directed by the Engineer, the Boring Contractor shall revise the details of their plan of operations and resubmit them for approval. Such revisions, when approved by the Engineer, will form the basis of an Addendum to the Traffic Permit. Work affected by the revisions shall not be undertaken until an Addendum to the Traffic Permit has been issued.

If the approved methods of operations or revisions thereto, submitted by the Boring Contractor for a Traffic Permit are not strictly adhered to by the Contractor, the Engineer shall have the right to revoke the permit, and when so revoked, all work which, in the opinion of the Engineer, will affect the maintenance and protection of traffic, shall be summarily discontinued. The Permit will not be renewed and such work shall not be resumed until the Engineer is assured and satisfied that the Boring Contractor will perform the work in conformity with the approved methods of operations. The Boring Contractor shall have no claim against the Authority or Engineer for losses or delays caused by the revocation of the Permit.

1.3. <u>Maintenance and Protection of Traffic</u>

When any portion of the work under this Contract requires one or more traffic lane(s) and/or one or more shoulder of an Authority roadway be closed, such closings shall be made only at the times, to the limits, and in the manner that the movement of traffic by the closings will be at a minimum, and that all traffic moving on portions of the roadway not closed will be able to flow smoothly, and will be protected from all hazards attendant on the Boring Contractor's operations because of the closings, all in accordance with the requirements of the latest edition of NJTA Manual for Traffic Control in Work Zones. NJTA's online Traffic Closure Portal (http://tplc.newjerseyturnpike.com) shall be used for all lane closings, shoulder closings, and slowdowns requests.

The Boring Contractor is advised that Authority facilities are in continuous operation 24 hours a day, 7 days a week, and that the work under this Contract has been planned to cause no interference or as little interference to traffic as possible. The Boring Contractor shall, therefore, plan their operations to permit the continuous flow of traffic along the roadways.

It is the intent of the Contract to limit toll lane and shoulder closings to an absolute minimum and that work requiring closings be carried out in an expeditious manner.

The work for maintenance and protection of traffic is a joint Contractor and Authority effort and consists in general of furnishing and/or placing traffic protection devices for closing lanes and shoulders; furnishing personnel immediately and solely employed for the maintenance of the devices and protection of the traveling public; the transportation of devices to and from the site of the Project; placing or installing the devices; moving devices from one position to another as required; all in accordance with the Traffic Permit, the Plans and the General and Special Provisions of the Contract. Confirm with the Maintenance Department if the Authority will provide maintenance and protection of traffic.

No signs except traffic protection signs and traffic direction signs specified or as directed by the Engineer shall be erected by the Boring Contractor or their subcontractors on or near the Authority right of way.

The safety measures outlined and prescribed shall be considered elementary only and not necessarily sufficient in every instance to guarantee the protection of the traveling public. Compliance with the safety measures and precautions prescribed in the Specifications and on the Plans shall not relieve the Boring Contractor of responsibility for taking all necessary measures to protect and safeguard the public, nor relieve them of responsibility for the installation of adequate safety measures and for the protection of the traveling public and their own personnel on Authority roadways and premises, shall rest with the Boring Contractor. The cost of safety measures for which payment is not specifically provided under scheduled items in the Proposal, shall be included in the prices bid for the various items scheduled in the Proposal.

1.3.1. Lane and Shoulder Closings

(a). Condition and Situation Requirements

The Boring Contractor’s personnel, vehicles or equipment shall not occupy any part of a toll lane, through lane, ramp, or shoulder until the lane or shoulder has been closed.

The Boring Contractor’s personnel, vehicles or equipment shall not occupy any area within 30 feet from the outside edge of a shoulder where there is no guide rail (typ.) until the shoulder has been closed. The storage of materials and equipment will be permitted within the Authority right of way only at specific locations to be designated by the Engineer which shall be not less than 30 feet from outside edge of shoulder or behind guide rail (typ.).

Whenever any equipment occupying a shoulder or through lane and not behind barrier curb will be within 3 feet of a traveled lane or will come within three feet when operated (such as a tractor, or a crane swinging), the lane adjacent to the shoulder shall also be closed.

Materials or equipment shall not be stored in a closed lane or shoulder unless protected by barrier curb.

(b). Times for Closings

Because of heavy traffic during morning and evening commuter rush hours, on weekends, over holidays, and during the summer vacation periods (between Memorial Day and Labor Day) the times or hours when a toll lane or lanes may be closed and work requiring toll lane closings may be performed, are limited.

Toll Lane Closings Permitted. Toll lanes may be closed, and work requiring toll lanes to be closed may be performed only during the times prescribed in the Special Provisions of the Contract.

Shoulder Closings Permitted. Shoulder closings by use of cones, as necessitated by work in progress, will be permitted at any time except that simultaneous closing of both the right and left shoulder of a roadway will not be permitted.

Work requiring the use of barrier curb shall be completed at the earliest time so that prompt removal of the curb can be accomplished.

Emergency Closings. When it becomes necessary, in the opinion of the Chief Engineer, to make prompt repairs to work in progress or to other facilities that are damaged, the lanes will be closed. In such event the Boring Contractor shall provide all the materials and manpower necessary, and shall work continuously on a 24 hour per day basis to complete the emergency repairs and again make all lanes available to use by public traffic. Compensation for emergency repairs of damage beyond the Boring Contractor's control will be paid on a cost-plus basis as specified in the New Jersey Turnpike Authority's Standard Specifications or on such other basis as agreed upon by the Contractor and the Engineer. All costs incurred as a result of emergency repairs of damage caused solely by the Contractor's procedures shall be borne entirely by the Contractor.

(c). Number and Length

During permissible lane closing hours, no more than one traffic lane may be closed at any one time in any one work area unless multiple traffic lane closings are permitted in the Special Provisions of the Contract.

All shoulder closings shall be of the shortest overall length necessary to protect traffic from a hazardous condition. It is essential that as much shoulder as possible be kept open for use by disabled vehicles.

(d). Methods

Toll lanes and shoulders shall be closed in accordance with the Specifications and with the typical closing procedure and traffic protection devices shown on the New Jersey Turnpike Authority Standard Drawings.

The Boring Contractor shall give the Engineer 48-hour prior notice of the proposed time to place or remove any toll lane closing.

The traffic protection devices (cones and/or pylons) for closing a toll lane or shoulder shall always be set up progressively in the direction of traffic from a truck equipped with not less than two approved six inch diameter flashing vehicle lights to warn traffic, and with the truck traveling in the lane or shoulder to be closed. The protection devices shall always be removed in the reverse order by the truck backing up in the closed toll lane or shoulder.

The Boring Contractor's personnel shall, while working on foot, wear a sleeveless vest the same as that specified below to be worn.

1.3.2. Movement of Contractor's Vehicles, Equipment and Personnel

(a). General

Pedestrians are not allowed on Authority roadways at any time; the Boring Contractor's employees shall not walk across any Authority roadway, nor walk along any Authority roadway except within areas coned off or otherwise closed to the traveling public.

The Boring Contractor shall be responsible for transporting all their personnel, in accordance with N.J.S.A. 39.4-69-Riding on Part Not Intended for Passengers Prohibited, to and from enclosed or closed-off work areas. Personal vehicles will not be permitted to park anywhere within Authority or private properties except in areas designated by the Engineer. Whenever the Boring Contractor's vehicles operate on any Authority roadway or ramp pavement which is open to traffic, travel shall always be with and not across or against traffic.

Whenever the Boring Contractor intends to transport oversize or slow moving equipment, or any equipment whose movement may be disruptive to the traveling public, on Authority roadways open to the public, they shall first notify the Engineer or the Engineer's duly authorized representative at least 24 hours in advance of the intended move and the Engineer will establish the time and the route to be taken. At least two approved flashing vehicle lights shall be mounted on all slow moving vehicles.

Vehicles shall enter and leave work areas in a manner which will not be hazardous to or interfere with traffic. During permissible times for lane closings or shoulder closings, automobiles operated solely for the transportation of supervisory personnel, or approved GFRs will be allowed access to the work site provided such vehicles are operated in a safe manner.

Vehicles shall not park or stop in roadways or on shoulders except within areas of toll lanes or shoulders coned off or otherwise closed to the traveling public.

Unless otherwise specified the Boring Contractor's vehicles will not be permitted to use Z-turns, median U-turns, grade separated U-turns, or make U-turns across the median or in any Toll Plaza area. Any vehicle making an illegal turn will be subject to a summons by the State Police.

When, in the opinion of the Engineer, the security of the Authority roadways might become endangered by an operation of the contractor, their subcontractors or suppliers which would permit unauthorized entry to or exit from Authority property, the Boring Contractor shall take immediate measures to restore the security of the Authority right of way.

(b). Vehicle Access to Work Areas

The Boring Contractor's vehicles entering or leaving a work area via the Authority roadways shall be operated in a safe manner without creating any hazard or danger to the traveling public. They shall leave and enter the traffic stream at designated points, as shown on the Plans, or as specified herein, or as directed by the Engineer.

Delivery of materials or personnel and movement of vehicles and equipment, into and out of a work area via the Authority roadways shall be made only during the times for closings prescribed above.

(c). Traffic Protection Devices

Whenever the Boring Contractor's work requires closing of any toll lane or shoulder, the Authority will furnish, at no cost to the Boring Contractor certain traffic protection devices required for the Project. These devices will be identified and listed in the Special Provisions of the Contract.

1.4. Authority's Utilities:

The Plans indicate the locations of some subsurface structures within the vicinity of the proposed borings. The Boring Contractor shall not proceed with their work at any one boring location until diligent inquiries have been made at the office of the Engineer, utilities and private companies and municipal authorities, to determine the existence and exact locations of subsurface structures. The Boring Contractor shall also comply with the State's Underground Facility Protection Act and notify the State's One Call System before performing any work. The Boring Contractor shall exercise extreme care in accurately locating all utilities and in carrying out operations, and shall be solely responsible for any damages caused to utilities and to the facilities affected by such utility damage, whether such utilities are shown on any available plan or not.

The Boring Contractor shall fill all holes caused by their operations and shall take every precaution against injuring paving, utilities, or private or public property, and shall promptly repair, at their own expense and to the satisfaction of the Engineer and the owners, any damage to such paving, utilities and property caused by their operations. This shall also include sodding of any areas where the grass is damaged.

Upon completion of the Boring Contractor's operations at each site, they shall remove their equipment therefore, including pulling all casing and shall clear the area of all debris and restore it to the condition existing before the start of their operations.

APPENDIX B1
Modified Burmister Soil Identification System

MODIFIED BURMISTER SOIL IDENTIFICATION SYSTEM

IDENTIFICATION OF SOILS

A. Object

Included herein is the Modified Burmister System for identification of soils that is to be used by the GE and GFR. It provides a concise and accurate description of the soil, yet is simple enough to determine the soil components by visual identification.

This system provides a description of the granular materials, Silt, Sand and Gravel, that is based upon particle size. The description of cohesive soils is based upon the plasticity. The criteria for soil identification using this system are noted later with examples.

B. Modified Burmister Soil Identification System

Following in outline form, are the criteria for the identification of soils. Figure B1-1 summarizes this system and Figure B1-2 shows the standard semi-log graph used for graphic presentation of soils identifications. The plot gives the Percentage Finer by Weight vs. Grain Size in Millimeters. Beneath the graph are the particle size limits of the soil components used for the identification system.

1. Particle Size Limits for Soil Components.

(a) Cobbles and Boulders: Greater than 3 inch diameter (76.2 MM)

(b) Gravel: 3 inch diameter (76.2 MM) to No. 10 sieve (2.0 MM)

(c) Sand: No. 10 Sieve (2.0 MM) to No. 200 Sieve (0.074 MM)

(d) Silt: Material passing the No. 200 Sieve (0.074 MM) of a non-plastic nature

(e) Clay: Material passing the No. 200 Sieve (0.074 MM) of plastic nature

(f) Miscellaneous: Materials such as mica, shells, organic silt, peat, decomposed bedrock in place, etc. These materials are described per se, without regard to grain size.

2. Particle Size Limits for Granular Soil Functions.

The above mentioned constituents in Categories b, c and d are further sub-divided into coarse, medium and fine components.

Following is a list of the component parts of the above mentioned constituents and their size limits:

(a) Gravel Coarse: Less than 3 inches (76.2 MM) Greater than 1 inch (25.4 MM)

Medium: Less than 1 inch (25.4 MM) Greater than 3/8 inch (9.52 MM)

Fine: Less than 3/8 inch (9.52 MM) Greater than No. 10 Sieve (2.0 MM)

(b) Sand - Coarse: Less than No. 10 Sieve (2.0 MM) Greater than No. 30 Sieve (0.59 MM)

Medium- Less than No. 30 Sieve (0.59 MM) Greater than 60 Sieve (0.25 MM)

Fine: Less than No. 60 Sieve (0.59 MM) Greater than No. 200 Sieve (0.074 MM)

(c) Silt - Coarse: Material passing the No. 200 Sieve that is free draining in character

Fine: Material passing the No. 200 Sieve that is slow draining in character

The predominant fraction of any constituent can be noted by a plus sign. For example:

Mainly coarse Gravel = “Coarse + to fine Gravel” or

Mainly fine Sand = “Medium to fine + Sand”

3. Quantitative Description of Granular Components.

Soils that are essentially granular in character are identified by the classification system outlined above and are described on this basis and by the percentages by weight of each component part. Descriptive adjectives therefore precede the name of the soil component, which cover a rather narrow range of percentages of that component, by weight. Following is a list of descriptive adjectives and the percentage range of the total soil sample that they represent.

a. Not described	Less than 1%
b. Trace	1-10%
c. Little	10-20%
d. Some	30-35%
e. And	35-50%

4. Description of Granular Soil.

The major constituent of the soil sample is written in upper case letters. The minor components are written, along with their descriptive adjectives in lower case letters. Commas separate the various components. The color of the soil precedes the description.

Example: A soil sample is composed of the following percentages by weight of the various components.

Gravel (Medium and Fine only)	25%
Sand (Coarse, Medium & Fine Combined)	70%
Silt (Coarse & Fine Combined)	5%

The soil color is brown.

The written description of the soil is as follows:

“Brown coarse to fine SAND, some medium to fine Gravel, trace Silt”

5. Description of Cohesive Soil.

For soils that are essentially cohesive in character, the clay-silt fraction is described on the basis of plasticity, since silt and clay in intimate combination cannot be easily separated. The soil is described then, on the basis of its plasticity index, which is a function of the types of clay minerals present, and of the clay-silt ratio. Following is a list of terms used to describe the clay-silt fraction of a soil, along with its range of plasticity indices.

Description	Plasticity Index
Silt	0
Clayey Silt	1 -5
Silt & Clay	5 -10
Clay & Silt	10-20
Silty Clay	20 - 40
Clay	Greater than 40
Organic Clayey Silt,	An organic soil,
Organic Silt & Clay, etc.	usually black in color.

