REPORT OF SUBSURFACE INVESTIGATION AND GEOTECHNICAL ENGINEERING SERVICES

Juniper Trail Bridge
Southern Shores, North Carolina

G E T Project No: EC16-246G
October 21, 2016

PREPARED FOR:

Town of Southern Shores

106 Capital Trace, Unit E • Elizabeth City, North Carolina 27909
Phone: (252)-335-9765
www.getsolutionsinc.com

GET Solutions, Inc.

Geotechnical • Environmental • Testing

October 21, 2016

TO: **Town of Southern Shores**
5375 N. Virginia Dare Trail
Southern Shores, NC 27949

Attn: Ms. Rachel Patrick

RE: Report of Subsurface Investigation and Geotechnical Engineering Services
Juniper Trail Bridge
Southern Shores, North Carolina
GET Project No: EC16-246G

Dear Ms. Patrick:

In compliance with your instructions, we have completed our Subsurface Investigation and Geotechnical Engineering Services for the above referenced project. The results of this study, together with our recommendations, are presented in this report.

Often, because of design and construction details that occur on a project, questions arise concerning subsurface conditions. **G E T Solutions, Inc.** would be pleased to continue its role as Geotechnical Engineer during the project implementation.

Thank you for the opportunity to work with you on this project. We trust that the information contained herein meets your immediate need, and should you have any questions or if we could be of further assistance, please do not hesitate to contact us.

Respectfully Submitted,
G E T Solutions, Inc.

Gerald W. Stalls Jr., P.E.
Senior Project Engineer
NC Reg. # 034336

Camille A. Kattan, P.E.
Principal Engineer
NC Reg. # 014103

Copies: (3) Client

106 Capital Trace Unit E • Elizabeth City, NC 27909 • Phone: (252)335-9765 • Fax: (252)335-9766
info@getsolutionsinc.com

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EXECUTIVE SUMMARY

This project is in its conceptual stage and the precise characteristics have yet to be finalized. At this time, the proposed construction at this site is planned to consist of replacing the existing five (5) to ten (10) year old culvert with a single or a two (2) span bridge. It is our understanding that the replacement of the culvert is due to a combination of subsurface erosion and/or deflection of the culverts that are resulting in displacement of the asphalt pavement section and/or the development of sink holes. As an alternative to constructing a new bridge, it may be selected to construct a pile supported structural slab to span over the existing culvert in order to allow it to remain.

The replacement bridge was previously proposed to be supported by either shallow foundations or deep foundations. Given the findings of the subsurface exploration procedures, it is our opinion that a shallow foundation system is not a feasible option considering that the footings would be founded on FILL soils. As such, the proposed bridge should be supported by deep foundations consisting of round timber or 12-inch pre-stressed pre-cast concrete piles. Although the maximum foundation loading conditions associated with the new bridge were not known at this time, it is expected that they will be required to support HS-20 loading conditions. However, based on our experience with similar projects, the maximum pile foundation loads are not expected to exceed about 20 to 25 tons for round timber piles or about 50 tons for 12-inch pre-cast concrete piles.

The structural slab option should be supported by deep foundations consisting of either round timber or 12-inch pre-stressed pre-cast concrete piles. Although the maximum loading conditions associated with a structural slab were not known at this time, it is expected that they will be required to support HS-20 loading conditions. Additionally, they are expected to vary relative to the selected pile type and potentially range from about 20 tons to 50 tons.

The finished deck elevation of the bridge or finished grade elevation of the alternative structural slab option is anticipated to coincide with existing site grade elevations. However, an excavation depth ranging from approximately 2 to 15 feet may be required in order to effectively remove the existing FILL and/or culvert system for the construction of a new bridge. At this time it is anticipated that the existing natural sloped embankments adjacent to the construction area will remain (side slopes). As such, fill operations are not expected to be required in order to establish the design grade elevations within the approach and embankment areas. Due to the conceptual stage of this project and the unknown necessity of sloped embankments on the canal side of the bridge option (end slopes), slope stability analysis has not been included in our scope of services at this time. Once the design has been selected and if needed, **G E T Solutions, Inc.** can complete a slope stability analysis upon request.

Our field exploration program included four (4) 30- to 50-foot deep Standard Penetration Test (SPT) borings drilled by **G E T Solutions, Inc.** within the proposed construction areas. The groundwater level was recorded at the boring locations and as observed through the wetness of the recovered soil samples during the drilling operations. The initial groundwater table was measured to occur at depths ranging from 14 to 18 feet below the varying existing grades at the boring locations, which visually appeared to correspond to the varying existing site grade elevations and the approximate water level within the existing canal. Existing site grade elevations were not available at this time. As such, the corresponding groundwater elevations could not be determined. The boreholes were backfilled upon completion for safety considerations. As such, the reported groundwater levels may not be indicative of the static groundwater level.

The following evaluations and recommendations were developed based on our field exploration and laboratory-testing program:

- Field testing program during construction to include, subgrade proofrolling, compaction testing, and test and production pile monitoring for axial capacity verification. All other applicable testing, inspections, and evaluations should be performed as indicated in the North Carolina State Building Code (2006 International Building Code with North Carolina Amendments) and/or the NCDOT Standard Specifications For Roads and Structures.

- Deep foundation design recommendations are provided herein for supporting the proposed single or multiple span bridge as well as the structural slab alternative.

- A seismic site classification of “D” is recommended for this site, based on which seismic designs should be incorporated. In order to substantiate the site classification and/or to determine if a site Class C can be used, if needed, a 100-foot deep SPT or CPT boring and/or soil shear wave velocity testing should be performed.

This summary briefly discusses some of the major topics mentioned in the attached report. Accordingly, this report should be read in its entirety to thoroughly evaluate the contents.

1.0 PROJECT INFORMATION

1.1 Project Authorization:

G E T Solutions, Inc. has completed our Geotechnical Engineering study for the proposed Juniper Trail Bridge project. The Geotechnical Engineering Services were conducted in general accordance with **G E T Solutions, Inc.** Proposal No. PEC16-175G, dated July 28, 2016. Authorization to proceed with the services was received in the form of an email from Ms. Rachael Patrick with the Town of Southern Shores on the date of July 28, 2016.

1.2 Project Description:

This project is in its conceptual stage and the precise characteristics have yet to be finalized. At this time, the proposed construction at this site is planned to consist of replacing the existing five (5) to ten (10) year old culvert with a single or a two (2) span bridge. It is our understanding that the replacement of the culvert is due to a combination of subsurface erosion and/or deflection of the culverts that are resulting in displacement of the asphalt pavement section and/or the development of sink holes. As an alternative to constructing a new bridge, it may be selected to construct a pile supported structural slab to span over the existing culvert in order to allow it to remain.

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If any of the noted information is incorrect or has changed, please inform G E T Solutions, Inc. so that we may amend the recommendations presented in this report, if appropriate.

1.3 Purpose and Scope of Services:

The purpose of this study was to obtain information on the general subsurface conditions at the proposed project site. The subsurface conditions encountered were then evaluated with respect to the available project characteristics. In this regard, engineering assessments for the following items were formulated:

1. General assessment of the soils revealed by the borings performed at the proposed development.

2. General location and description of potentially deleterious material encountered in the borings that may interfere with construction progress or structure performance, including existing fills or surficial/subsurface organics.

3. Soil subgrade preparation, including stripping, grading and compaction. Engineering criteria for placement and compaction of approved structural fill material.

4. Construction considerations for fill placement, subgrade preparation, and foundation excavations.

5. Feasibility of utilizing a deep foundation system for support of the proposed bridge or alternative structural slab option. Design parameters required for the foundation systems, including foundation sizes, allowable bearing pressures, foundation levels and expected total and differential settlements.

6. Seismic site classification provided based on the results of the SPT borings performed at the project site as well as our experience in the project area.

The scope of services did not include an environmental assessment for determining the presence or absence of wetlands or hazardous or toxic material in the soil, bedrock, surface water, groundwater or air, on or below or around this site. Prior to development of this site, an environmental assessment is advisable.

2.0 FIELD AND LABORATORY PROCEDURES

2.1 Field Exploration:

In order to explore the general subsurface soil types and to aid in developing associated foundation and retaining wall design parameters, four (4) 30- to 50-foot deep Standard Penetration Test (SPT) borings (designated as B-1 through B-4) were drilled within the conceptual limits of the proposed construction area.

The SPT borings were performed utilizing mud-rotary drilling techniques with a CME 45 truck mounted drill rig. Standard Penetration Tests were performed in the field in general accordance with ASTM D 1586. The tests were performed continuously from the existing ground surface to a depth of 12 feet and at 5-foot intervals thereafter to the boring termination depth. The soil samples were obtained with a standard 1.4" I.D., 2" O.D., 30-inch long split-spoon sampler. The sampler was driven with blows of a 140 lb. hammer falling 30 inches with the use of an automatic hammer. The number of blows required to drive the sampler each 6-inch increment of penetration was recorded and is shown on the boring logs. The sum of the second and third penetration increments is termed the SPT N-value (uncorrected for automatic hammer). A representative portion of each disturbed split-spoon sample was collected with each SPT, placed in a glass jar, sealed, labeled, and returned to our laboratory for review. Following the exploration procedures, the borings were backfilled with a neat cement grout mix in accordance with NCDENR requirements for aquifer protection. More specific information regarding boring depths and locations is provided in the following table (Table I – Boring Schedule).

