REPORT OF GEOTECHNICAL EXPLORATION

City of Pembroke Wastewater Treatment Plant Expansion

Sims Road, Pembroke, Georgia

WSP Project No. US0039300.2246

Prepared for:

Mr. Oscar Garcia
M.E. Sack Engineering
515 N. Main Street
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9/26/2024

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Subject: Report of Geotechnical Investigation
City of Pembroke Wastewater Treatment Plant Expansion
Sims Road
Pembroke, Georgia 31313
WSP Project No. US0039300.2246

Dear Mr. Oscar Garcia:

WSP USA Environment & Infrastructure Inc. (WSP) is pleased to submit this subsurface exploration and geotechnical engineering evaluation report for the above referenced property. This exploration was conducted in general accordance with our proposal dated August 29, 2024. This report briefly discusses our understanding of the project, describes our exploratory procedures and results, and presents our conclusions and recommendations related to the project design and construction.

We appreciate the opportunity of working with you on this project and look forward to our continued association during the construction phases of the project. Please contact us if you have any questions about this report or if we may be of further service.

Sincerely,

WSP USA Environment & Infrastructure Inc.

GEORGIA REGISTERED No. PE042627 PROFESSIONAL ENGINEER YANBO HUANG

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Table of Contents

Appendix

1.0 Introduction

WSP has completed a geotechnical exploration for the design phase services of a proposed expansion to the existing wastewater treatment plant located north of Sims Road in Pembroke, Georgia. The objective of this study was to explore the general subsurface conditions at the site of the proposed development and to analyze these conditions as they relate to foundation, pavement, and earth retaining structure design and construction. This report discusses our understanding of the project, describes our exploratory procedures and presents our conclusions and recommendations.

2.0 Project Information

We understand that the proposed expansion will be located at the existing City of Pembroke Wastewater Treatment Plant Property which is located on a 239.42-acre parcel identified by the Bryan County Tax Assessor as Parcel ID 011 056. The portion of the property to be developed with the expansion is currently grassed and is occupied by a gravel driveway and two existing structures associated with the wastewater treatment plant. Existing lagoons are present to the north and west of the expansion site. We were provided with a set of plans which included existing conditions and site plans, civil drawings, and mechanical drawings prepared by the client and dated July 22, 2024. Based on the provided information, we understand the proposed expansion will include an operations building, an MCC building, a building for a backup power generator, #1 and #2 SBR basins, a Post EQ Basin, a Belt Press Building, a Chemical Building, a Blower Building, Tertiary Filtration, UV disinfection, a Reclaimed Water Pump Station, an Influent Pump Station, a Grit Separation System, Headworks, Reject Pump Stations, roadways, 4”, 8”, and 10” force mains, and other miscellaneous improvements.

Structural loading information was not provided to WSP. Based on the provided plan, for the SBR basin 1, digester basin, Post EQ basin and SBR basin 2, the bottom of tank is at elevation 98.2 ft and the HWL is at 119.2 ft. We estimated the bearing pressure is about 1400 psf for the basins. For other structures and buildings, we have assumed that the maximum structural loads will not exceed 100 kips per column, 5 kips per linear foot for load-bearing walls, and 500 pounds per square foot for slabs.

Based on the provided plan, the elevation of existing ground surface is in the range of 94 to 105 ft. The bottom of tank is at elevation 98.2 ft for the SBR basin 1, digester basin, Post EQ basin and SBR basin 2. The top of base is at elevation 84.7 ft for the wet well of influent pump station and reject pump station.

We understand the proposed tank was proposed to be supported on a mat foundation for the SBR basin 1, digester basin, Post EQ basin and SBR basin 2. The remainder of the proposed improvements were anticipated be supported on conventional shallow foundation systems.

3.0 Field Exploration

In order to explore the general subsurface conditions in the areas of the planned construction, a total of fourteen (14) CPT soundings and seven (7) hand auger borings were performed at the approximate locations shown on the attached boring location plan. Three of the CPT soundings (CPT-1, CPT-4, and CPT-9) were repeated in order to confirm the data obtained in the initial soundings. The repeated CPT soundings have been identified as CPT-1a, CPT-4a, and CPT-9a. The CPT soundings were pushed to a maximum depth of 56 feet below existing site grades within the proposed building area and the hand augers borings were drilled to approximately 5 feet for the proposed pavement and force main.

All Cone Penetration Testing (CPT) soundings were performed using a geoprobe drill rig utilizing an instrumented cone. Prior to the commencement of drilling operations, an 811 ticket was created to check for the presence of underground utilities at the site. In addition, a WSP utilized a subcontracted private utility locator to clear the boring locations prior to drilling.

The sounding and hand auger boring locations were staked in the field by WSP's driller by measuring distances from existing landmarks and using a handheld GPS device. The sounding and hand auger boring locations are shown on the Boring Location Plan (Figure 2) in the Appendix and should be considered approximate. Existing topographic information was not provided to WSP. All sounding depths were recorded as from the ground surface at the time of drilling.

The Cone Penetration Test Sounding Records, in the Appendix, graphically show the penetration resistances and present the soil descriptions for each of the CPT soundings. The stratification lines and depth designations on the sounding records represent the approximate boundaries between soil types. In some instances, the transition between types may be gradual.

4.0 Site and Subsurface Conditions

4.1 Area and Site Geology

The subject site is located within the Coastal Plain Physiographic Province. The Coastal Plain is a wedge-shaped deposit of Cretaceous and younger sediments which range in thickness from near zero at the contact with the Piedmont Physiographic Province (the Fall Line) to the northwest, to thousands of feet at the coast. Published USGS geologic mapping indicates the site is classified under Wilcomico Shoreline Complex - marsh and lagoonal facies.

4.2 Subsurface Conditions

4.2.1 General

The subsurface conditions discussed in the following paragraphs and those shown on the Cone Penetration Test Sounding Records and Hand Auger Boring Logs represent an interpretation of the sounding and other data using normally accepted geotechnical engineering judgments considering local geology and experience.

The Sounding Records and Hand Auger Boring Logs represent our interpretation of the field conditions based on an engineer's review of cone penetration testing data. The groundwater conditions indicated on the CPT Sounding Records were estimated based on the pore water pressure. The lines designating the interfaces between various strata represent approximate boundaries only, as transitions between materials may be gradual. Soil conditions may vary between and away from the sounding locations.

4.2.2 Subsurface Soil Conditions

Based on the sounding data and the samples obtained from the hand auger borings, the natural soils are typical of the Coastal Plain in this area and consist of layers of fine-grained materials (clays/silts), sandy/clayey silts, and silty/clayey sands. The depositional processes would produce some mixing of these materials, so interbedding and transitions will likely be present.

Topsoil ranging in thickness from approximately 4 to 11 inches was encountered in all hand auger borings except HA-6. HA-6 was located at the bottom of the existing drainage ditch. Below the topsoil, the surface soils generally consisted of alternating strata of relatively clean to silty/clayey sands and clays. Throughout the site, clayey soils were encountered at various depths. Besides the clayey soils encountered at depths of 1 to 3 feet, a layer of soft clay was encountered at depths of 16 to 25 feet, ranging in thickness from about 3 to 8 feet. In addition, another soft clay layer was encountered at an approximate depth of 32 feet in CPT-9/9a and continued to the sounding termination depths of 40 feet. In general, below the soft clay layer, relatively clean to silty/clayey sands were encountered to the sounding termination depths.

CPT refusal was encountered at approximately 50 feet below existing grades at sounding CPT-1, 56 feet below existing grades at CPT-1a and CPT-2, and 49 feet below existing grades at CPT-3.

4.2.3 Groundwater

Groundwater was encountered in hand auger borings HA-3, HA-5, and HA-6 at depths ranging from approximately 0.3 feet to 4.8 feet below existing grades (elevation 93.2 to 97.7 feet). It should be noted that the groundwater levels encountered during this exploration could be affected by the proximity of the nearby wastewater treatment lagoons. Groundwater levels can fluctuate with changes in tides, weather, climate, local drainage, and with construction activity in the area. Since groundwater level variations are anticipated, design drawings and specifications should accommodate such possibilities and construction planning should be based on the assumption that variations will occur.

5.0 Conclusions and Recommendations

The following conclusions and recommendations are based on the previously discussed project information, our observations at the site, interpretation of the field data obtained during the exploration, and our experience with similar subsurface conditions. Subsurface conditions in unexplored locations may vary from those encountered at the specific sounding locations. If the construction scheme should vary significantly from that previously described, we request the opportunity to review these recommendations and amend them if necessary.

5.1 General Assessment

In general, our geotechnical study has determined that site improvement techniques will be necessary to make the site suitable for the proposed SBR Basin, Digester, and Post EQ Basin, which we anticipate will be supported on a rigid mat foundation. Site improvement techniques are necessary due to the very soft clay layer encountered by the borings. Data obtained from the CPT soundings and past experience with similar materials indicates that this material is very compressible and will consolidate under the anticipated loading from the planned construction. Preloading with surcharge would typically be the most practical and cost-effective method of site improvement considering the subsurface conditions for this project. However, due to the time required for preloading and costs associated with surcharging, other ground improvement options may be beneficial for schedule and cost considerations. Discussion of preloading, alternative ground improvement options and other recommendations are presented in the following report sections.

5.2 Groundwater Conditions

Our hand auger borings indicated groundwater to be about 0.3 to 4.8 feet below the existing ground surface. We anticipate that groundwater issues may occur in areas where excavations will be required below existing grades. If groundwater is encountered during construction, some form of temporary dewatering will be required. Conventional dewatering methods, such as pumping from sumps and/or trenching, should likely be adequate for temporary removal of groundwater encountered during excavations at the site.

In some areas, the current groundwater level may be shallow enough that dewatering will be required during initial fill placement. During construction, groundwater levels should be maintained at least 2 feet below the working surface to allow for proper compaction of the fill materials. Surface water should be directed away from the structural areas.

We do not anticipate that the groundwater level on the site will be significantly affected during the preloading operation. If wick drains are employed, the compressing soil will force groundwater out through the wick drains and sand layer and will need to be treated as surface water for control.

5.3 Settlement Estimates and Preloading Recommendations

Settlement analyses were performed for the various structures and loads anticipated for the facility in conjunction with the subsurface conditions encountered in the borings. The settlement calculations were based on the assumption that no more than 2 feet of fill would be required to achieve finished site grades. Settlement for the rigid mat foundations was based on the estimated bearing pressure of 1400 psf. Column footing, wall footing, and floor slab settlements were based on an assumed allowable bearing pressure of 2000 psf with foundation dimensions based on assumed loads of 100 kips per column, 5 kips per linear foot for load bearing walls, and 500 psf for slabs. If the structural loads are greater than the assumed values, we can rerun our analysis and revise our recommendations accordingly. All structure settlements shown do not include the settlement anticipated based on general site fill to raise grades. The following table provides a summary of the conditions analyzed along with the estimated settlement values.

Area	Loads	Estimated Settlement (inches)
SBR Basin, Digester, and Post EQ Basin	Rigid Mat Foundation: 1400 psf	0.5 to 3
Other Buildings and Structures	Columns: 100 kips Walls: 5 kips per linear foot Slabs: 500 psf	<1 <1 <1

Based on the anticipated settlements, as provided in the table above, it will be necessary to preload the SBR Basin, Digester, and Post EQ Basin area to reduce the settlements experienced by the Basin structure. Preloading with surcharge is a site improvement technique performed by placing a quantity of soil on the site and foundation soils that is equivalent to the anticipated sustained loadings that will be applied by the proposed structures. The object of preloading is to consolidate the soft soils prior to construction of the proposed structures, thus reducing the settlements experienced by the structures. The preloading soil is generally left in place for some time to allow for consolidation of the soft soil to occur. The time required for consolidation is dependent on the quantity of soil used for the preload and the characteristics of the consolidating soil. Settlement monitoring points are typically installed to provide information on the amount of consolidation that has occurred and when it is acceptable to remove the preload and start construction of the structures. The time for the consolidation process to occur may be decreased by installing vertical wick drains. Preloading reduces the amount of settlement experienced by the proposed structure but does not eliminate settlement entirely. The preloaded area will generally experience some rebound when the preload is removed and then some settlement when the structure is constructed.

To consolidate the subsurface soils and minimize the settlements experienced by the Basin structure, we recommend the Basin structure area be preloaded with a surcharge of 12 feet of soils above the finished grade. The total unit weight of soil is assumed to be 120 psf for the surcharge soils. The preloaded area should extend outside the structure perimeter (a minimum of 10 feet) depending planned grades and site constraints. The slope of the surcharge should be 2(Horizontal):1(vertical) to minimize erosion. The preloading time should be determined based on the results of settlement monitoring. Based on the thickness of the clay layer and the recommended amount of surcharge placed, we anticipate a minimum of 3-month waiting period may be required for the Basin structure area.

As previously stated, wick drains may be utilized to speed up the consolidation process and reduce the waiting period. The design of a wick drain system (type of drain, spacing and depth of drains, etc.) is usually part of a specialty geotechnical contractor's scope of work. Wick drains should be installed prior to placing fill required to bring the site to grade. A 18-inch-thick layer of clean sand (SP) should be spread over the areas to be preloaded to allow for water flow out of the wick drains due to the consolidation process. The wick drains should extend to the depths of 32 to 33 feet below the existing ground surface. This sand layer should be connected to a drainage trench around the preload area to transport the water away. Once the sand layer is in place, the fill and surcharge should be placed. The fill material (if any) used to raise the structure pad areas to the design final grades should be compacted according to the recommendations presented in this report.

Settlement monitoring points (settlement plates) should be installed at selected locations around the structure pads prior to any fill placement to evaluate the settlement during fill placement and preloading. We recommend that the elevations be surveyed to the nearest 0.01 foot twice a week, beginning at the time of monument seating. Precise elevation data will be necessary for accurate settlement analyses. There are many potential things which could influence the accuracy of the settlement data; therefore, we recommend considering the following when obtaining the data:

- Obtain the readings during the same time of day to reduce thermal differentials, which can potentially cause the settlement monitor to change in elevation (expand/contract).
- Use the same equipment and personnel to obtain the data each time.

- The settlement monuments should be marked/shaped such that the surveyor's rod is placed in approximately the same location each time.
- Protect the monuments from damage due to construction activity.
- Document activities around the monuments that might influence the rate of settlement such as adding additional fill in the area, setting equipment on nearby foundations, etc.
- Assure the benchmark is accurate. The benchmark should be established in a stable area well outside the fill zone.

The monitoring data should be evaluated weekly for analysis and evaluation. Once fill and preload placement is complete and four (4) weeks of settlement data has been obtained, a preliminary evaluation of the settlement magnitude and time frame can be provided to the design/construction team. If the settlement appears to have effectively transitioned from the primary to secondary phase before the scheduled waiting period is complete, the actual required waiting period may be reduced at the discretion of the geotechnical engineer.

5.4 Ground Improvement Options

In addition to the previously discussed site improvement options of preloading or preloading with wick drains, alternative ground improvement techniques could be evaluated. In general, rigid inclusions and/or aggregate piers could be used to reduce the settlement potential and increase the bearing capacity of the soils to allow for shallow foundation support where appropriate. Since majority of the settlement is from the soft clayey soils at the depths between 24 and 32 ft, we recommended the rigid inclusion or aggregate piers be extended to at least below the soft clayey soils.

Rigid inclusions are vertical elements (columns) typically made of grout used to reduce settlement and increase bearing capacity. The columns are normally constructed by vibrating a mandrel through weak, compressible soils to underlying firm/dense soils. Concrete or grout is then pumped through the mandrel as it is extracted while maintaining a positive head of grout. The top of the inclusions typically terminates in a load transfer platform made of granular material allowing structure loads to be transferred through the weak soils to more competent underlying soils thus reducing potential settlement.

Aggregate piers are also vertical elements but are typically composed of dense aggregate. Aggregate piers in coastal geologic conditions are often installed using a displacement system as previously described using a mandrel. The mandrel is driven or vibrated to the required depth and then open-graded aggregate is placed into the annular space. As the mandrel is withdrawn, the aggregate is densified by the mandrel. This process continues in 2-foot-thick lifts until the entire pier is constructed to the surface.

Rigid inclusion and aggregate pier systems are design-build systems and the installer (specialty geotechnical contractor) of either system should provide detailed design calculations sealed by a professional engineer. The design calculations should demonstrate that the soil improved system is estimated to control long-term total and differential settlements. The specialty contractor should warrant their work as well as the maximum total and differential settlements they predict.

5.5 Site Preparation

Following completion of the recommended site improvement techniques, we have recommended the following site preparation procedures. All vegetation, including stumps and root systems, organic topsoil and other deleterious materials should be stripped from proposed construction areas. After clearing and stripping, areas intended to support the structures, including new fill should be carefully assessed by a qualified geotechnical engineer or his representative. This assessment should include proofrolling to locate soft or weak subgrades that may need repair. Any areas that pump or deflect excessively should be addressed as appropriate. Please note, due to the uneven ground surface, the volume of topsoil may be greater than the volume calculated by area times the topsoil thickness indicated on the boring logs.

Areas intended to support new fill and/or structures should be proofrolled with a fully-loaded, tandem-axle, dump truck or other pneumatic-tired vehicle of similar size and weight under the observation of the geotechnical engineer. The purpose of proofrolling is to locate soft, weak, or excessively wet surficial soils present at the time of construction. Proofrolling consists of trafficking the site with a fully loaded, tandem axle dump truck or pneumatic tired vehicle of similar size and weight (20 tons). Proofrolling should detect shallow, soft, wet, or otherwise unsuitable soils. Materials judged to be unstable during proofrolling operations will require remedial action and should be treated as recommended by the engineer. The geotechnical engineer can recommend treatment based on the planned construction in the area and severity of the issues discovered. These recommendations often include harrowing and disking the upper one foot of exposed surface to alter moisture content followed by recompaction of the harrowed materials.

If earthwork is conducted in hot, dry weather favorable for drying soils, issues with wet unstable soils tend to be less of a concern. However, if the required treatment effort and volumes for stabilization become widespread or grading is performed during unfavorable weather conditions, additional measures such as lime or cement stabilization might be required to dry and stabilize the soils in wet or unfavorable weather.

5.6 Excavation and Earthwork Construction Considerations

Upon completion of filling and grading, care should be taken to maintain the subgrade moisture content prior to construction of floor slabs and pavements. Construction traffic over the completed subgrade should be avoided to the extent practical. The site should also be graded to prevent ponding of surface water on the prepared subgrades or in excavations. If the subgrade should become frozen, desiccated, saturated, or disturbed, the affected material should be removed or these materials should be scarified, moisture conditioned, and recompacted prior to floor slab construction and observed by a WSP representative.

Surface water should not be allowed to pond on the site and soak into the soil during construction. Construction staging should provide drainage of surface water and precipitation away from the building area. Any water that collects over or adjacent to construction areas should be promptly removed, along with any softened or disturbed soils. Surface water control in the form of sloping surfaces, drainage ditches and trenches, and sump pits and pumps will be important to avoid ponding and associated delays due to precipitation and seepage.

Groundwater was encountered in the hand auger borings at depths as shallow as 0.3 feet below existing grades. If groundwater is encountered during construction, some form of temporary or permanent dewatering may be required. Conventional dewatering methods, such as pumping from sumps or trenching, should likely be adequate for temporary removal of any groundwater encountered during excavations at the site.

Based on the provided plan, we have anticipate the maximum fill height will be less than 2 feet to achieve finished elevations in the proposed building footprint and pavement areas.

All excavations should be sloped or braced as required by Occupational Health and Safety Administration (OSHA) regulations to provide stability and safe working conditions. The grading contractor, by his contract, is usually 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. All excavations should comply with applicable local, state, and federal safety regulations, including the current OSHA Excavation and Trench Safety Standards.

Construction site safety is the sole responsibility of the contractor who controls the means, methods, and sequencing of construction operations. Under no circumstances shall the information provided herein be interpreted to mean that WSP is assuming any responsibility for construction site safety or the contractor's activities; such responsibility shall neither be implied nor inferred.

5.7 Fill Placement

Fill to replace undercut areas or achieve finished grades should be non-plastic soils with less than 25 percent passing the number 200 sieve in building and pavement areas. Fill should be free of deleterious materials and rock fragments larger than about 3 inches in any dimension. All structural fill should be placed in maximum 8-inch thick loose lifts and compacted to at least 95 percent of the soil's maximum dry density as determined by the Modified Proctor compaction test (ASTM D 1557). Soil moisture during placement should be maintained within 3 percent of the optimum moisture content. The upper 2 feet of fill beneath foundations, slabs, or pavements should be compacted to 98 percent. Fill should be placed in horizontal lifts and adequately keyed into stripped and scarified subgrade soils. In confined areas such as utility trenches or over anchor blocks, portable compaction equipment and thin lifts of 3 to 4 inches may be required to achieve specified degrees of compaction.

Fill placement should be observed by a qualified soils technician under the supervision of the geotechnical engineer and that frequent fill density and moisture tests be performed to verify that the specified degree of compaction is being achieved. Areas that do not meet the compaction specifications should be reworked to achieve compliance and retested.

5.8 Foundation Recommendations

Structural loading information was not available at the time of this proposal. We have assumed a design bearing pressure of 1400 psf for the rigid mat foundation for the SBR Basin, Digester, and Post EQ Basin.

We have further assumed maximum column loads of 100 kips, maximum wall loads of 5 kips per linear foot, and maximum slab loads of 500 psf for the remainder of the proposed structures and improvements associated with this project. Our settlement analysis has been based on these structural loading assumptions.

From the settlement analysis result, the total settlement was estimated to be in the range of 0.5 to 3 inches for the mat foundation of SBR Basin, Digester, and Post EQ Basin under the bearing pressure of 1400 psf. Majority of the settlement are from the clayey soils at the depths between 24 and 32 ft. Therefore, to mitigate the risk of excessive settlement, ground improvement (e.g., preloading with surcharge, aggregate piers or rigid inclusion) is recommended for the mat foundation.

For the other structures and buildings, the total settlement was estimated to be less than one inch under the assumed structural loads. Therefore, conventional, shallow column and wall foundations bearing on coastal plain soil or fill compacted in accordance with the recommendations in this report may be designed for an allowable soil bearing pressure of 2,000 psf.

The allowable foundation bearing pressures apply to dead loads plus design live load conditions. The design bearing pressure may be increased by one-third when considering total loads that include wind or seismic conditions. The weight of the foundation concrete below grade may be neglected in dead load computations. Interior footings should bear a minimum of 12 inches below finished grade. Finished grade is the lowest adjacent grade for perimeter footings and floor level for interior footings.