The description, based on plasticity index, is the same as for ordinary clays and silts.

Example: A soil sample contains the following percentage by weight of these soil components:

Gray Gravel	15%	(fine Gravel only)
Gray Sand	30%	(all components)
Gray Silt-Clay	50%	(Plasticity Index of 15)
Shell Material	5%	

The above soil sample is described as follows:

"Gray CLAY & SILT, some coarse to fine Sand, little fine Gravel, trace Shells."

6. Shorthand - For Field Use Only

Symbol	Word
C	Clay
$	Silt
S	Sand
G	Gravel
O$	Organic Silt
c	Coarse
m	Medium

Symbol	Word
f	Fine
t	Trace
l	Little
s	Some
and	And

C. Visual Identification of Soils.

To facilitate visual identifications of soils in the field, the following methods are recommended. The use of these methods will provide easy and accurate identifications of soils with a little practice.

The best approach is to identify each sample by following a series of steps, as outlined below:

1. Color

   Determine the color or colors if the sample is mottled.

2. Gravel Content

   Determine the percent of the total sample that is gravel by separating out the gravel and estimating by eye. Allowance should be made for fine gravel that was missed when picking over the sample.

3. Gravel Gradation

   The size of the largest piece of gravel should be determined. Use an average dimension, not the maximum dimension. Estimate by eye the predominant size. If there is a predominant size, note with a plus sign as shown in Section 2.

4. Sand Content

   After separating the gravel, take a pinch of the remaining soil and rub it between the fingers and thumb. The presence of coarse and medium sand can be noted by a gritty feeling. By rubbing in the palm of the hand with a finger, the soil can be dried. The sand grains can then be distinguished and the percentage of sand in the Sand-Silt-Clay portion of the sample noted. This percentage should be corrected for the whole sample, to include the gravel, as illustrated below.

Gravel separated out	30%
Remaining Sand-Silt-Clay portion of sample	70%
Sand content of Sand-Silt-Clay portion of sample	50%
Sand content of whole sample = 50 x 70	35%

5. Sand Gradation

   By drying a pinch of the sample in the palm of the hand as previously noted, the gradation of the sample can be determined. An easy way to distinguish between fine Sand and Silt is that the individual grains of the Sand can be distinguished by eye whereas the individual grains of Silt cannot.

6. Silt-Clay Content

Moisten a 1/2 inch diameter ball made from the Sand-Silt-Clay portion of the sample. Shake this ball in the palm of the hand and notice if moisture appears on the surface of the ball. If moisture appears, no clay is present; if no moisture appears, there is clay in the sample. If moisture appears on the surface of the ball, gently squeeze the ball between the forefinger and the thumb till the moisture disappears, then release. If moisture reappears, the sample is coarse Silt; if no moisture appears, or appears very slowly the sample is coarse and fine Silt.

For soils that contain both Silt and Clay, or Clay only, the plasticity index can be estimated by a rather simple test. A 1/2 inch diameter ball of the soil is made. The consistency or strength of the ball is brought to that of modeling clay by drying or adding moisture to the ball of soil. A piece of the ball is then rolled into a thread on a flat surface, and the diameter at which the thread crumbles indicates the Clay-Silt content, as noted below:

Thread Diameter	Plasticity Index	Identification
¼”	0	SILT
¼”	1- 5	Clayey SILT
1/8”	5-10	SILT & CLAY
1/16”	10 - 20	CLAY & SILT
1/32”	20 - 40	Silty CLAY
1/64”	40	CLAY

It is helpful to have a 1/2 inch diameter ball of modeling clay available when performing the test. By squeezing the clay in one hand, and the soil in the other with the index finger and thumb, a good check on the consistency can be made by comparison.

7. Another test to help determine the soil plasticity is the resistance of a piece of dried soil to crushing by finger pressure. A soil specimen is molded to the consistency of putty, adding water if necessary. The moist pat of soil is allowed to dry (in oven, sun, or air) and is then crumbled between the fingers. Soils with slight dry strength crumble readily with very little finger pressure. Silt soils have almost no dry strength. Organic soils and clayey silt soils of low plasticity have slight dry strength. Soils of medium dry strength require considerable finger pressure to powder the sample. Silt and clay and clay and silt soils exhibit medium dry strength. Soils with high dry strength can be broken but cannot be powered by finger pressure. High dry strength is indicative of silty clay or clay soils as well as some organic clays of high plasticity.

An additional aid in classifying soil types that consist of various components of sand, silt and clay is the sedimentation test in a glass jar or test tube. The various percentages of the components are estimated from the sedimentation test which is performed as follows:

A small quantity of soil is placed in the bottom one-half or one-quarter of a test tube or jar such that it is compact without any large air spaces. The height of sample is then noted. A supply of water is then added to the test tube and the test tube is vigorously shaken until the soil sample is entirely in suspension. All sand particles should settle out of the suspension within 30 seconds after the shaking has stopped. The silt particles will settle out of the suspension within 30 minutes after the shaking has stopped. The relative depth of sand and silt to the original depth of sample will provide for an estimate of the percentage of sand and silt in the total sample. The clay will still be all in suspension at the end of 30 minutes and it may take up to 24 hours for all the clay to settle out. Therefore, there is no advantage in performing the test more than 30 minutes.

Figure B1-1

MODIFIED BURMISTER SOIL IDENTIFICATION METHOD

1. SOIL MATERIAL Composition, Gradation, and Plasticity Characteristics

a) Soil Components and Soil Fractions

Sieve	3"	1"	3/8"	No. 10 2 mm	No. 30	No. 60	No. 200 0.076 mm	0.02 mm
Granular Components Fractions	GRAVEL coarse	medium	fine	SAND coarse	medium	fine	SILT coarse	fine
Clay Soil Components							CLAY-SOIL Defined and named on a Plasticity Basis	

b) Identifying Terms for Granular Soils
Composition and Proportion Terms for Components

Component		Proportions Terms	Defining Range of Percentages
Principal Components - GRAVEL, SAND, SILT (all capitals)			
Minor Components -	Gravel	and	35 to 50%
	Sand	some	20 to 35%
	Silt	little	10 to 20%
		trace	1 to 10%

Gradation Terms for Granular Soils				ORGANIC SOILS
coarse	to	fine	all fractions are more than 10%	Plasticity Basis, as
coarse	to	medium	fine less than 10%	
medium	to	fine	coarse less than 10%	Organic SILT, H. PI
medium			coarse and fine less than 10%	
fine			coarse and medium less than 10%	Organic SILT, L. PI

PLUS or minus signs used to indicate nearer upper or lower limits.

c) Identifying Terms for CLAY, Soils. Plasticity Basis for Combined Silt
And Clay Components, Expressing the Relative Dominance of Clay.

Overall plasticity	Plasticity Index			Principal Component	Minor Component
Non-Plastic		0		SILT	Silt
Slight	1	to	5	Clayey SILT	Clayey Silt
Low	5	to	10	SILT & CLAY	Silt & Clay
Medium	10	to	20	CLAY & SILT	Clay & Silt
High	20	to	40	Silty CLAY	
Very High	more than		40	CLAY	

Example: Soil 60% coarse to fine Sand, 25% medium to fine Gravel, 15% Clayey Silt and color-brown.

Identification: Br. Coarse to fine SAND, some medium to fine Gravel, little Clayey Silt.

Reference:
1) D.M. Burmister, "Principles and Techniques of Soil Identification" 29th Highway Research Board Proceedings, 1949.
2) "Identification and Classification of Soils - An appraisal and Statement of Principles," ASTM Special Publication No. 113, 1951

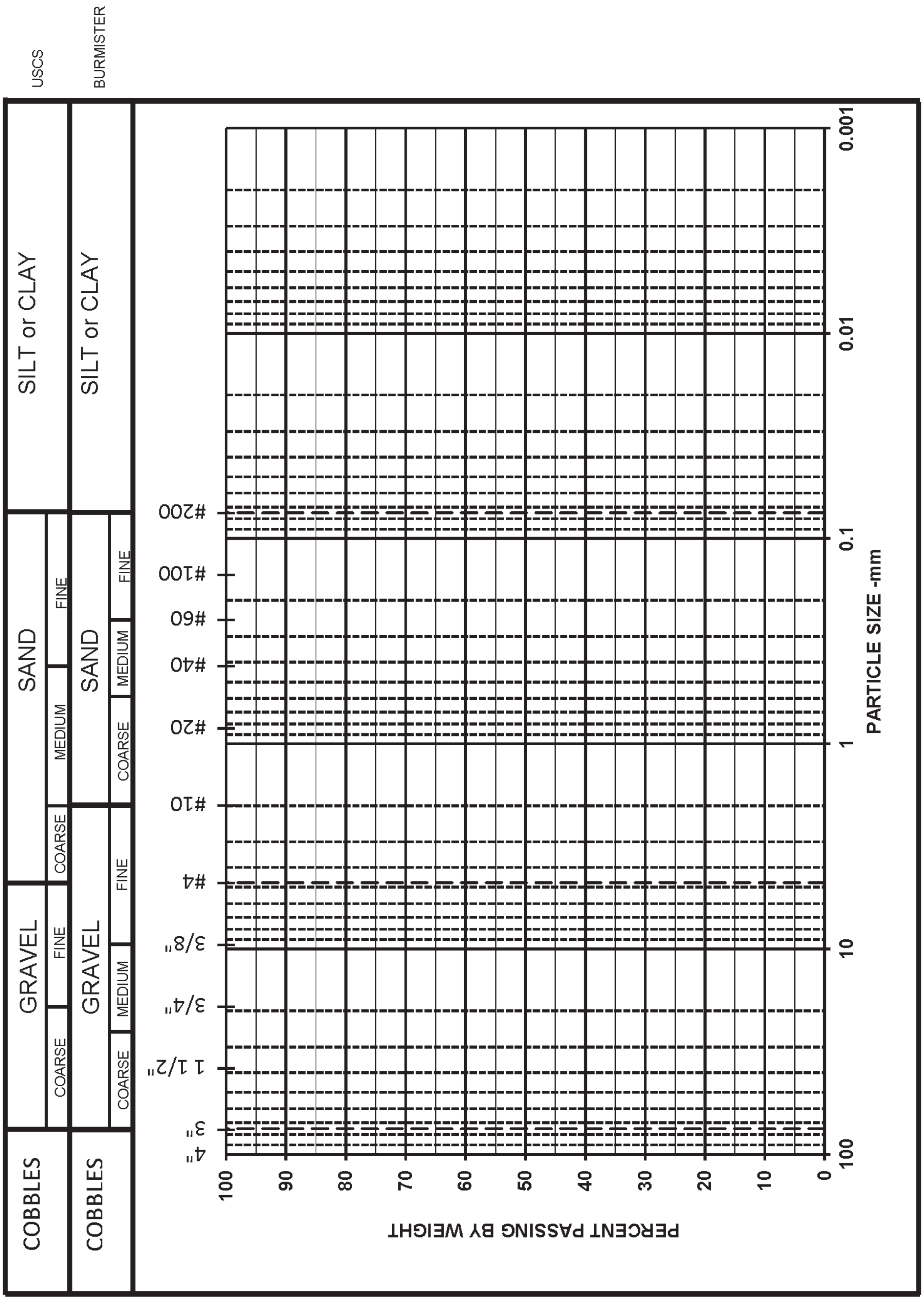

USCS
BURMISTER
SILT or CLAY
SILT or CLAY
SAND
FINE
MEDIUM
COARSE
SAND
FINE
MEDIUM
COARSE
GRAVEL
FINE
COARSE
GRAVEL
FINE
MEDIUM
COARSE
COBBLES
COBBLES
#200
#100
#60
#40
#20
#10
#4
3/8"
3/4"
1 1/2"
3"
4"
0.001
0.01
0.1
1
10
100
PARTICLE SIZE -mm
100
90
80
70
60
50
40
30
20
10
0
PERCENT PASSING BY WEIGHT

Figure B1-2

APPENDIX B2
Rock Identification System

ROCK CLASSIFICATION AND LOGGING

IDENTIFICATION OF ROCKS

A. Object

The basic objectives of classifying and logging core are to provide accurate and concise record of the important geological and physical characteristics of engineering significance. Data reported in geologic logs not only must be accurate, consistently recorded, and concise, but also must provide quantitative and qualitative descriptions.

Most engineering rock shear strength parameters, published in recent AASHTO and FHWA documents, are functions of the two rock classification systems Rock Mass Rating (RMR) Geomechanics Classification and Geological Strength Index (GSI) Classification Systems.

Core logs are to be prepared such that necessary information to estimated shear strength parameters of rock. In addition, further need for additional exploration or testing, final design criteria, treatment design, methods of construction, and the evaluation of structure performance may depend on core logs.

Rock core logs with adequate descriptions of recovered cores and samples shall be prepared through visual or hand specimen examination of the core with the aid of simple field tests using the following classification.

The order of classification terminology for identification of rock core samples will follow the following order:

Core Data	Run	Elevation	
		Sample Number	
		Depth Range	
		Core Time	
	Water	Gain/Loss	
	Rock Core	Number of Pieces	
		Recovery	
		RQD	
Rock Description	General	Rock Type	
		Color	
		Weathering	
		Strength	
	Fabric	Grain Size	
		Grain Shape	
	Structure	Fracture Spacing	
		Bedding Spacing	
		Foliation Angle	
Discontinuity Description	Planes	Fracture Type	
		Fracture Aperture	
	Joints	Joint Surface	Shape
			Roughness
		Joint Infilling	
		Water Content	
		Joint Roughness Coefficient	
Lab Testing	Strength		
	Unit Weight		
	Other Tests		

B. Rock Classification

1. Rock Core Data

(a) Recovery

The core recovery is the length of rock core recovered from a core run. The recovery ratio is the ratio of the length of core recovered to the total length of the core drilled on a given run, expressed as a percentage. Core length should be measured along the core centerline. When the recovery is less than the length of the core run, the non-recovered section should be assumed to be at the end of the run unless there is reason to suspect otherwise (e.g., weathered zone, drop of rods, plugging during drilling, loss of fluid, and rolled or re-cut pieces of core). The correct procedure for measuring the recovery is illustrated in Figure B2-1.

(b) Rock Quality Designation (RQD)

The RQD is a quantitative measure that represents a modified core recovery percentage. By definition the RQD is the sum of the lengths of all pieces of sound core over 4 inches long divided by the length of the core run (Deere 1963). The correct procedure for measuring RQD is illustrated in Figure B2-1.