Table I – Boring Schedule

Boring Number	Boring Depth (feet)	Boring Location
B-1	50	Approximately 20 feet North of the Center of Culvert; Northbound Lane
B-2	30	Approximately 20 feet South of the Center of Culvert; Southbound Lane
B-3	30	Approximately 45 feet North of the Center of Culvert; Southbound Lane
B-4	50	Approximately 40 feet South of the Center of Culvert; Northbound Lane

The boring locations were established by **G E T Solutions, Inc.** and GHK Development, Inc. and subsequently located in the field by a representative of **G E T Solutions, Inc.** by measuring from the existing site features. The approximate boring locations are shown on the attached "Boring Location Plan" (Appendix I).

2.2 Laboratory Testing:

Representative portions of all soil samples collected during drilling were sealed in glass jars, labeled and transferred to our laboratory for classification and analysis. The soil classification was performed by a Geotechnical Engineer in accordance with ASTM D2488.

A total of six (6) representative soil samples were selected and subjected to laboratory testing, which included natural moisture and -#200 sieve wash testing and analysis, in order to corroborate the visual classification. These test results are provided in the following table (Table II – Laboratory Test Results) and are also presented on the "Boring Log" sheets (Appendix III).

Table II - Laboratory Test Results

Boring No.	Depth (Ft)	Natural Moisture Content (%)	-#200 Sieve (%)	USCS Classification
B-1	8-10	12.5	3.8	SP
B-1	28-30	22.3	3.1	SP
B-2	18-20	15.9	1.1	SP
B-3	4-6	1.7	0.7	SP
B-4	0.7-2	2.3	1.7	SP
B-4	23-25	21.3	2.4	SP

3.0 SITE AND SUBSURFACE CONDITIONS

3.1 Site Location and Description

The project site is located along Juniper Trail and north of Sweetgum Lane in Southern Shores, North Carolina. At the time of our field services, the proposed construction areas were observed to consist of an existing canal crossing having a single corrugated metal culvert. Based on our site observations, the existing site grade elevations within the anticipated limits of construction appeared to have changes generally ranging from less than 1-foot in 50 linear feet to about 5 feet in 50 linear feet.

Figure 1: Juniper Trail Bridge Vicinity Map

More specifically, the existing elevations within the roadway alignments and at the culvert crossing appeared to be relatively level. However, the approach areas were visually estimated to have changes in existing elevations on the order of about 5 feet in 50 linear feet. Finally, the existing elevations along the sides of the construction area were estimated to have changes of approximately 5 to 6 feet in less than 40 feet.

The project site generally includes the existing asphalt paved Juniper Trail roadway alignment and is bisected by an existing canal having a width of approximately 10 to 15 feet. Furthermore, the site is bordered to the north and south by grass covered areas followed by lightly wooded residential parcels with existing residences as well as their associated concrete paved entrance driveways. The site location and its associated surrounding characteristics are illustrated in the Google Earth imagery (© 2016 Google) provided in “Figure 1” above.

3.2 Subsurface Soil Conditions

The results of our soil test borings generally indicated that the existing pavement section was composed of asphalt paving ranging from 2 to 5 inches in thickness underlain by GRAVEL (GP) ranging from 5 to 40 inches in thickness. More specific information regarding the existing pavement section thicknesses is provided in the following table (Table III – Existing Pavement Section Summary).

Table III – Existing Pavement Section Summary

Boring Number	Asphalt Thickness	Aggregate Base Type and Thickness
B-1	3 Inches	GRAVEL (GP) 18 Inches
B-2	2 Inches	GRAVEL (GP) 40 Inches
B-3	5 Inches	Not Encountered
B-4	3 Inches	GRAVEL (GP) 5 Inches

Underlying the existing pavement section, the encountered soils were generally noted to consist of FILL (SAND: SP, SP-SM; and/or GRAVEL: GP) and extended to depths ranging from 13.5 to 18 feet. As an exception, the GRAVEL (GP) aggregate base and FILL were not encountered at the location of boring B-3. The subsurface soils encountered beneath the existing pavement section and/or FILL soils, where present, were generally noted to consist of SAND (SP, SP-SM) having varying amounts of Silt. Finally, the granular soils encountered at the location of boring B-1 at a depth of 20 feet and extending to about 23.5 feet below existing grades were noted to contain Organics. More detailed information regarding the surficial material and subsurface soil descriptions and thicknesses are provided in the following table (Table IV – Subsurface Soil Summary).

Table IV - Subsurface Soil Summary

AVERAGE DEPTH (Feet)	STRATUM	DESCRIPTION	RANGES OF SPT(1) N-VALUES
Borings: B-1 through B-4			
0 to 0.17 – 0.42	Existing Asphalt	2 to 5 Inches of Asphalt	
0.17 – 0.42 to 0.67 – 3.5	Existing Aggregate Base	5 to 40 Inches of GRAVEL (GP); Not Encountered at Boring B-3	
0.67 – 3.5 to 13.5 – 18	FILL	GRAVEL (GP) with Sand and/or SAND (SP, SP-SM) having varying amounts of Silt and/or Gravel; Not Encountered at Boring DCP-3 Only	-
13.5 – 18 to 30 – 50	I	SAND (SP, SP-SM) with varying amounts of Silt. All borings were terminated within this stratum.	4 to 55
20 to 23.5	IA	SAND (SP-SM) with Silt and Organics; Boring B-1 Only	18

Note (1): SPT = Standard Penetration Test, N-Values in Blows-per-foot (Uncorrected).

It is noted that the topsoil designation references the presence of surficial organic laden soil, and does not represent any particular quality specification. It is recommended that this material be tested for approval prior to use.

The subsurface description is of a generalized nature provided to highlight the major soil strata encountered. The records of the subsurface exploration are included in Appendix III (Boring Log sheets) and in Appendix IV (Generalized Soil Profile), which should be reviewed for specific information as to the individual borings. The stratifications shown on the records of the subsurface exploration represent the conditions only at the actual boring locations. Variations may occur and should be expected between boring locations. The stratifications represent the approximate boundary between subsurface materials and the transition may be gradual.

3.3 Groundwater Information

The groundwater level, where encountered, was recorded at the boring locations as observed through the wetness of the recovered soil samples during the drilling operations. The initial groundwater table was measured to occur at depths ranging from 14 to 18 feet below the varying existing grades at the boring locations, which visually appeared to correspond to the varying existing site grade elevations and the approximate water level within the existing canal. Existing site grade elevations were not available at this time. As such, the corresponding groundwater elevations could not be determined.

The varying groundwater level is anticipated to be a result of the varying existing site grade elevations occurring within the proposed construction areas. The boreholes were backfilled upon completion for safety considerations as well as in accordance with NCDENR requirements for aquifer protection. As such, the reported groundwater levels may not be indicative of the static groundwater level.

Groundwater conditions will vary with environmental variations and seasonal conditions, such as the frequency and magnitude of rainfall patterns, as well as man-made influences, such as existing swales, drainage ponds, underdrains and areas of covered soil (paved parking lots, sidewalks, etc.). Seasonal groundwater fluctuations of $\pm$ 3 feet are common in the project's area; however, greater fluctuations have been documented. We recommend that the contractor determine the actual groundwater levels at the time of the construction to determine groundwater impact on the construction procedures.

4.0 EVALUATIONS AND RECOMMENDATIONS

This project is currently in its conceptual stage and it is unknown if a new single or dual span bridge is to be constructed and supported by deep foundations. Due to the unknown settlement potential (total and differential) developed by the existing FILL, shallow foundation supported abutments are not recommended. Alternatively, a pile supported structural slab may be designed and constructed to allow the existing culvert system to remain in place. As such, the following sections of this report provide design and construction recommendations associated each of the above noted options.

Our recommendations are based on the previously discussed project information, our interpretation of the soil test borings and laboratory data, and our observations during our site reconnaissance. If the proposed construction should vary from what was described, we request the opportunity to review our recommendations and make any necessary changes.

4.1 Deep Foundations

The recommendations presented in Sections 4.2.1 through 4.2.13 of this report are provided in the event that the proposed single or dual span bridge is to be supported by a deep foundation system following the necessary demolition and excavation procedures required to remove the existing pavement section, FILL, and subsurface culvert.

4.1.1 Driven Pile Foundation Design Recommendations:

Subsurface 1.5 to 4.5-foot thick deposits of FILL consisting of GRAVEL (GP) encountered at borings B-2 and B-4 at depths ranging from 13.5 and 12 feet below existing grades (respectively) may hinder pile foundation installations. Furthermore, the presence of Organic laden subsurface conditions at the location of boring B-1 indicates that this site is susceptible to more significant Organic conditions (i.e. stumps and/or root mat) that may inhibit pile driving procedures. As such, the pile foundation installation operations should be performed with caution and care to prevent pile damage that would decrease the potential allowable pile capacities. Furthermore, pre-augering and/or spudding of the pile locations may be required during installation procedures.