Footings, foundations, and masonry walls should be reinforced as necessary to reduce the potential for distress caused by differential foundation movement. The use of joints at openings or other discontinuities in masonry walls is recommended.

Foundation excavations should be observed by the geotechnical engineer. If the soil conditions encountered differ from those presented in this report, supplemental recommendations will be required.

The base of all foundation excavations should be free of water and loose soil prior to placing concrete. Concrete should be placed soon after excavating to reduce bearing soil disturbance. Should the soils at bearing level become excessively dry, disturbed, or saturated, or frozen, the affected soil should be removed prior to placing concrete. Place a lean concrete mud-mat over the bearing soils if the excavations must remain open over night or for an extended period of time. It is recommended that the geotechnical engineer be retained to observe and test the soil foundation bearing materials.

If unsuitable bearing soils are encountered in footing excavations, the excavations should be extended deeper to suitable soils and the footings could bear directly on these soils at the lower level or on lean concrete backfill placed in the excavations. We recommend the upper 2 feet of subgrade beneath the footing bearing elevations be compacted to at least 98 percent of the Modified Proctor maximum dry density as discussed in Section 5.7. Overexcavation for compacted backfill placement below footings should extend laterally beyond all edges of the footings at least 8 inches per foot of overexcavation depth below footing base elevation. The overexcavation should then be backfilled up to the footing base elevation with well-graded granular material placed in lifts of 8 inches or less in loose thickness and compacted to at least 95 percent of the material's maximum Modified Proctor dry density (ASTM D 1557).

We recommend widths of not less than 24 inches for footings for ease of construction and to reduce the possibility of localized shear failures. In addition, exterior footing bottoms should be at least 18 inches below exterior grades for protection against frost damage. A qualified geotechnical engineer should observe all footing excavations and assess whether the foundations are placed on a competent bearing stratum.

5.9 Floor Slab Recommendations

A subgrade prepared and tested as recommended in this report should provide adequate support for lightly loaded floor slabs.

Where appropriate, saw-cut control joints should be placed in the slab to help control the location and extent of cracking. For additional recommendations refer to the ACI Design Manual. Joints or any cracks that develop can be sealed with a waterproof, non-extruding compressible compound.

The floor slab design should include a base course comprised of at least 4-inch of free draining, compacted granular materials. The granular materials may be graded aggregate base (GAB) or sands with fine content less than 5 percent. The use of a vapor retarder or barrier should be considered beneath concrete slabs on grade that will be covered with wood, tile, carpet, or other moisture sensitive or impervious coverings, or when the slab will support equipment sensitive to moisture. When conditions warrant the use of a vapor retarder, the slab designer and slab contractor should refer to ACI 302 and ACI 360 for procedures and cautions regarding the use and placement of a vapor retarder/barrier.

On most project sites, the site grading is generally accomplished early in the construction phase. During construction the subgrade may be disturbed due to utility excavations, construction traffic, desiccation, rainfall, etc. As a result, the floor slab subgrade may not be suitable for placement of base rock and concrete and corrective action may be required.

We recommend the area underlying the floor slab be rough graded and then thoroughly proofrolled with a loaded tandem axle dump truck prior to final grading and placement of base rock. Particular attention should be paid to high traffic areas that were rutted and disturbed earlier and to areas where backfilled trenches are located. Areas where unsuitable conditions are located should be repaired by removing and replacing the affected material with properly compacted fill. All floor slab subgrade areas should be moisture conditioned and properly compacted to the recommendations in this report immediately prior to placement of the base rock and concrete.

The slab should be isolated from column and wall foundations to reduce the risk of cracking because of the differential loading.

5.10 Pump Station Wet Well Soil Parameters

We understand that the proposed construction will include several pump stations which will include a reclaimed water pump station, influent pump station, and reject pump station. Based on the provided plans, we estimated that the wet wells of the pump stations will bear at depths up to approximately 15 feet below finished grades. Based on the data obtained from the CPT soundings in the vicinity of the proposed pump station locations, we have recommended in-situ soil parameters for use in the design of the wet wells, which are presented in the tables below.

Reclaimed Water Pump Station (CPT-8)								
Depth (ft)	Elevation (ft)	Soil Type	N60	Cohesion (psf)	Friction Angle (Degrees)	Lateral Earth Pressure Coefficients		
						At-rest, K_o	Active, K_a	Passive, K_p
0 to 3	98 to 95	Silty Sands	12	--	31	0.48	0.32	3.12
3 to 5.5	95 to 92.5	Silty Clays	3	300	--	1	1	1
5.5 to 7.5	92.5 to 90.5	Silty Clays	12	1100	--	1	1	1
7.5 to 17.5	90.5 to 80.5	Silty Sands	37	--	34	0.44	0.28	3.54
17.5 to 20	80.5 to 78	Silty Clays	5	400	--	1	1	1
20 to 28	78 to 70	Silty Sands	15	--	32	0.47	0.31	3.25
28 to 31	70 to 67	Silty Sands	35	--	34	0.44	0.28	3.54

Reject Pump Station (CPT-9/CPT-9a)								
Depth (ft)	Elevation (ft)	Soil Type	N60	Cohesion (psf)	Friction Angle (Degrees)	Lateral Earth Pressure Coefficients		
						At-rest, K_o	Active, K_a	Passive, K_p
0 to 2	97.5 to 95.5	Silty Sands	15	--	32	0.47	0.31	3.25
2 to 6	95.5 to 91.5	Silty Clays	9	1000	--	1	1	1
6 to 16	91.5 to 81.5	Silty Sands	40	--	34	0.44	0.28	3.54
16 to 20	81.5 to 77.5	Silty Clays	7	700	--	1	1	1
20 to 31	77.5 to 66.5	Silty Sands	22	--	33	0.46	0.29	3.39
31 to 36	66.5 to 61.5	Silty Clays	5	400	--	1	1	1
36 to 38	61.5 to 59.5	Silty Sands	8	--	30	0.5	0.33	3.00
38 to 40	59.5 to 57.5	Silty Clays	4	350	0	1	1	1

Influent Pump Station (CPT-11)								
Depth (ft)	Elevation (ft)	Soil Type	N60	Cohesion (psf)	Friction Angle (Degrees)	Lateral Earth Pressure Coefficients		
						At-rest, K_o	Active, K_a	Passive, K_p
0 to 2	98.5 to 96.5	Silty Sands	22	--	33	0.46	0.29	3.39
2 to 6	96.5 to 92.5	Silty Clays	9	1000	--	1	1	1
6 to 13	92.5 to 85.5	Silty Sands	40	--	34	0.44	0.28	3.54
13 to 17	85.5 to 81.5	Silty Clays	9	1000	--	1	1	1
17 to 26	81.5 to 72.5	Silty Sands	15	--	32	0.47	0.31	3.25
26 to 31	72.5 to 67.5	Silty Clays	4	350	--	1	1	1
31 to 36	67.5 to 62.5	Silty Sands	10	--	31	0.48	0.32	3.12
36 to 40	62.5 to 58.5	Silty Sands	22	--	33	0.46	0.29	3.39

5.11 Pavements

Pavements should be supported on structural fill or suitable native soils. Some of the native clayey soils may not be ideal for support of pavements, and pavements supported on those soils could be subject to more frequent maintenance costs. We recommend that the pavement areas be thoroughly proofrolled as described previously in the Site Preparation section of the report. A modulus of subgrade reaction of about 120 pounds per cubic inch (pci) or a California Bearing Ratio (CBR) value of 8 may be considered for well-prepared subgrade consisting of site soils. The structural fill bedding material should be compacted to a minimum of 98 percent of Modified Proctor dry density (ASTM D 1557).

Pavements for parking and driveways restricted to automobile traffic (light duty) typically consist of 2.5 inches of asphaltic concrete over a 6-inch graded aggregate base. Pavements for trucks traffic (heavy duty) typically consist of 4 inches of asphaltic concrete over 8 inches of graded aggregate base. Rigid Portland Cement Concrete (PCC) pavements are usually more suitable in the areas where tight turns and maneuvering of the trucks is expected. Typical PCC sections usually consist of 5 inches of PCC over a minimum of 4 inches of graded aggregate base for light duty section and 7 inches of PCC over a minimum of 4 inches of graded aggregate base for heavy duty section.

Pavement design should be based on traffic or other loading conditions. If project specific traffic loads are provided later, more specific pavement recommendations may be provided, but we do not anticipate those would vary significantly from what is provided above. The pavement thickness was analyzed based on a pavement design life of 20 years. However, some maintenance repairs are typically required within a period of 8 to 10 years.

5.12 Seismic Design Parameters – Seismic Site Classification

The International Building Code (IBC) 2018 and ASCE 7-16 describes six Site Class Definitions that range from hard rock (A) to potentially unstable soil (F). Each site class is described by the average shear wave velocity, standard penetration resistance, or soil undrained shear strength in the top 100 feet of the site profile. The shear wave velocity is related to the soil column shear modulus, whereas the standard penetration resistance and undrained shear strength can be empirically related to the shear wave velocity. Each site class is associated with amplification factors that represent the effects that site stiffness (shear modulus) has on the presumed earthquake bedrock motion.

The seismic site class per IBC 2018/ASCE 7-16 for the Site is estimated to be Site Class D based on the CPT data from this study and the general geology of the area.

5.13 Qualifications of Recommendations

Our evaluation of foundation design and construction conditions has been based on our understanding of the site, the available project information, our assumptions, and the data obtained during our field exploration as described herein. The general subsurface conditions used were based on interpolation of the subsurface data at our soundings. The design recommendations in this report have been developed based on the previously described project characteristics and subsurface conditions. If project criteria or locations change, we must be permitted to determine if our recommendations are still applicable or if they must be modified. The findings of such a review will be presented in a supplemental report.

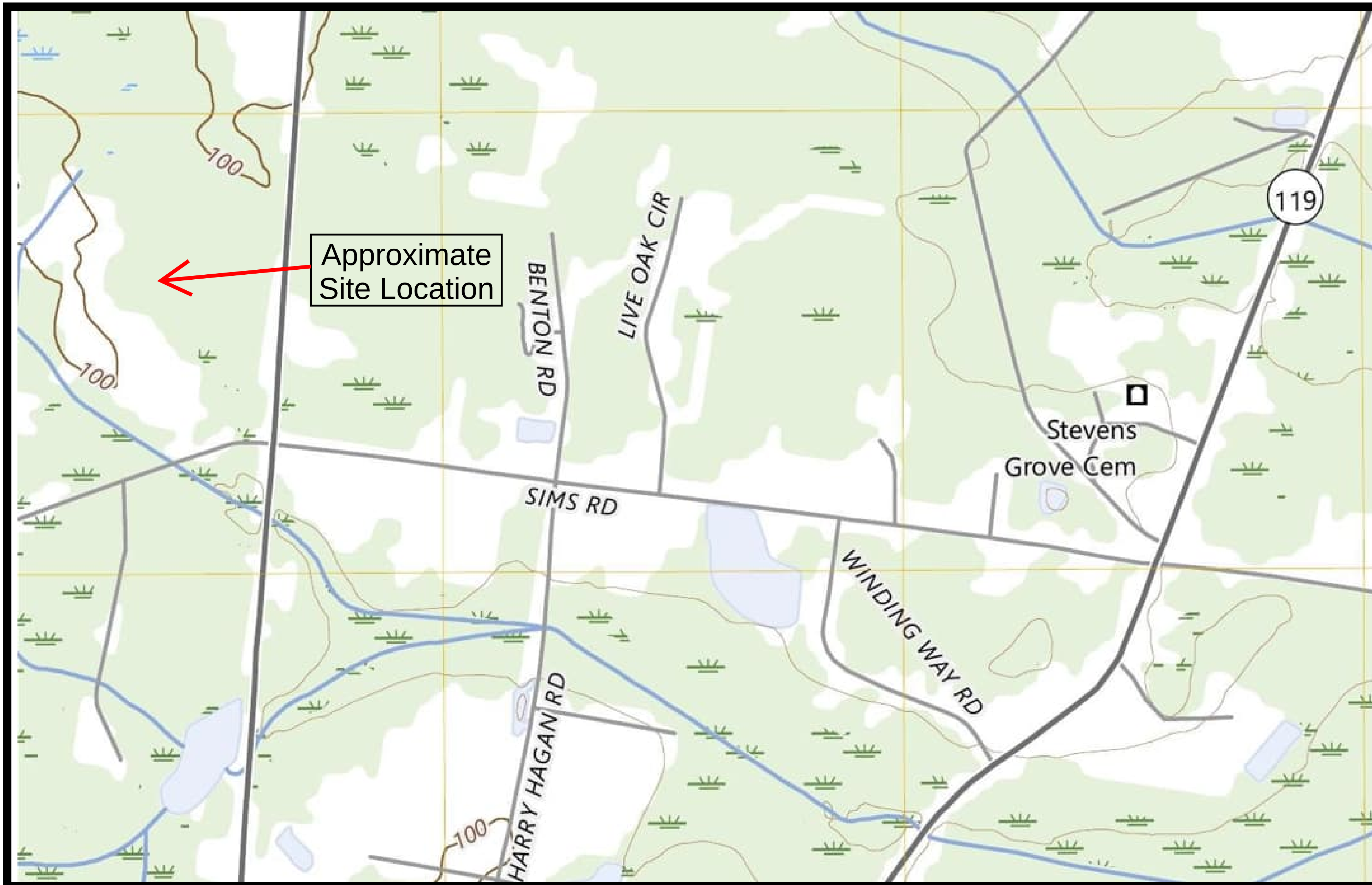
Subsurface conditions in unexplored locations may vary from those encountered at specific sounding location. The nature and extent of variations may not become evident until the course of construction. If such variations then appear evident, it will be necessary to re-evaluate the recommendations of this report after on-site observations of the conditions.

Regardless of the thoroughness of a subsurface exploration, there is the possibility that conditions will differ from those at the sounding location, that conditions are not as anticipated by the designers, or that the construction process has altered the soil conditions. Therefore, experienced geotechnical engineers must observe earthwork and foundation construction to assess if the conditions anticipated in design exist.

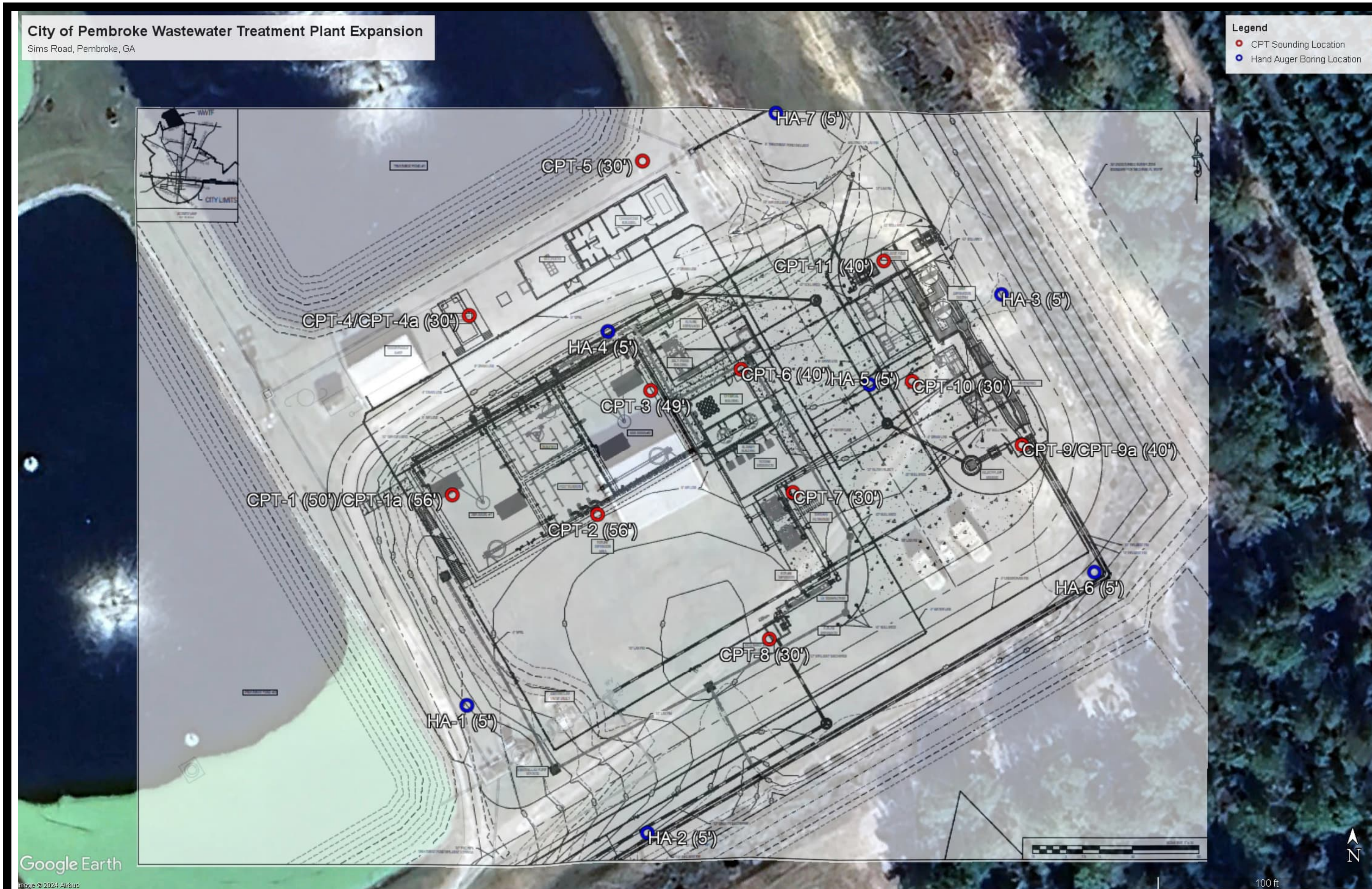
Our professional services have been performed, our findings derived, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices. This warranty is in lieu of all other warranties either express or implied. This company is not responsible for the conclusions, opinions or recommendations of others based on these data.

Appendix

Figure 1 - Site Location Plan
Figure 2 – Boring Location Plan
Cone Penetration Test Sounding Records (14)
Hand Auger Boring Logs (7)
Key to Symbols and Descriptions
Laboratory Test Results (5)
GBA Information About Geotechnical Reports



City of Pembroke **Wastewater Treatment Plant Expansion** **Sims Road** **Pembroke, Georgia**	WSP USA Environment and Infrastructure, Inc. 2000 Business Center Drive, Suite 235 Savannah, Georgia 31405	**SITE LOCATION PLAN** Project No. US0039300.2246 | DATE: September 2024 | FIGURE 1



City of Pembroke **Wastewater Treatment Plant Expansion** **Sims Road** **Pembroke, Georgia**	WSP USA Environment and Infrastructure, Inc. 2000 Business Center Drive, Suite 235 Savannah, Georgia 31405	**BORING LOCATION PLAN** Project No. US0039300.2246 / DATE: September 2024 / FIGURE 2

WSP USA
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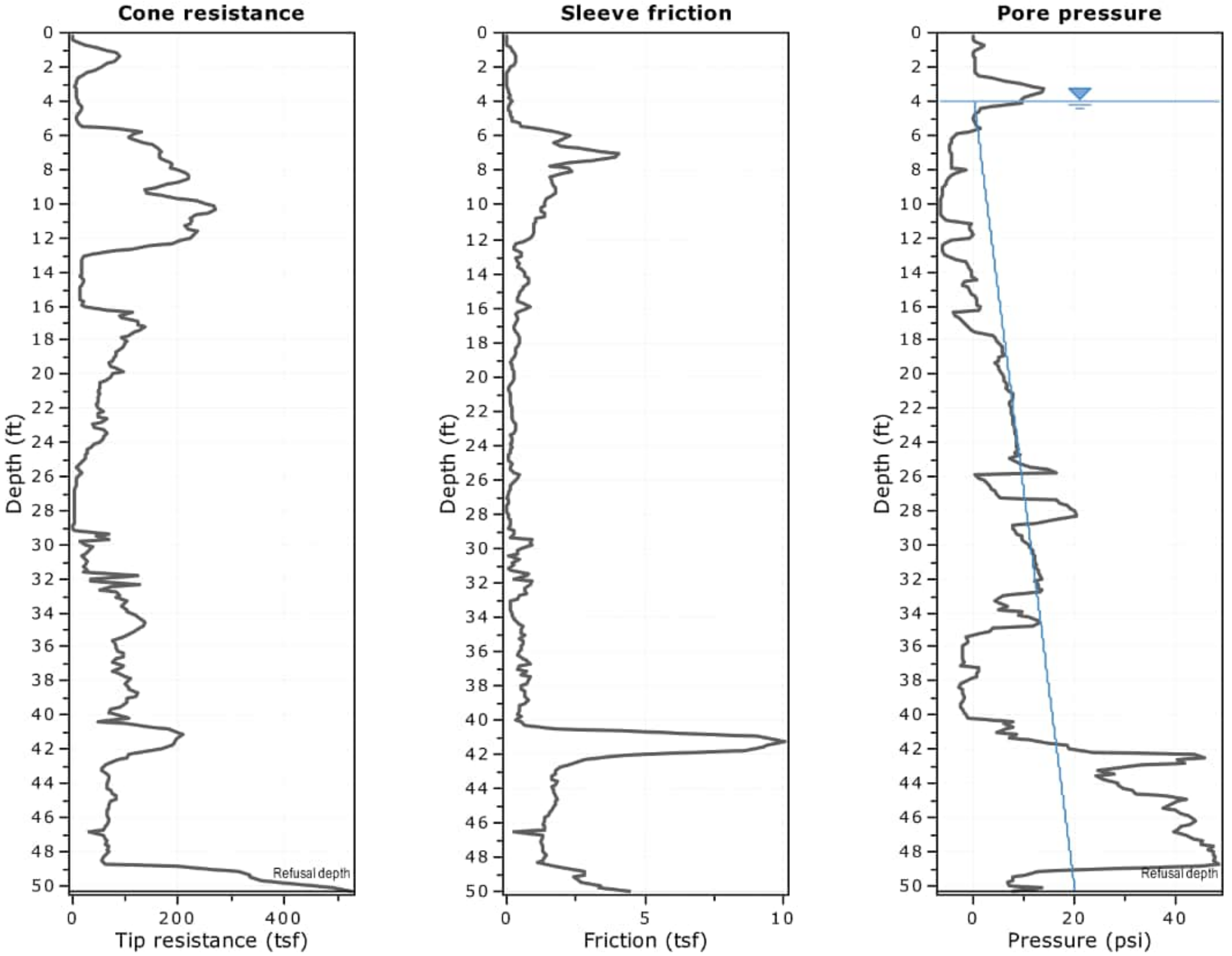
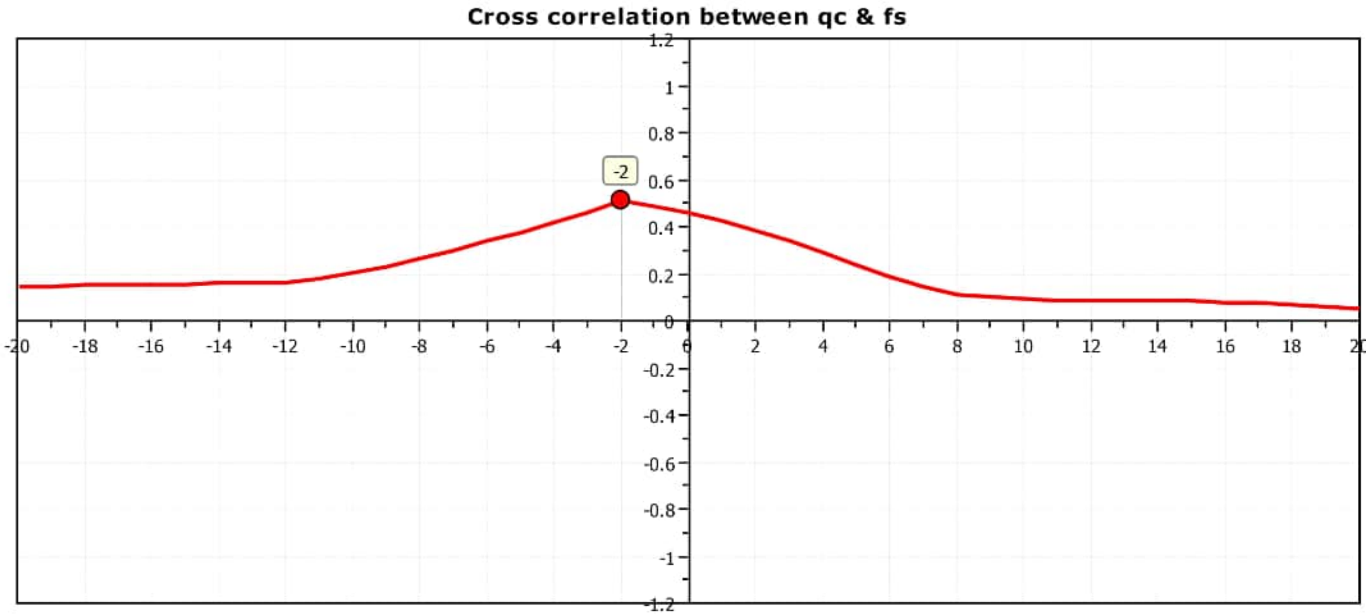
Savannah, Georgia
www.wsp.com