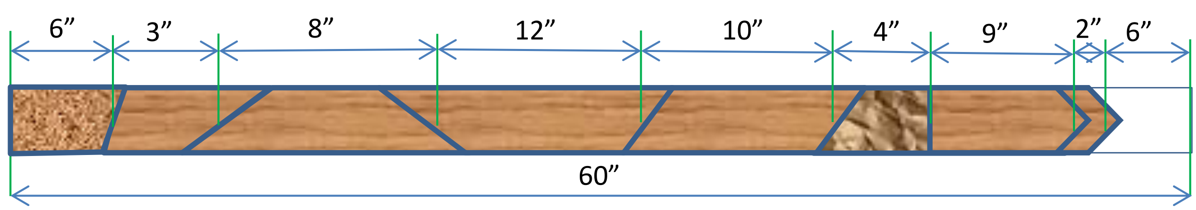


Figure B2-1

Core Run = 60”

Recovery = 6” + 3” + 8” + 12” + 10” + 4” + 9” + 2” = 54”

Recovery % = 54”/60” x 100 = 90 %

RQD = 8” + 12” + 10” + 9” = 39”

RQD % = 39”/60” x 100 = 65 %

2. Rock Description

(a) General

i. Rock Type

Division	Class	Type
Igneous	Coarse-Grained (Intrusive)	Granite
		Diabase
	Fine-Grained (Extrusive)	Basalt
Sedimentary	Calcareous	Limestone
		Dolomite
	Siliceous	Conglomerate
		Sandstone
		Quartzite
		Claystone
		Mudstone
		Siltstone
		Argillite
		Shale
		Chert
Metamorphic	Foliated	Slate
		Phyllite
		Schist
		Amphibolite
		Hornfers
	Nonfoliated	Marble
		Metaquartzite
		Serpentinite
		Gneiss

ii. Color

Record the color of rock core in wet condition using terminology consistent with those used for logging of soil samples. Use descriptive terminology such as light, dark, banded, mottled, streaked and stained if appropriate for a more complete description of rock mass.

iii. Weathering

Weathering can be indicated visually by changes in color and texture of the body of the rock, color and condition of the fracture filings and surfaces, physical properties such as hardness based on the guidelines provided in the tables below.

Term	Description	Grade
Unweathered	No visible sign of rock material weathering, perhaps slight discoloration on major discontinuity surfaces.	I
Slightly Weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering and be somewhat weaker externally than in its fresh condition.	II
Moderately Weathered	Less than half of the rock material is decomposed and/ or disintegrated to a soil. Fresh or discolored rock is present either as a continuous framework or as corestones.	III
Highly Weathered	More than half of the rock material is decomposed and/ or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones.	IV
Completely Weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.	V
Residual Soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.	VI

Note: See table below for definition of rock weathering terminology used in the above rock weathering descriptions.

Term	Discoloration Extent	Fracture Condition	Material Condition	Grade
Fresh	No discoloration or oxidization	Slight to no discoloration	Unchanged	I
Slightly Weathered	Discoloration/oxidation penetrates a short distance away from fracture	Discolored; may contain soil or altered mineral filling	Partial Discoloration	II
Moderately Weathered	Significant discoloration/ oxidation. Penetrates a significant distance	Discolored; may contain soil or altered mineral filling	Partial to complete discoloration; Parent rock/ minerals beginning	III

Term	Discoloration Extent	Fracture Condition	Material Condition	Grade
	away from fracture		to decompose into soil	
Completely Weathered	Throughout	-	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact	IV
Residual Soil	Throughout	-	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.	V

(Modified From U.S. Department of the Interior, Bureau of Reclamation (2001), *Engineering Geology Field Manual, 2nd Edition, Chapter 4, table 4.4*, and Brown, E.T., 1981, Rock Characterization Testing and Monitoring, ISRM Suggested Methods, p., 31)

Definition of Rock Weathering Terminology

Term	Definition
Fresh	No visible sign of weathering of rock material.
Discolored	The color of the original fresh rock material is changed. The degree of change from the original color should be indicated. If the color change is confined to particular mineral constituents, this should be mentioned.
Decomposed	The rock is weathered to the condition of a soil in which the original material fabric is still intact, but some or all of the mineral grains are decomposed.
Disintegrated	The rock is weathered to the condition of a soil in which the original material fabric is still intact. The rock is friable, but the mineral grains are not decomposed.

iv. Strength

Strength of rock can be described based on field identification method or based on laboratory uniaxial compressive strength based on the guidelines in the following table.

Description	Field Identification	Approximate Range of Uniaxial Compressive Strength (psi)	Grade
Extremely Weak Rock	Indented by thumbnail	36 - 150	R0
Very Weak Rock	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife.	150 - 725	R1
Weak Rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer.	725 - 3,600	R2
Medium Strong Rock	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer.	3,600 - 7,250	R3
Strong Rock	Specimen requires more than one blow of geological hammer to fracture it.	7,250 - 14,500	R4
Very Strong Rock	Specimen requires many blows of geological hammer to fracture it.	14,500 - 36,000	R5
Extremely Strong Rock	Specimen can only be chipped with geological hammer.	>36,000	R6

(b) Fabric

i. Grain Size

Criteria for Sedimentary Grain Size

Description	Criteria	Group
Very Fine-Grained	Individual grains cannot be distinguished with naked eye; < 0.0029” (0.075 mm)	VF
Fine-Grained	Not visible to barely visible with naked eye; 0.0029”-0.0165” (0.075 mm – 0.425 mm)	FG
Medium grained	Barely to easily visible with naked eye; 0.0165”-0.0787”	MG

	(0.425 mm – 2.0 mm)	
Coarse-Grained	Easily visible; 0.0787”-0.187” (2.0 mm – 4.75 mm)	CG
Very Coarse-Grained	Grains sizes are greater than popcorn kernels ; > 0.187” (4.75 mm)	VC

Criteria for Igneous and Metamorphic Grain Size

Description	Criteria	Group
Pegmatitic	Average crystal size greater 3/8”	P
Coarse-grained	Average crystal size (A) is 3/16” < A <3/8”	CG
Medium Grained	Average crystal size (A) is 1/32” < A < 3/16”	MG
Fine-grained	Average crystal size (A) is 1/250” < A < 1/32”	FG
Aphanitic	Crystal size not visible with naked eye	A

(From Soil and Rock Logging, Classification, and Presentation Manual, 2010, State of California Department of Transportation Division of Engineering Services; Geotechnical Services, Figure 2-27, pg. 27)

ii. Grain Shape

Criteria for Grain Shape

Description	Criteria	Group
Angular	Showing very little evidence of wear. Grain edges and corners are sharp. Secondary corners are numerous and sharp	AG
Subangular	Showing definite effects of wear. Grain edges and corners are slightly rounded off. Secondary corners are slightly less numerous and slightly less sharp than in angular grains	SA
Subrounded	Showing considerable wear. Grain edges and corners are rounded to smooth curves. Secondary corners are reduced greatly in number and highly rounded	SR
Rounded	Showing extreme wear. Grain edges and corners are smoothed off to broad curves. Secondary corners are few in number and rounded	RD
Well Rounded	Completely worn. Grain edges or corners are not present. No secondary edges or corners are present	WR

(c) Structure

Fracture is a terminology used for describing any natural breaks excluding shear zones in geologic material. Bedding features provide the rock anisotropic properties or represent potential failure surfaces. Joint (bedding or foliation joints), bedding plane separation due to stress relief, random fracture (not belonging to a joint set), and mechanical breaks during sampling can be described in terms of fracture/bedding spacing and fracture surface type based on the tables below.

i. Fracture Spacing

Discontinuity Type	**Description**	**Spacing**	**Group**
Fracture Spacing (Joints, Faults, Other Fractures)	Extremely Close	<3/4 in	ECF
	Very Close	3/4 in-2-1/2 in	VCF
	Close	2-1/2 in-8 in	CF
	Moderate	8 in-2 ft.	MF
	Wide	2 ft. - 6 ft.	WF
	Very wide	6 ft. -20 ft.	VWF

ii. Bedding Spacing

Discontinuity Type	**Description**	**Spacing**	**Group**
Bedding Spacing (may include Foliation or Banding)	Thinly Laminated	<1/4 in	TLB
	Laminated	1/4 in -1/2 in	LB
	Very Thin	1/2 in-2 in	VTNB
	Thin	2 in-1 ft.	TNB
	Medium	1 ft. - 3 ft.	MB
	Thick	3 ft.- 10 f	TKB
	Very Thick/Massive	> 10 ft.	VTKB

iii. Foliation Angle

3. Discontinuity Description

(a) Planes

i. Fracture Type

Fracture	Description	Group
Joint	A discontinuity in which there has been no to very little observable relative movement. Joints may be open, healed, or filled; and surfaces may be striated due to minor movement	J
Fault	A discontinuity along which there has been an observable amount of displacement. Faults are rarely single planar units; normally they occur as parallel or sub-parallel sets of discontinuities along which movement has taken place to a greater or less extent.	Ft
Shear	A structural break where differential movement has occurred along a surface or zone of failure; characterized by polished surfaces, striations, slickensides, gouge, breccia, mylonite, or any combination of these.	Sh
Foliation	Parallel orientation of platy minerals or minerals banding in metamorphic rock	Fo
Vein	Fractures filled with secondary crystallization	V
Bedding	A separation along bedding planes after exposure due to stress relief or slaking parallel to the surface of deposition, which may or may not have physical expression. Fractures which are parallel to bedding are termed bedding joints or bedding plane joints.	B

ii. Fracture Aperture

Descriptor	Fracture Width (FW) (in)	Group
Tight	No visible Separation	T
Slightly open	<1/32	SO
Moderately Open	1/32 ≤FW ≤ 1/8	MO
Open	1/8 ≤ FW ≤ 3/8	O
Moderately Wide	3/8 ≤ FW ≤ 1	MW
Wide	1 ≤ FW	W

(From U.S. Department of the Interior, Bureau of Reclamation (2001), *Engineering Geology Field Manual, 2nd Edition, modified from Chapter 5 Table 5.5*)

(b) Joints

i. Joint Surface

Criteria for Describing Surface Shape of Joint

Shape	Description	Group
Planar		P
Undulating		U
Wavy	Waviness is the result from large scale Undulations contained within a rock mass	WU
Stepped		ST
Irregular	No Discernable pattern to surface of joint	IR

(This table should be used in conjunction with criteria for describing roughness of the surface of the joint for a more complete description of the fracture.)

Criteria for Describing Roughness of Surface

Roughness	Description	Group
Slickensided	Surface smooth, glassy finish with visual evidence of striations	SL
Smooth	Surface appears smooth and feels so to the touch	SM
Slightly Rough	Asperities on the discontinuity surfaces are distinguishable and can be felt	SR
Rough	Some ridges and side angle steps are evident, asperities are clearly visible, & discontinuity surface feels very abrasive	RG
Very Rough	Near vertical steps and ridges occur on the discontinuity surface	VR

ii. Joint Infilling

Joint Infilling Materials

Material	Identification	Notation
Chlorite	Light to dark green, grayish green. Slightly greasy texture. Predominately found in metamorphic rocks	Cl

Material	Identification	Notation
Calcite	Common infilling material found in a variety of colors. Will effervesce when exposed to weak acid solution. Low hardness that can be scratched by copper penny	Ca
Gypsum	Light colored mineral typically found in sedimentary rocks. No major physical identification characteristics and requires lab testing to identify	Gy
Mica	Found as dark colored (Biotite) or light colored (Muscovite). Can be peeled apart in thin sheets and has a lustrous appearance. Typically found in Metamorphic rocks	Mc
Iron-bearing Minerals	Typically occur as Hematite (He) and Magnetite (Mt). Common rock forming minerals that when exposed to moisture stains red to reddish brown. Typically found as staining on walls of discontinuities.	Fe
Pyrite	Yellowish Gray to gold in color. Sulfur bearing mineral typically found as weathered flakes with distinctive gold lustrous appearance	Py
Quartz	Most common rock forming mineral found in a large variety of colors. Hard mineral that can't be scratched by pocket knife	Qtz
Talc	Light colored mineral typically found in metamorphic rocks. Talc is very soft and can be scratched with fingernail. Greasy texture	Tl
Clay	See Appendix B-1 for identification	C
Silt	See Appendix B-1 for identification	M
Sand	See Appendix B-1 for identification	S
Gravel	See Appendix B-1 for identification	G

Descriptive Terminology for Strength of Joint Filling Material

Grade	Description	Field Identification	Approximate Range of Uniaxial Compressive Strength (psi)
S1	Very Soft Clay	Easily penetrated several inches by fist.	500
S2	Soft Clay	Easily penetrated several inches by thumb.	500 – 1,000
S3	Firm Clay	Can be penetrated several inches by thumb with moderate effort.	1,000 – 2,000
S4	Stiff Clay	Readily indented by thumb but penetrated only with great effort.	2,000 – 5,000
S5	Very Stiff Clay	Readily indented by thumbnail.	5,000 – 10,000

Grade	Description	Field Identification	Approximate Range of Uniaxial Compressive Strength (psi)
S6	Hard Clay	Indented with difficulty by thumbnail.	>10,000

Note: Grades S1 to S6 apply to cohesive soils such as clays, silty clays, and combination of silts and clays with sand, generally slow draining. If non-cohesive fillings are identified, qualitatively identify, e.g., fine sand.