We conducted pile capacity analyses using static formulas with coefficients recommended by Geoffrey Myerhoff and George Sowers. The analyses include the contributions of shaft friction and end bearing to the pile capacity. The piles are expected to derive the majority of their capacity from end bearing in the deeper sand layer at the depths presented in the following table (Table V – Pile Capacities).

The allowable capacity for the piles includes a safety factor of at least 2.0 to allow for a pile load test program that relies primarily on dynamic testing. The capacity of a group of piles spaced at least 3 pile diameters apart, center to center, can be taken as the sum of the individual capacities with no reduction factor. If closer pile spacing is anticipated, the geotechnical engineer should be contacted to evaluate the efficiency of the specific pile group. The final order lengths and tip elevations will be adjusted based on the results of the test piles and load test programs.

Table V – Pile Capacities

Pile Type & Dimensions	Installation Method	Embedment Depth (ft)	Allowable Capacity in Compression (tons)	Allowable Capacity in Tension (tons)
Round Timber (minimum 8-inch tip diameter)	Driving	25-30	25	7
Precast Pre-Stressed Concrete (12-inch Square)	Driving	25-30	50	15

Note (1): The pile embedment depths noted above are referenced from below the current site grade elevations occurring at each specific boring location.

The noted pile embedment depths and pile tip elevations are required to achieve the allowable capacities. Any reduction in the length of embedment will correspond to a reduction in the allowable design capacity, unless otherwise directed by the geotechnical engineer after the pile-testing program.

Some piles may encounter the necessary penetration resistance and/or practical refusal at a pile embedment depth shallower than that noted above. This may occur as a result of densification occurring from the vibrations generated during pile installation of adjacent piles. In the event that refusal occurs, the Geotechnical Engineer will determine the tension capacities achieved relative to the penetration resistance, embedment depth, and pile type.

In order to minimize the reduction in capacity due to group action of the piles, it is recommended that the piles be installed with a center-to-center spacing of at least 3 feet. The driven piles should be advanced with an impact hammer to their design tip elevations. If for some reason during construction, pile "refusal" is encountered before piles reach their design tip elevation, the Geotechnical Engineer should be retained to review field records and reports before assuming the pile can adequately support the design capacity. If the pile driving hammer is not properly matched to the pile type, size, and subsurface conditions, it may reach practical refusal before the pile reaches the design tip elevation, or the required capacity.

The natural granular soil materials typically exhibit time-dependent strength characteristics, consequently shaft friction and end bearing support tend to increase from initial installation through a process termed "soil setup". Essentially, the dynamics of driving piles through these materials will cause excess pore pressures to develop, thereby decreasing driving resistance during initial pile installation. The pile capacities developed during driving are usually much lower than the design values. However, once driving is complete these excess pore pressures dissipate with time (and soil setup occurs) and the bearing capacity of the pile increases. Based upon our experience with similar projects in the area, 72 hours is usually required for the pore pressures to dissipate and soil setup to occur.

In order to confirm the required tip elevations, we recommend conducting a Test Pile Program prior to ordering production piles. The piles should be advanced by driving with an impact hammer to their design tip elevations. If for some reason during construction, pile driving "refusal" is encountered before the piles reach their design tip elevations, the Geotechnical Engineer should be retained to review driving records and field reports to determine whether the pile can adequately support the design loads. If the pile driving hammer is not properly matched to the pile type, size and subsurface conditions, it may reach practical refusal before the pile reaches the design tip elevation, or the required capacity.

4.1.2 Driven Pile Group Settlement:

Based on the results of load tests performed on piles in similar soil conditions, it is anticipated that the total butt settlements (including elastic shortening) will not exceed about ½-inch, which is the settlement necessary to mobilize the soil/pile capacity in combination with the pile tip settlements due to the stress increase in the underlying soils.

In order to minimize the reduction in capacity due to group action of the piles, it is recommended that the piles be installed with a center-to-center spacing of at least 3 feet.

4.1.3 Timber Piles:

It is recommended that the timber piles meet the requirements of ASTM D-25 for timber tip bearing piles. The piles should be clean peeled and pressure treated in accordance with the requirements of AWPA C3. The timber pile design stresses should be established in accordance with ASTM D-2899 and the local applicable Building Codes. Additionally, we recommend the timber piles be treated with ACA (ammoniated copper arsenate) or CCA (copper chrome arsenate) due to the location of the proposed structure in a temperature zone coastal environment. Prior to driving, it is recommended that timber piles be relatively free of defects and have a water content greater than approximately 20 percent (to minimize “breaking”) and less than about 50 percent (to minimize “brooming”).

4.1.4 Driven Test Piles:

We recommend that a test pile program be implemented for the purpose of assisting in the development of final tip elevations and to confirm that the contractor’s equipment and installation methods are acceptable. The test program should involve test piles to provide an indication of various driving and/or installation conditions. We recommend at least two (2) test piles (1 per abutment) be performed for the proposed structure. It is important to note the relationship between the required testing and our design assumptions. We chose safety factors based upon the recommended pile testing program. The selected pile foundations should be installed while monitored by a **G E T Solutions, Inc.** representative to obtain the design capacities previously noted in Section 4.2.1 of this report.

The test pile lengths and locations should be selected by the geotechnical engineer. The piles should be driven using the drive system submitted by the contractor and approved by the geotechnical engineer. Test pile lengths should be at least five (5) feet longer than anticipated production pile lengths to ensure that the required capacity is developed, to allow for refinement of estimated capacities, and for dynamic and static testing reasons. The indicator piles installed during the Test Pile Program which satisfy the geotechnical engineer’s requirements for proper installation may also be used as permanent project piles.

The contractor should include in his equipment submittal a Wave Equation Analyses (using GRLWEAP™ software) modeling the behavior of the test piles during driving, or what is termed by GRL as a "Driveability Study." The primary intent of the Wave Equation Analyses is to estimate the feasibility of the contractor's proposed pile driving system with respect to installing the piles. Since the results of the Wave Equation Analyses are dependent on the chosen hammer, the pile type and length, and the subsurface conditions, it is likely that at least one Wave Equation Analysis per bridge will be required. Depending on the difference in size between the abutment piles for a given bridge and the subsurface conditions, separate Wave Equation Analyses may be required.

Pile driving equipment should not be mobilized for the test piles until the Wave Equation Analyses have been submitted and approved by the geotechnical engineer. If the contractor's proposed pile driving system is rejected, subsequent submittals of alternative drive systems should also include appropriate Wave Equation Analyses that are subject to the approval of the geotechnical engineer. The Wave Equation Analyses are also used to estimate:

- Compressive and tensile stresses experienced by the modeled pile during driving
- The total number of blows required to install the pile
- Driving resistance (in terms of blows per foot) within the various soil strata the pile is embedded in
- Driving time

The results of the WEAP analyses are highly dependent on the many input parameters related to the soil conditions, static pile capacity estimates, as well as specific characteristics associated with different makes and models of pile driving hammers.

4.1.5 Dynamic Testing

Dynamic testing was developed as a method of improving upon the reliability of the wave equation and other dynamic predictions by actually measuring the acceleration and strain of a pile during driving. This technique was developed in the mid-1960's and has been continually refined. The use of dynamic pile testing has permitted the possibility of checking the driving stresses in the pile and the hammer performance during pile driving. It is also possible to estimate the static capacity of the pile based upon the strain and acceleration measurements taken during pile driving.

The test pile installation should be monitored by the Geotechnical Engineer using the PDA, an electronic device that records driving stresses and pile/soil interactions, among other things. The PDA results will confirm that the pile driving system (hammer type/energy, cushion type/ thickness, etc.) can successfully install the piles without over stressing them in compression or tension.

No sooner than 5 to 7 days after installation, all of the test piles should be re-struck while being monitored with the PDA. This test establishes the “static capacity” of the pile. The initial hammer blow during re-strike activities is critical to the quality of dynamic data with respect to capacity interpretation. The contractor should make every effort to insure an initial high-energy blow of the hammer. After several blows during re-strike activities, pore pressures increase, soil setup diminishes, and ultimate pile capacities (as recorded by the PDA) decrease. Loss of estimated static capacity following repeated hammer blows is the reason the initial blows are critical.

The dynamic data recorded by the PDA during restrike testing should be further refined by using CAPWAP® analysis. CAPWAP® analysis, not the initial assessment of capacity determined by the PDA, should be the basis of static pile capacity estimates. Interpretation of CAPWAP® data, in the context of the subsurface soil conditions and previous static pile capacity estimates, should allow the Geotechnical Engineer to estimate ultimate pile capacities and recommend appropriate production pile lengths.

Our previous experience with the PDA indicates that a significant cost savings may be realized if the PDA is properly utilized to monitor the installation of test piles, confirm pile capacity in production installations, and monitor potentially damaging stresses during driving. The use of the PDA permits the confirmation of allowable compression and uplift capacities and pile integrity on several piles for a cost similar to or less than that of a single full-scale static load test. We recommended the owner retain the services of the Geotechnical Engineer to perform the dynamic testing, not the installation contractor, to avoid possible conflicts of interest.