CPT-01
Total depth: 50.36 ft, Date: 9/9/2024
Surface Elevation: 103.00 ft
Cone Operator: BM

Project: **Wastewater Treatment Plant**
Location: **Pembroke, Georgia**



The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



WSP USA
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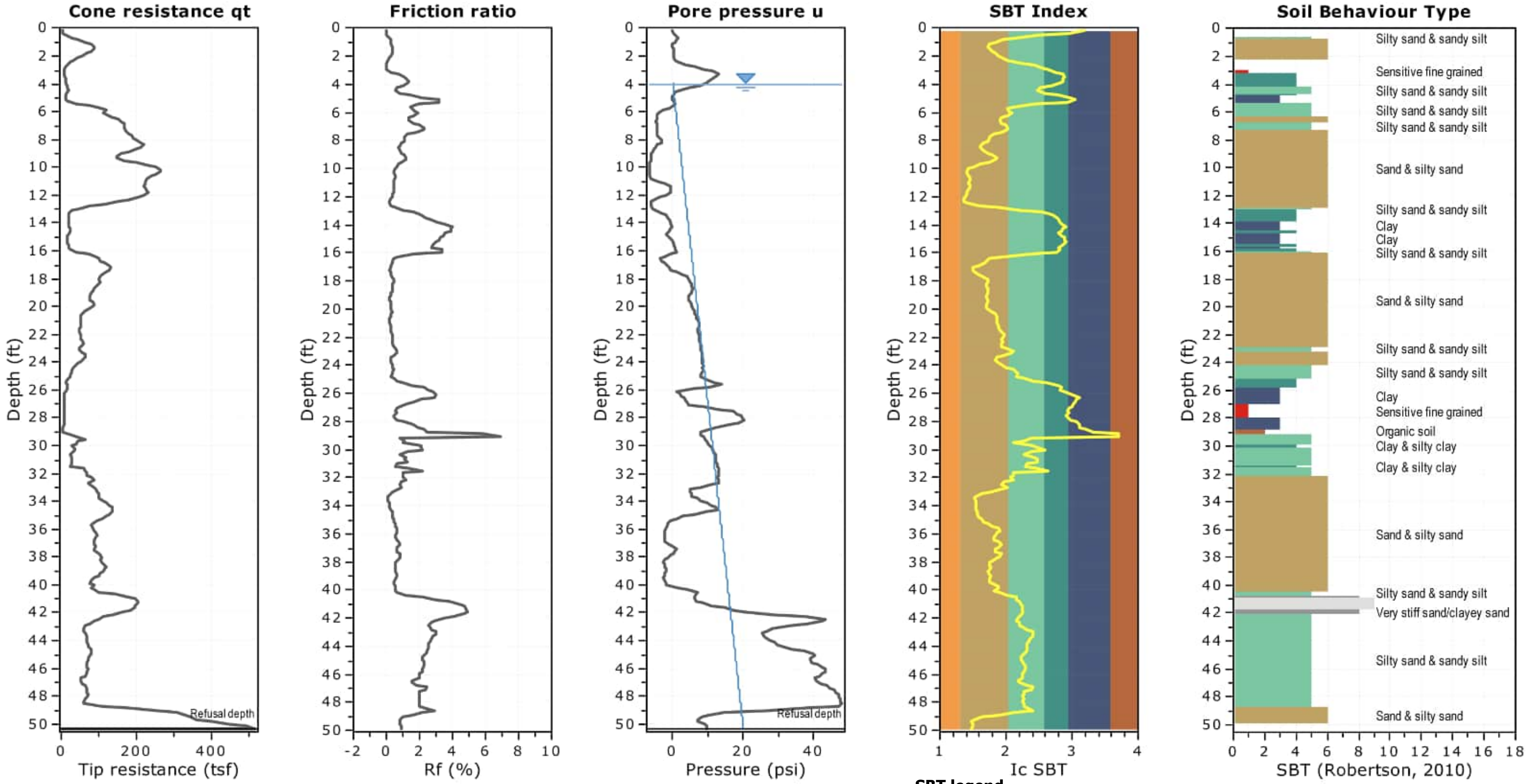
CPT-01
Total depth: 50.36 ft, Date: 9/9/2024
Surface Elevation: 103.00 ft
Cone Operator: BM

Project: **Wastewater Treatment Plant**
Location: **Pembroke, Georgia**



SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

WSP USA
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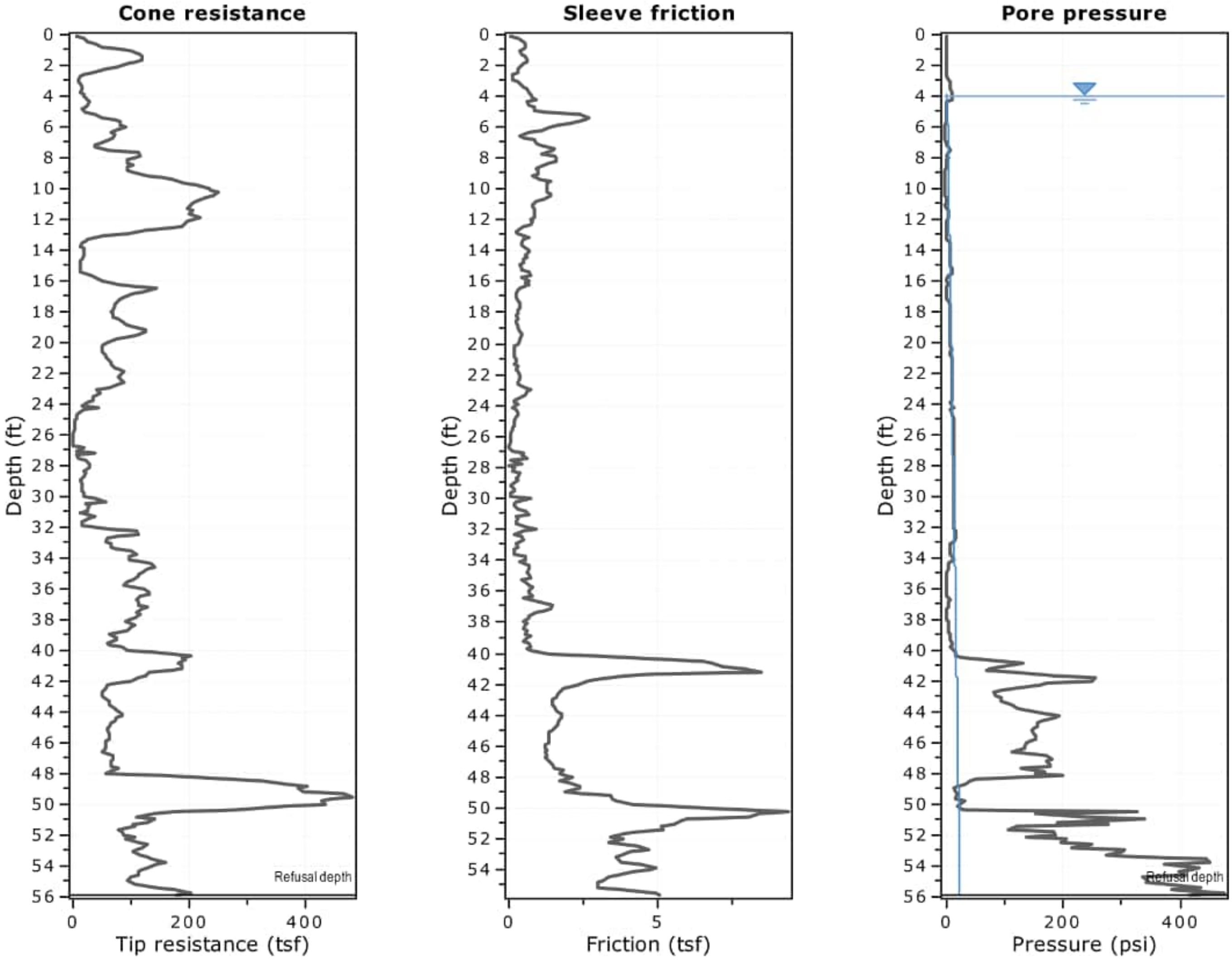
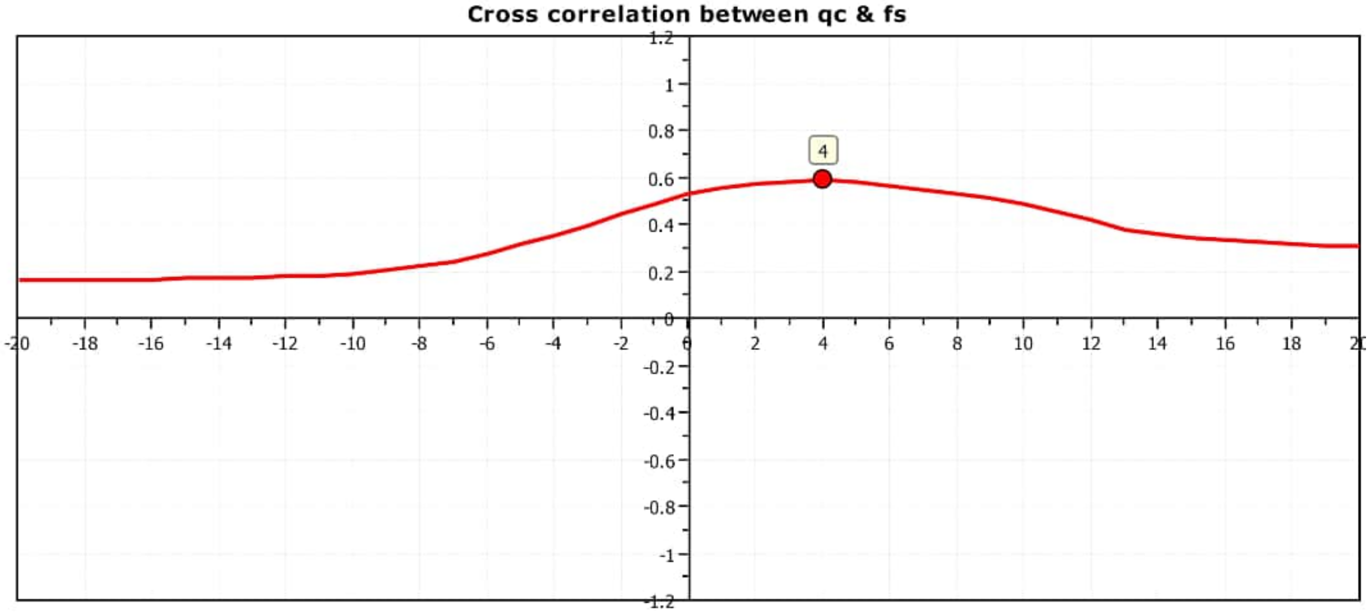
CPT-01a
Total depth: 55.94 ft, Date: 9/16/2024
Surface Elevation: 103.00 ft
Cone Operator: BM

Project: **Wastewater Treatment Plant**
Location: **Pembroke, Georgia**



The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).

Cross correlation between qc & fs



WSP USA
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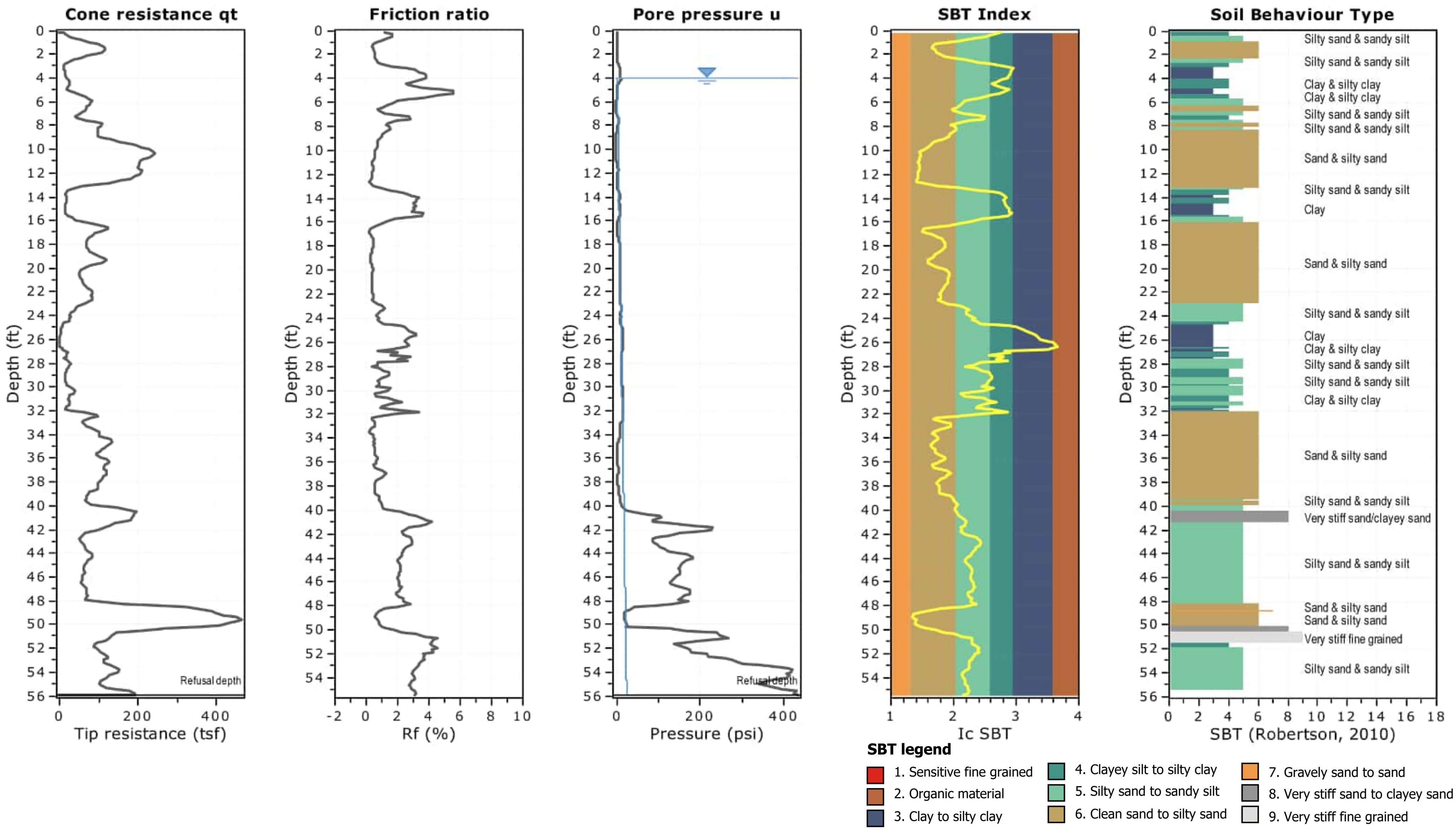
CPT-01a

Total depth: 55.94 ft, Date: 9/16/2024
Surface Elevation: 103.00 ft
Cone Operator: BM

Project: Wastewater Treatment Plant
Location: Pembroke, Georgia



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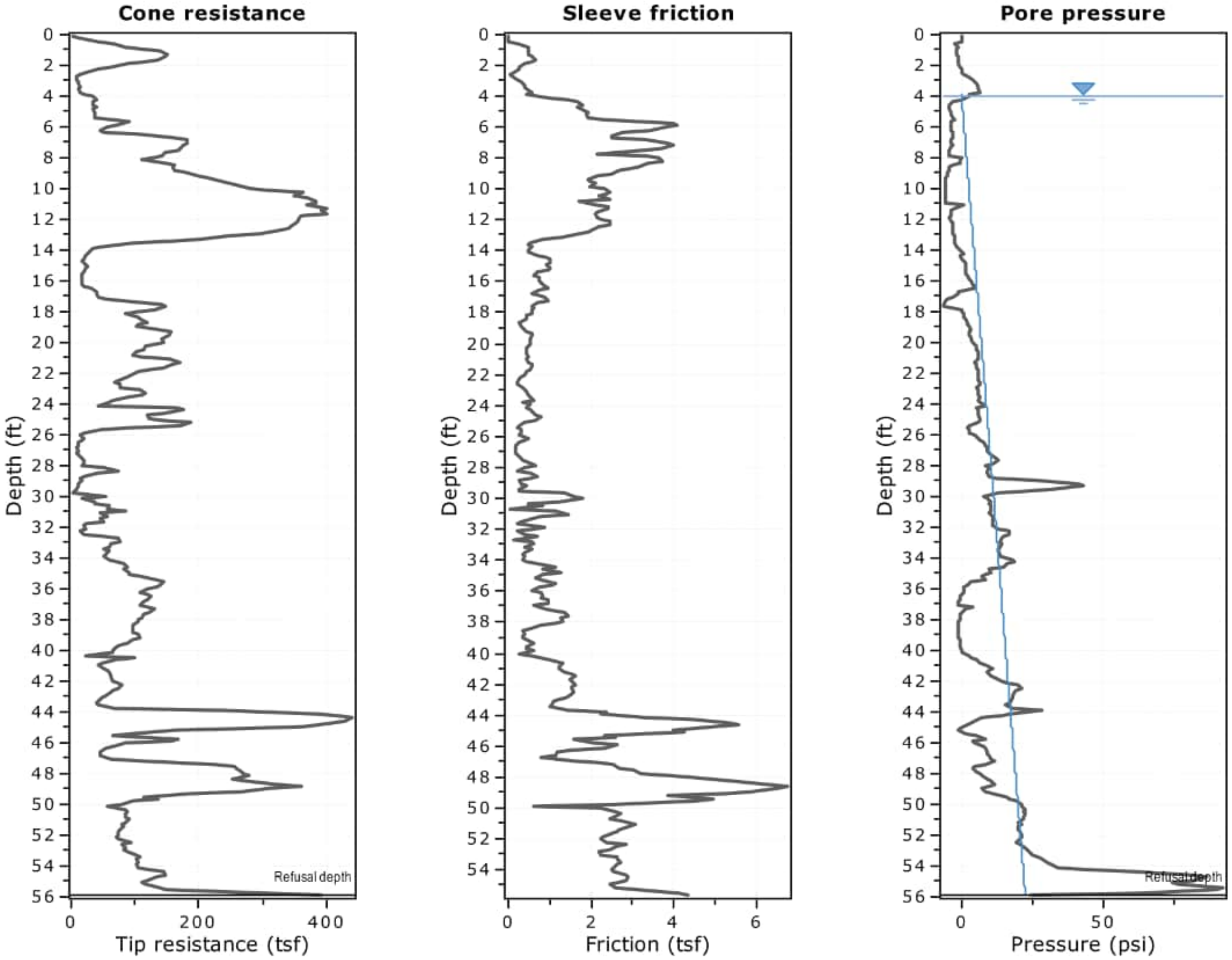
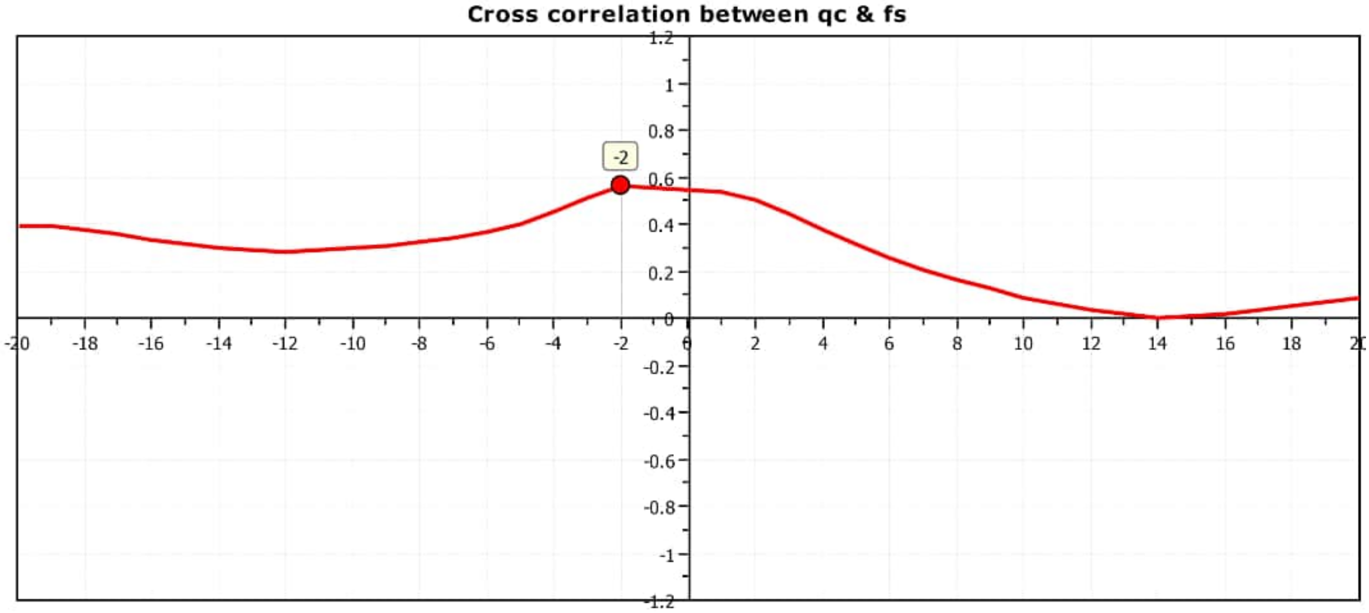
CPT-02
Total depth: 55.94 ft, Date: 9/9/2024
Surface Elevation: 98.50 ft
Cone Operator: BM