iii. Water Content

Descriptive Terminology for Water content in Rock Fracture

Description	Criteria	Group
Dry	Fracture is dry tight with no evidence of previous water flow or staining.	D
Moist	The fracture filling (where present) is damp and evidence of minimal amounts of free water is visible.	M
Wet	The fracture has sizeable amounts of free water and filling material (where present) shows signs of piping and or washing out.	W

iv. Joint Roughness Coefficient

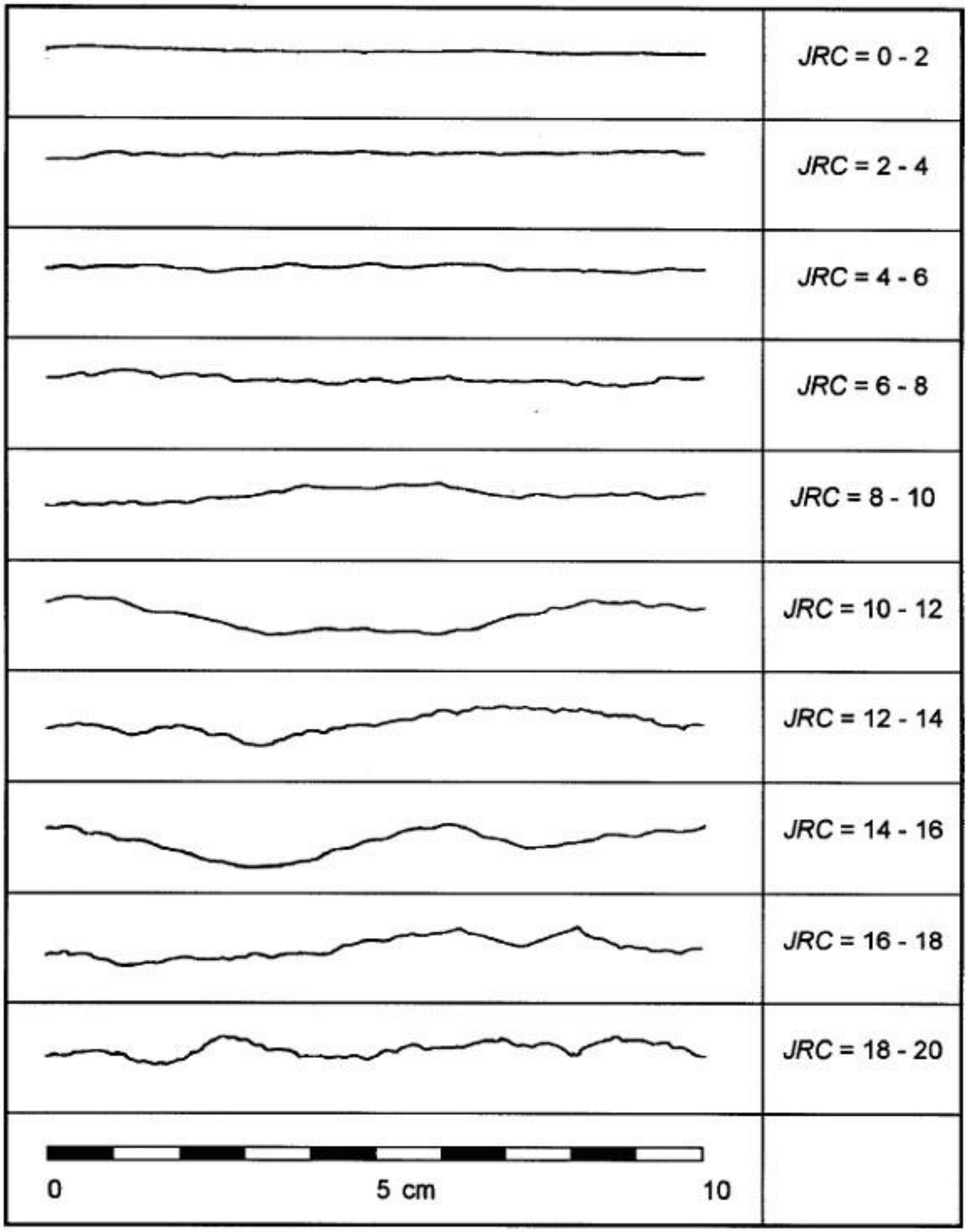


Figure B2-2

APPENDIX B3
USDA Soil Identification System

USDA SOIL CLASSIFICATION AND LOGGING

IDENTIFICATION OF SOILS

A. Object

The basic objectives of classifying and logging core are to provide accurate and concise record of the important geological and physical characteristics of engineering significance. Data reported in geologic logs not only must be accurate, consistently recorded, and concise, but also must provide quantitative and qualitative descriptions.

B. USDA Classification

1. Particle Size Definition

Soil	**Fraction**	**Actual Sizes**
Boulders		> 600 mm
Stones		600mm to 250 mm
Cobbles		250 mm to 76 mm
Gravel	course	76 mm to 20mm
	medium	20 mm to 5 mm
	fine	5 mm to 2mm
Sand	very coarse	2.0 to 1.0 mm
	coarse	1.0 mm to 0.5 mm
	medium	0.5 mm to 0.25 mm
	fine	0.25 mm to 0.10 mm
	very fine	0.10 mm to 0.05 mm
Silt		0.05 mm to 0.002 mm
Clay		< 0.002 mm

2. Soil Texture

This is the numerical proportion (percent by weight) of sand, silt and clay in soil.

Texture Classes/ Subclasses	**Definition**
Sands	More than 85% percent sand, the percentage of silt plus 1.5 times the percentage of clay is less than 15.
Coarse sand	*A total of 25 % or more very coarse and coarse sand and less than 50% any other single grade of sand.*
Sand	*A total of 25 % or more very coarse, coarse, and medium sand, a total of less than 25 percent very coarse and coarse sand, and less than 50% fine sand and less than 50 % very fine sand.*
Fine Sand	*50% or more fine sand: or a total of less than 25% very coarse, coarse, and medium sand and less than 50% very fine sand.*
Very Fine Sand	*50 % or more fine sand.*
Loamy Sands	Between 70 and 91 percent sand and the percentage of silt plus 1.5 times the

	percentage of clay is 15 or more; and the percentage of silt plus twice the percentage of clay is less than 30.
Loamy coarse sand	*A total of 25 % or more very coarse, and coarse sand and less than 50 % any other single grade of sand.*
Loamy sand	*A total of 25 % or more very coarse, coarse, and medium sand and a total of less than 25 % very coarse and coarse sand, and less than 50 % fine sand and less than 50 % very fine sand.*
Loamy fine sand	*50 % or more fine sand; or less than 50 % very fine sand and a total of less than 25% very coarse, coarse, and medium sand.*
Loamy very fine sand	*50% or more very fine sand.*
Sandy loams	7 to 20 % clay, more than 52% sand, and the percentage of silt plus twice the percentage of clay is 30 or more; or less than 7 % clay, less than 50 % silt, and more than 43% sand.
Coarse sandy loam	*A total of 25% or more very coarse and coarse sand and less than 50 % any other single grade of sand.*
Sandy loam	*A total of 30% or more very coarse, coarse, and medium sand, but a total of less than 25% very coarse and coarse sand and less than 30% fine sand and less than 30% very fine sand; or a total of 15 % or less very coarse, coarse, and medium sand, less than 30% fine sand and less than 30% very fine sand with a total of 40% or less fine and very fine sand.*
Fine sandy loam	*30% or more fine sand and less than 30 % percent very fine sand; or a total of 15 to 30% very coarse, coarse, and medium sand; or a total of more than 40 percent fine and very fine sand, one half or more of which is fine sand, and a total of 15% or less very coarse, coarse, and medium sand.*
Very fine sandy Loam	*30 % or more very fine sand and a total of less than 15% very coarse, coarse, and medium sand; or more than 40% fine and very fine sand, and more than half of which is very fine sand, and a total of less than 15% very coarse, coarse, and medium sand.*
Loam	7 to 27 % clay, 28 to 50 % silt, and 52 % or less sand.
Silt loam	50 % or more silt and 12 to 27 % clay, or 50 to 80 % silt and less than 12 % clay.
Silt	80% or more silt and less than 12 % clay.
Sandy clay loam	20 to 35 % clay, less than 28 % silt, and more than 45 % sand.
Clay loam	27 to 40 % clay and more than 20 to 46 percent sand.
Silt Clay loam	27 to 40 % clay and 20 % or less sand.
Sandy clay	35 % or more clay and 40 % or more sand.
Silty clay	40 % or more clay and 40 % or more silt.
Clay	40 % or more clay, 45 % or less sand, and less than 40 % silt.

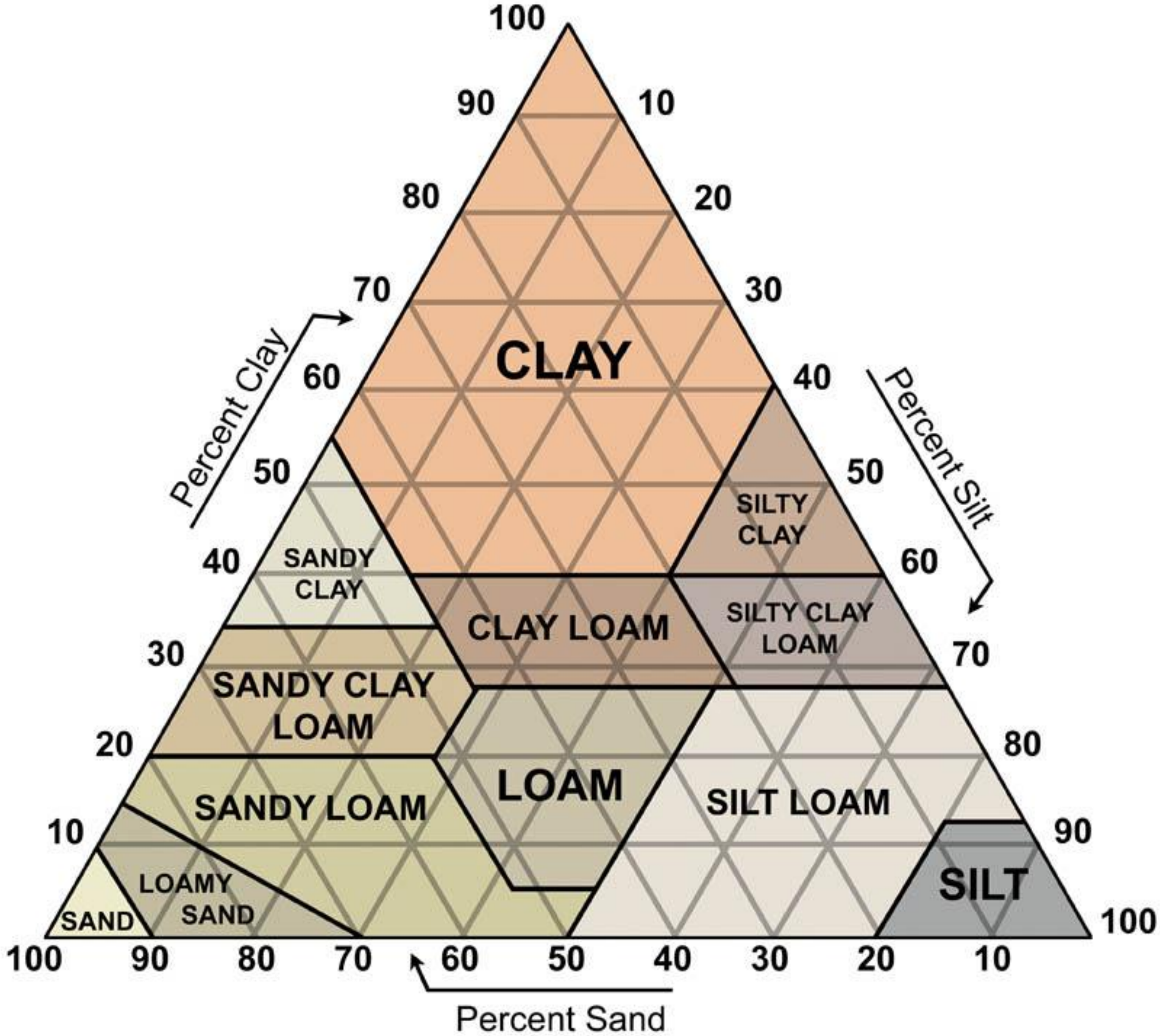


Figure B3-1

3. Grouping of Soil Texture Classes

General terms	Texture	Texture classes
Sandy soil materials	Coarse-textured	Sands (coarse sand, sand, fine sand, very fine sand) Loamy sands (loamy coarse sand, loamy sand, loamy fine sand, loamy very fine sand)
Loamy soil materials	Moderately coarse-textured	Coarse sandy loam, sandy loam, fine sandy loam
	Medium-textured	Very fine sandy loam, loam, silt loam, silt
	Moderately fine-textured	Clay loam, sandy clay loam, silty clay loam
Clayey Soils	Fine textured	Sandy clay, silty clay, clay

4. Rock Fragments- Size and Quantity

Fragment content % by volume	Rock fragment Modifier usage
<15	No texture adjective is used (noun only; e.g., loam)
15 to <35	Use adjective for appropriate size; e.g., gravelly
35 to <60	Use "very" with the appropriate size adjective; e.g., very gravelly.
60 to <90	Use "extremely" with the appropriate size adjective; e.g., extremely gravelly.

5. Soil Color

Identify the colors of the soil matrix with Munsell notation (Hue, Value, Chroma)

Munsell Notation	Consists of about 250 different colored papers or chips arranged on hue cards according to their Munsell notation. The color system uses three elements of color: hue, value and chroma.
Hue	Measure of the chromatic composition of light that reaches the eye.
	5 principal hues: red (R), yellow(Y), green (G), blue (B) and purple (P).
	Intermediate hues are yellow-red (YR), green-yellow (GY), blue-green (BG), purple-blue (PB), and red-purple (RP).
	Each of the 10 major hue is divided into 4 segments of equal visual steps. Yellow-red (YR) hue are identified as 2.5 YR, 5 YR, 7.5 YR, and 10 YR. The standard chart for soil has separate hue cards from 10 R through 5Y.
Value	Indicates the degree of lightness or darkness of a color in relation to a neutral gray scale.
	Pure black (0/), Gray (5/), white (10/). A card of the color chart for soil has a series of chips arranged vertically to show the lightest to the darkest shade of the hues.
Chroma	Indicates the degree of saturation of neutral gray by the spectral color. /0 Neutral colors to /8 as the strongest expression of color used for soil.
	The color chips are arranged horizontally by increasing chroma from left to right on the color card.
	At the extreme left of the card are symbols such as N6/. These are colors of zero Chroma. They have no hue and no chroma and range in values from black (N 2/) to white (N 8/). Gray is N 5/.

E.g., Pale brown 10 YR 6/3, Very dark brown 10 YR 2/2.