4.1.6 Establishing Pile Driving Criteria:

Prior to driving production piles, the geotechnical engineer should establish the criteria for pile installation. The criteria will be based on the test pile operations. The intent of establishing driving criteria is to facilitate installation of the production piles without damage and to provide a means of establishing when piles have achieved the design capacities. The driving criteria may include: hammer type, hammer energy, ram weight, pile cushion and thickness, hammer cushion type and thickness, required tip elevations and driving resistance necessary to achieve capacities, and possibly predrilling/jetting recommendations (if the test pile results warrant the need).

4.1.7 Allowable Driving Stresses

Guidelines from the Prestressed Concrete Institute (PCI), American Society of Civil Engineers (ASCE), and the American Association of State Highway Transportation Officials (AASHTO) indicate that maximum compressive stresses, imposed on driven precast concrete piles during installation, should be less than the following equation: 0.85 x f'_c (concrete compressive strength, psi) - f_{pe} (effective pre-stressing after losses from relaxation). The three groups differ on the maximum tensile stresses. PCI recommends 6 x square root of f'_c + f_{pe} ; AASHTO and ASCE recommend 3 x square root f'_c + f_{pe}. We recommend using the consensus value for the maximum compressive stress, and the ASCE/AASHTO recommended value for the maximum tensile stress.

4.1.8 Hammer Types and Energies

In comparing hammers of equal energy, the Prestressed Concrete Institute (PCI) states that hammers with heavier rams and lower impact velocities are less likely to cause damaging stresses in concrete piles. Hammers with proportionally higher ram weights and short stroke heights (low impact velocities) are usually air, steam and hydraulic driven, and not diesel fueled. It has been our experience that air, steam and hydraulic hammers are more appropriate for the installation of precast concrete piles than similarly sized (in terms of energy) diesel hammers. We recommend that the contractor use an air, steam or hydraulic driven hammer whose ram weight is roughly equal to 0.5 to 1.0 times the weight of the pile itself. The actual determination of an acceptable ram weight should be determined through the results of the Test Pile Program.

4.1.9 Driven Pile Installation Monitoring:

The geotechnical engineer should observe the installation of the test piles and all production piles. The purpose of the geotechnical engineer's observations is to determine if production installations are being performed in accordance with the previously derived Pile Driving Criteria. Continuous driving and installation records should be maintained for all piles. Production piles should be driven utilizing the approved system established as a result of the Test Program. In order to develop the allowable pile capacities provided in Section 4.5 of this report it is considered necessary to install the round timber piles by means of driving from the existing site grade elevations to the proposed tip elevations.

The field duties of the geotechnical engineer (or a qualified engineer's representative) should include the following:

1. Being knowledgeable of the subsurface conditions at the site and the project-specific Pile Driving Criteria.

2. Being aware of aspects of the installation including type of pile driving equipment and pile installation tolerances.

3. Keeping an accurate record of pile installation and driving procedures.

4. Documenting that the piles are installed to the proper depth indicative of the intended bearing stratum. Also documenting that appropriate pile splicing techniques are used, if necessary.

5. Recording the number of hammer blows for each foot of driving.

6. Generally confirming that the pile driving equipment is operating as anticipated. Record the energy rating of the hammer.

7. Informing the geotechnical engineer of any unusual subsurface conditions or driving conditions.

8. Notifying the contractor and structural engineer when unanticipated difficulties or conditions are encountered.

9. Confirming from visual appearance that the piles are not damaged during installation and observing the piles prior to installation for defective workmanship. The geotechnical engineer should review all driving records prior to pile cap construction.

4.1.10 Adjacent Structures:

When considering the suitability of a driven pile foundation, consideration should be given to the integrity of nearby structures. Due to the large amount of energy required to install driven deep foundations, vibrations of considerable magnitude are generated. These vibrations may affect nearby structures. These structures can, due to their proximity, be detrimentally affected by the construction unless proper protection measures are taken. In addition, experience has shown that these construction features will often lead adjacent property owners to conclude that damage to their property has taken place, even though none has occurred. It is therefore recommended that a thorough survey of the adjacent property be made prior to starting construction. This will help to better evaluate real claims and refute groundless nuisance claims. The survey should include, but not be limited to, the following:

1. Visually inspect adjacent structures, noting and measuring all cracks and other signs of distress. Take photographs as needed.

2. Visually inspect adjacent pavements, noting and measure any significant cracks, depressions, etc. Take photographs as needed.

3. Establish several bench marks along foundation walls on adjacent structures. Both vertical and horizontal control should be employed.

4. Determine if equipment in any adjacent building is sensitive to vibration, and if so, establish proper control and monitoring system.

4.2 Pile Supported Structural Slab

The recommendations presented in Section 4.2.1 of this report are provided in the event that the proposed construction to occur at this project site is to consist of a pile supported structural slab and allow the existing culvert to remain in place. However, it is noted that subsurface erosion below the existing pavement section has occurred and resulting in deformation of the asphalt surface. Accordingly, it is recommended that the Engineer of Record and/or a Structural Engineer evaluate the existing culvert to determine its suitability to remain in place. In the event that the existing culvert is failing, deflecting, and/or allowing significant subsurface erosion to occur, this should be considered in determining the suitability of a pile supported structural slab system. The design of the structural slab should be accomplished under the direction of a Licensed Structural Professional Engineer registered in North Carolina.

The design finished grade elevations of the pile supported structural slab are anticipated to coincide with the existing site grade elevations. As such, the construction areas are not anticipated to require the placement of structural fill to establish the design grades. In the event that structural fill is required to establish the design grade elevations, recommendations for fill quality, placement, and compaction can be provided upon request.

Following the removal of the existing asphalt and/or underlying GRAVEL (GP) aggregate base, where encountered, the result exposed subgrade should be densified with a large static smooth drum roller. Care should be used when operating the compactors near existing structures and/or above the existing culvert to avoid transmission of the vibrations that could cause settlement damage or disturb occupants. In this regard, it is recommended that the vibratory roller remain at least 25 feet away from existing structures; these areas should be compacted with small, hand-operated compaction equipment.

4.2.1 Deep Foundation Design and Construction Recommendations

The proposed structural slab should be supported by a deep foundation system designed and constructed consistent with that indicated in Sections 4.1 and 4.1.1 through 4.1.10 of this report. In the event that significantly lighter compression capacities are required, a supplemental deep foundation analysis can be provided up on request.

4.3 Seismic Evaluation

It is noted that, in accordance with the NC Building Code; Chapter 16, this site would be classified as a site Class D, based on which seismic designs should be incorporated. This recommendation is based on the data obtained from the 50-foot deep SPT borings, our experience in the project, as well as the requirements indicated in the 2012 North Carolina State Uniform Building Code. In order to substantiate the site classification and/or to determine if a site Class C can be used, if needed, a 100-foot deep SPT or CPT boring and/or soil shear wave velocity testing should be performed.

5.0 CONSTRUCTION CONSIDERATIONS

5.1 Drainage and Groundwater Concerns

It is expected that dewatering may be required for excavations that extend near or below the existing groundwater table. Dewatering above the groundwater level could probably be accomplished by pumping from sumps. Dewatering at depths below the groundwater level may require well pointing.

It is recommended that all building and site drainage (such as surface water drains, roof down spouts, etc.) be designed to include a discharge location away from the foundation areas to promote rapid positive drainage. Furthermore, we recommend the ground surface be sloped away from the foundation for a minimum distance of 10 feet and that all downspouts (if applicable) be connected to tightline drains that discharge to a suitable location down slope of the foundations.

It would be advantageous to construct all fills early in the construction. If this is not accomplished, disturbance of the existing site drainage could result in collection of surface water in some areas, thus rendering these areas wet and very loose. Temporary drainage ditches should be employed by the contractor to accentuate drainage during construction.

5.2 Site Utility Installation

The base of the utility trenches should be observed by a qualified inspector prior to the pipe and structure placements to verify the suitability of the bearing soils. If unstable bearing soils are encountered during installation, some form of stabilization may be required to provide suitable bedding. This stabilization is typically accomplished by providing additional bedding materials (NCDOT No. 57 stone). In addition, depending on the depth of the utility trench excavation, some means of dewatering may be required to facilitate the utility installation and associated backfilling.

All utility excavations should be backfilled with structural fill, as described in Section 4.3 of this report. Additional information regarding the suitability of the existing soils for re-use as backfill and/or fill is provided in Section 4.4 of this report.

5.3 Excavations

In Federal Register, Volume 54, No. 209 (October, 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its “Construction Standards for Excavations, 29 CFR, part 1926, Subpart P”. This document was issued to better insure the safety of workmen entering trenches or excavations. It is mandated by this federal regulation that all excavations, whether they be utility trenches, basement excavation or footing excavations, be constructed in accordance with the new (OSHA) guidelines. It is our understanding that these regulations are being strictly enforced and if they are not closely followed, the owner and the contractor could be liable for substantial penalties.

The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor’s responsible person, as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor’s safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations.