Project: **Wastewater Treatment Plant**
Location: **Pembroke, Georgia**



The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



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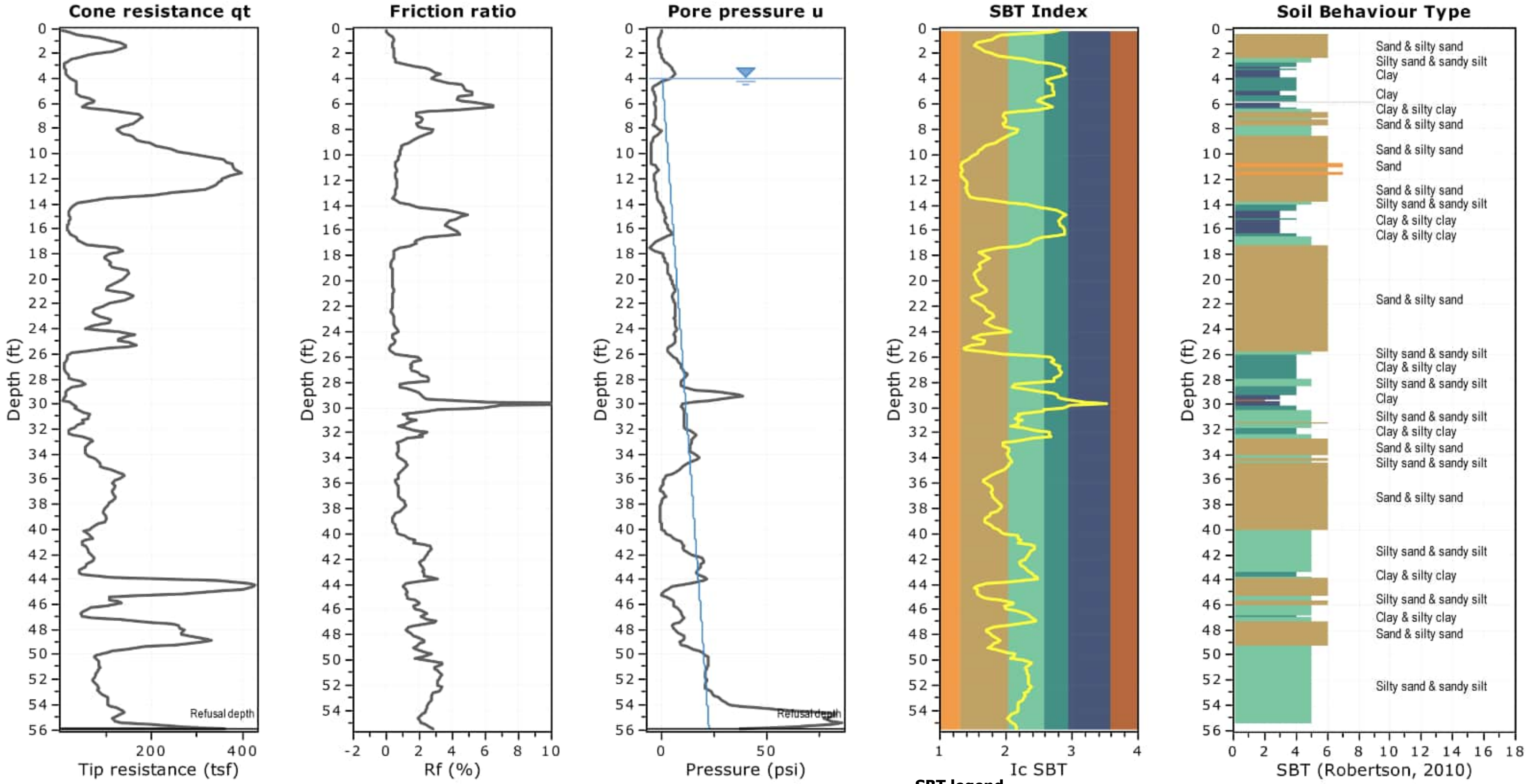
CPT-02
Total depth: 55.94 ft, Date: 9/9/2024
Surface Elevation: 98.50 ft
Cone Operator: BM

Project: **Wastewater Treatment Plant**
Location: **Pembroke, Georgia**



SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

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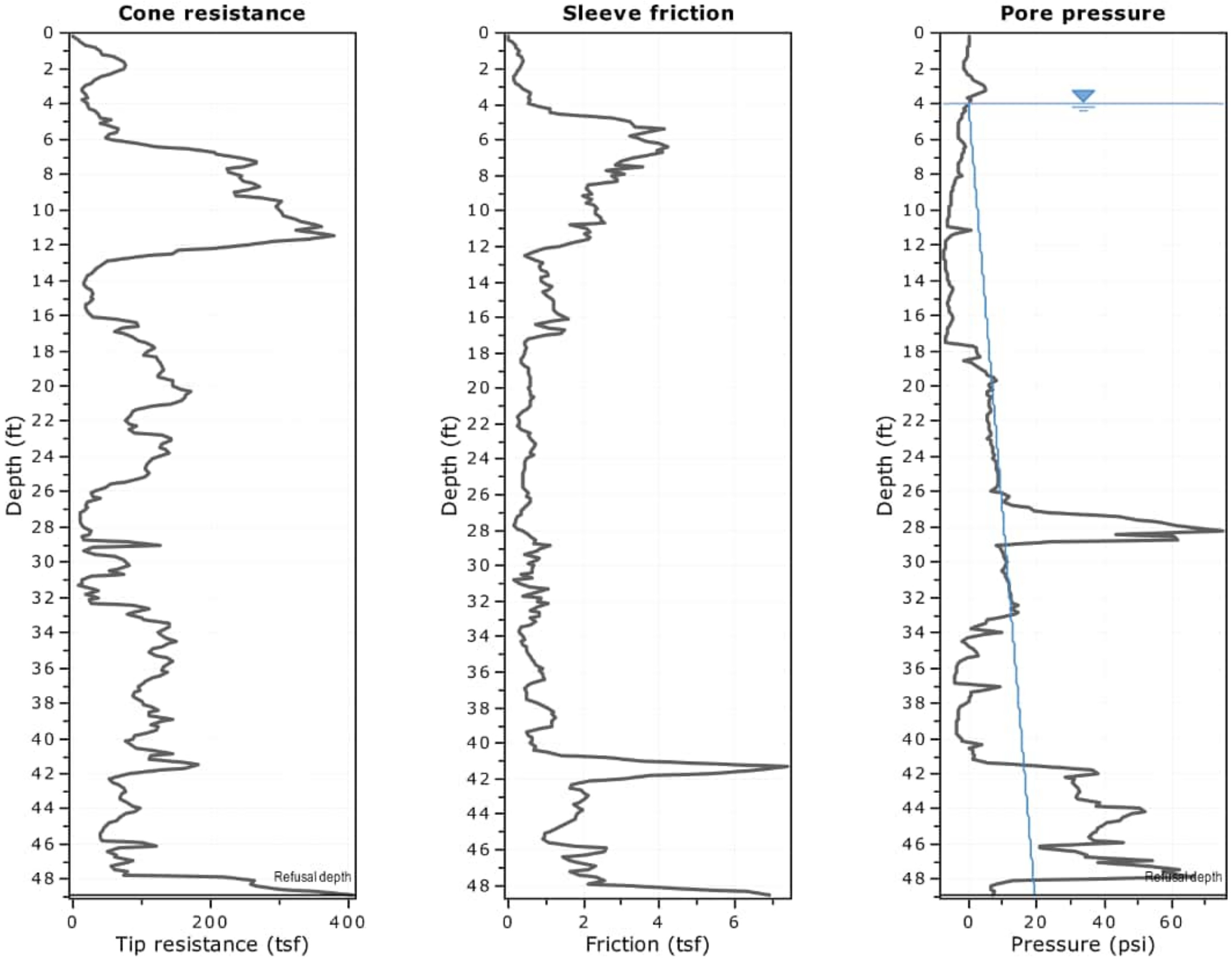
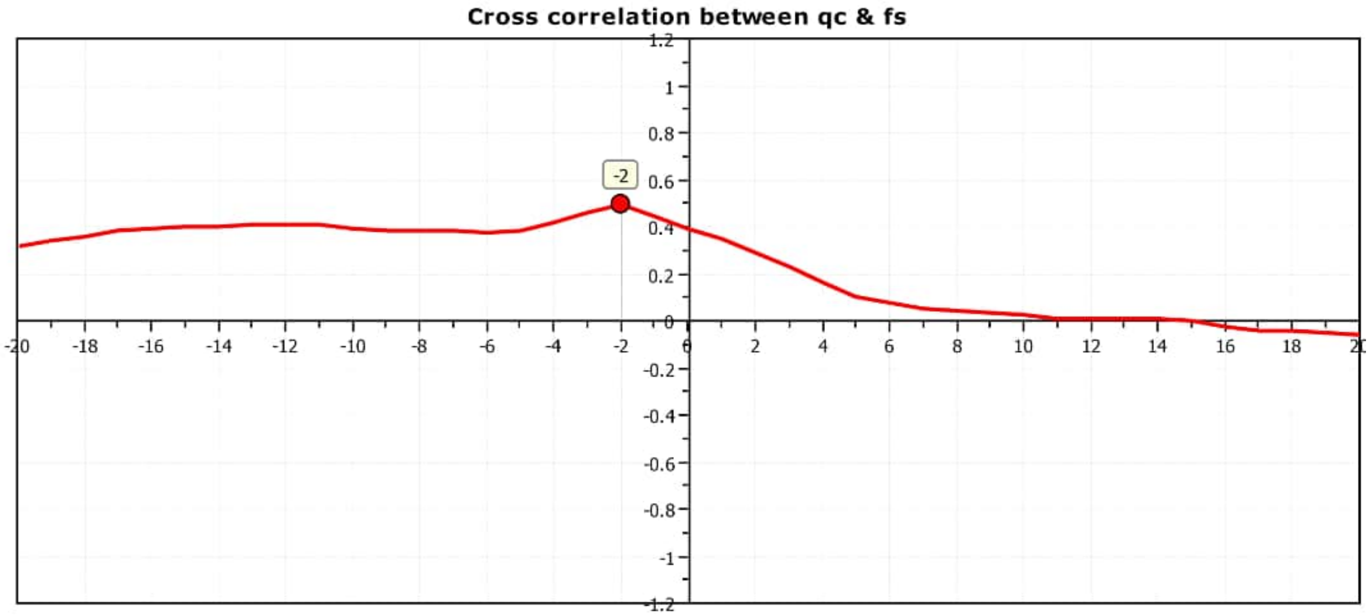
CPT-03
Total depth: 48.88 ft, Date: 9/9/2024
Surface Elevation: 101.50 ft
Cone Operator: BM

Project: **Wastewater Treatment Plant**
Location: **Pembroke, Georgia**



The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



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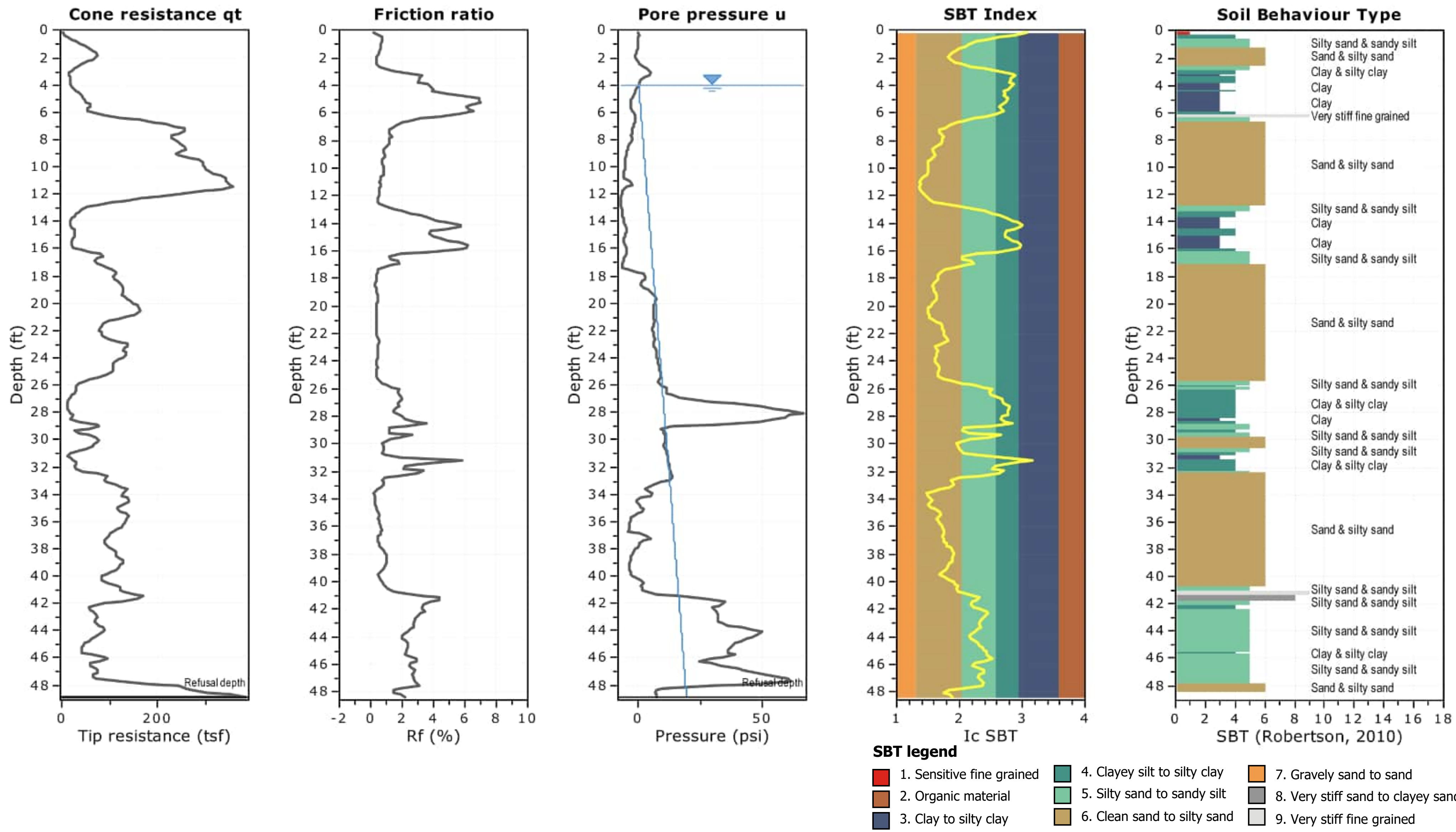
CPT-03

Total depth: 48.88 ft, Date: 9/9/2024
Surface Elevation: 101.50 ft
Cone Operator: BM

Project: Wastewater Treatment Plant
Location: Pembroke, Georgia



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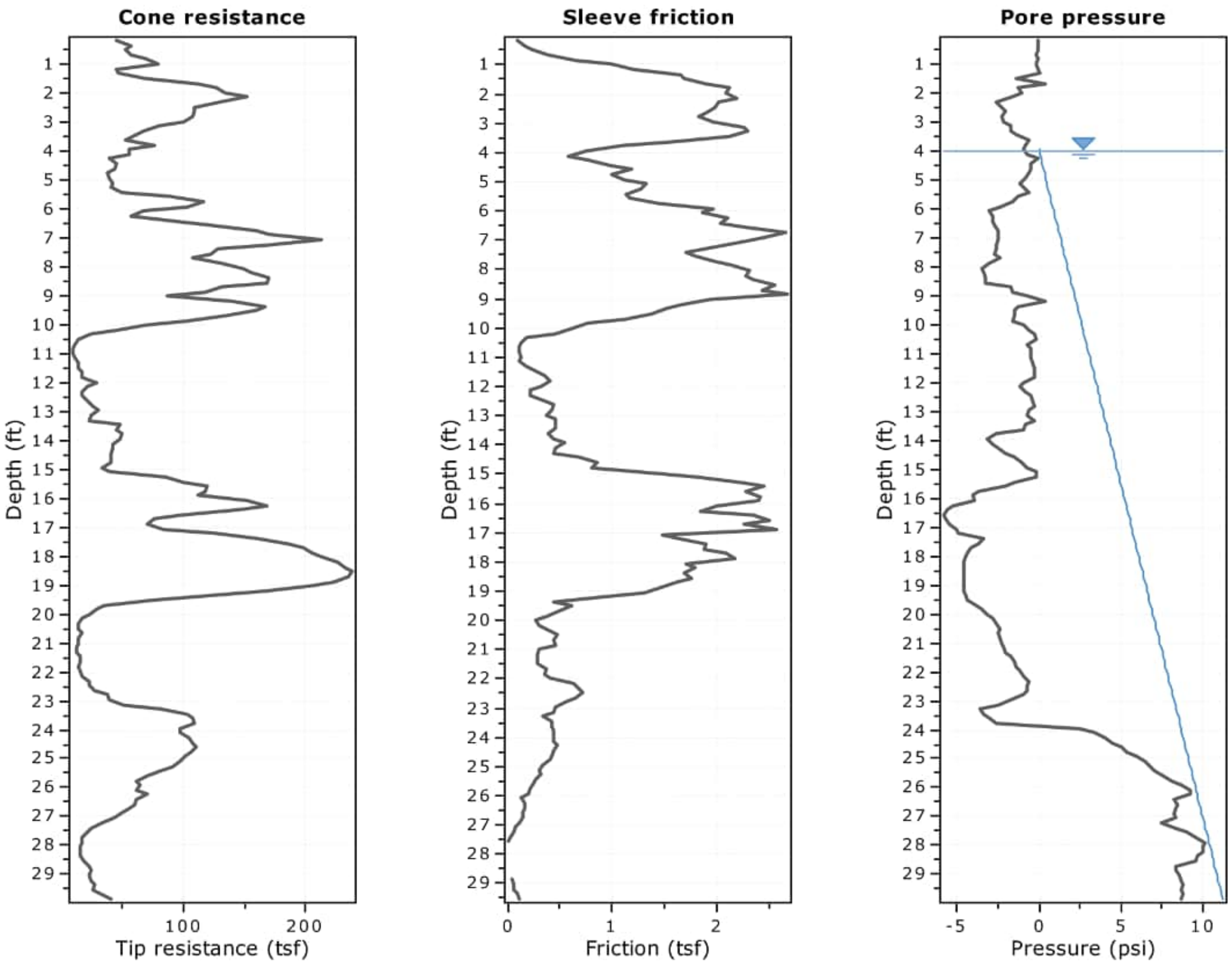
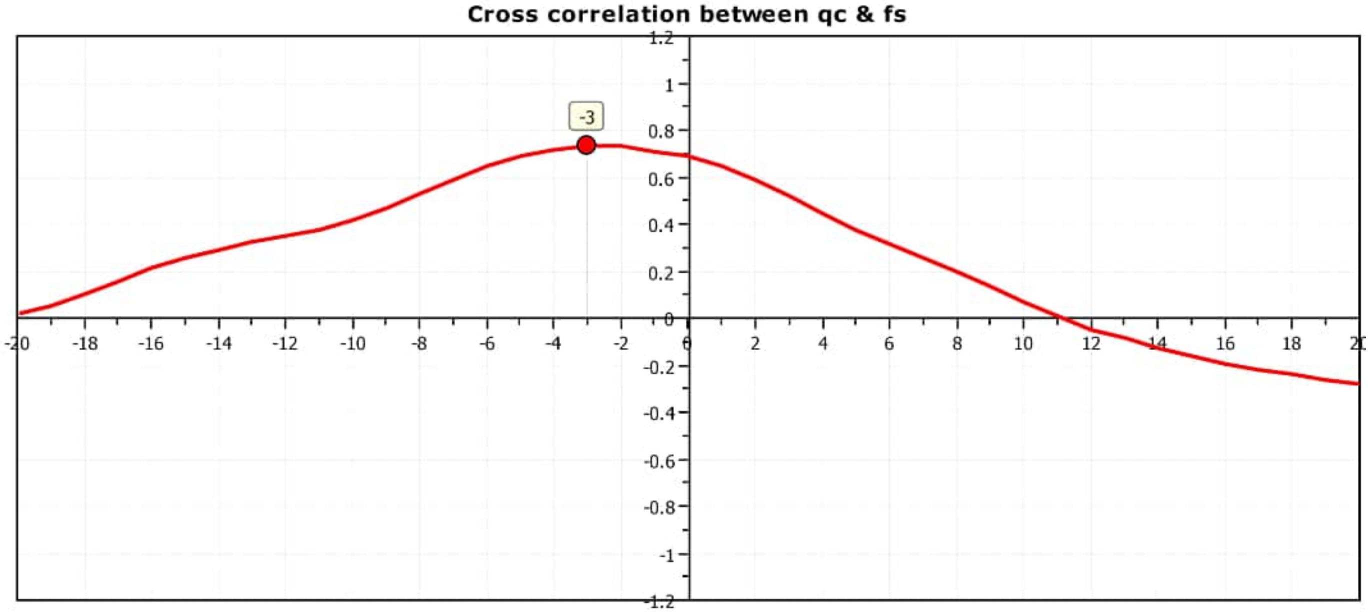
CPT-04