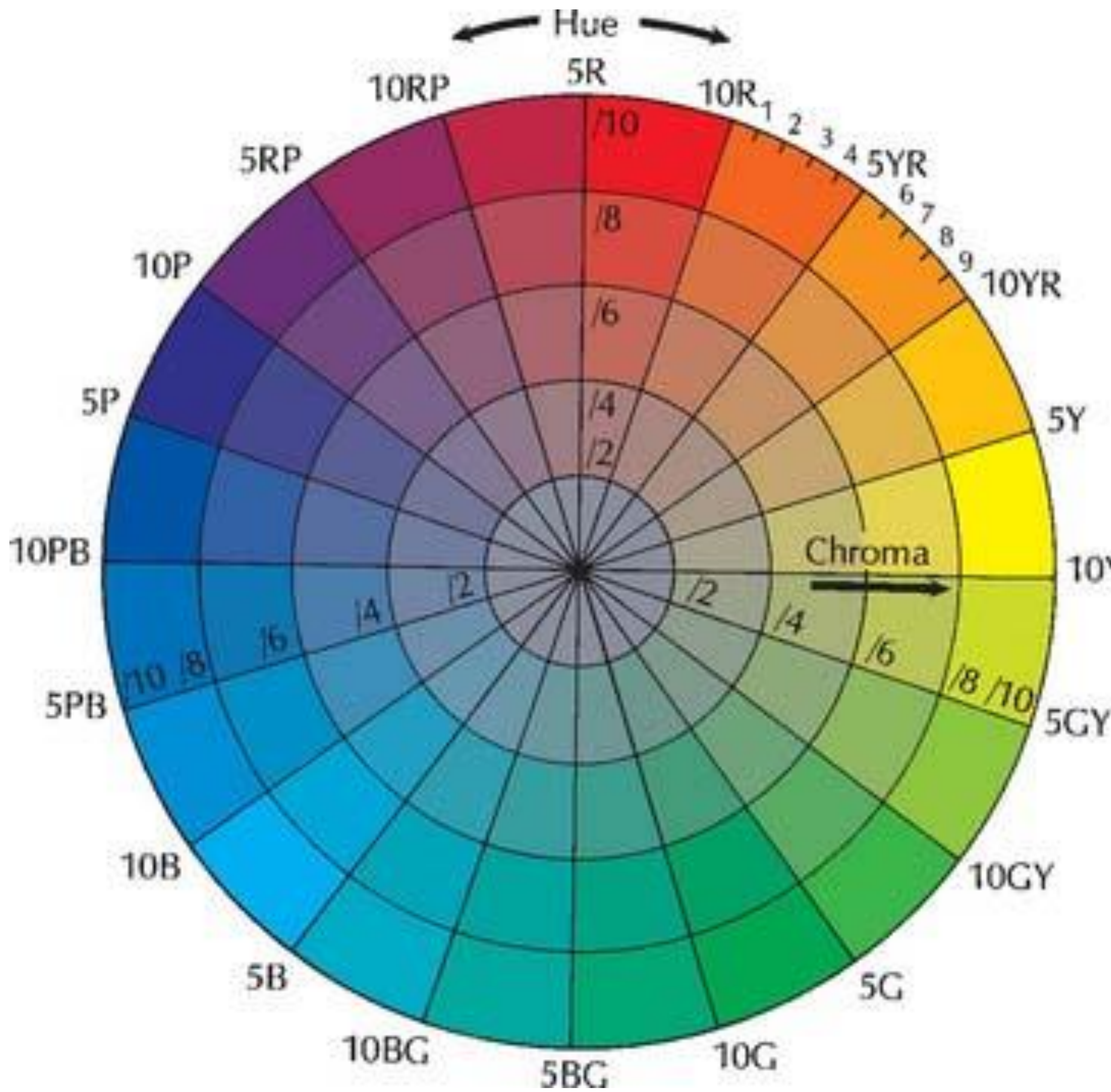


Figure B3-2

6. Mottling

Mottling refers to repetitive color changes that cannot be associated with compositional properties of the soil. Redoximorphic features are a type of mottling that is associated with wetness. Mottles are described by quantity, size, contrast, color and other attributes in that order.

Mottling	Measure	Composition
Quantity	few	less than 2%
	common	2 to 20%
	many	more than 20%
Size	fine	smaller than 5mm

Mottling	Measure	Composition
	medium	5 mm to 15 mm
	coarse	larger than 15 mm
Contrast	faint	Evident only on close examination. Faint mottles commonly have the same hue as the color to which they are compared and differ by no more than 1 unit of chroma or 2 units of value. Some faint mottles of similar but low chroma and value differ by 2.5 units (one card) of hue
	distinct	Readily seen but contrast only moderately with the color to which they are compared. Distinct mottles commonly have the same hue as the color to which they are compared but differ by 2 to 4 units of chroma or 3 to 4 units of value; or differ from the color to which they are compared by 2.5 units (one card) of hue but by no more than 1 unit of chroma or 2 units of value
	prominent	Contrast strongly with the color to which they are compared. Prominent mottles are commonly the most obvious color feature of the section described. Prominent mottles that have medium chroma and value commonly differ from the color to which they are compared by at least 5 units (two pages) of hue if chroma and value are the same; at least 4 units of value or chroma if the hue is the same; or at least 1 unit of chroma or 2 units of value if hue differs by 2.5 units (one card)

APPENDIX C
Geotechnical Field Representative (GFR) Manual

GEOTECHNICAL FIELD REPRESENTATIVE (GFR) MANUAL

INTRODUCTION

This publication provides a guide to field monitoring, preparation of boring logs, sample preservation and compilation of boring contract records. Soil identification is covered in Appendix B and should be utilized in the preparation of boring logs.

ORGANIZATION

All borings are to be performed under the supervision of a GFR located in the field with the boring equipment. The GFR is to be under the supervision of the GE who will make periodic visits to the boring work.

Prior to initiating any field exploration, the GFRs and the Drilling Superintendent must be thoroughly briefed relative to anticipated subsurface conditions, boring locations, boring numbering, boring depth criteria, sampling procedures, boring access and other provisions required of the field exploration. Such information and instruction shall be covered in the boring contract plans and specifications.

The GE shall be contacted by telephone daily, or as directed, during the course of the Exploration. These reports are to document conditions encountered and to check on possible changes in the boring program and sampling procedures. No boring rig should be allowed to demobilize or otherwise leave the site without the GE’s knowledge. In the event that the GE is unavailable for a decision, which cannot be made by the GFR, then the person next in command above the GE who is associated with the Project should be consulted.

QUALITY ASSURANCE MONITORING

The GFR is responsible for QA monitoring of all field activities. The GFR shall have copies of all relevant procedures relative to the work. The GFR shall be knowledgeable about the procedures and with the GE identify any missing procedures, criteria, or issues related to the quality of the work. This shall be followed with a pre-work meeting with the Boring Contractor to review all quality procedures. It should be recognized that ASTM and other similar procedures are excellent in their content, but may be missing detailed information relative to the work.

The boring method can have a significant impact on data quality. The GFR shall monitor the boring operations to confirm that the borehole (including bottom of the borehole) is sufficiently stable to allow sampling and testing.

The GFR is responsible for the following field activities:

1. Quality assurance monitoring that all boring, sampling, and testing is performed in accordance with Project standards
2. Preparation of accurate boring logs, daily reports, and other documentation
3. Determination of pay quantities including daily reconciliation with the Boring Contractor
4. Accurate location and elevations of all borings
5. Documentation of any field operations that may impact the quality or reporting of field data
6. Documentation of any interaction with third parties, including clients, residents, etc.
7. Final abandonment of the borings and site clean up

The GFR is also responsible for observing and recording drilling operation issues that may provide additional subsurface information. These include but are not limited to:

- Water losses
- Resistance to drilling (advance rate and down pressure, particularly in hard soils or rock)
- Dropping of drill rod

BORING RECORDS

The boring log is the basis for every foundation analysis and it is, therefore, important that a complete and accurate record of all aspects of the subsurface exploration be maintained. Exhibit 5-2 to Exhibit 5-6 are the templates for the boring logs, and Exhibit C-1 to Exhibit C-5 give a sample boring logs. The following items should be carefully observed and recorded on the field boring log.

1. Dates and times of beginning and completion of work.
2. Identifying number and location of test boring.
3. Ground surface and elevation at the boring and source of reference.
4. Diameter and description of casing.
5. Total length of each size of casing.
6. Length of casing extending below ground surface at the completion of the boring.
7. Weight, number of blows, and the length of drop of hammer used to drive the casing each successive foot.
8. Water level observation with remarks on possible tidal variations. (All measurements from original ground surface.)
9. Depth to top of each different material penetrated.
10. Depth to the bottom of sampler at start of driving for each sample and depth to which the sampler was driven.
11. Sampler type and dimensions.
12. Weight of hammer, hammer type, the length of drop used to drive the split spoon sampler, and the number of blows required to drive the sampler, measured in 6-inch intervals, throughout its full depth of penetration.
13. Methods and forces used to push sampler tube when not driven.
14. Length of sample recovered.
15. Loss or gain of drilling fluid or mud.
16. Any sudden dropping of drill rods or other abnormal behavior.
17. An accurate record of any change in the original boring location.
18. Identification of the subsoils and bedrock including color, moisture, structure, condition, etc.
19. Type of drilling operation used to advance hole.

20. Comparative resistance to drilling.

On the field boring log form all data on the drilling and sampling should be noted along with the identification of the soil samples obtained and soil strata changes. It is best to include too much information rather than too little. A sample boring logs are attached that provides guidance (see Exhibits C-1 through C-5). The GE should be consulted to be sure sufficient information is provided on the boring logs.

Generally, the driller should be the entity best able to detect changes in strata and drilling resistance during the course of drilling. There should, therefore, be a close liaison between the driller and the GFR at all times. When changes in strata are indicated by the driller in between scheduled sampling intervals, samples should be taken at the strata change wherever such is feasible.

There are a number of abbreviations and terms that are useful in preparing boring logs that may be utilized. Abbreviations for soil identifications should only be used when lack of space prohibits their being written out in full.

STANDARD ABBREVIATIONS

Soil Identification

Gravel	G	**Silty Clay**	$yC
Sand	S	**Clay**	C
Silt	$	**Peat**	Pt
Clayey Silt	Cy$	**fine**	f
Silt & Clay	$&C	**medium**	m
Clay & Silt	C&$	**coarse**	c

Color

light gray	LtGr	**tan**	Tn
gray	Gr	**yellow**	Yl
dark gray	DkGr	**green**	Grn
black	Bk	**blue**	Bl
brown	Br	**red**	Rd

Samplers

Split spoon	SS	**Diamond core size**	BX, NX
Shelby Tube	ST	**Torvane**	Tv
Piston Sampler	PS	**Field Vane**	Fv
Denison Sampler	DS	**Pocket penetrometer**	Pp

Modifications

organic	org	**seam, seams**	sm, sms
calcareous	calc	**streaks**	stks

ferrous	fer	**nodules**	nod
lignitic	lig	**laminated**	lam
very	v	**slickensided**	sls
slightly	sl	**interbedded**	intbdd
at	@	**intermixed**	intmx
with	w/		

<u>TERMS CHARACTERIZING SOIL STRUCTURE</u>

Slickensided	-	surfaces that are slick and glossy in appearance or polished.
Fissured	-	extensive crack or cracks.
Sensitive	-	pertaining to cohesive soils that are subject to appreciable loss of strength when remolded.
Varved	-	alternating thin layers of silt (or fine sand) and clay.
Laminated	-	composed of thin layers and texture, 1 cm or less, in thickness.
Interlayered	-	composed of alternate layers of different soil types.
Parting	-	a very thin layer one or two grains thick.
Calcareous	-	containing appreciable quantities of calcium carbonate.

BORING LOCATIONS AND GROUND SURFACE ELEVATION

Accurate boring location and ground surface elevation are a critical part of the boring and to the interpretation of the boring results. It is the GFR's responsibility, in conjunction with the GE, to assure that this information is accurate.

<u>Location</u>: The GRF shall determine the boring location to the greatest accuracy feasible (to an accuracy of approximately one-half foot). This can be done by several methods:

- Referencing (taping) to site features shown on the Boring Location Plan. The GFR shall be responsible for confirming that the reference site features match the boring location plan.
- Use of hand held GPS unit.
- Reference (taping) to the proposed boring location stake, if the stake was determined by accurate methods as discussed above.
- Survey of the actual boring location. This is the most accurate method, and is required for as built locations. However, it should be noted that survey may occur several months after the boring operation. It is the GFR's responsibility to ensure that the survey team finds the actual boring location.

Ground Surface Elevation: Determination of accurate ground surface elevation is often problematic. Typically, interpolation from topographic information does not provide an adequate elevation. An accuracy of 0.1 foot is required. The following are considered accurate methods:

- Survey of the actual boring location. Note the requirement to assure that the survey team finds the actual boring location.
- Referencing to site feature with a known elevation. If the elevation difference is not too much (less than approximately 5 feet) the elevation difference can be field determined (with care) using hand levels and folding rules.
- Referencing to staked boring location, if the elevation was determined an accurate method. The same limitations and methods as discussed in the previous paragraph are applicable.

Documentation: Documentation of both location and elevation determination. This should include detailed recording of dimension, reference featured, methods, etc. The notes should be attached to the Daily Report.

Daily Geotechnical Field Representative Report

Daily Geotechnical Field Representative Reports (DGFRR) should include, in chronological order, the following:

1. Job number, name, location, date, weather conditions, client, owner and contractor representatives.
2. Arrival and departure of all personnel involved.
3. Record all delay and down-times, their causes and eventual conclusion.
4. Summaries of any discussions, conversations and meetings relevant to the Project work including any instructions and change orders.
5. Summarize all work, progress for the day. Include, whenever possible, a location diagram of work areas for that day.
6. Record all contacts made, the names of the parties contacted, and who they represent.
7. Tabulate daily pertinent data such as water level readings, boring footage (rock and soil), and footage for observation well installation, etc.
8. Document all equipment used and maintain an accurate record of all expenses incurred.

A sample of a DGFRR is attached for guidance; see Exhibit C-6.

Following is a list of necessary equipment and supplies to be used by the GFR.

1. Clip board.
2. Boring plans and Specifications
3. Boring log forms.
4. Daily report and other type of forms.
5. 6-foot Folding Engineers ruler.
6. Pocket knife.
7. Optional equipment:

   100-foot Measuring tape

Pocket penetrometer, Torvane

Hand level

Flagging tape

Indelible black ink marker pen

Geologic Hammer

Litmus Paper

SITE RECONNAISSANCE

It is important that a site reconnaissance be made. Any conditions which may affect design considerations or construction should be noted and logged into the DGFRR and verbally reported to the GE. Examples might be: the presence of buildings or old foundations left over from demolition; man made or sanitary landfills; indications of sinkholes, depressions or open caves; existing rock outcrops; surface drainage; etc. Careful documentation throughout the field operations is critical. The GFR shall prepare sketches with reference dimensions if feasible.

SOIL BORINGS

The Authority preference for advancing soil borings shall be mud-rotary with casing employed for borehole stability where applicable. However, when deemed acceptable by the GE other methods may be permissible.

The GFR has several responsibilities with respect to the drilling operation:

- The GFR should be knowledgeable about the drilling operation and be aware of issues which could affect data quality. If the GFR is not familiar all aspects, he should notify the GE and request assistance and training.
- The bottom of the bore hole is where the samples are attained. The bottom can be affected by unbalanced hydrostatic forces among other issues.
- The GFR should be aware of the length of all tools that use in the drilling operation, including rods, drill bits, and samplers. With this information, and working with the driller, the GFR can determine that the "drilled to" depth and the sample depth are the same. If they are not, this could indicate bottom instability.

<u>SPT Sampling</u>: The GRF shall be familiar with the requirements of the Standard Penetration Test (ASTM D1586). The Authority preference is to use an Automatic Hammer. The GFR should inspect the hammer with the driller to assure proper operation.

Immediately upon removal from the hole the split barrel sampler should be split open to provide for visual inspection of the intact sample by the GFR. It may be necessary to remove mud from side of the sample, or possibly cut the lengthwise so that soil and soil structure can be observed. After examining the exposed soil, a digital photo of the sample shall be taken and incorporated into the Phase B Geotechnical Engineering Report. The soil photographs shall meet the following criteria:

- Image quality is adequate to discern soil type, grain size and color

- The photo taken perpendicular to the sample from above with sample in full view.
- A folding rule placed along the sampler for scale.
- The sampling jar cap with complete markings set aside sampler and within the photo for identification.
- The drilling fluid should be scraped to reveal the natural material's color and grain size distribution.
- The entire sample contents of the split spoon shall be photographed.