We are providing this information solely as a service to our client. **G E T Solutions, Inc.** is not assuming responsibility for construction site safety or the contractor’s activities; such responsibility is not being implied and should not be inferred.

6.0 REPORT LIMITATIONS

The recommendations submitted are based on the available soil information obtained by **G E T Solutions, Inc.** and the information supplied by the client for the proposed project. If there are any revisions to the plans for this project or if deviations from the subsurface conditions noted in this report are encountered during construction, **G E T Solutions, Inc.** should be notified immediately to determine if changes in the foundation recommendations are required. If **G E T Solutions, Inc.** is not retained to perform these functions, **G E T Solutions, Inc.** can not be responsible for the impact of those conditions on the geotechnical recommendations for the project.

The Geotechnical Engineer warrants that the findings, recommendations, specifications or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area. No other warranties are implied or expressed.

After the plans and specifications are more complete the Geotechnical Engineer should be provided the opportunity to review the final design plans and specifications to assure our engineering recommendations have been properly incorporated into the design documents, in order that the earthwork and foundation recommendations may be properly interpreted and implemented. At that time, it may be necessary to submit supplementary recommendations. This report has been prepared for the exclusive use of the Town of Southern Shores and their consultants for the specific application to the proposed Juniper Trail Bridge project located in Southern Shores, North Carolina.

APPENDICES

APPENDIX I	BORING LOCATION PLAN
APPENDIX II	SUMMARY OF SOIL CLASSIFICATION
APPENDIX III	BORING LOGS
APPENDIX IV	GENERALIZED SOIL PROFILES

APPENDIX I

BORING LOCATION PLAN

LEGEND
— APPROXIMATE BORING LOCATION

GET Solutions, Inc.
Geotechnical • Environmental • Testing

Project Name: JUNIPER TRAIL BRIDGE SOUTHERN SHORES, NORTH CAROLINA	
Project No.: EC16-246G	Drawn By: GS
Date: 10/21/16	Figure No.: 1
BORING LOCATION PLAN	SCALE: NOT TO SCALE

APPENDIX II

SUMMARY OF SOIL CLASSIFICATION

Project: Juniper Trial Bridge
Project Location: Southern Shores, North Carolina
Project Number: EC16-246G

GET Solutions, Inc.
Geotechnical • Environmental • Testing

Key to Soil Symbols and Terms

TERMS DESCRIBING CONSISTENCY OR CONDITION

COARSE-GRAINED SOILS (major portions retained on No. 200 sieve): includes (1) clean gravel and sands and (2) silty or clayey gravels and sands. Condition is rated according to relative density as determined by laboratory tests or standard penetration resistance tests.

Descriptive Terms	Relative Density	SPT Blow Count
Very loose	0 to 15 %	< 4
Loose	15 to 35 %	4 to 10
Medium dense	35 to 65 %	10 to 30
Dense	65 to 85 %	30 to 50
Very dense	85 to 100 %	> 50

FINE-GRAINED SOILS (major portions passing on No. 200 sieve): includes (1) inorganic and organic silts and clays, (2) gravelly, sandy, or silty clays, and (3) clayey silts. Consistency is rated according to shearing strength, as indicated by penetrometer readings, SPT blow count, or unconfined compression tests.

Descriptive Terms	Unconfined Compressive Strength kPa	SPT Blow Count
Very soft	< 25	< 2
Soft	25 to 50	2 to 4
Medium stiff	50 to 100	4 to 8
Stiff	100 to 200	8 to 15
Very stiff	200 to 400	15 to 30
Hard	> 400	> 30

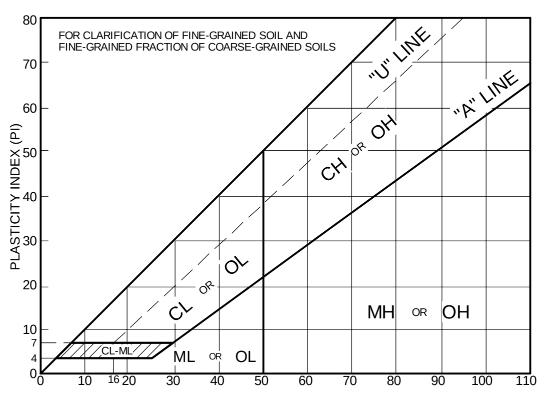
GENERAL NOTES

1. Classifications are based on the United Soil Classification System and include consistency, moisture, and color. Field descriptions have been modified to reflect results of laboratory tests where deemed appropriate.

2. Surface elevations are based on topographic maps and estimated locations.

3. Descriptions on these boring logs apply only at the specific boring locations and at the time the borings were made. They are not guaranteed to be representative of subsurface conditions at other locations or times.

Major Divisions			Group Symbols	Typical Names	Laboratory Classification Criteria		
Coarse-Grained soils (More than half the material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravel (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain size curve, Depending on percentage of fines (fraction smaller than No. 200 sieve) coarse-grained soils are classified as follows: Less than 5 percent....... GW, GP, SW, SP More than 12 percent...... GM, GC, SM, SC 6 to 12 percent...... Borderline case4s requiring dual symbols**	$C_U = \frac{D_{60}}{D_{10}}$ greater than 4; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW	
		Gravel with fines (Appreciable amount of fines)	GM* d / u	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are border-line cases requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-silt mixtures		Atterberg limits above "A" line or P.I. greater than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines		$C_U = \frac{D_{60}}{D_{10}}$ greater than 6; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP	Poorly-graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM* d / u	Silty sands, sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are border-line cases requiring use of dual symbols
			SC	Clayey sands, sand-clay mixtures		Atterberg limits above "A" line or P.I. greater than 7	
Fine-Grained soils (More than half the material is smaller than No. 200 sieve size)	Silts and Clays (Liquid limit less than 60)		ML	Inorganic silts and very fine sands, rock floor, silty or clayey fine sands or clayey silts with slight plasticity	Plasticity Chart		
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
			OL	Organic silts and organic silty clays of low plasticity			
	Silts and Clays (Liquid limit greater than 60)		MH	Inorganic silts, micaceous or disto-maceous fine sandy or silty soils, organic silts			
			CH	Inorganic clays of high plasticity, fat clays			
			OH	Organic clays of medium to high plasticity, organic silts			
	Highly Organic Soils		Pt	Peat and other highly organic soils			



Plasticity Chart

Material		Particle Size mm	Sieve sizes
Silt or clay		< 0.074	< #200
Sand	Fine	0.074 to 0.42	#200 to #40
	Medium	0.42 to 2.00	#40 to #10
	Coarse	2.00 to 4.76	#10 to #4
Gravel	Fine	4.76 to 19.1	#4 to 3/4 in.
	Coarse	19.1 to 76.2	3/4 in. to 3 in.
Cobble		76.2 to 304.8	3 in. to 12 in.
Boulders		304.8 to 914.4	12 in. to 36 in.

* Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg Limits: suffix d used when L.L. is 23 or less and the P.I. is 6 or less; the suffix is used when L.L. is greater than 26.

** Borderline classifications used for soils possessing characteristics of two groups are designated by combinations of groups symbols. For example; GW-GC, well-graded gravel-sand mixture with clay binder.

MAJOR DIVISIONS					TYPICAL NAMES
COARSE GRAINED SOILS More than Half > #200 sieve	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW		WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES
			GP		POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES
		GRAVELS WITH OVER 15% FINES	GM		SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES
			GC		CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS WITH LITTLE OR NO FINES	SW		WELL GRADED SANDS, GRAVELLY SANDS
			SP		POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH OVER 15% FINES	SM		SILTY SANDS, POOORLY GRADED SAND-SILT MIXTURES
			SC		CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS More than Half < #200 sieve	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML		INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL		INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL		ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH		INORGANIC SILTS, MICACEOUS OR DIATOMACIOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH		INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH		ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			Pt		PEAT AND OTHER HIGHLY ORGANIC SOILS

	Modified California	RV	R-Value
	Split Spoon	SA	Sieve Analysis
	Pushed Shelby Tube	SW	Swell Test
	Auger Cuttings	TC	Cyclic Triaxial
	Grab Sample	TX	Unconsolidated Undrained Triaxial
	Sample Attempt with No Recovery	TV	Torvane Shear
CA	Chemical Analysis	UC	Unconfined Compression
CN	Consolidation	(1.2)	(Shear Strength, ksf)
CP	Compaction	WA	Wash Analysis
DS	Direct Shear	(20)	(with % Passing No. 200 Sieve)
PM	Permeability	▽	Water Level at Time of Drilling
PP	Pocket Penetrometer	▼	Water Level after Drilling(with date measured)

SOIL CLASSIFICATION CHART AND KEY TO TEST DATA

Juniper Trial Bridge
Southern Shores, North Carolina

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(1) GET - CLASSIFICATION LEGEND - - 10/14/16 15:58 - G:\GINT\PROJECTS\EC16\EC16-246G JUNIPER TRAIL BR.GPJ

APPENDIX III

BORING LOGS

RECORD OF SUBSURFACE EXPLORATION

GET Solutions, Inc.
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Virginia Beach
204 Grayson Road
Virginia Beach, VA 23642
757-518-1703

Williamsburg
1592-E Penniman Road
Williamsburg, VA 23185
757-564-6452

Elizabeth City
106 Capital Trace Unit E
Elizabeth City, NC 27909
252-335-9765

Jacksonville
415-A Western Blvd
Jacksonville, NC 28546
910-478-9915

BORING ID B-1

PROJECT NAME: **Juniper Trial Bridge**
PROJECT NUMBER: **EC16-246G**
CLIENT: **Town of Southern Shores**
SURFACE ELEVATION (MSL) (ft):
PROJECT LOCATION: **Southern Shores, North Carolina**
LOGGED BY: **GS**
BORING LOCATION: **See Attached Boring Location Plan**
DATE STARTED: **8/8/2016**
DRILLING METHOD(S): **Rotary wash "mud"**
DATE COMPLETED: **8/8/2016**
GROUNDWATER*: INITIAL (ft) ▽: **15** AFTER ____ HOURS (ft) ▼: ____ CAVE-IN (ft): ____
DRILLER: **GET Solutions, Inc.**

The initial groundwater readings are not intended to indicate the static groundwater level.