Total depth: 29.86 ft, Date: 9/9/2024
Surface Elevation: 105.00 ft
Cone Operator: BM

Project: **Wastewater Treatment Plant**
Location: **Pembroke, Georgia**



The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



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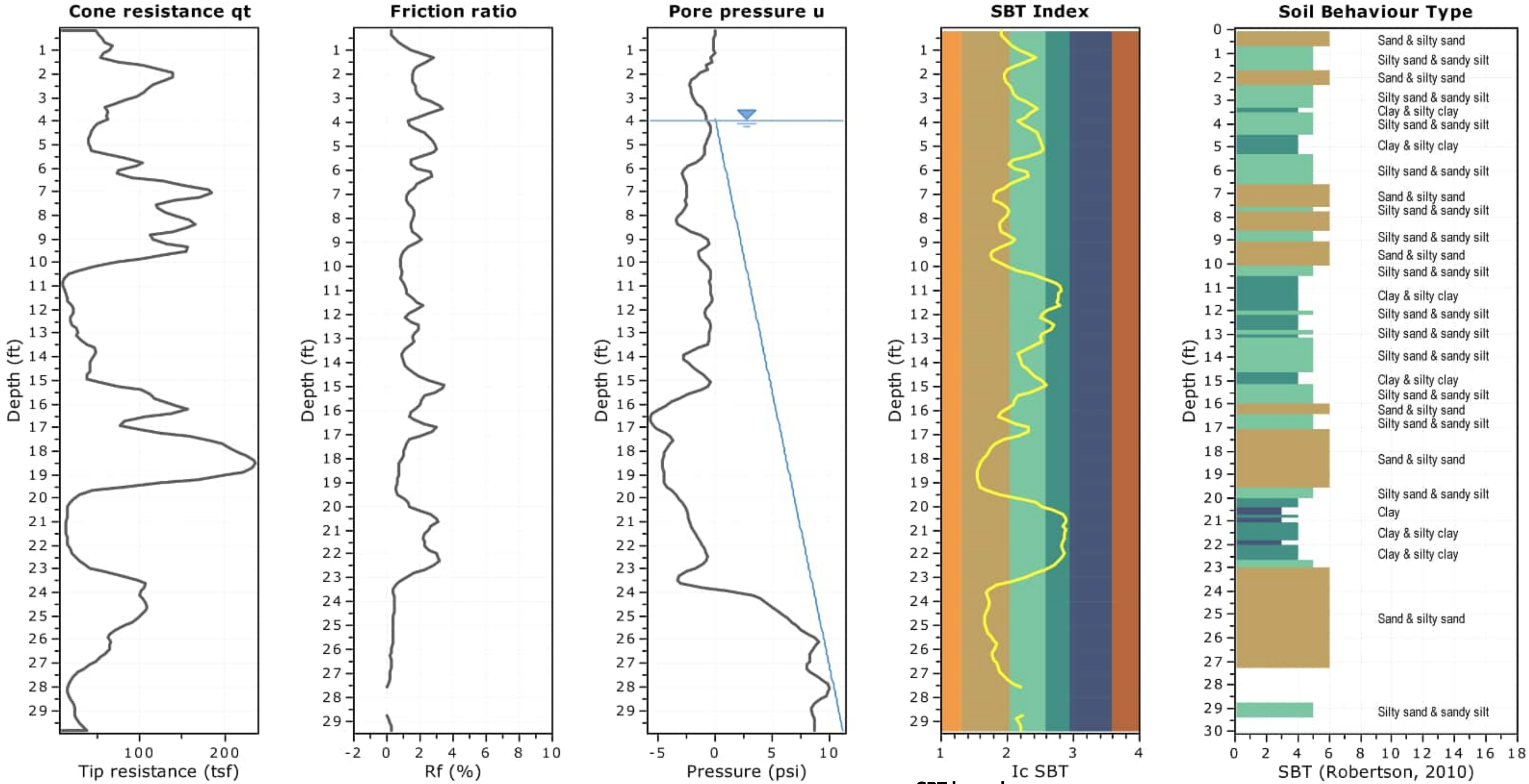
CPT-04
Total depth: 29.86 ft, Date: 9/9/2024
Surface Elevation: 105.00 ft
Cone Operator: BM

Project: **Wastewater Treatment Plant**
Location: **Pembroke, Georgia**



SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

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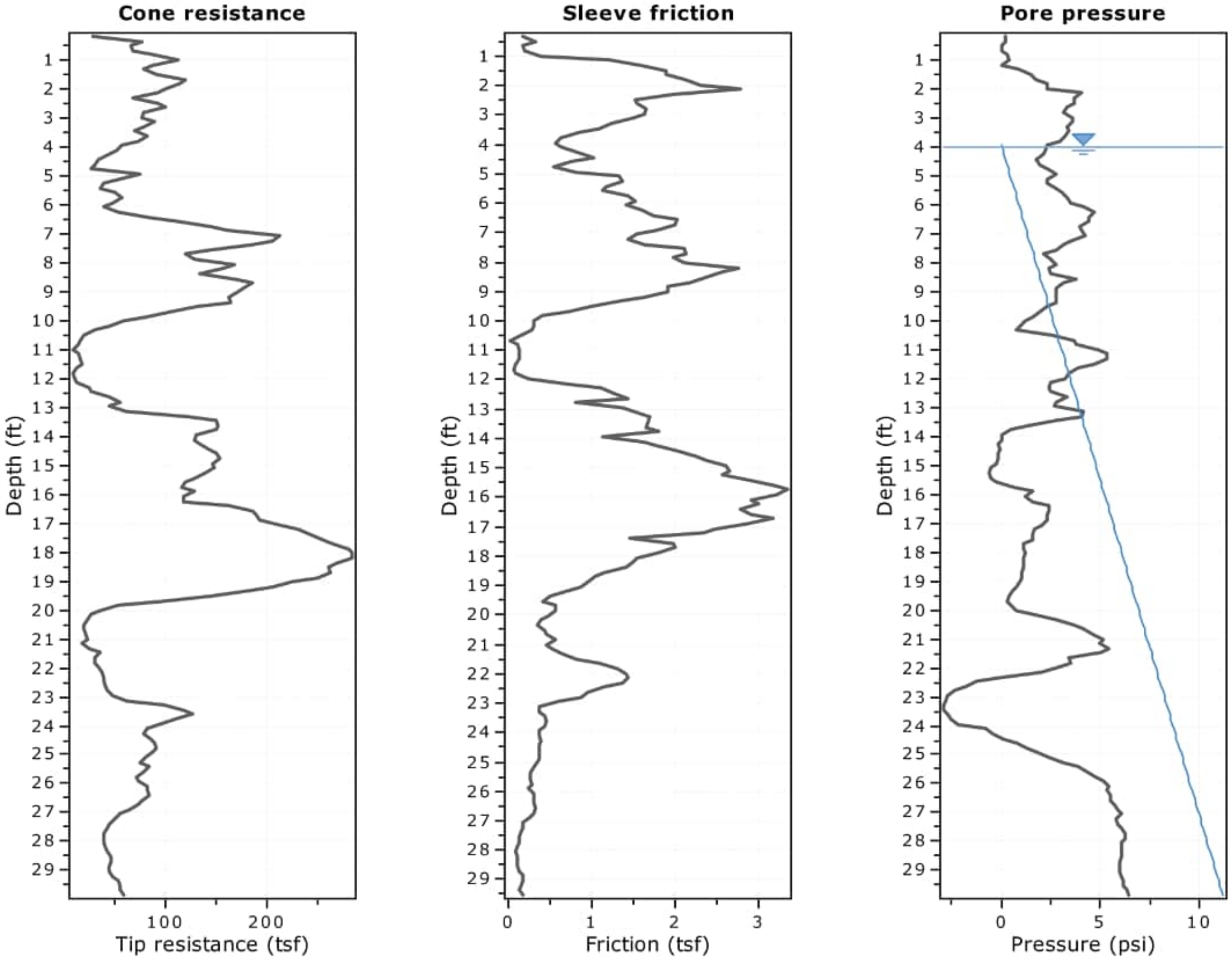
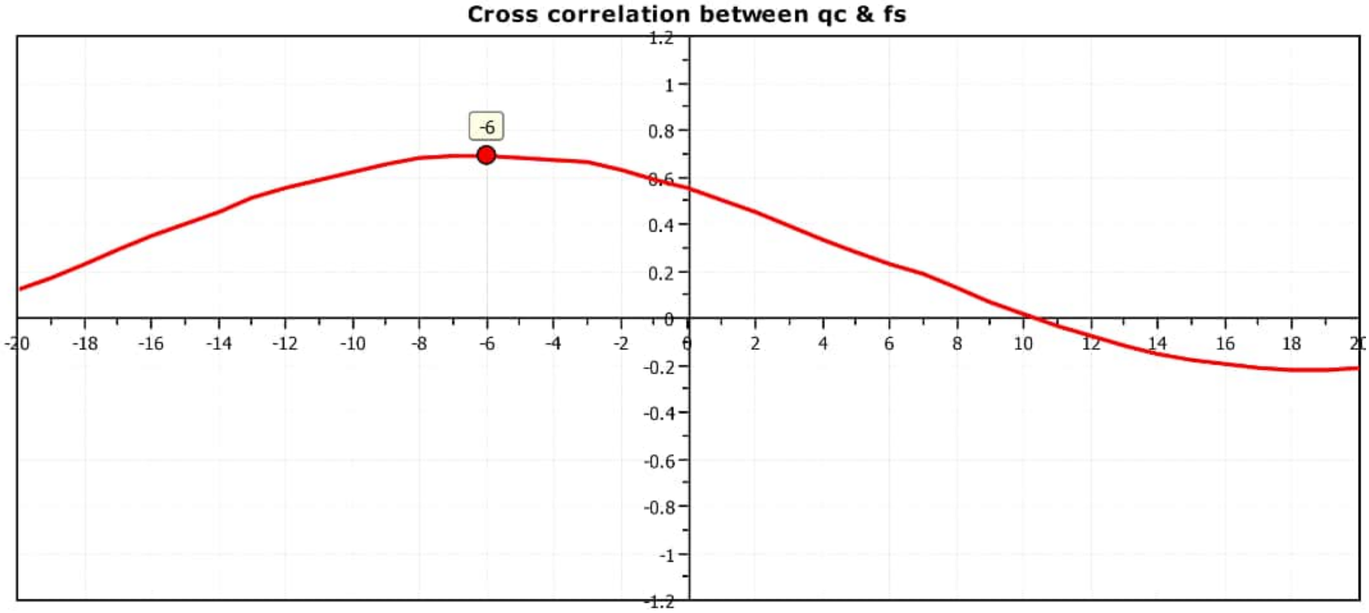
CPT-04a
Total depth: 29.86 ft, Date: 9/16/2024
Surface Elevation: 105.00 ft
Cone Operator: BM

Project: **Wastewater Treatment Plant**
Location: **Pembroke, Georgia**



The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



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CPT-04a

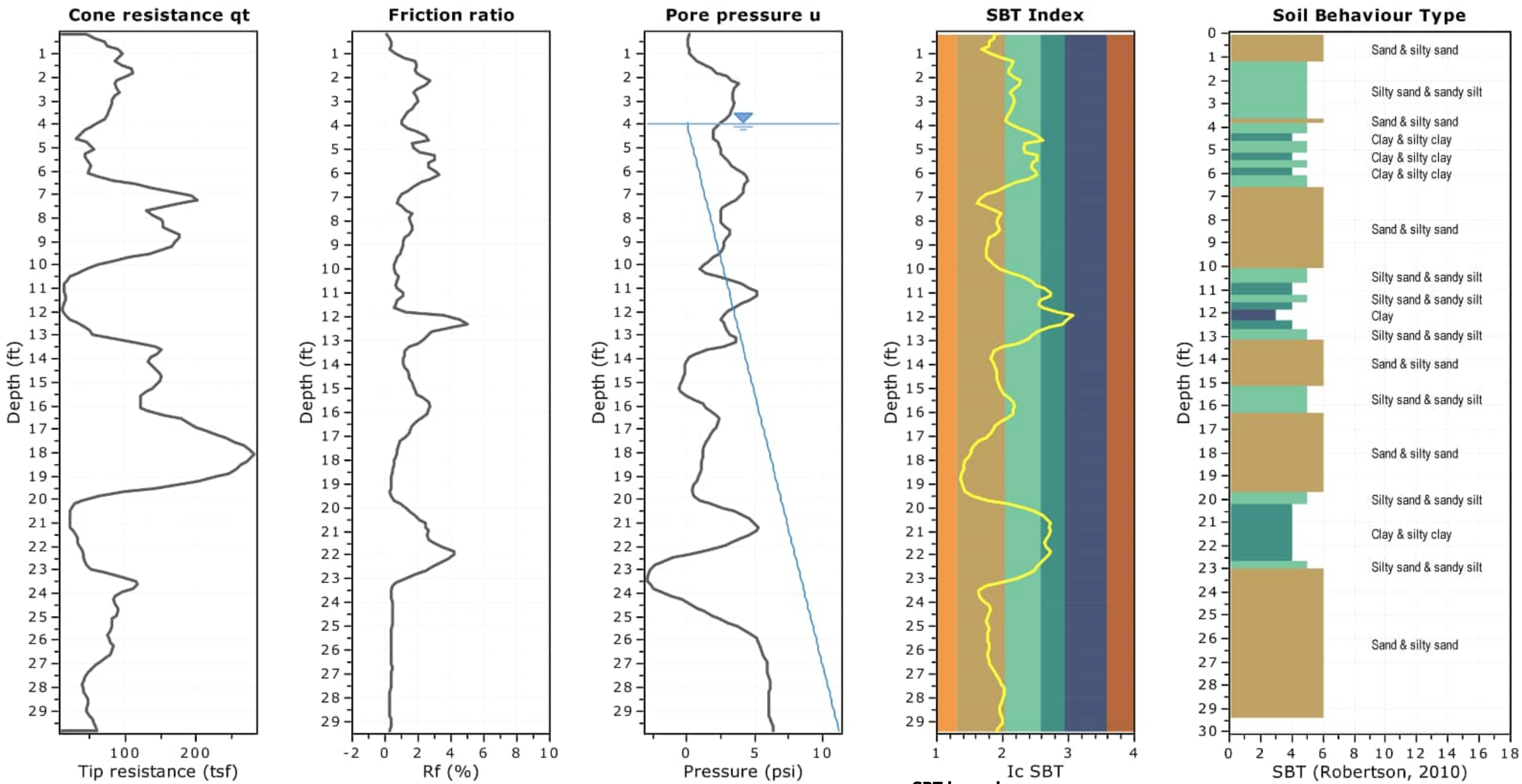
Total depth: 29.86 ft, Date: 9/16/2024

Surface Elevation: 105.00 ft

Cone Operator: BM

Project: **Wastewater Treatment Plant**

Location: **Pembroke, Georgia**



SBT legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty clay
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to clayey sand
- 9. Very stiff fine grained

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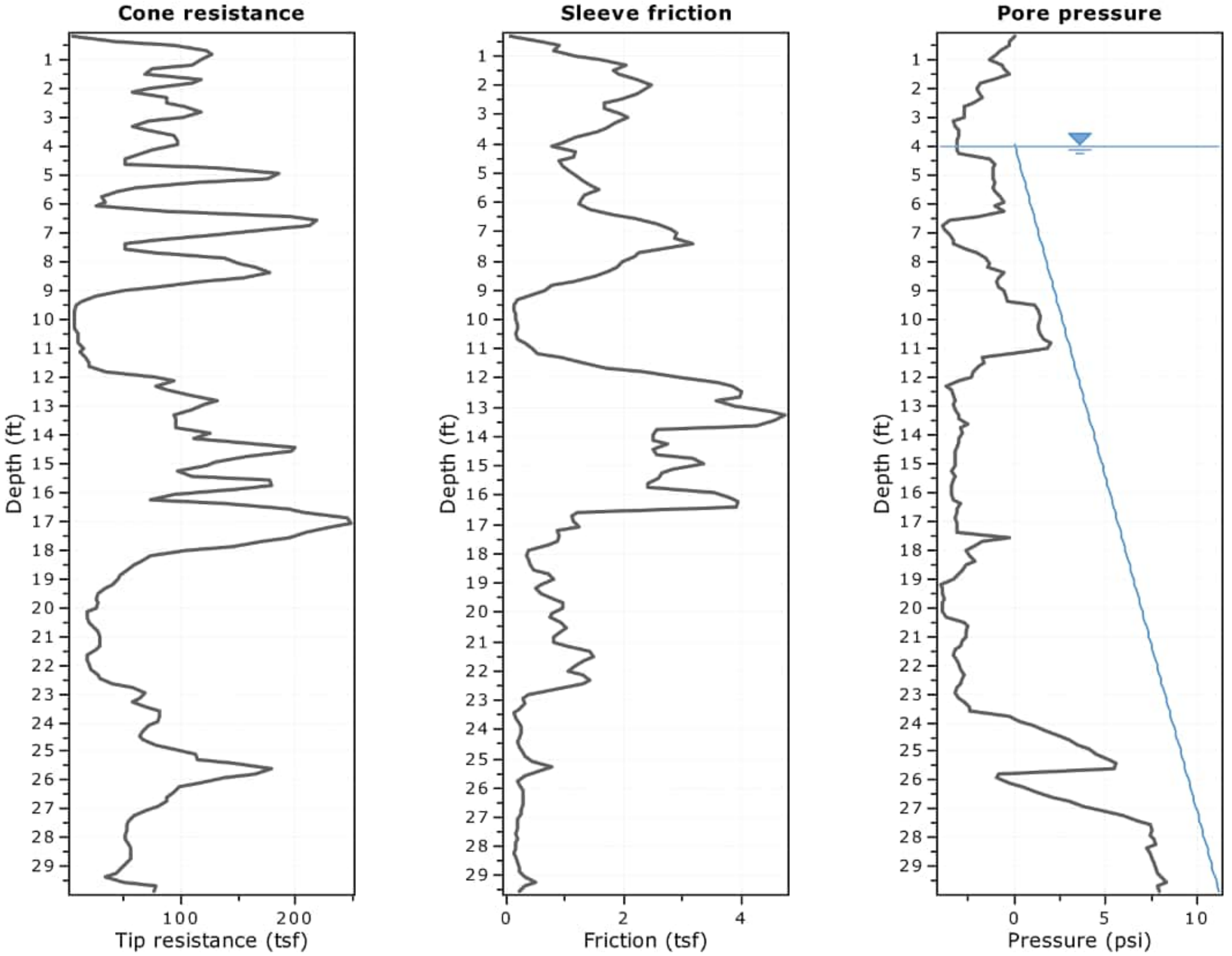
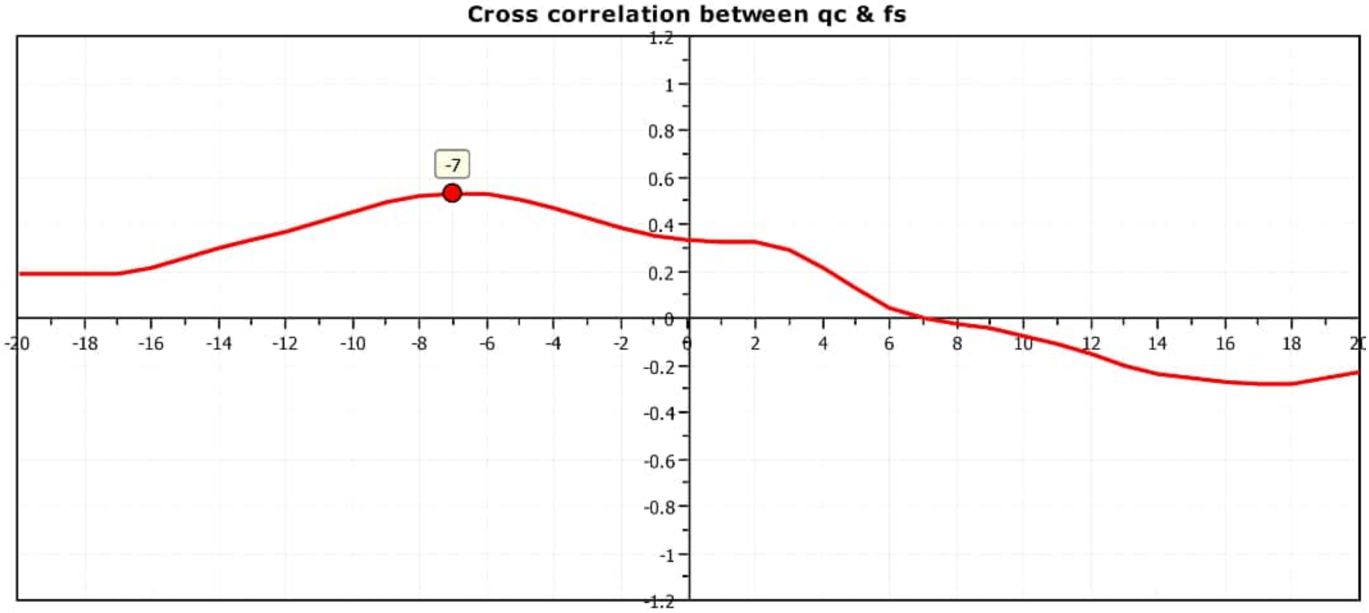
CPT-05
Total depth: 29.86 ft, Date: 9/9/2024
Surface Elevation: 105.00 ft
Cone Operator: BM

Project: **Wastewater Treatment Plant**
Location: **Pembroke, Georgia**



The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).