Before the samples are selected, be sure a complete record of the sample is recorded, including interbedded zones, soil structure, etc. Samples should be selected to represent different soil types in the sampler, if present. In no cases, shall soils with different characteristics be placed in the same sample jar.

Samples should then be tightly sealed in screw-top glass jars or bottles at least 3-1/2 inches high, approximately 1-1/2 inch inside diameter at the mouth, and with inside diameter of the jar no more than ¼-inch larger than that at the mouth. The jars shall be provided with metal screw caps containing a rubber or waxed-paper gasket.

Samples shall be placed in the jars in the condition in which they are removed from the split barrel sampler without squeezing, mashing or otherwise excessively distorting the sample. Samples which have been recovered and preserved should be numbered consecutively; i.e. S1, S2, etc. If the sample from the split barrel is divided into subsamples because of material change, then the sample designation number will be followed by a letter designation assigned alphabetically from top to bottom; i.e. S1A, S1B, etc. If no sample is recovered it will be designated by an "NR" and no sample number assigned. No jar need be placed in the jar box to show N position.

Each sample jar and box shall be labeled as follows (see Exhibit 5-7 for examples of soil sample labeling).

a. <u>Sample Jar Lids:</u> Each sample jar lid shall contain the following information:

i. NJTA Project Name
ii. NJTA Project Contract Number
iii. Boring Number
iv. Sample Number; denoted as S-1, S-2, S-3, etc.
v. Sample depth
vi. SPT Blow Counts for each 6-inches of penetration for a total of 24-inches
vii. Sample recovery length
viii. Date sample was taken

b. <u>Sample Jar Boxes</u>: Sample jar boxes shall contain the following information on the top and on one of each of the long and short sides of the box:

i. Geotechnical Engineering Firm
ii. NJTA Project Name
iii. NJTA Project Contract Number
iv. Boring and Sample Numbers; denoted BR-1 (S-1 to S-10), BR-2 (S-1 to S-16), etc.
v. Dates Samples were taken

vi. Initials of the GFR

Undisturbed Samples

As shown in the GEP, the driller may be required to take undisturbed samples. These samples are often collected using a three-inch O.D. open-type “Shelby” tube sampler with sample tubes 30 inches long and provided with a positive ball check valve in its head. Such samples shall be obtained by pushing or jacking the sampler into undisturbed soil at the bottom of the hole. Wherever possible, the equipment for advancing the sampler shall measure the force required to penetrate the soil. The GFR shall record the force required to penetrate the soil. The GFR shall record this force, depth of penetration and length of sample recovered. These samples shall be sealed in the tubes in which they are obtained and carefully labeled to show location and depth of sample (i.e.: U-1, U-2, etc.). When there are problems with obtaining undisturbed samples using Shelby tubes, undisturbed soil samples shall be recovered by means of special piston- type samplers.

When ready to take Shelby or piston-type samples, all loose and disturbed materials shall be removed to the bottom of the casing or of the open hole. This final cleaning should be accomplished with a device in which washwater is fully deflected in an upward direction. No washing with downward directed jets should be permitted within four inches of the intended top of the undisturbed sample. Cleaning out should be done in such a manner that the soil immediately below the bottom of the casing is as nearly undisturbed as possible. The sampling device connected to the drilling rod should then be lowered slowly to the bottom of the hole and the sampler forced into the soil for a distance of not less than 24 inches or more than 27 inches.

In the operation of securing the undisturbed samples, the samplers should be forced into the soil at a rate of four to five inches per second. The samplers should be pushed or jacked downward, and not to be driven unless the character of the soil is such that driving with the hammer is absolutely necessary and is approved by the Project engineer.

The sampler with its contained soil sample should be rotated, and then carefully removed from the hole. The thin-walled tube containing the sample should be detached from the driving head. A portion of the undisturbed sample should always be carefully removed from both ends of a tube (a minimum of ½ inch thickness) and squared and preserved whether the sample is sealed in the tube or extruded in the field and preserved in cartons. The removed samples from the top and tip of the tube should then be described on the boring log along with any other information which may be helpful in determining subsurface conditions. If stated in the GEP, it may be necessary to perform Pocket Penetrometer and field Torvane tests on the bottom of the recovered undisturbed samples. Care must be taken to prevent sample disturbance to the thin-walled tube samples.

The ends of the tube are wiped clean and the end spaces filled with hot paraffin or hot melted beeswax. Snug-fitting metal or plastic caps should be placed on the ends of the tube. The caps should be sealed with friction tape. Finally, the ends of the tube should be dipped in hot paraffin or beeswax to provide airtight seals.

Undisturbed samples, designated by a “U” should be numbered according to their occurrence in the sampling sequence: i.e., U-1, U-2, etc. The boring log should provide the type of undisturbed sampling types (Shelby Tube, piston sampler, etc.)

Each Undisturbed (Thin-Walled Tube) Sample shall contain the following information:

i. Geotechnical Engineering Firm
ii. NJTA Project Contract Number
iii. NJTA Project Name

iv. Boring Number
v. Sample Number should be designated by “U” and numbered according to occurrence in the boring sequence; denoted as S-1, S-2, U-1, S-3, S-4, S-5, U-2, …
vi. Sample depth
vii. Sample recovery length
viii. Label the top and bottom of the tube
ix. Date sample was taken
x. Initials of the GFR

Undisturbed soil tube should be clearly and permanently marked to show the top and bottom of the tube. Undisturbed samples should be handled and transported in a cushioned rack with the top of the sample always upright. It should be delivered to the laboratory with extreme care in order to minimize disturbance effects which may render laboratory test results useless.

During the winter months, precautions must be taken to prevent undisturbed samples from freezing during handling and shipping; if allowed to freeze, the samples will be worthless for strength or consolidation testing.

Tubes for undisturbed samples are to be provided by the driller, and should be of steel, seamless brass or hard aluminum. Sample tubes should have a machine-prepared sharp cutting edge with a flat bevel to the outside wall of the tube. The cutting edge shall be drawn in to provide an inside clearance beyond the cutting edge of 0.015 inch± - 0.005 inch.

When recovery of samples by use of Shelby tubes is poor, then undisturbed soil samples are to be recovered by means of a thin-wall piston-type sampling device with piston rods that extend to the ground surface, or a self-contained hydraulically-operated piston sampler, such as the “Osterberg” sampler. The sampler selected should be designed to utilize sample tubes with a three-inch outside diameter. When samplers, utilizing piston rods extending to the ground surface, are used, positive locking of the piston rods with respect to the surface of the ground must be provided to prevent upward or downward motion of the piston during the advance of the sampling tube and the piston rods must be positively locked to the drill pipe at the surface during removal of the sampler for the depth to which it penetrated undisturbed soil. If the piston rods are locked to the mast of a truck-mounted drill rig, the rig should be blocked and anchored to the ground in such a manner as to prevent motion of the rig during the sampling operations.

If specifically approved in advance by the GE, samples may be recovered in hard soils by an open-type, thin-wall sampling device.

In very soft soils, a weighted drilling mud may be required, whether or not casing is used, in order to maintain a pressure on the soil as nearly equal as possible to that existing before the drilling operations.

Under certain conditions, continuous sampling with three-inch diameter “Shelby” tubes may be required in cohesionless materials encountered in 3½ inch undisturbed sample Borings.

ROCK CORING AND FIELD LOGGING

When it has been determined that bedrock has been encountered, and not a boulder or very dense soils, then it may be feasible to core. The decision to core is not to be determined by the results of the Standard Penetration Test alone, but may be used as an indication that coring is possible. Knowledge of local geologic conditions, known or anticipated, and soil samples recovered, must be considered in any decision as to whether to begin coring. However, coring this zone may produce very low core recoveries. For this reason, the GEP shall describe in detail how refusal will be determined and how this transition zone will be investigated so as to maximize information. Acoustic Televiewer (ATV) or Optical Televiewer (OTV) logging may also be used to better define this zone.

Once the driller is set up to core, the GFR should document the following information on the boring log:

1. Type of core barrel, diameter (ID), drill bit type and condition (which should be good);
2. Note any circulating fluid losses, depth and time of occurrence and any actions or reasons resulting in loss of core;
3. The starting and completion depth (to the nearest tenth of a foot) of each run, with no core run length to exceed 5.0 feet unless approved by the GE;
4. The core run designation (i.e. C-1, C-2, etc.) and the recovery;
5. The type of rock recovered, color, the core recovery, RQD (Rock Quality Designation) and any other related information, see Exhibit C-4.

In addition to the above, the GFR should insure that:

1. Casing has been sealed into bedrock;
2. Coring equipment used is of a type that would maintain continuous contact between the core bit and the rock being drilled.

Each core should be packed in well-constructed wooden boxes, provided by the Boring Contractor, with dividing strips to hold the cores in position and in the order in which they were recovered from each hole. Wooden blocks should be placed in the box to separate the core runs and should be marked to identify the core depth. When the core recovered is fragmented, all pieces of a size less than the core diameter should be put in plastic bags and placed in the core box in its appropriate position within that core run. When desirable to maintain sample moisture wrap the core in plastic to prevent drying. In the GEP, specify whether core will require wrapping. Core boxes should be marked on the inside and the outside as describe below and as shown in Exhibit 5-8.

Rock Core Boxes - Top: The outside top of a Rock Core box shall contain the following information:

i. Geotechnical Engineering Firm
ii. NJTA Project Contract Number
iii. NJTA Project Name
iv. Boring Number
v. Core run and depth; denoted as C-1 45'-50', C-2 50'-55', etc.
vi. Core run recovery in percent of total 5' run
vii. Core run RQD in percent of total 5' run
viii. Date cores was taken
ix. Initials of the GFR

Rock Core Boxes - Inside Lid: The inside of the Rock Core box lid shall be divided into four compartments by drawing three lines to mimic the compartments of the core box and contain the following information:

i. Boring Number
ii. Core run and depth
iii. Core run recovery in percent of total 5’ run
iv. Core run RQD in percent of total 5’ run
v. Label the top and bottom of the core run starting in the left corner of the upper compartment and work to the right. When compartment is filled, move down to the left corner of the next compartment and work to the right.
vi. Draw break line in between cores where wooden block separates core runs in the compartments
vii. Indication of natural and mechanical fractures

BORING TERMINATION

The GEP should have a clear explanation of boring termination. This should include a minimum depth assigned to each boring location. The GEP should also give the GFR guidance about when to extend Borings deeper. The following present conditions when Borings might not be terminated at the designated completion depths:

1. The boring is in soft clays or organic silts or some compressible stratum;
2. Sampling blow counts have been decreasing significantly or are very low to begin with (i.e. fewer than 10 to 20 blows per foot);
3. A void is encountered just before or at the design completion depth;
4. Unanticipated subsurface conditions have been encountered;
5. Minimum criteria for terminating a boring as established by the GE have not been met (i.e. blow count, core recovery, RQD, particular stratum, etc.)

Before proceeding further with the boring, the GFR should consult with the GE for further instructions. If the GE is unavailable, then consult the person next above in the chain of command, such as another engineer, the project manager or principal-in-charge.

GROUNDWATER LEVEL EXPLORATION

Groundwater levels should be recorded when first encountered during drilling, at the start of work each morning for Borings in progress and at the completion of each boring. Groundwater levels should also be recorded at the end of the field exploration project. The date and approximate time after boring completion should also be recorded for each water level reading. All water level observations should be summarized on the boring logs in the spaces provided. Observations should be made of ground water levels at the start of each day and in all completed holes. Any unusual water conditions and gain or loss of water in boring operations should be recorded completely in the boring logs. Whenever required by the GE, bore holes should be bailed for observations of groundwater conditions. When the open hole drilling method is used, and natural or commercial drilling mud utilized to stabilize the hole, the hole may have to be flushed thoroughly with clean water at the completion of the boring for the purpose of observing groundwater levels.

Groundwater level observations can be made in an open hole by filling the hole with clean water to a point above the natural groundwater level and observing the drop in the level of water in the hole. This may be followed by bailing the hole to a point below the natural groundwater level and observing the rise in the level of water in the hole. All individual measurements of the water level in holes should state the time elapsed since the last filling or bailing of the hole.

<u>INSTALLATION OF STAND-PIPE OBSERVATION WELLS</u>

Stand-pipe observation wells provide long term ground water observation and are often required by the Contract plans and specifications. Upon reaching the completion depth of a boring it may be part of the drilling program that an observation well be installed. In Borings advanced with casing or hollow-stem augers, the well pipe, usually 1 inch to 2 inch diameter PVC pipe, may be inserted with its screen tip into the casing or hollow stem augers prior to their being withdrawn. In the event that the grain size gradation of the stratum into which the well tip screen has been placed, is finer than the screen opening size, then the screen should be packed with graded granular material to avoid plugging of the screen. Once the well has been installed, it should be backflushed with clean water to clear the screen and the well-developed. Upon removal of casing or augers and completion of backfilling, it may be desirable to again backflush by applying a slightly increased hydrostatic pressure and monitoring its drop to assure the screen is still open. When drilling with mud, those Borings which require observation wells should be drilled with biodegradable mud if possible. If no biodegradable mud is available, then the hole should be thoroughly backflushed after the pipe and screen have been installed and tested, after backfilling. When non-biodegradable muds are used, boring wall permeability may be obstructed and may result in unreliable water level readings. No observation well installed using a non-biodegradable mud should be accepted unless the driller demonstrates that the well is in working order or unless directed by the GE. A sample Monitoring Well Log is provided as Exhibit C-4.

<u>ABANDONED BORINGS</u>

Borings should not be abandoned before reaching the final depth ordered except with the approval of the GE or his representative. No payment will be made for Borings abandoned by reason of an accident or negligence attributable to the driller.

Borings abandoned before reaching the required depth, due to an obstruction or other reasonable cause not permitting completion of the boring by standard procedures, shall be replaced by a supplementary boring adjacent to the original one and carried to the required depth. Penetration to the bottom depth of the abandoned boring may be made by any means selected by the driller and approved by the GFR unless payment is being rendered for the overlapped portion of the bore hole, in which case standard drilling procedures should be used. Samples to be taken in the supplementary boring should commence from the last sampling elevation at which the original boring was abandoned in the manner specified for the original boring. This will establish a sampling continuity between the two Borings.

All borehole and monitoring well casings shall be cut off a minimum of 2 ft. below the ground surface or removed completely. Monitoring wells shall be abandoned in accordance with NJAC 7:9D-3.1. Boreholes shall be abandoned in accordance with NJAC 7:9D-3.4 which allows boring less than 25 feet to be backfilled with drill cuttings. Borings greater than 25 feet should be grouted using a tremie grout method. The top surface shall receive the same treatment type and thickness as the existing condition (i.e. topsoil, crushed stone, and ballast). Boreholes in pavements shall be grouted using a tremie grout method and the upper portion backfilled with concrete to the same thickness as the existing pavement.

Test pits shall be backfilled in minimum 12-inch thick lifts and compacted by repeatedly striking the soil with the excavator bucket. If and where directed, the GFR may require additional compaction provisions based upon the sensitivity of the area to settlement.