TEST RESULTS: Plastic Limit X——X Liquid Limit; Water Content - ●; Penetration - (hatched bar); scale 10 20 30 40 50 60 70

Elevation (ft)	Depth (ft)	STRATA DESCRIPTION	Sample ID	Sample Type	Sample Recovery (in.)	Blow Counts (N-Values)	%<#200
	0.3	3 Inches Asphalt					
	1.8	18 Inches GRAVEL (GP; FILL)	1	SS	0	0-0-0-0 (0)	
		Tan, moist to wet, poorly graded fine SAND (SP) to poorly graded fine SAND (SP-SM) with Silt (FILL)	2	SS	16	3-3-3-3 (6)	
	5		3	SS	19	2-3-3-4 (6)	
			4	SS	20	3-2-2-3 (4)	
			5	SS	12	2-3-3-3 (6)	4
	10		6	SS	16	2-2-2-3 (4)	
		Trace to Little fine Gravel from 13.5 Feet	7	SS	6	2-2-2-2 (4)	
▽	15	Wet from 15 Feet					
	18.0	Gray, wet, poorly graded fine to medium SAND (SP) to poorly graded fine to medium SAND (SP-SM) with Silt, very loose	8	SS	12	1-2-2-3 (4)	
	20.0	Dark Brown, wet, poorly graded fine to medium SAND (SP-SM) with Silt and Organics, medium dense					
	23.5	Gray, wet, poorly graded fine to medium SAND (SP) to poorly graded fine to medium SAND (SP-SM) with Silt, medium dense to dense	9	SS	14	2-5-13-15 (18)	
	25						
			10	SS	18	10-17-20-25 (37)	3
	30						
			11	SS	18	12-16-17-18 (33)	
	35						

Sample Type(s):
SS - Split Spoon

Notes:

RECORD OF SUBSURFACE EXPLORATION

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204 Grayson Road
Virginia Beach, VA 23642
757-518-1703

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1592-E Penniman Road
Williamsburg, VA 23185
757-564-6452

Elizabeth City
106 Capital Trace Unit E
Elizabeth City, NC 27909
252-335-9765

Jacksonville
415-A Western Blvd
Jacksonville, NC 28546
910-478-9915

BORING ID B-1

PROJECT NAME: **Juniper Trial Bridge**
CLIENT: **Town of Southern Shores**
PROJECT LOCATION: **Southern Shores, North Carolina**
BORING LOCATION: **See Attached Boring Location Plan**
DRILLING METHOD(S): **Rotary wash "mud"**
GROUNDWATER*: INITIAL (ft) ▽: **15** AFTER ____ HOURS (ft) ▼: ____ CAVE-IN (ft): ____
The initial groundwater readings are not intended to indicate the static groundwater level.

PROJECT NUMBER: **EC16-246G**
SURFACE ELEVATION (MSL) (ft): ____
LOGGED BY: **GS**
DATE STARTED: **8/8/2016**
DATE COMPLETED: **8/8/2016**
DRILLER: **GET Solutions, Inc.**

Elevation (ft)	Depth (ft)	STRATA DESCRIPTION	Sample ID	Sample Type	Sample Recovery (in.)	Blow Counts (N-Values)	%<#200
		Gray, wet, poorly graded fine to medium SAND (SP) to poorly graded fine to medium SAND (SP-SM) with Silt, medium dense to dense (layer continued from previous page)					
	40		12	SS	20	12-18-16-22 (34)	
	43.0	Gray, wet, Silty fine SAND (SM), medium dense to dense					
	45		13	SS	18	14-18-17-20 (35)	
	50.0		14	SS	19	9-10-14-17 (24)	
	50	Boring terminated at 50 feet below existing grade.					

TEST RESULTS
Plastic Limit X———X Liquid Limit
Water Content - ●
Penetration - ////
10 20 30 40 50 60 70

Sample Type(s):
SS - Split Spoon

Notes:

This information pertains only to this boring and should not be interpreted as being indicitive of the site.

RECORD OF SUBSURFACE EXPLORATION

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204 Grayson Road
Virginia Beach, VA 23642
757-518-1703

Williamsburg
1592-E Penniman Road
Williamsburg, VA 23185
757-564-6452

Elizabeth City
106 Capital Trace Unit E
Elizabeth City, NC 27909
252-335-9765

Jacksonville
415-A Western Blvd
Jacksonville, NC 28546
910-478-9915

BORING ID B-2

PROJECT NAME: **Juniper Trial Bridge**
CLIENT: **Town of Southern Shores**
PROJECT LOCATION: **Southern Shores, North Carolina**
BORING LOCATION: **See Attached Boring Location Plan**
DRILLING METHOD(S): **Rotary wash "mud"**
GROUNDWATER*: INITIAL (ft) ▽: **14** AFTER ____ HOURS (ft) ▼: ____ CAVE-IN (ft): ____
The initial groundwater readings are not intended to indicate the static groundwater level.

PROJECT NUMBER: **EC16-246G**
SURFACE ELEVATION (MSL) (ft):
LOGGED BY: **GS**
DATE STARTED: **8/8/2016**
DATE COMPLETED: **8/8/2016**
DRILLER: **GET Solutions, Inc.**

Depth (ft)	Strata Description	Sample ID	Sample Type	Sample Recovery (in.)	Blow Counts (N-Values)	%<#200	Test Results
0.2	2 Inches Asphalt						
0.2–3.5	40 Inches GRAVEL (GP; FILL)	1	SS	0	1-1-1-1 (2)		
		2	SS	12	1-1-1-1 (2)		
3.5–13.5	Tan, moist to very moist, poorly graded fine SAND (SP) to poorly graded fine SAND (SP-SM) with Silt (FILL)	3	SS	18	1-2-1-2 (3)		
		4	SS	19	2-2-2-2 (4)		
	With Trace to Little Gravel from 8 Feet	5	SS	10	3-5-6-3 (11)		
		6	SS	10	9-16-2-2 (18)		
13.5–18.0	Tan-Gray, very moist to wet, GRAVEL (GP) with Sand (FILL) Wet from 14 Feet	7	SS	6	1-2-2-2 (4)		
18.0–30.0	Gray, wet, poorly graded fine SAND (SP) to poorly graded fine SAND (SP-SM) with Silt, very loose to medium dense	8	SS	12	2-2-2-2 (4)	1	
		9	SS	24	2-10-15-13 (25)		
		10	SS	10	12-13-7-10 (20)		
30.0	Boring terminated at 30 feet below existing grade.						

TEST RESULTS: Plastic Limit x—x Liquid Limit; Water Content - ●; Penetration - (hatched bar). Scale: 10 20 30 40 50 60 70

Sample Type(s):
SS - Split Spoon

Notes:

This information pertains only to this boring and should not be interpreted as being indicitive of the site.

RECORD OF SUBSURFACE EXPLORATION

GET Solutions, Inc.
Geotechnical • Environmental • Testing

Virginia Beach	Williamsburg	Elizabeth City	Jacksonville
204 Grayson Road	1592-E Penniman Road	106 Capital Trace Unit E	415-A Western Blvd
Virginia Beach, VA 23642	Williamsburg, VA 23185	Elizabeth City, NC 27909	Jacksonville, NC 28546
757-518-1703	757-564-6452	252-335-9765	910-478-9915

BORING ID B-3

PROJECT NAME: **Juniper Trial Bridge**
CLIENT: **Town of Southern Shores**
PROJECT LOCATION: **Southern Shores, North Carolina**
BORING LOCATION: **See Attached Boring Location Plan**
DRILLING METHOD(S): **Rotary wash "mud"**
GROUNDWATER*: INITIAL (ft) ∇: **16** AFTER ____ HOURS (ft) ▼: ____ CAVE-IN (ft): ____
The initial groundwater readings are not intended to indicate the static groundwater level.