Cross correlation between qc & fs



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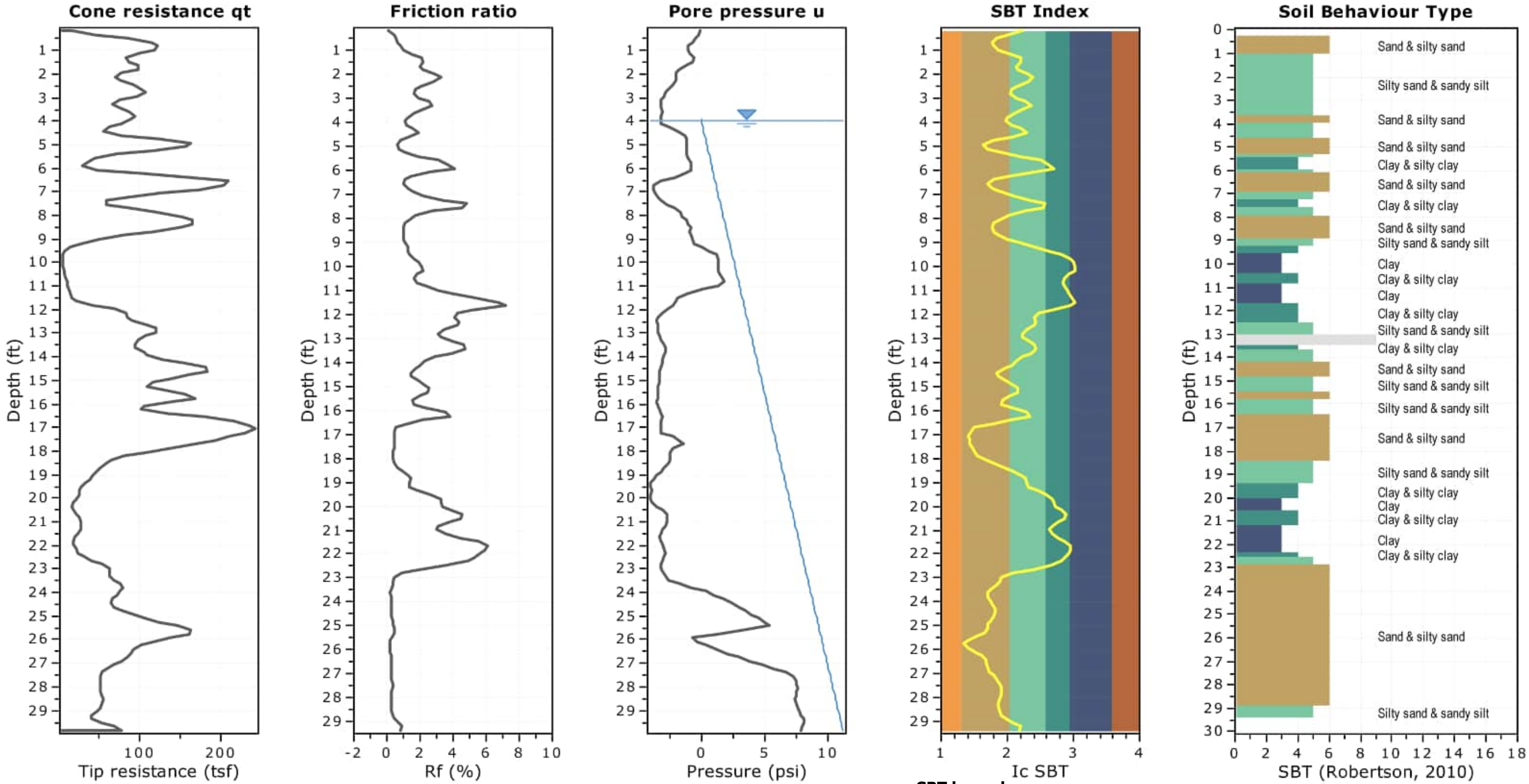
CPT-05
Total depth: 29.86 ft, Date: 9/9/2024
Surface Elevation: 105.00 ft
Cone Operator: BM

Project: **Wastewater Treatment Plant**
Location: **Pembroke, Georgia**



SBT legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty clay
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to clayey sand
- 9. Very stiff fine grained

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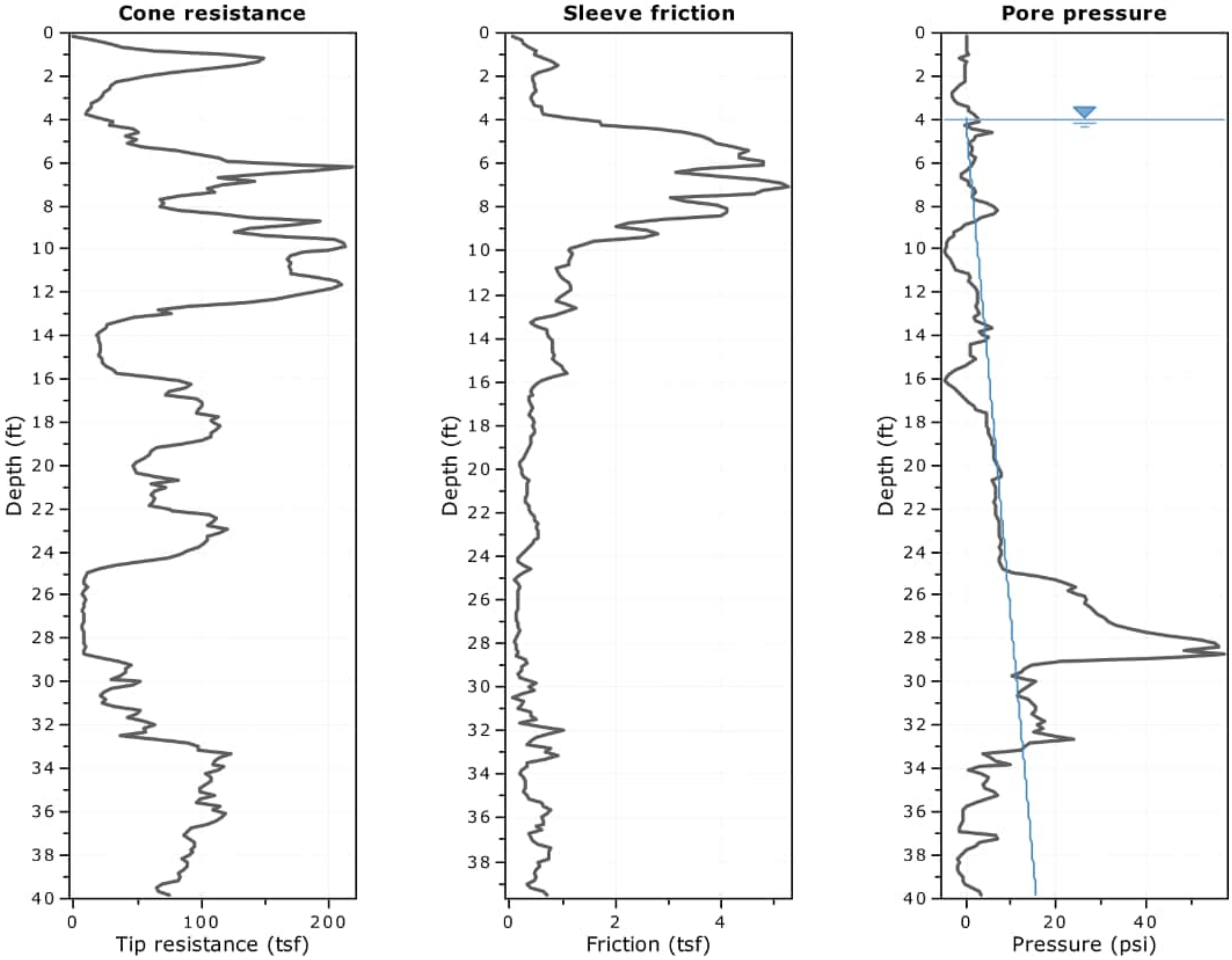
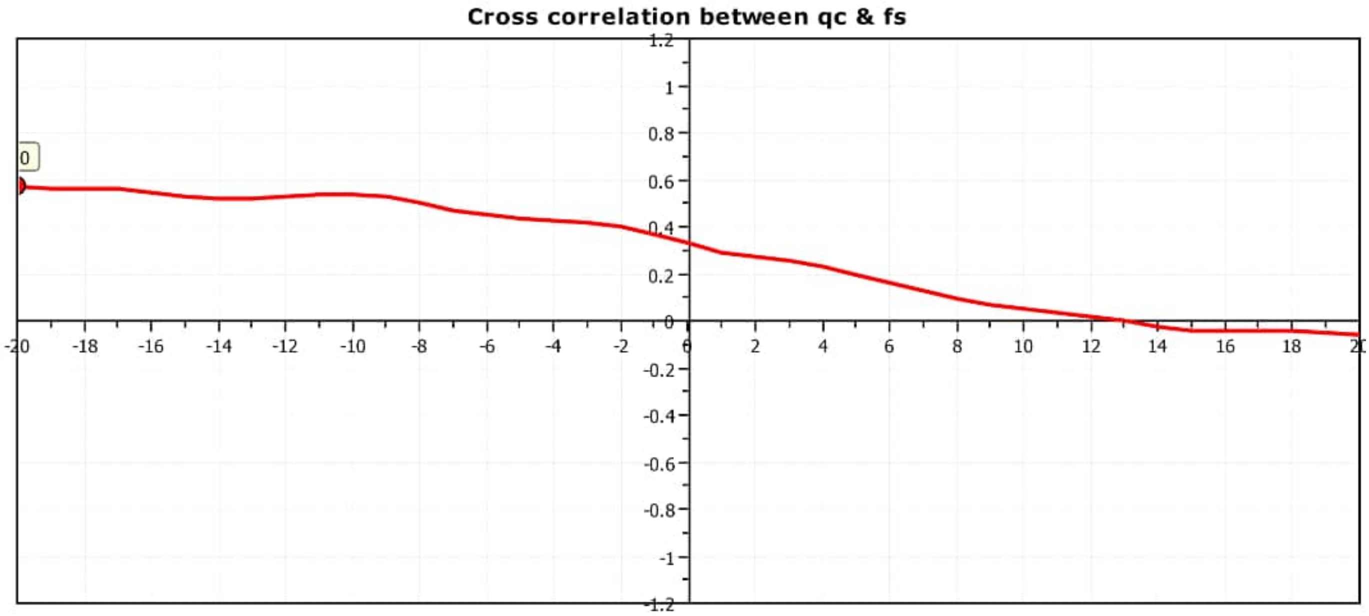
CPT-06
Total depth: 39.86 ft, Date: 9/9/2024
Surface Elevation: 98.50 ft
Cone Operator: BM

Project: **Wastewater Treatment Plant**
Location: **Pembroke, Georgia**



The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



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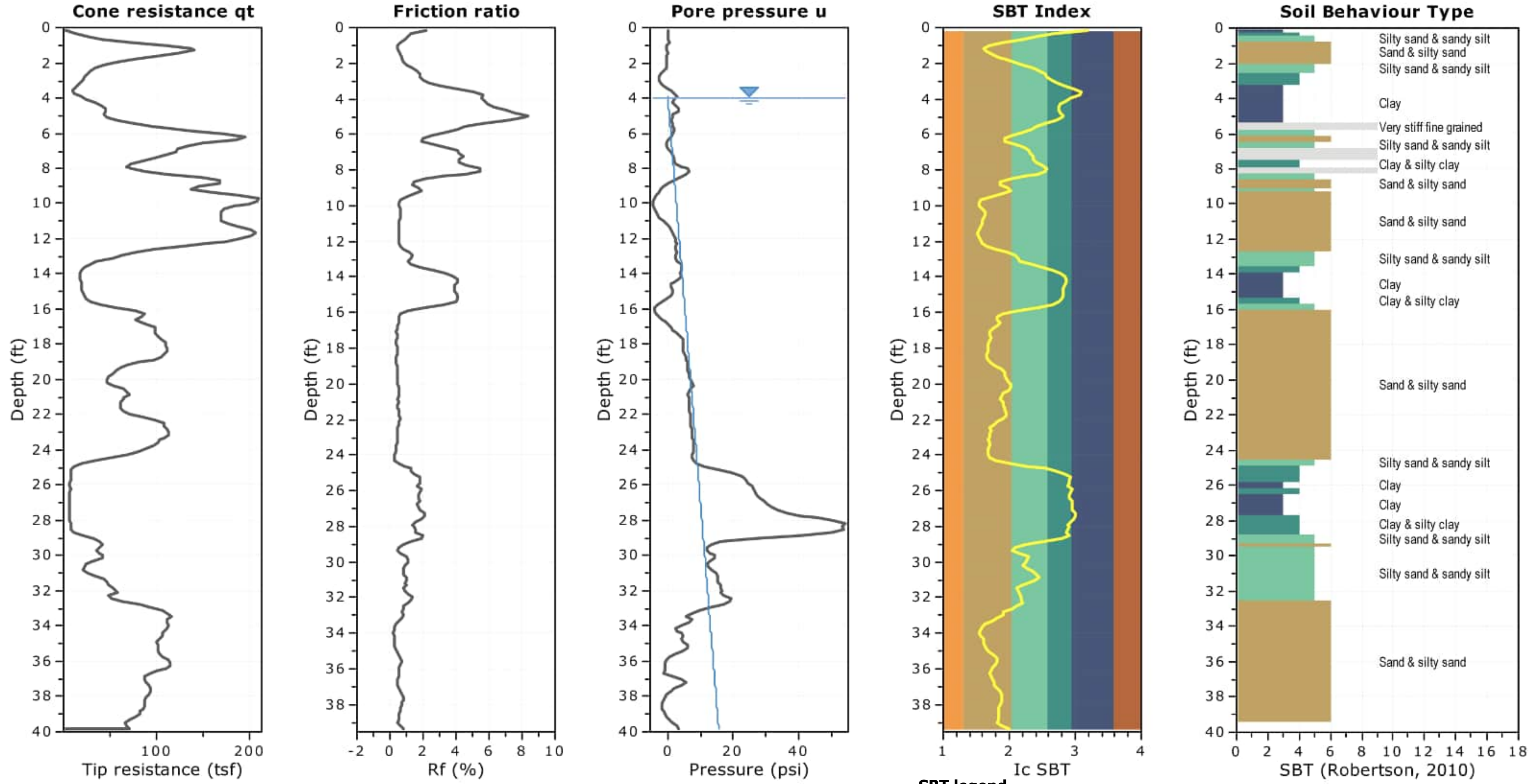
CPT-06
Total depth: 39.86 ft, Date: 9/9/2024
Surface Elevation: 98.50 ft
Cone Operator: BM

Project: **Wastewater Treatment Plant**
Location: **Pembroke, Georgia**



SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

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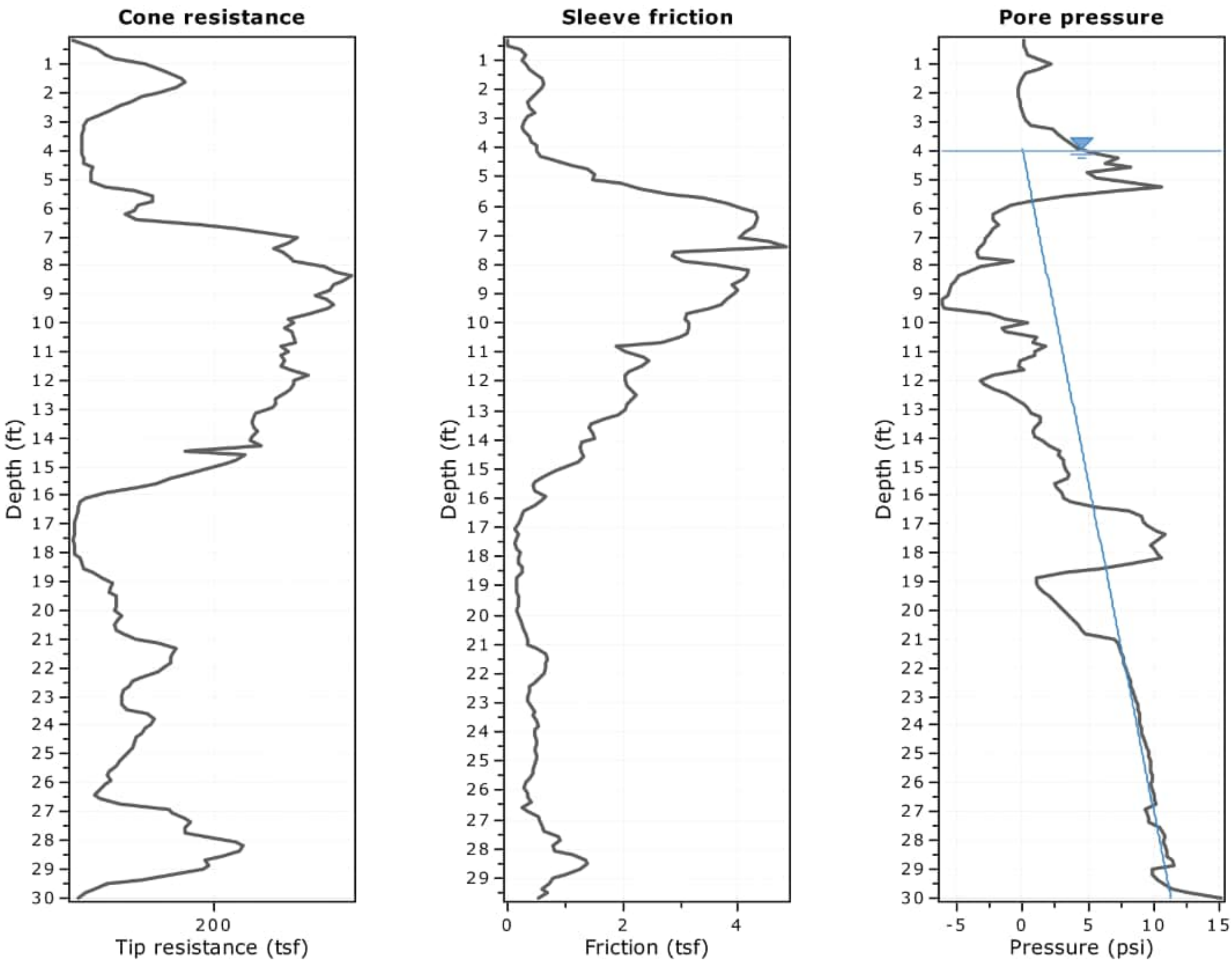
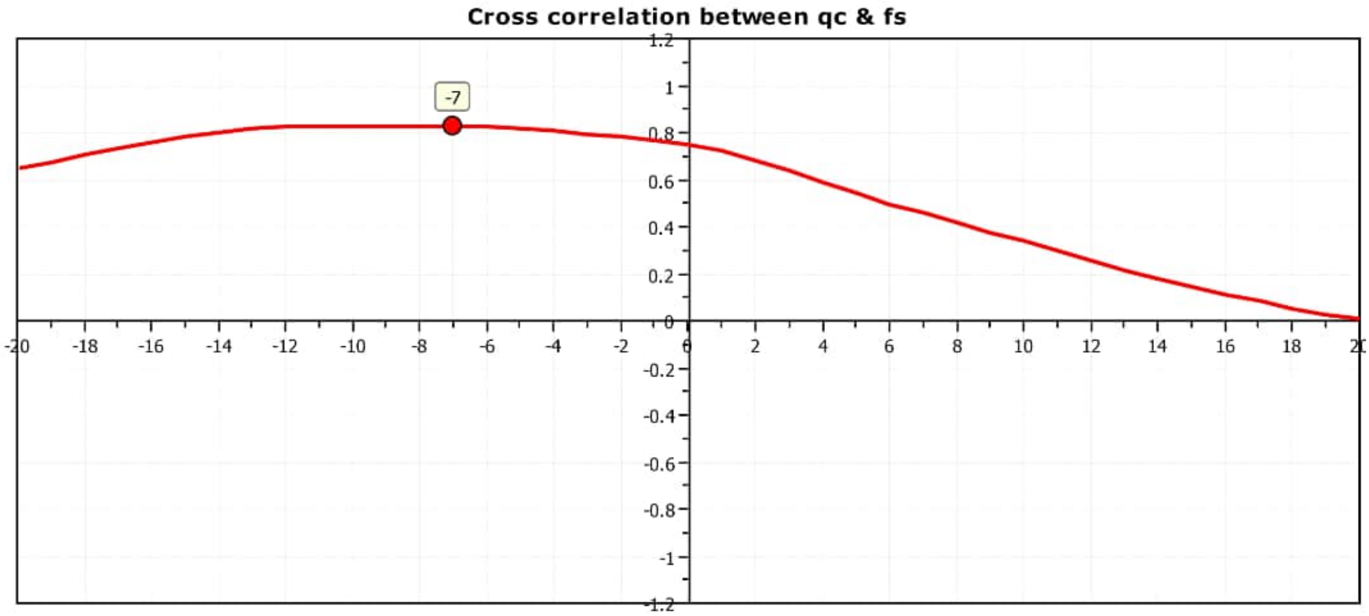
CPT-07

Total depth: 30.02 ft, Date: 9/9/2024
Surface Elevation: 98.00 ft
Cone Operator: BM

Project: **Wastewater Treatment Plant**
Location: **Pembroke, Georgia**



The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



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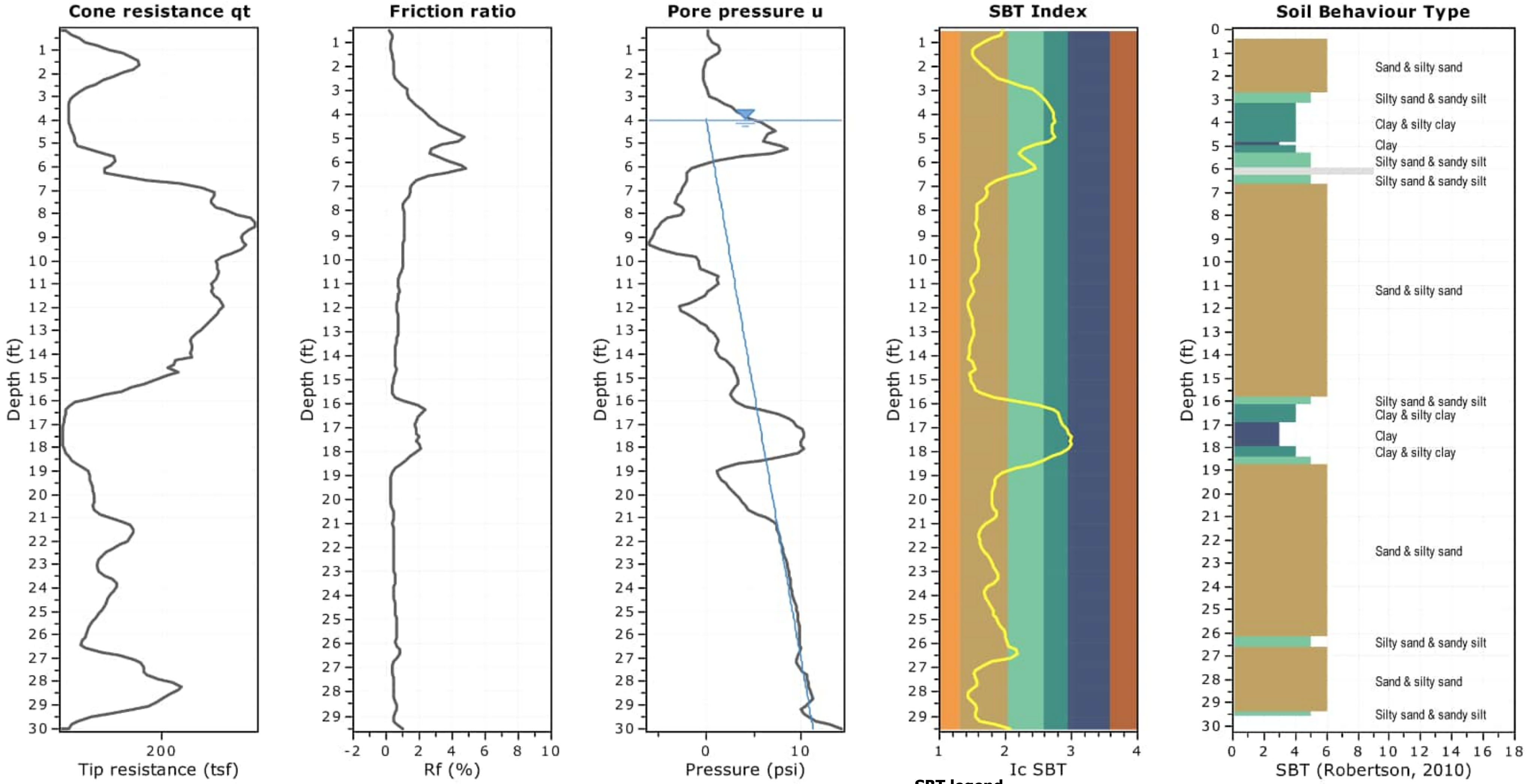
CPT-07