All drilling mud and cuttings shall be hosed off or disposed of beyond developed areas, wherever feasible, and in a legal and environmentally approved manner. Test locations shall be re-graded to match the existing conditions and grassed areas shall be seeded.

Upon completion of the work the driller should remove his rigs, all equipment, unused material and soil removed from the holes and leave the site in a clean condition satisfactory to the owner and Engineer.

HNTB

GEOTECHNICAL BORING LOGS FOR

Boring No. BR-1
Sheet No. 1 of 2

New Jersey Turnpike Authority
(Owner)

NJTA Interchange 15E
(Project)

XYZ Drilling, Inc.
(Contractor)

Contract No. T500.250 Purpose Bridge Structure No. 1455-23
Location Newark, NJ RDWY. Turnpike SND STA. 109+35 OFF. 35' LT.

Rig No. 8	Type CME75		Driller Dave Mueller		Helper Dan Smith		
DATE	07/01/10						
TIME STARTED	07:00 am						
TIME FINISHED	03:00 pm						
WEATHER	Sunny 80						
DEPTH REACHED	42 ft						

GROUND ELEVATION 34.5 ft M.L.W. ELEVATION
ZERO OF BORING LOG 34.5 ft ELEVATION GROUND WATER 26.3 ft

PAY QUANTITIES										
LINEAL FEET OF BORING					SAMPLES			LINEAL FEET OF ROCK CORE		
2-½ in	3 in	4 in			ORD. DRY	UNDIST. DRY		1-7/8" ID (NQ)	2-1/8" ID (NX)	
ITEM	ITEM	ITEM	ITEM	ITEM	ITEM	ITEM	ITEM	ITEM	ITEM	ITEM
---	---	42'	---	---	10	1	---	---	---	---

	Type		Type	Size	Weight of Hammer	Average Fall	Hammer Type
Drilling Mud	Revert	Casing:	HW	4"	140 lb	30"	Auto
Ordinary Dry Samples O.D.	2"	I.D.		1-3/8"	140 lb	30"	Auto
Undisturbed Samples		Type	Shelby tube	Length 30"	O.D. 3"	I.D.	2-7/8"

GROUND WATER READINGS							
DATE	07/01/10	07/02/10					
TIME	03:10 pm	07:30 am					
DEPTH	12.5 ft	8.2 ft					

GENERAL REMARKS:
1. Boring moved 10' west to avoid utility.
2. Drilling mud introduced from 11'.
3. Boring tremie grouted on 07/02/10 @ 10:30 AM.

NORTHING: 695428.6 EASTING: 611603.8

All elevations refer to the NAVD 88 datum. Horizontal locations refer to the NJ State Plane coordinate system as per the NAD 83 datum.

The subsurface information shown hereon was obtained for NJTA design and estimate purposes. It is made available to authorized users only that may have access to the same information available to the NJTA. It is presented in good faith, but is not intended as a substitute for investigations, interpretation, or judgement of such authorized users.

INSPECTOR Gearge Sable GEOTECHNICAL ENGINEER Sam Connor

Exhibit C-1: Sample Boring Log

HNTB

BORING LOG

Boring No. BR-1
Sheet No. 2 of 2

CONTRACT NO. XX345 RDWY. Turnpike SND STA. 109+35 OFF. 35' LT.

Elev. (ft) 34.5	Blows on Casing	Blows on Spoon For 6-in Penetration		Sample No.	Sample Depth (ft)	Log	Material & Remarks
	27	27	11	S-1	0' - 2'		Brown cf SAND, some cf Gravel, trace Silt
	32	12	13				Rec=12"
	206	2	2	S-2	2' - 4'		Red c+m SAND, little Silt & Clay, trace f Gravel
	76	3	14				Rec=15"
	93	32	69	S-3	4' - 6'	Sand	*Gray f SAND, and Silt, trace f Gravel
	70	42	28				Rec=14"
	72						
▼	56						
	78						
24.5	85						
	54	2	3	S-4	10' - 12'		Yellowish brown Silty CLAY, trace f Sand
		5	4			Silty Clay	Rec=21"
	M						
	U						
19.5	D						
	U	WOR	WOH	S-5	15' - 17'		Dark brown PEAT
	S	1	1			Peat	Rec=18"
	E			U-1	17' - 19'		Shelby Tube Pushed
15.5	D						Rec=22"
		3	1	S-6	19' - 21'		Brown CLAY & SILT, trace+ f Sand
		2	2			Clay & Silt	Rec=24"
10.5							
				C-1	25' - 27'	Boulder	Boulder
7.5							Rec=14"
		37	29	S-7	27' - 28.3'		Gray cf GRAVEL, some cf Sand
		100/4"					Rec=18"
		11	50	S-8	30' - 32'	Gravel	* Gray cm+f Sand, trace+ Silt, and+ mf+ Gravel
		67	44				Rec=16"
-0.5							
		34	49	S-9	35' - 37'		Brownish red cf GRAVEL, some mf Sand, trace Silt
		54	54				(Decomposed Shale) Rec=12"
						Decomposed Rock	
		52	49	S-11	40' - 41.2'		*Brownish mf sand, and Silt & Clay, some f Gravel
-6.7		100/2"					(Decomposed Shale) Rec=12"
						41.2 ft	End of soil sampling at 41.2 feet and start coring rock.
							See Rock Core Log for details.

* Indicates that soil description has been verified based on laboratory results.

Exhibit C-1: Sample Boring Log

ROCK CORING LOG

Boring No. BR-1
Sheet No. 3 of 3

CONTRACT NO. T3954 RDWY. NBHCE STA. 45+50 OFF. 25' LT

Core Data										Rock Description									Discontinuity Description								Lab Testing			
Run				Water	Rock Core					General				Fabric		Structure			Planes		Joints									
																					Joint Surface									
Elev.	Sample no.	Depth (ft)	Core Time (ft/min)	Gain/Loss	No. of Pieces	Recovery		Rock Quality Designation, RQD		Rock Type	Color	Weathering	Strength	Grain Size	Grain Shape	Fracture Spacing	Bedding Spacing	Foliation Angle	Fracture Type	Fracture Aperture	Shape	Roughness	Joint Infilling	Infilling Strength	Water Content	Joint Roughness Coefficient	Strength (ksi)	Unit Weight (pcf)	Slake Durability (%)	Other (Specify)
						in	%	in	%	(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)	(j)	(k)	(l)	(m)	(n)	(o)	(p)	(q)	(r)	(s)	(t)	(u)
55.5	C-1	50	2.0		22	57	95	33	55	Shale	Br	IV	R3	VF		CF	TNB		B	SO	P	SR	Fe		M	4			92	1.5
		51	2.5									III	R3	VF		CF	TNB		B	SO	P	SR	Ca		M	4	3.5	148	95	1.0
		52	3.0									II	R4	VF		CF	TNB		B	T	P	SM	-		M	4				
		53	3.0									II	R4	FG		VCF	MB		B	T	IR	SM	C	S2	M	5				
		54	2.5									II	R4	FG		VCF	MB		B	T	IR	SM	-		M	5	3.3	149		
50.5	C-2	55	3.0		8	60	100	48	80	Shale	Br	II	R4	FG		VCF	MB		B	T	IR	SM	-		M	3				
		56	3.0									II	R4	FG		VCF	MB		B	T	P	SM	C	S2	M	4				
		57	3.0									II	R4	FG		VCF	MB		B	T	P	SM	-		M	4	4.5	155		
		58	3.0									II	R4	FG		VCF	MB		B	T	IR	SM	Ca		M	5				
		59	3.5									II	R4	FG		VCF	MB		B	T	IR	SM	-		M	4				

(a) Amphibolite, Argillite, Basalt, Chert, Claystone, Conglomerate, Diabase, Dolomite, Gneiss, Granite, Hornfers, Limestone, Marble, Metaquartzite, Mudstone, Phyllite, Quartzite, Sandstone, Schist, Serpentinite, Shale, Siltstone, Slate

(c) I-Unweathered, II- Slightly Weathered, III-Moderately Weathered, IV-Highly Weathered, V-Completely Weathered, VI- Residual Soil

(d) R0-Extremely Weak, R1-Very Weak, R2-Weak, R3-Medium Strong, R4-Strong, R5-Very Strong, R6- Extremely Strong

(e) Sedimentary: VF- Very Fine, FG-Fine, MG-Medium, CG-Coarse, VC-Very Coarse; Igneous & Metamorfic: P- Pegmatitic, CG-Coarse, MG-Medium, FG-Fine, A- Aphanitic

(f) AG- Angular, SA- Subangular, SR- Subrounded, RD-Rounded, WR- Well Rounded

(g) ECF- Extremely Close, VCF-Very Cloase, CF-Close, MF-Moderate, WF-Wide, VWF-Very Wide

(h) TLB-Thinly Laminated, LB- Laminated, VTNB-Very Thin, TNB-Thin, MB-Medium, TKB-Thick, VKTB-Very Thick (massive)

(j) J-Joint, Ft-Fault, Sh-Shear, Fo-Foliation, V-Vein, B-Bedding

(k) T-Tight, SO-Slightly Open, MO-Moderately Open, O-Open, MW-Moderately Wide, W-Wide

(l) P-Planar, U-Undulating, WU-Wavy, ST-Stepped, IR-Irregular

(m) SL- Slickensided, SM-Smooth, SR-Slightly Rough, RG-Rough, VR-Very Rough

(n) Ca-Calcite, Fe-Iron, Qtz-Quartz, C-Clay, M-Silt, S-Sand, G-Gravel

(o) S1-Very Soft, S2-Soft, S3-Firm, S4-Stiff, S5-Very Stiff, S6-Hard

(p) D-Dry, M-Moist, W-Wet

(u) CAI as per ASTM D7625

Exhibit C-2: Sample Rock Coring Log

HNTB

GEOTECHNICAL BORING LOGS FOR

Boring No. DB-1
Sheet No. 1 of 3

New Jersey Turnpike Authority
(Owner)

NJTA Interchange 15E
(Project)

XYZ Drilling, Inc.
(Contractor)

Contract No. XX345 Purpose Stormwater Basin Structure No. 1455-23
Location Newark, NJ RDWY. Turnpike SND STA. 109+35 OFF. 35' LT.

Rig No. 8 Type CME75 Driller Dave Mueller Helper Dan Smith

DATE	07/01/10					
TIME STARTED	07:00 am					
TIME FINISHED	03:00 pm					
WEATHER	Sunny 80					
DEPTH REACHED	12 ft					

GROUND ELEVATION 53.3 ft M.L.W. ELEVATION
ZERO OF BORING LOG ELEVATION GROUND WATER

PAY QUANTITIES										
LINEAL FEET OF BORING					SAMPLES			LINEAL FEET OF ROCK CORE		
2-½ in	3 in	4 in			ORD. DRY	UNDIST. DRY		1-7/8" ID (NQ)	2-1/8" ID (NX)	
ITEM	ITEM	ITEM	ITEM	ITEM	ITEM	ITEM	ITEM	ITEM	ITEM	ITEM
---	---	14'	---	---	10	---	---	---	---	---

	Type		Type	Size	Weight of Hammer	Average Fall	Hammer Type
Drilling Mud		Casing:	HW	4"	140 lb	30"	Auto
Ordinary Dry Samples O.D.	2"	I.D.		1-3/8"	140 lb	30"	Auto

Undisturbed Samples Type Length O.D. I.D.

GROUND WATER READINGS

DATE							
TIME							
DEPTH							

GENERAL REMARKS:
1. Boring moved 10' west to avoid utility.
2. Boring tremie grouted on 07/02/10 @ 10:30 AM.

NORTHING: 695428.6 EASTING: 611603.8

All elevations refer to the NAVD 88 datum. Horizontal locations refer to the NJ State Plane coordinate system as per the NAD 83 datum.

The subsurface information shown hereon was obtained for NJTA design and estimate purposes. It is made available to authorized users only that may have access to the same information available to the NJTA. It is presented in good faith, but is not intended as a substitute for investigations, interpretation, or judgement of such authorized users.

INSPECTOR Gearge Sable GEOTECHNICAL ENGINEER Sam Connor

Exhibit C-3: Sample Boring Log for Stormwater Facilities

BORING LOG

BORING NO. DB-1
SHEET NO. 2 OF 3

CONTRACT NO. XX345 RDWY. Turnpike SND STA. 109+35 OFF. 35' LT

Elev. (ft) 53.3	Blows on Spoon	Sample No.	Sample Depth (ft)	Log	Material & Remarks
	1	S-1	0 - 2	LOAMY SAND	3" Topsoil; Dark Yellowish Brown (10YR 4/4) coarse grained Loamy SAND, single grained, very loose; no mottles, no Gravel, moist, (m-f(+) SAND, little Silt). Rec.: 14"
	2				
	1				
51.3	2				
	4	S-2	2 - 4	SAND	Light Olive Brown (2.5Y 5/3) coarse grained SAND, single grained, loose; no mottles, no Gravel, moist, (m-f SAND, trace Silt). Rec.: 18"
	5				
	4				
	3				
	1	S-3	4 - 6	SILTY CLAY	Reddish Gray (10R 5/1) fine grained Silty CLAY, massive, medium stiff; many, medium, distinct, Yellowish Brown (10YR 5/8) mottles, no Gravel, moist. (SILT & CLAY) Rec.: 18"
	2				
	2				
47.3	4				
	1	S-4	6 - 8	CLAY LOAM	Gray (10YR 5/1) fine grained Clay LOAM, massive, medium stiff; common, coarse, distinct, Strong Brown (7.5YR 4/6) mottles, no Gravel, moist, (CLAY & SILT, some(+) m-f Sand). Rec.: 16"
	2				
	4				
45.3	4				
	2	S-5	8 - 10	SANDY LOAM	Gray (N5) coarse grained Sandy LOAM, massive, loose; no mottles, no Gravel, (m-f SAND, some Clayey Silt). Rec.: 8"
	3				
	3				
43.3	3				
	4	S-6	10 - 12	CLAY	Very Dark Gray (N3) fine grained CLAY, massive, medium stiff; no mottles, no Gravel, moist, (Silty CLAY, trace(+) f Sand, micaceous). Rec.: 6"
	4				
	3				
41.3	3				

*Indicates that soil description has been verified based on laboratory results.