PROJECT NUMBER: **EC16-246G**
SURFACE ELEVATION (MSL) (ft):
LOGGED BY: **GS**
DATE STARTED: **8/8/2016**
DATE COMPLETED: **8/8/2016**
DRILLER: **GET Solutions, Inc.**

Elevation (ft)	Depth (ft)	STRATA DESCRIPTION	Sample ID	Sample Type	Sample Recovery (in.)	Blow Counts (N-Values)	%<#200
	0.4	5 Inches Asphalt					
		Tan, moist to wet, poorly graded fine SAND (SP) to poorly graded fine SAND (SP-SM) with Silt, medium dense	1	SS	13	5-6-7 (11)	
			2	SS	22	4-6-7-7 (13)	
			3	SS	22	5-7-7-7 (14)	1
			4	SS	24	5-5-6-6 (11)	
			5	SS	16	6-9-10-10 (19)	
			6	SS	13	3-8-8-8 (16)	
			7	SS	16	2-6-8-10 (14)	
		Gray-Brown from 14 Feet					
∇		Wet from 16 Feet					
		Gray from 18 Feet	8	SS	12	5-7-10-16 (17)	
			9	SS	12	8-12-14-16 (26)	
			10	SS	14	8-10-12-14 (22)	
	30.0	Boring terminated at 30 feet below existing grade.					

TEST RESULTS: Plastic Limit X—X Liquid Limit; Water Content - ●; Penetration - (hatched bar); scale 10 20 30 40 50 60 70

Sample Type(s):
SS - Split Spoon

Notes:

This information pertains only to this boring and should not be interpreted as being indicitive of the site.

RECORD OF SUBSURFACE EXPLORATION

GET Solutions, Inc.
Geotechnical • Environmental • Testing

Virginia Beach
204 Grayson Road
Virginia Beach, VA 23642
757-518-1703

Williamsburg
1592-E Penniman Road
Williamsburg, VA 23185
757-564-6452

Elizabeth City
106 Capital Trace Unit E
Elizabeth City, NC 27909
252-335-9765

Jacksonville
415-A Western Blvd
Jacksonville, NC 28546
910-478-9915

BORING ID B-4

PROJECT NAME: **Juniper Trial Bridge**
PROJECT NUMBER: **EC16-246G**
CLIENT: **Town of Southern Shores**
SURFACE ELEVATION (MSL) (ft):
PROJECT LOCATION: **Southern Shores, North Carolina**
LOGGED BY: **GS**
BORING LOCATION: **See Attached Boring Location Plan**
DATE STARTED: **8/8/2016**
DRILLING METHOD(S): **Rotary wash "mud"**
DATE COMPLETED: **8/8/2016**
GROUNDWATER*: INITIAL (ft) ▽: **18** AFTER ____ HOURS (ft) ▼: ____ CAVE-IN (ft): ____
DRILLER: **GET Solutions, Inc.**

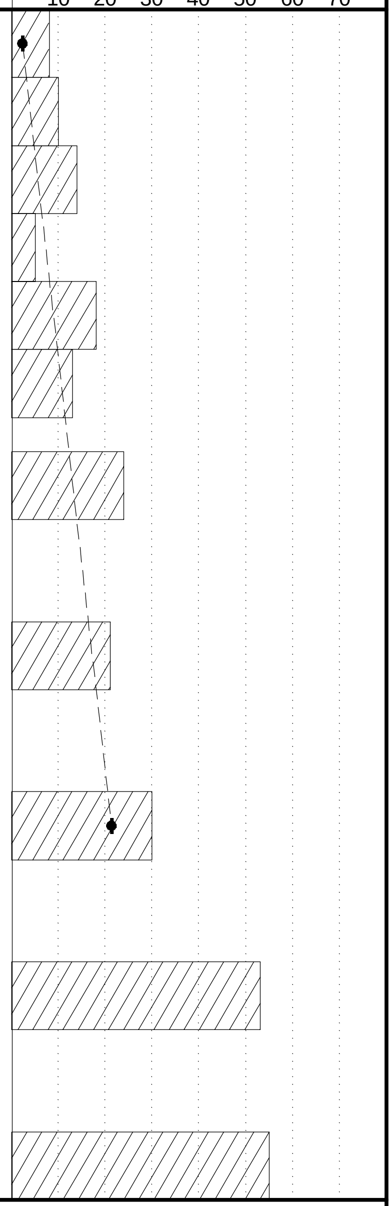
The initial groundwater readings are not intended to indicate the static groundwater level.

Elevation (ft)	Depth (ft)	STRATA DESCRIPTION
	0.3	3 Inches Asphalt
	0.7	5 Inches Gravel (GP: FILL)
		Tan, moist to very moist, poorly graded fine SAND (SP) to poorly graded fine SAND (SP-SM) with Silt (FILL)
	12.0	Gray, very moist, GRAVEL (GP) with Sand (FILL)
	13.5	Gray, very moist to wet, poorly graded fine SAND (SP) to poorly graded fine SAND (SP-SM) with Silt, medium dense to very dense
▽	18	Wet from 18 Feet

Sample ID	Sample Type	Sample Recovery (in.)	Blow Counts (N-Values)	%<#200
1	SS	12	3-5-7 (8)	2
2	SS	16	4-5-5-5 (10)	
3	SS	24	5-6-8-9 (14)	
4	SS	10	2-2-3-6 (5)	
5	SS	14	4-9-9-8 (18)	
6	SS	14	3-4-9-10 (13)	
7	SS	12	5-11-13-21 (24)	
8	SS	19	6-10-11-13 (21)	
9	SS	24	7-14-16-18 (30)	2
10	SS	22	12-23-30-27 (53)	
11	SS	20	15-22-33-33 (55)	

TEST RESULTS

Plastic Limit X——X Liquid Limit
Water Content - ●
Penetration - ////



Sample Type(s):
SS - Split Spoon

Notes:

This information pertains only to this boring and should not be interpreted as being indicitive of the site.

RECORD OF SUBSURFACE EXPLORATION

GET Solutions, Inc.
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Jacksonville
415-A Western Blvd
Jacksonville, NC 28546
910-478-9915

BORING ID B-4

PROJECT NAME: **Juniper Trial Bridge**
CLIENT: **Town of Southern Shores**
PROJECT LOCATION: **Southern Shores, North Carolina**
BORING LOCATION: **See Attached Boring Location Plan**
DRILLING METHOD(S): **Rotary wash "mud"**
GROUNDWATER*: INITIAL (ft) ▽: **18** AFTER ____ HOURS (ft) ▼: ____ CAVE-IN (ft): ____
The initial groundwater readings are not intended to indicate the static groundwater level.

PROJECT NUMBER: **EC16-246G**
SURFACE ELEVATION (MSL) (ft):
LOGGED BY: **GS**
DATE STARTED: **8/8/2016**
DATE COMPLETED: **8/8/2016**
DRILLER: **GET Solutions, Inc.**

TEST RESULTS: Plastic Limit X——X Liquid Limit; Water Content - ●; Penetration - (hatched bar); scale 10 20 30 40 50 60 70

Elevation (ft)	Depth (ft)	STRATA DESCRIPTION	Strata Legend	Sample ID	Sample Type	Sample Recovery (in.)	Blow Counts (N-Values)	%<#200
		Gray, very moist to wet, poorly graded fine SAND (SP) to poorly graded fine SAND (SP-SM) with Silt, medium dense to very dense (layer continued from previous page)						
				12	SS	20	13-23-24-25 (47)	
	40							
	43.0	Gray, wet, Silty fine SAND (SM), dense		13	SS	19	13-20-18-16 (38)	
	45							
				14	SS	24	11-18-15-18 (33)	
	50.0	Boring terminated at 50 feet below existing grade.						

Sample Type(s):
SS - Split Spoon

Notes:

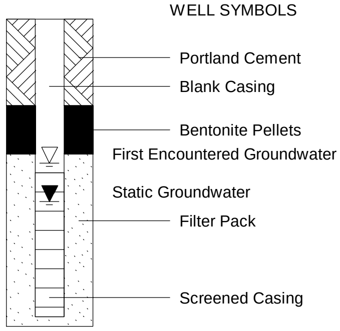
MAJOR DIVISIONS				TYPICAL NAMES
COARSE-GRAINED SOILS MORE THAN HALF IS COARSER THAN NO. 200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS WITH LESS THAN 15% FINES	GW	WELL-GRADED GRAVELS WITH OR WITHOUT SAND
			GP	POORLY-GRADED GRAVELS WITH OR WITHOUT SAND
		GRAVELS WITH 15% OR MORE FINES	GM	SILTY GRAVELS WITH OR WITHOUT SAND
			GC	CLAYEY GRAVELS WITH OR WITHOUT SAND
	SANDS MORE THAN HALF COARSE FRACTION IS FINER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LESS THAN 15% FINES	SW	WELL-GRADED SANDS WITH OR WITHOUT GRAVEL
			SP	POORLY-GRADED SANDS WITH OR WITHOUT GRAVEL
		SANDS WITH 15% OR MORE FINES	SM	SILTY SANDS WITH OR WITHOUT GRAVEL
			SC	CLAYEY SANDS WITH OR WITHOUT GRAVEL
FINE-GRAINED SOILS MORE THAN HALF IS FINER THAN NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50% OR LESS		ML	INORGANIC SILTS OF LOW TO MEDIUM PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
			OL	ORGANIC SILTS OR CLAYS OF LOW TO MEDIUM PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50%		MH	INORGANIC SILTS OF HIGH PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
			CH	INORGANIC CLAYS OF HIGH PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
			OH	ORGANIC SILTS OR CLAYS OF HIGH PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
HIGHLY ORGANIC SOILS			PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