Total depth: 30.02 ft, Date: 9/9/2024
Surface Elevation: 98.00 ft
Cone Operator: BM

Project: **Wastewater Treatment Plant**
Location: **Pembroke, Georgia**



SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

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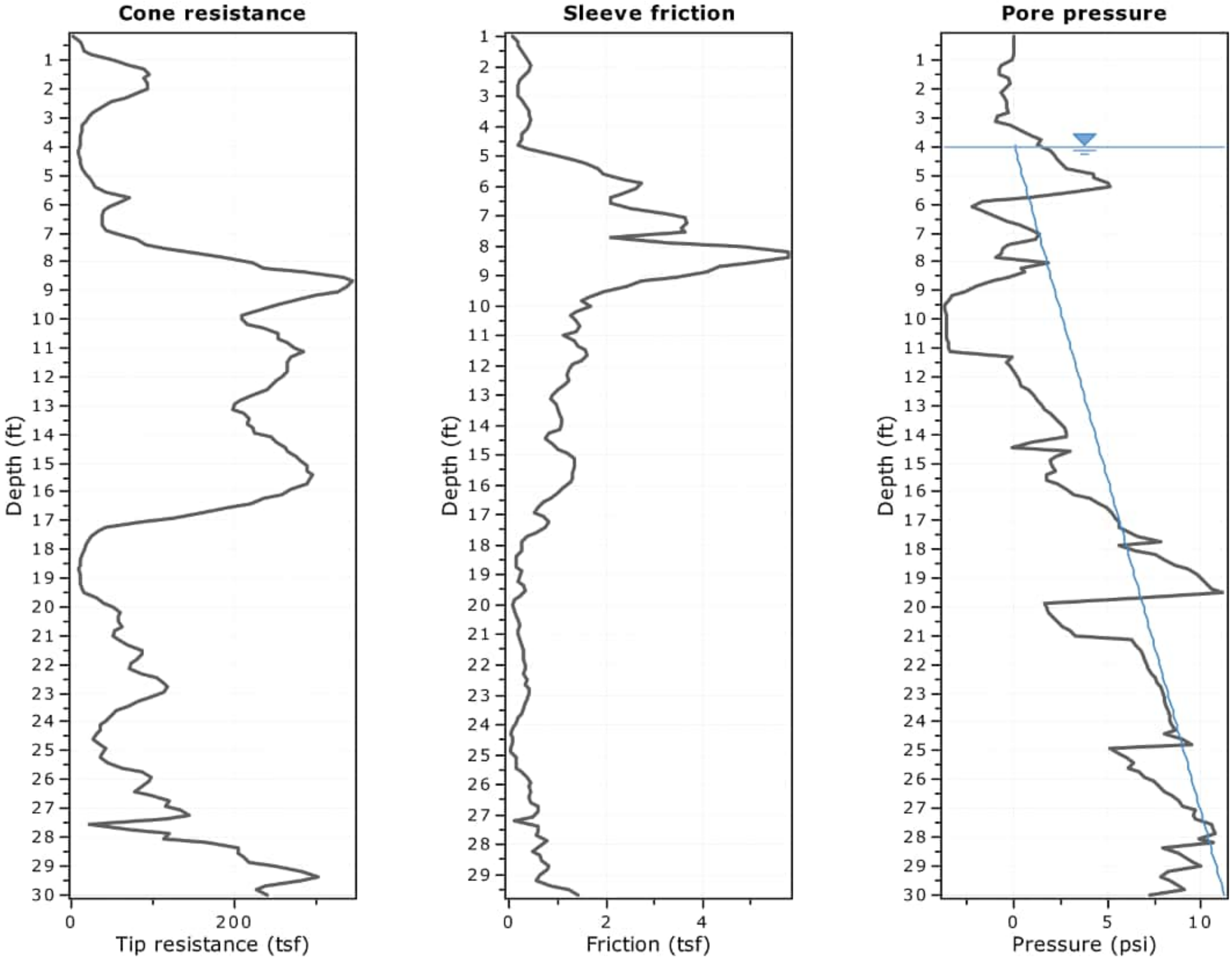
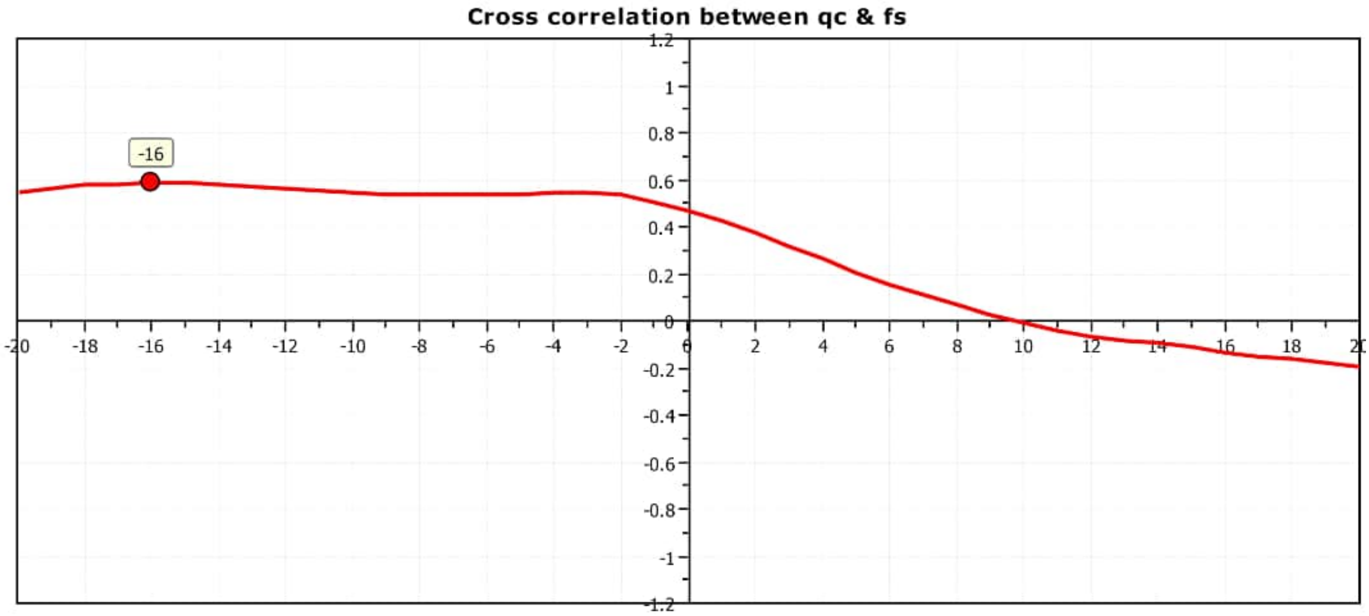
CPT-08

Total depth: 30.02 ft, Date: 9/9/2024
Surface Elevation: 98.00 ft
Cone Operator: BM

Project: **Wastewater Treatment Plant**
Location: **Pembroke, Georgia**



The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



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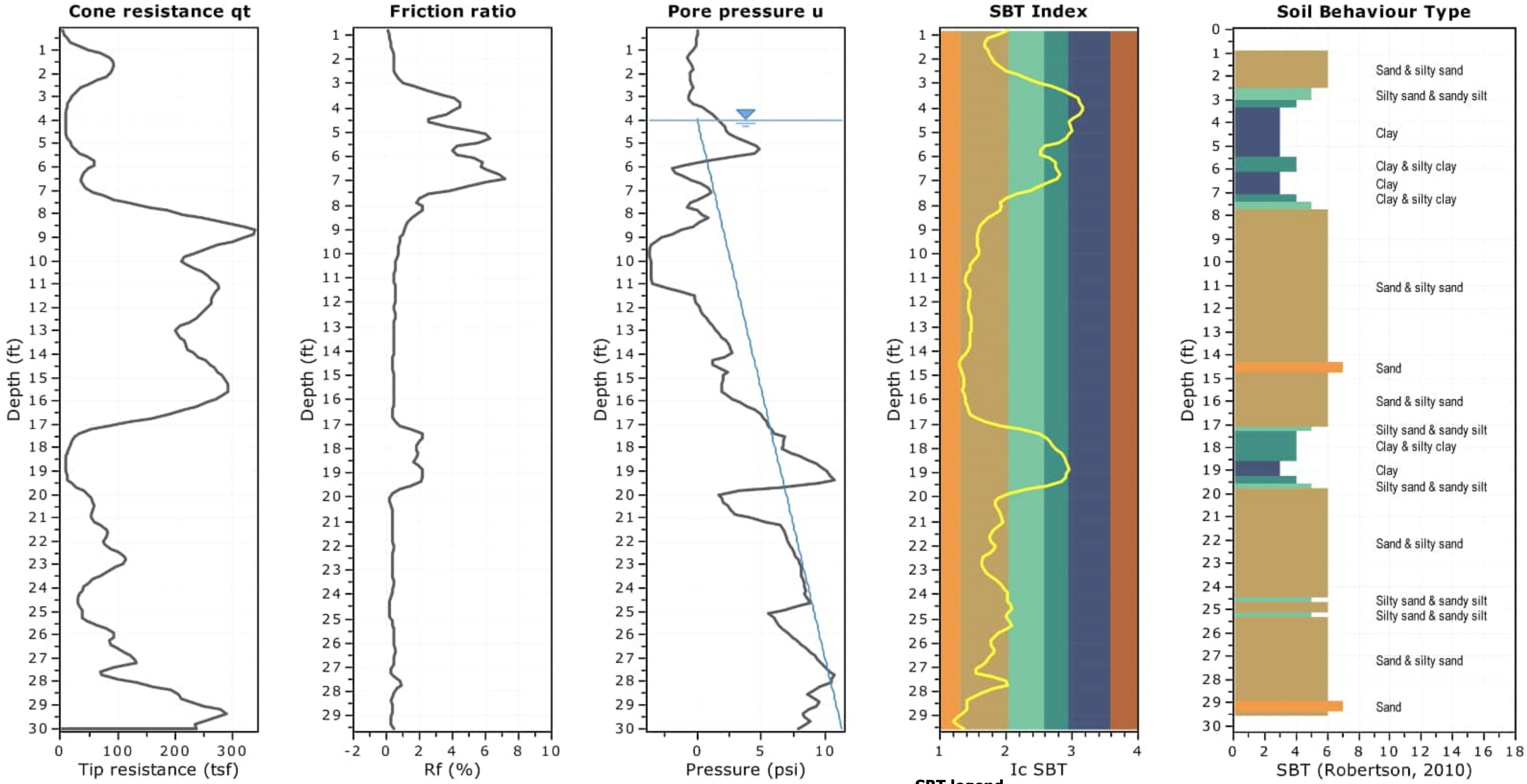
CPT-08

Total depth: 30.02 ft, Date: 9/9/2024
Surface Elevation: 98.00 ft
Cone Operator: BM

Project: **Wastewater Treatment Plant**
Location: **Pembroke, Georgia**



SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

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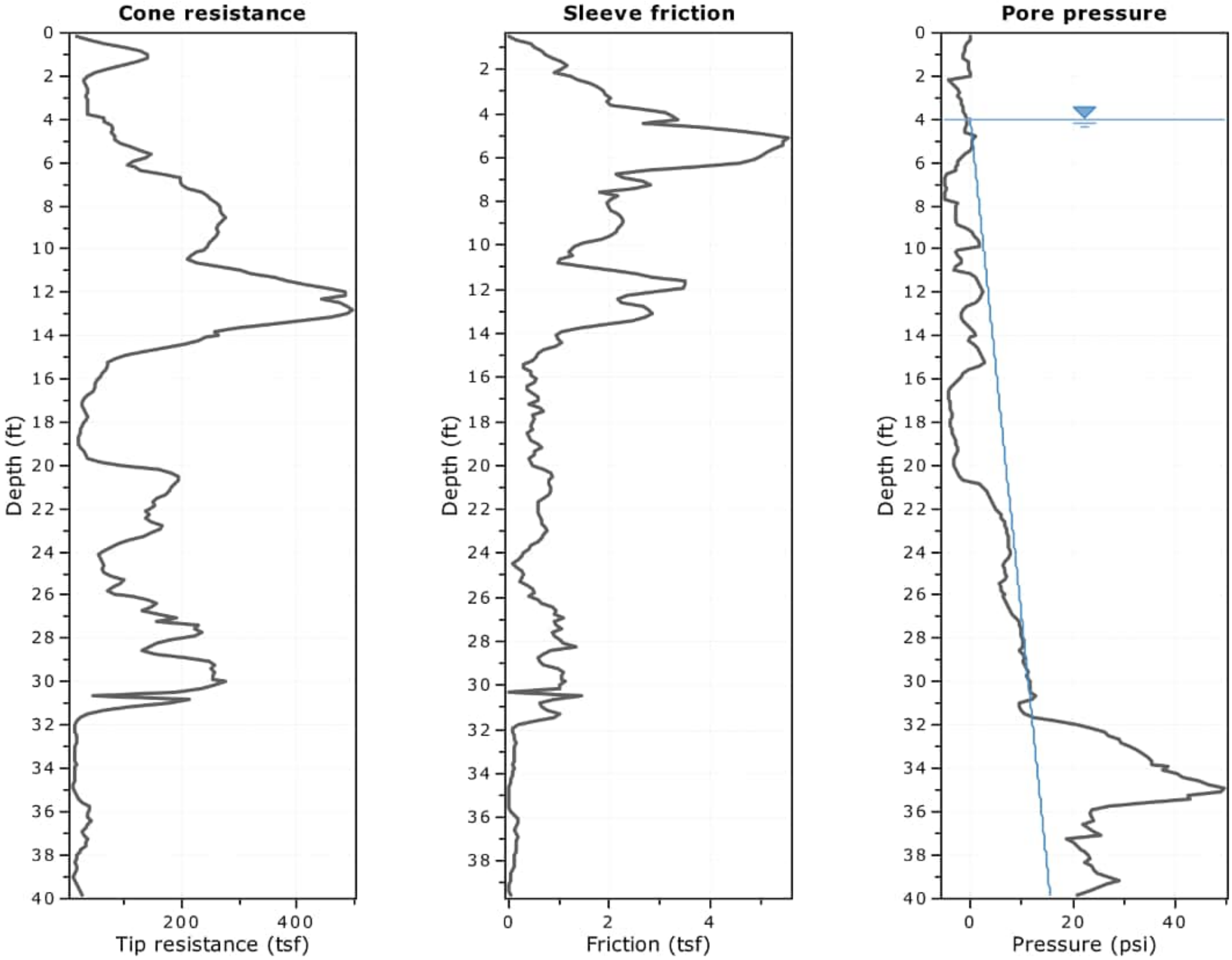
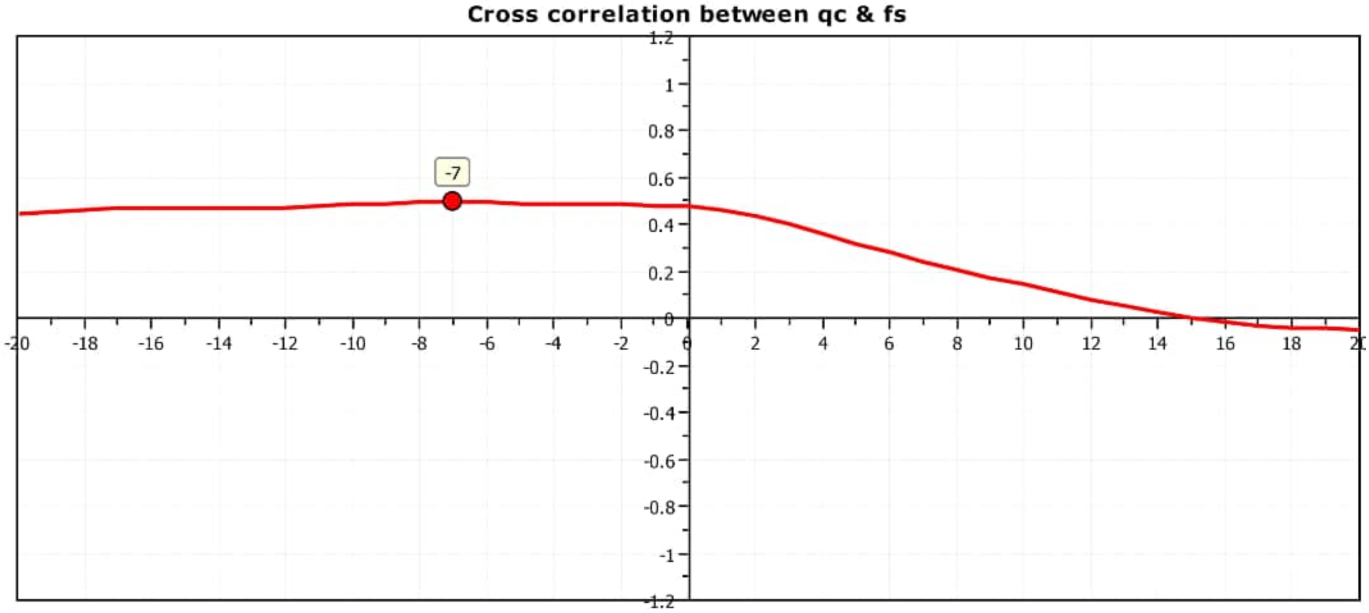
CPT-09
Total depth: 39.86 ft, Date: 9/9/2024
Surface Elevation: 97.50 ft
Cone Operator: BM

Project: **Wastewater Treatment Plant**
Location: **Pembroke, Georgia**



The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).

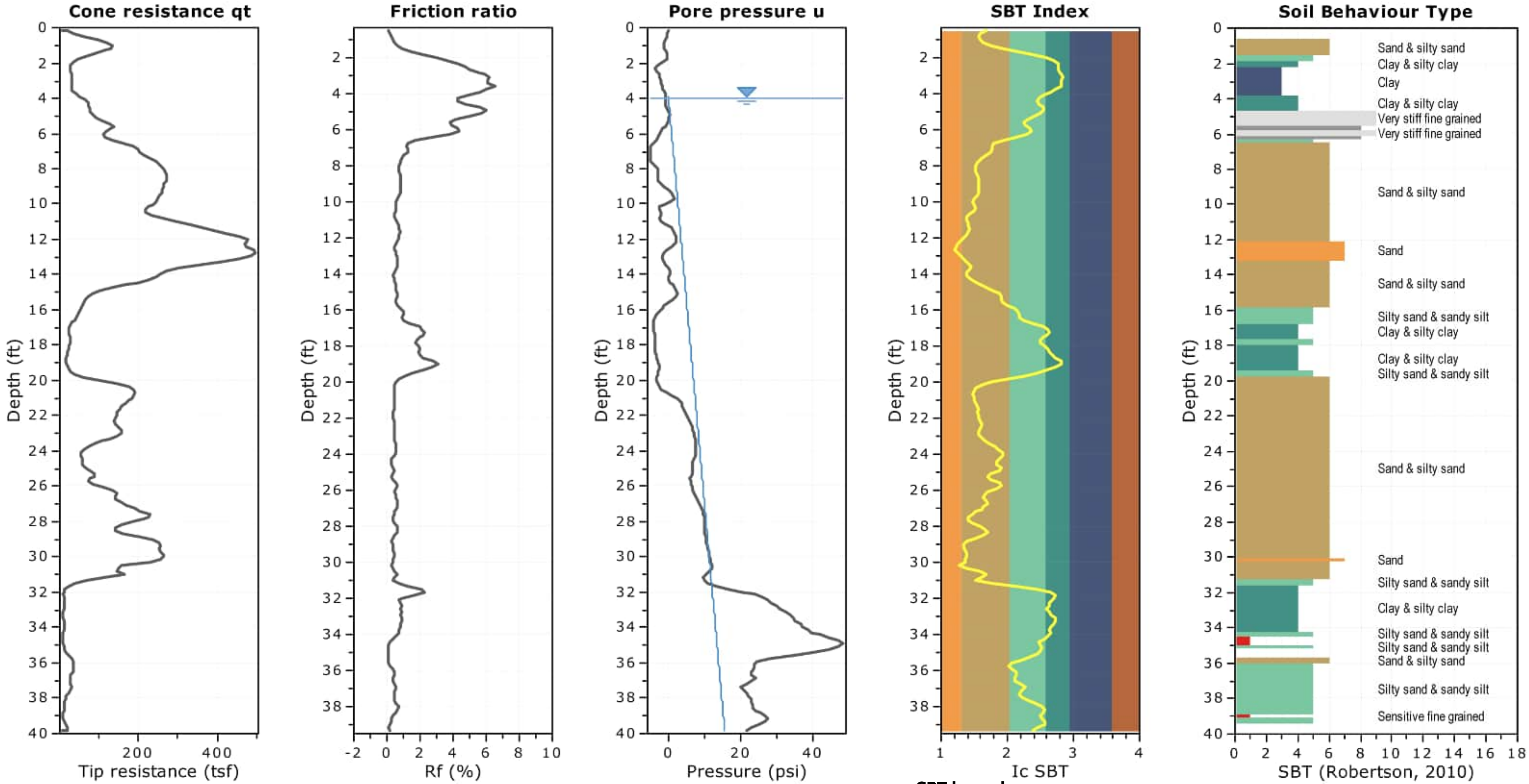
Cross correlation between qc & fs



WSP USA
2000 Buisness Center Drive, Suite 235
Savannah, Georgia
www.wsp.com

CPT-09
Total depth: 39.86 ft, Date: 9/9/2024
Surface Elevation: 97.50 ft
Cone Operator: BM