Exhibit C-3: Sample Boring Log for Stormwater Facilities

BORING LOG

BORING NO. DB-1
SHEET NO. 3 OF 3

CONTRACT NO. T500.250 RDWY.Turnpike SND STA. 109+35 OFF. 35'LT

Elev. (ft) 41.3	Blows on Spoon	Sample No.	Sample Depth (ft)	Log	Material & Remarks
	WOH	S-7	12 - 14	SILT LOAM	Gray (2.5Y 5/1) medium grained Silt LOAM, massive, medium stiff; many, coarse, distinct, Reddish Brown (2.5YR 4/4) mottles, no Gravel, moist. (SILT, and f Sand) Rec.: 24"
	2				
	2				
39.3	4				
	2	S-8	14 - 16	SILTY CLAY LOAM	Reddish Gray (10R 5/1) fine grained Silty Clay LOAM, massive, medium stiff; many, coarse, distinct, Strong Brown (7.5YR 5/6) mottles, no Gravel, moist. (Clayey SILT, trace(+) f Sand) Rec.: 18"
	3				
	4				
37.3	6				
	6	S-9	16 - 18	LOAM	Yellowish Brown (10YR 5/4) medium grained LOAM, massive, soft; no mottles, no Gravel, moist. (Clayey SILT, some(+) f Sand) Rec.: 16"
	6				
	6				
35.3	6				
					End of Boring at 18 feet

*Indicates that soil description has been verified based on laboratory results.

Exhibit C-3: Sample Boring Log for Stormwater Facilities

HNTB

New Jersey Turnpike Authority

Observation Well Installation Log

WELL NO.: RD-11 (OW)

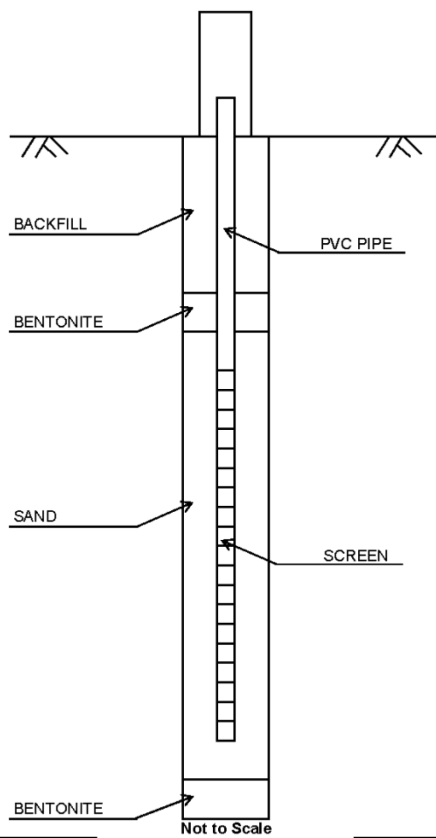
PROJECT NAME: NJTA Interchange 15E — CONTRACT NO.: T500.250

DATE INSTALLED: 2/25/2015 — INSPECTOR: George Sable — HOLE DEPTH: 20'

CONTRACTOR: XYZ Drilling — DRILLER: Dave Mueller — HELPER: Dan Smith

LOCATION: Newark, NJ — GROUND ELEVATION: 26.9

EASTING: 587021.8 NORTHING: 448165.1 STATION: OFFSET: BASELINE:



Sand BACKFILL MATERIAL

0.0 FT = TOP OF BENTONITE SEAL

3.0 FT = TOP OF SAND DEPTH

5.5 FT = TOP OF SCREEN DEPTH

10.0 FT = LENGTH OF SCREEN

15.5 FT = BOTTOM OF SCREEN DEPTH

17.0 FT = BOTTOM OF SAND DEPTH

20.0 FT = BOTTOM OF HOLE DEPTH

WELL READINGS

DATE	BY	DEPTH(FT)	EL.
3/3/2011	HJ	7.2	19.7
3/22/2011	JCJ	6.9	20.1
4/7/2011	JCJ	7.2	19.7
4/15/2011	PDT	7.1	19.8

WELL DATA

ITEM	DESCRIPTION
PVC Well Casing Inside Diameter	2" I.D.
Lock Installed	No
Standpipe or Flushmount	Standpipe
Bags of Sand Used	1 x 50LB
Bags of Bentonite Used	1/4 bucket
Development	

Depth recorded is depth below ground surface

All elevations refer to the NAVD 86 datum. Horizontal locations refer to the NJ State Plane coordinate system as per the NAD 83 datum.

NOTES:

1. Estimated water level 11' at installation.
2. Revert and easy mud used during drilling.

Exhibit C-4: Sample Observation Well Log

HNTB

NEW JERSEY TURNPIKE AUTHORITY
FIELD TEST PIT LOG

TEST PIT NO. TP-7
SHEET 1 OF 1
DATE: START 12/14/15
END 12/14/15

Contract No. T500.250 Purpose Bridge Structure No. 1455-23
Location Newark, NJ RDWY Turnpike SND STA. 102+35 OFF. 35' L
NORTHING 611603.8 EASTING 695428.6 Ground Elevation 34.5 ft
EQUIPMENT USED CAT Backhoe
INSPECTOR George Sable DRILLERS NAME/COMPANY XYZ Drilling
PIT DIMENSIONS: LENGTH 6.0 ft ; WIDTH 4.0 ft ; DEPTH 8.0 ft
WATER LEVEL DEPTH: NOT ENCOUNTERED [X]
CHECKED BY: Roy Roger ; DATE: 12/14/15

DEPTH (FT)	SAMPLE NO. AND TYPE	POCKET PENT/ TORVANE (TSF)	WATER CONTENT	DESCRIPTION	REMARKS
0.0 – 2.5	G-1		Dry	Brown cmf SAND, some(-) cm(+)f Gravel, trace(+) Silt (roots) (Topsoil) 2.5	
2.5 – 4.0	G-2		Dry	Brown black cmf(+) SAND, little Silt, trace(-) f Gravel 4.0	Bag sample collected.
4.0 – 6.0	G-3	1.0/ 0.5	Dry	Gray SILT, little cmf Sand, little(-) mf Gravel (mica)	
6.0 – 7.0	G-4	2.5/ 1.0	Moist	6.0 Gray brown Clayey SILT, some(-) cmf (+) Sand 7.0	
7.0 – 8.0				Light brown mf SAND, little Silt, trace f Gravel 8.0	Permeability test performed at 8ft.
8.0 – 10.0				End of Test Pit at 8 ft.	

NOTE: All elevations refer to the NAVD 88 datum. Horizontal locations refer to the NJ State Plane coordinate system as per the NAD 83 datum.

Exhibit C-5: Sample Test Pit Log

	SUBSURFACE EXPLORATION DAILY REPORT	Date	Day	Team Color
		Report No.	Page	
Project		Job No.		
Exploration Contractor		Weather		

EQUIPMENT ON JOB

DRILL RIG			
NO.	TYPE	MAKE	HAMMER TYPE
1			
2			

BORING INFORMATION

BORING NO.	DATE STARTED	IN PROGRESS	DATE COMPLETED	TOTAL DEPTH	GROUND ELEV.	CASING (C) OR MUD (M)	SAMPLES TAKEN		
							JAR	UNDIST.	ROCK CORE

Visitors ______________________ Representing ______________________

REMARKS

By initialing each box below I attest to the following.

- ☐ I have completed the log and indicated:
  Blow counts
- ☐ Recoveries
- ☐ Soil description
- ☐ Indicated sample moisture
- ☐ Utilized a pocket penetrometer on cohesive soils, where appropriate
- ☐ Utilized a torvane on cohesive soils, where appropriate
- ☐ Tightly sealed all jars and taped jars containing cohesive soils
- ☐ Core boxes properly labeled and samples wrapped.
- ☐ Undisturbed tubes have all relevant data (Project No., Boring No., Tube No., Depth and Date).
- ☐ Undisturbed tubes have been properly retrieved and recorded on the logs
- ☐ Undisturbed tubes do not have evidence that the spilt spoon intruded on the sample (i.e. donut)
- ☐ Undisturbed tubes have been properly wadded with a moist (not wet) rag to fill any void in tube, and wax applied to a clean tube interior to ensure a good seal
- ☐ Remained on-site until departure of drillers and witnessed that site was left in original condition
- ☐ Box labeled both sides with contents
- ☐ Samples listed above were turned over to the driller

If the answer is "No", please explain: ______________________

Inspector ____________ Date ____________ Driller ____________ Date ____________

Exhibit C-6: Subsurface Exploration Daily Report

APPENDIX D
Laboratory Testing for Soils and Rocks

LABORATORY TESTING FOR SOILS

	Tests	Reference
I.	Identification Tests:	
A.	Mechanical Analysis	
1.	Sieve Analysis (with grain size curve)	ASTM D 422
2.	Percent passing #200 Sieve	ASTM D 1140
3.	Hydrometer Analysis including Specific Gravity (with grain size curve)	ASTM D 422
B.	Index Properties	
1.	Preparation of Sample for Testing: Wet Preparation	ASTM D 2217
2.	Liquid Limit - with flow curve	ASTM D 4318
3.	Plastic Limit	ASTM D 4318
4.	Shrinkage Limit	ASTM D 427
C.	Specific Gravity	ASTM D 854
D.	Water Content Determination	ASTM D 2216
E.	Maximum & Minimum Density of Granular Soil (Dry State)	ASTM D 4253 / ASTM D 4254
F.	Visual identification and classification of Jar Samples	Ref. D.1
G.	Visual identification and log of undisturbed tube samples - including opening of tubes	Ref. D.1
H.	Natural Dry Density and Water Content Determination of Shelby Tube Samples	Ref. D.2
II.	Permeability Tests:	
A.	Permeability of granular Soils (Constant Head)	ASTM D 2434
B.	Permeability of undisturbed sample in 2.5 inch Dia. consolidation apparatus with a maximum pore water back pressure of 60 psi, reporting permeability (K20), natural water content and dry density.	Ref. D.2
III.	Strength Tests:	
A.	Unconfined compression test on undisturbed soil sample, including maximum stress and strain at failure, visual identification, initial water content, dry density, stress-strain curve and failure sketch. Minimum rate of strain 1% per minute.	ASTM D 2166
B.	Direct Shear Test (Consolidated-Quick) on undisturbed soil sample for each normal load, including trimming, visual identification and consolidation of sample, initial and final water contents, dry density and stress-strain curve. Rate of shear one-half percent per minute.	ASTM D 3080 and Ref. D.2

C. Triaxial Compression Test for 2.8 inch Dia. or 1.4 inch Dia. undisturbed or remolded soil sample.

1. Unconsolidated-Undrained for each lateral pressure including visual identification initial water content, dry density, stress-strain curve and failure sketch. Minimum rate of strain 1 percent per minute. — ASTM D 4767

2. Consolidated-Undrained for each lateral pressure at maximum of 24 hour consolidation, with or without back pressure including visual identification, initial and final water contents, strain dry density, stress— curve and failure sketch. Minimum rate of strain 1 percent per minute. — ASTM D 4767/ Ref. D.3

3. Consolidated Drained for each lateral pressure at maximum of 24 hour consolidation, with or without back pressure including visual identification, initial and final water contents, dry density, stress- strain curve and failure sketch. Minimum rate of strain 1 percent per minute. — Ref. D.3

D. Test Method for Consolidated Undrained Direct Simple Shear Testing of Cohesive Soils. — ASTM D 6528

IV. Consolidation Tests:

A. Consolidation test for 2.5 inch and not less than 0.75 inch high sample on undisturbed sample. — ASTM D 2435
For one load cycle and each load increment imposed for a maximum of 24 hours, and unloading to zero, including preparation, initial and final water contents, dry density, void ratio/log pressure curve or unit strain/log pressure curve. — Ref. D.4

B. Each unloading/reloading cycle consisting of two decrements and two increments. — Ref. D.4

C. For each additional day required for consolidation to define secondary consolidation. — Ref. D.4

D. For permeability test — Ref. D.4

E. One-Dimensional Consolidation Properties of Saturated Cohesive Soils Using Controlled Strain Loading. — ASTM D4186

V. Compaction Test:

A. Moisture/Density Relations of Soils using 10 pound Rammer and 18 inch Drop including sample preparation and moisture density curve (modified Proctor). — ASTM D 1557 / AASHTO T180

B. Moisture/Density Relations of Soils using 5 pound Rammer and 12 inch Drop including sample — ASTM D 698 / AASHTO T99 preparation and moisture density curve (Standard Proctor).

C. Standard Method of Test for California Bearing Ratio (CBR)with Stress Penetration Curve (cylinder soaked up to 3 days)including required sample preparation and compaction either (a) or (b) above. Moisture Density relations determined by ASTM D 1557/AASHTO T-180. — ASTM D 1883 / AASHTO T193

VI. Corrosion Aggressivity:

A. Test Method for pH of Soils — AASHTO T299

B. Test Method for pH of Peat Materials — ASTM D 2976

C. Resistivity — AASHTO T288

D. Chlorite Content — AASHTO T291

E. Sulfate Content — AASHTO T290

VII. Organic Content Test:

A. Test Method for Organic Content of Peat Samples by Dry Mass — AASHTO T267

VIII. Tests on Rock Samples:

A. Test for Triaxial Compressive Strength of Undrained Rock Core Specimens without Pore Pressure Measurements (Method A) — ASTM D 7012

B. Elastic Moduli of Undrained Rock Core Specimens in Triaxial Compression without Pore Pressure Measurements Specimens (Method B) — ASTM D 7012

C. Test Method for Unconfined Compressive Strength Testing of Intact Rock Core Specimens (Method C) — ASTM D 7012

D. Test for Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression (Method D) — ASTM D 7012

E. Determination of the Point Load Strength Index of Rock — ASTM D 5731

F. Standard Test Method for Slake Durability of Shales and Similar Weak Rocks — ASTM D 4644

G. Standard Test Method for Laboratory Determination of Abrasiveness of Rock using the CERCHAR Method — ASTM D 7625

H. Standard Test Method for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine — ASTM C 131

IX. Tests on Acid Producing Soil Samples:

A. Sulfate and pH Tests for acid producing soils shall be performed in accordance with Section 7 of the NJDEP Flood Hazard Technical Manual.

REFERENCES

D.1 Burmister, D. M. PRINCIPLES AND TECHNIQUES OF SOIL IDENTIFICATION. Proceedings Highway Research Board: Dec. 1949.

D.2 Lambe, T. W. SOIL TESTING FOR ENGINEERS. John Wiley and Sons: 1951.

D.3 Bishol, A. W. and Henkel, D. J. THE MEASUREMENT OF SOIL PROPERTIES IN THE TRIAXIAL TEST. 2nd Ed. Edward Arnold Ltd.

D.4 Burmister, D. M. THE APPLICATION OF CONTROLLED TEST METHODS IN CONSOLIDATION TESTING. ASTM Symposium of Consolidation Testing of Soils, Special Technical Publication No. 126: 1951.

APPENDIX E
Table of Content for Reports

PHASE A GEOTECHNICAL ENGINEERING REPORT

PROJECT NAME

OPS NO.: ***PROJECT NO.:***

TABLE OF CONTENTS

INDEX OF TABLES

INDEX OF FIGURES

APPENDIX

PHASE B GEOTECHNICAL ENGINEERING REPORT

PROJECT NAME

OPS NO.: ***PROJECT NO.:***

TABLE OF CONTENTS

INDEX OF TABLES

INDEX OF FIGURES

APPENDIX