SYMBOLS KEY

SAMPLE TYPES

Split Spoon

WELL SYMBOLS



ABBREVIATION KEY

CA	-	CHEMICAL ANALYSIS (CORROSIVITY)
CD	-	CONSOLIDATED DRAINED TRIAXIAL
CN	-	CONSOLIDATION
CU	-	CONSOLIDATED UNDRAINED TRIAXIAL
DS	-	DIRECT SHEAR
PP	-	POCKET PENETROMETER (TSF)
(3.0)	-	(WITH SHEAR STRENGTH IN KSF)
RV	-	R-VALUE
SA	-	SIEVE ANALYSIS: % PASSING #200 SIEVE
(200)	-	(WITH % PASSING NO. 200 SIEVE
SW	-	SWELL TEST
TC	-	CYCLIC TRIAXIAL
TV	-	TORVANE SHEAR
UC	-	UNCONFINED COMPRESSION
(1.5)	-	(WITH SHEAR STRENGTH IN KSF)
UU	-	UNCONSOLIDATED UNDRAINED TRIAXIAL
WA	-	WASH ANALYSIS
(200%)	-	(WITH % PASSING NO. 200 SIEVE)

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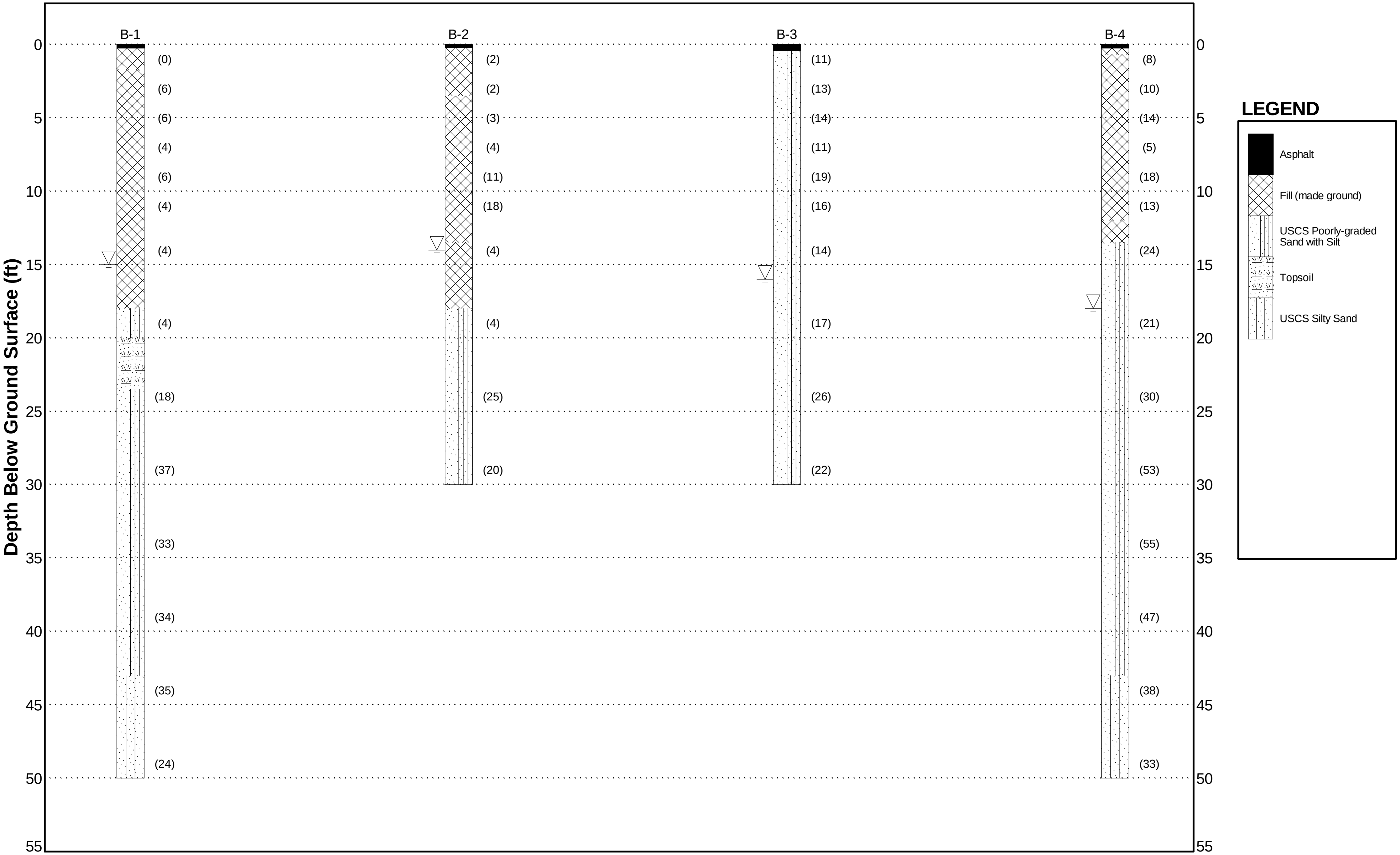
(2) GET - BORING LEGEND - - 10/14/16 15:58 - G:\GINT\PROJECTS\EC16\EC16-246G JUNIPER TRAIL BR.GPJ

APPENDIX IV

GENERALIZED SOIL PROFILE

GENERALIZED SOIL PROFILE

PROJECT NAME: **Juniper Trial Bridge**
PROJECT NUMBER: **EC16-246G**
PROJECT LOCATION: **Southern Shores, North Carolina**
CLIENT: **Town of Southern Shores**



(Numerical Value) = Sample N-Value

(2) GET - BORING LEGEND - - 10/14/16 15:58 - G:\GINT\PROJECTS\EC16\EC16-246G JUNIPER TRAIL BR.GPJ

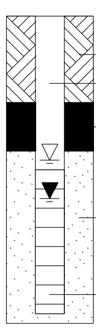
MAJOR DIVISIONS				TYPICAL NAMES
COARSE-GRAINED SOILS MORE THAN HALF IS COARSER THAN NO. 200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS WITH LESS THAN 15% FINES	GW	WELL-GRADED GRAVELS WITH OR WITHOUT SAND
			GP	POORLY-GRADED GRAVELS WITH OR WITHOUT SAND
		GRAVELS WITH 15% OR MORE FINES	GM	SILTY GRAVELS WITH OR WITHOUT SAND
			GC	CLAYEY GRAVELS WITH OR WITHOUT SAND
	SANDS MORE THAN HALF COARSE FRACTION IS FINER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LESS THAN 15% FINES	SW	WELL-GRADED SANDS WITH OR WITHOUT GRAVEL
			SP	POORLY-GRADED SANDS WITH OR WITHOUT GRAVEL
		SANDS WITH 15% OR MORE FINES	SM	SILTY SANDS WITH OR WITHOUT GRAVEL
			SC	CLAYEY SANDS WITH OR WITHOUT GRAVEL
FINE-GRAINED SOILS MORE THAN HALF IS FINER THAN NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50% OR LESS		ML	INORGANIC SILTS OF LOW TO MEDIUM PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
			OL	ORGANIC SILTS OR CLAYS OF LOW TO MEDIUM PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50%		MH	INORGANIC SILTS OF HIGH PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
			CH	INORGANIC CLAYS OF HIGH PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
			OH	ORGANIC SILTS OR CLAYS OF HIGH PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
HIGHLY ORGANIC SOILS			PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

SYMBOLS KEY

SAMPLE TYPES

Split Spoon

WELL SYMBOLS



ABBREVIATION KEY

CA	-	CHEMICAL ANALYSIS (CORROSIVITY)	(200)	-	(WITH % PASSING NO. 200 SIEVE
CD	-	CONSOLIDATED DRAINED TRIAXIAL			
CN	-	CONSOLIDATION	SW	-	SWELL TEST
CU	-	CONSOLIDATED UNDRAINED TRIAXIAL	TC	-	CYCLIC TRIAXIAL
DS	-	DIRECT SHEAR	TV	-	TORVANE SHEAR
PP	-	POCKET PENETROMETER (TSF)	UC	-	UNCONFINED COMPRESSION
(3.0)	-	(WITH SHEAR STRENGTH IN KSF)	(1.5)	-	(WITH SHEAR STRENGTH IN KSF)
RV	-	R-VALUE			
SA	-	SIEVE ANALYSIS: % PASSING #200 SIEVE	UU	-	UNCONSOLIDATED UNDRAINED TRIAXIAL
			WA	-	WASH ANALYSIS
			(200%)	-	(WITH % PASSING NO. 200 SIEVE)

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