Project: **Wastewater Treatment Plant**
Location: **Pembroke, Georgia**



SBT legend

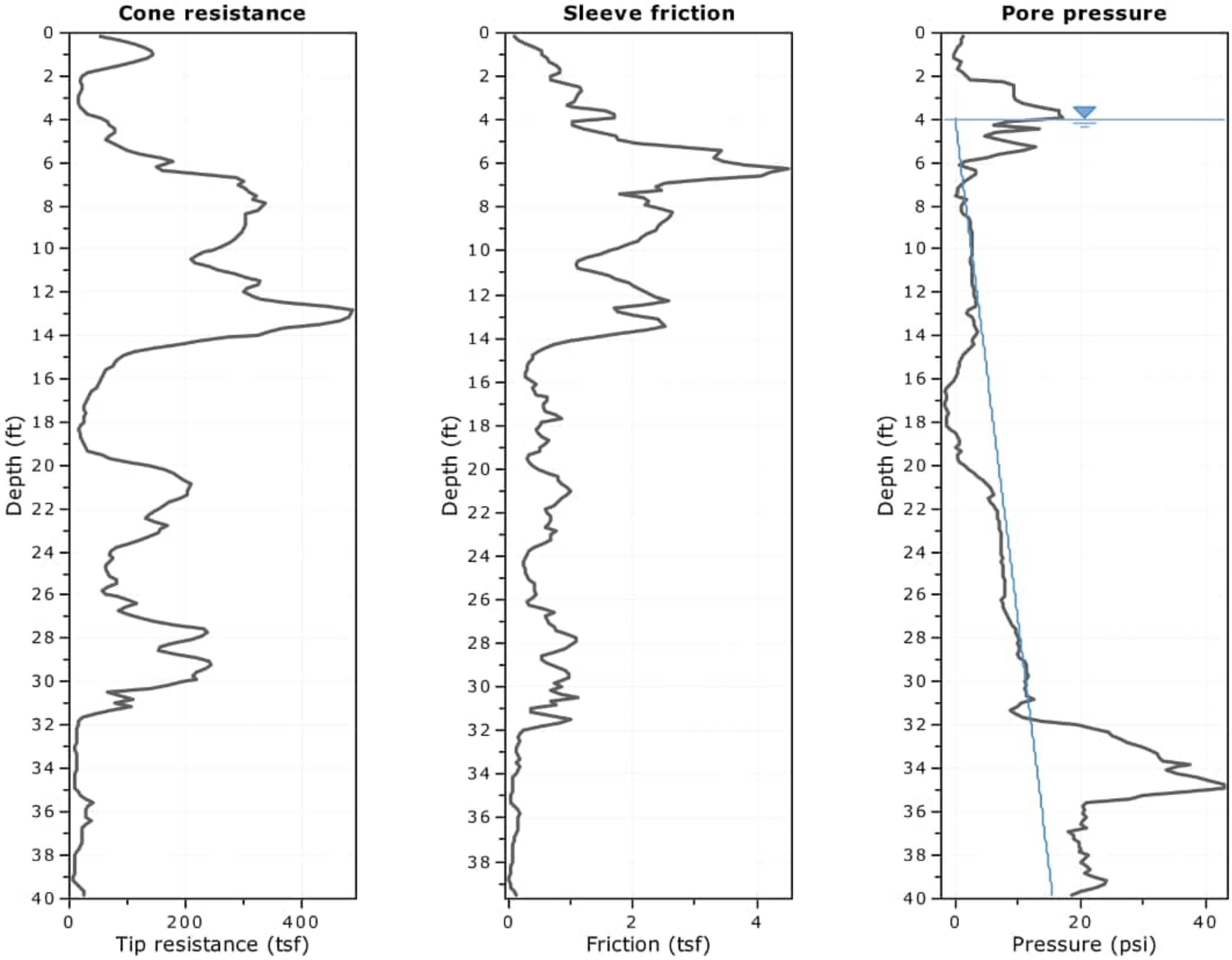
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

WSP USA
2000 Buisness Center Drive, Suite 235
Savannah, Georgia
www.wsp.com

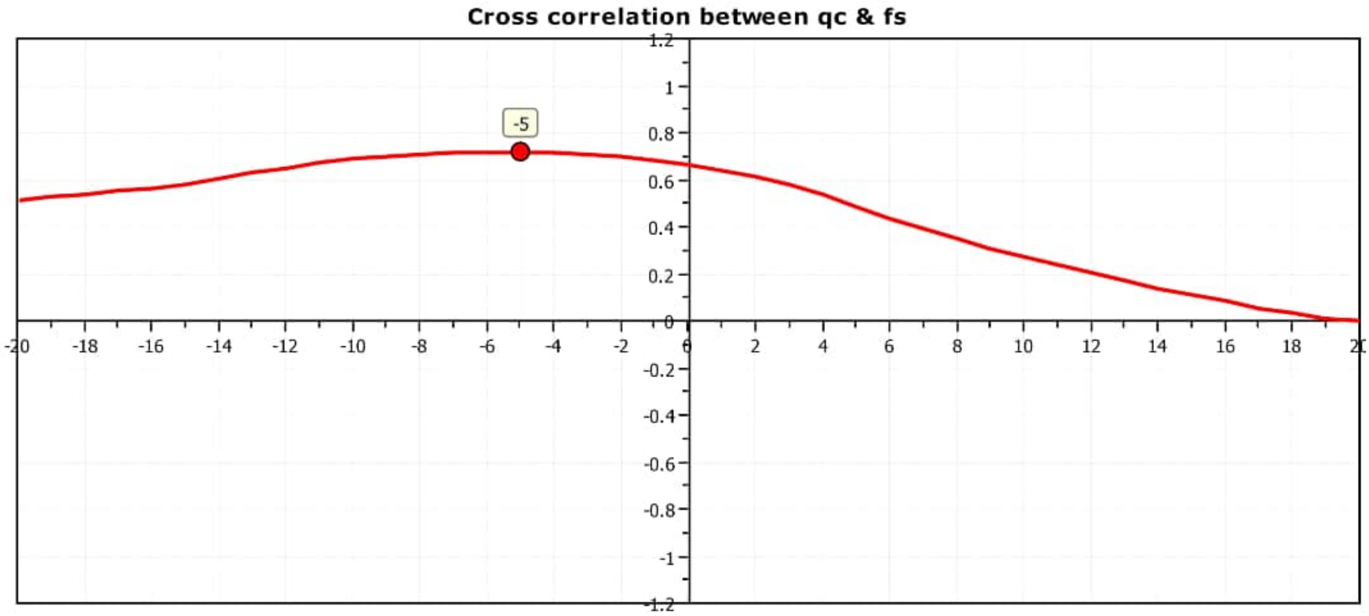
CPT-09a

Total depth: 39.86 ft, Date: 9/16/2024
Surface Elevation: 97.50 ft
Cone Operator: BM

Project: **Wastewater Treatment Plant**
Location: **Pembroke, Georgia**



The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



WSP USA
2000 Buisness Center Drive, Suite 235
Savannah, Georgia
www.wsp.com

CPT-09a

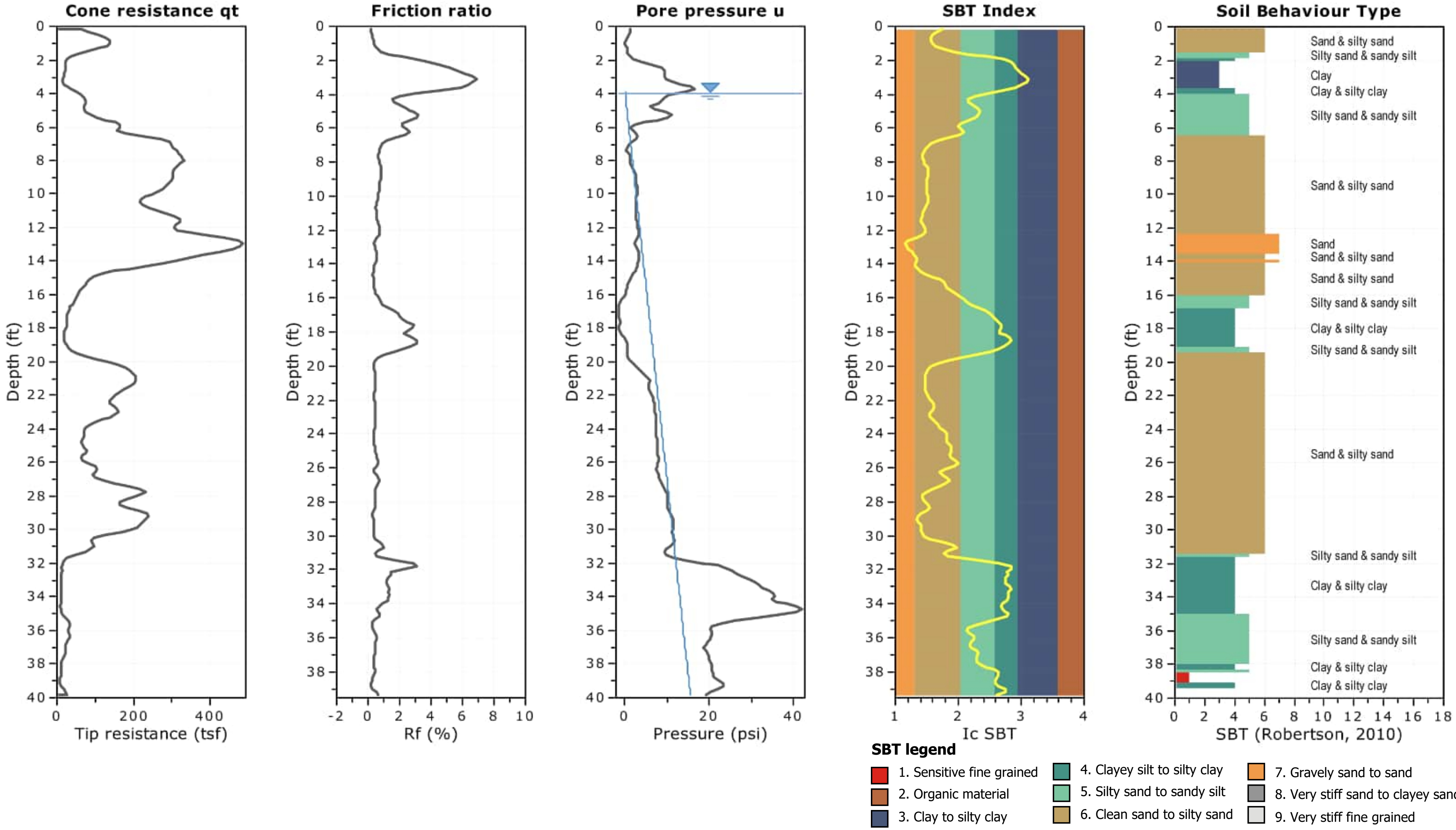
Total depth: 39.86 ft, Date: 9/16/2024

Surface Elevation: 97.50 ft

Cone Operator: BM

Project: **Wastewater Treatment Plant**

Location: **Pembroke, Georgia**

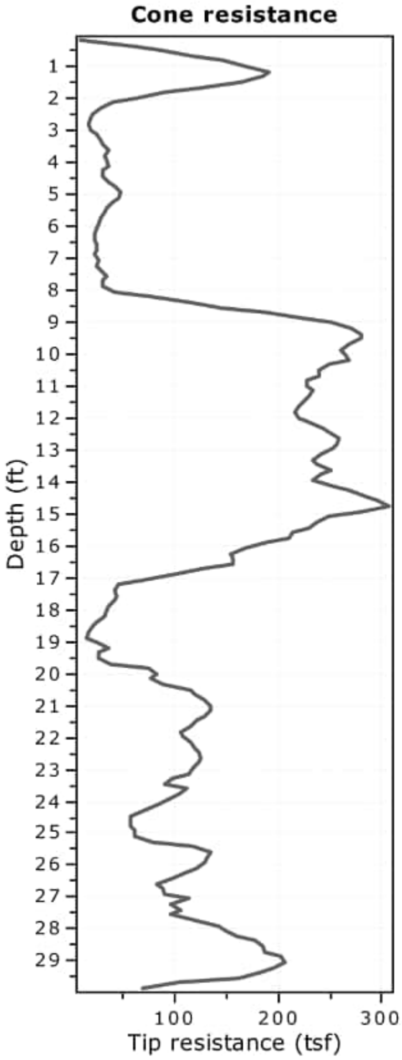


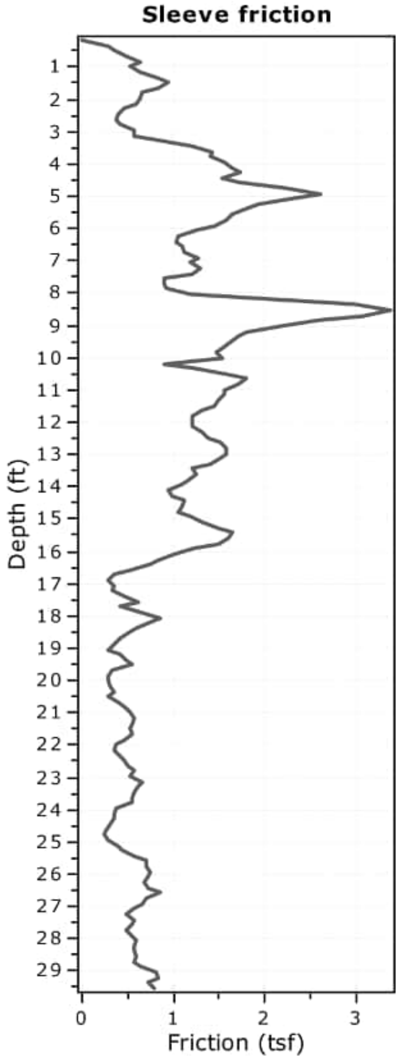
WSP USA
2000 Buisness Center Drive, Suite 235
Savannah, Georgia
www.wsp.com

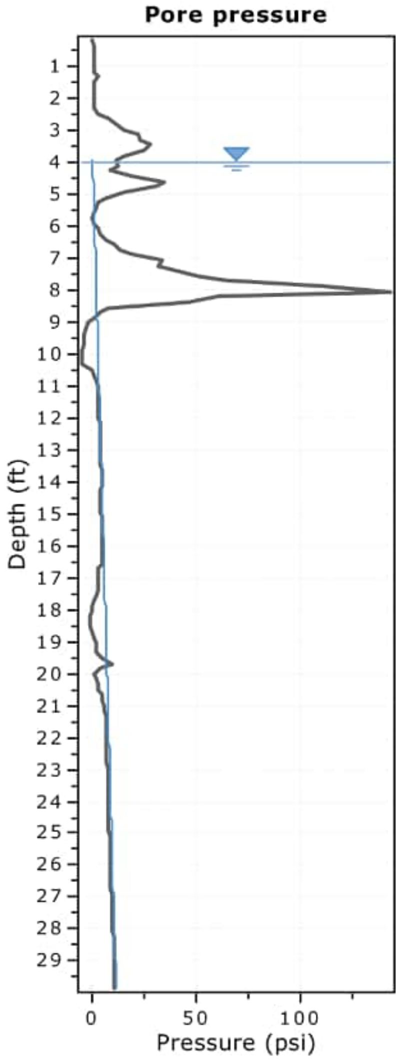
CPT-10

Total depth: 29.86 ft, Date: 9/8/2024
Surface Elevation: 98.00 ft
Cone Operator: BM

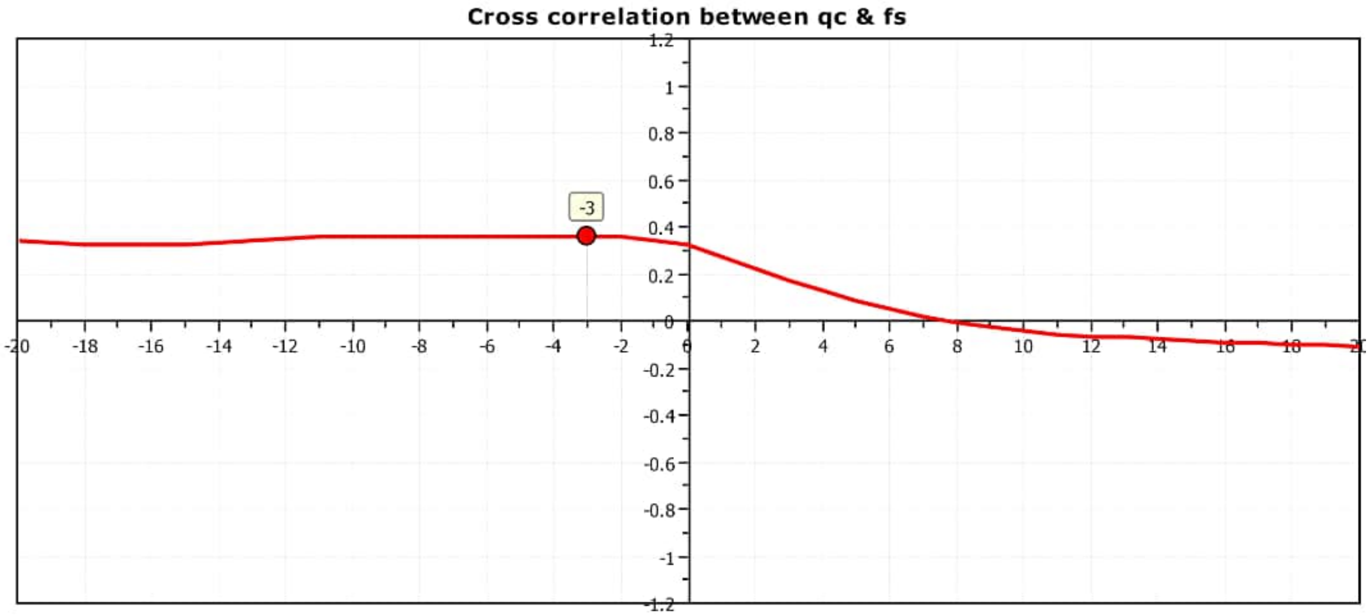
Project: Wastewater Treatment Plant
Location: Pembroke, Georgia







The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



WSP USA
2000 Buisness Center Drive, Suite 235
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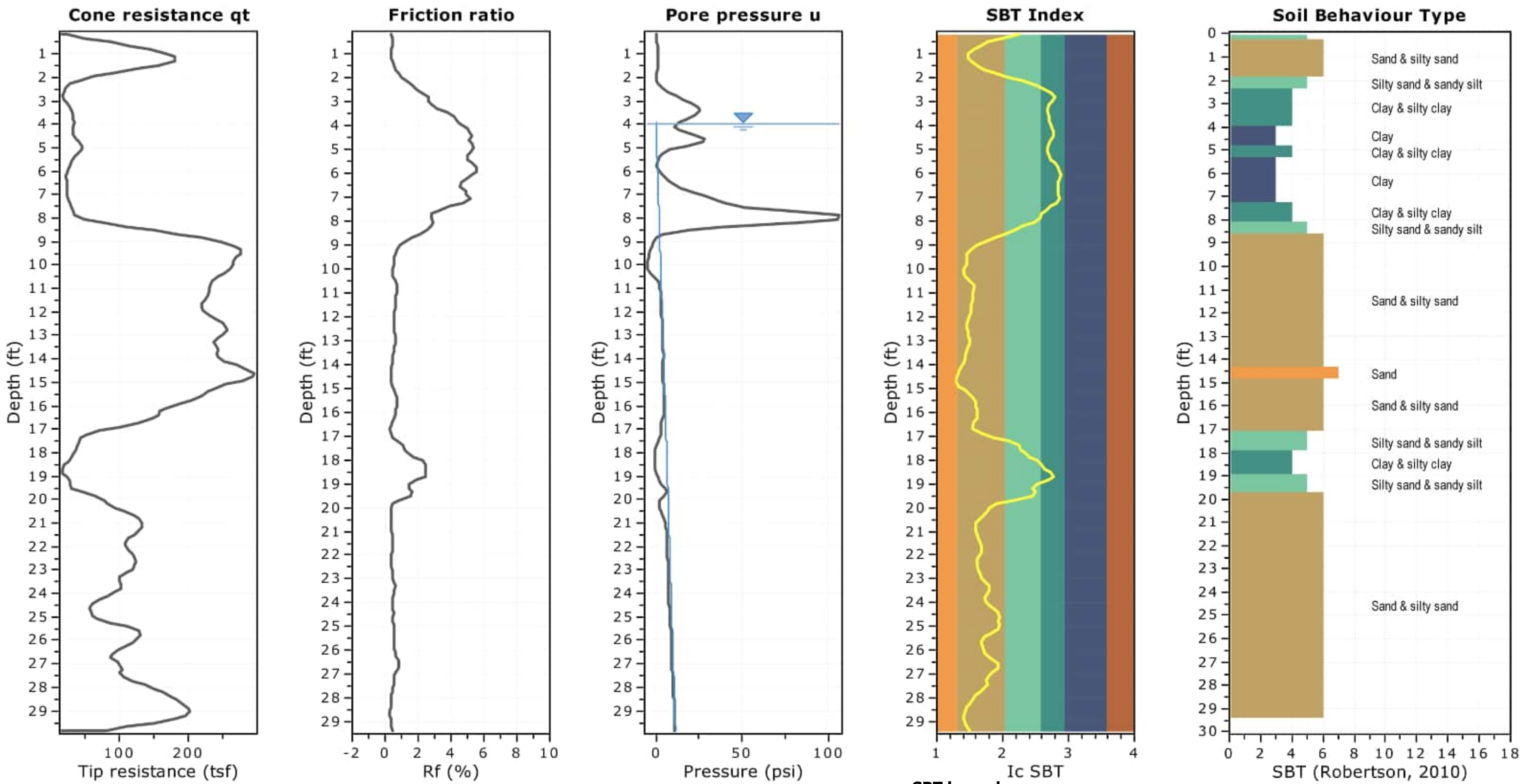
www.wsp.com

CPT-10
Total depth: 29.86 ft, Date: 9/8/2024
Surface Elevation: 98.00 ft
Cone Operator: BM

Project: **Wastewater Treatment Plant**
Location: **Pembroke, Georgia**



SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

WSP USA
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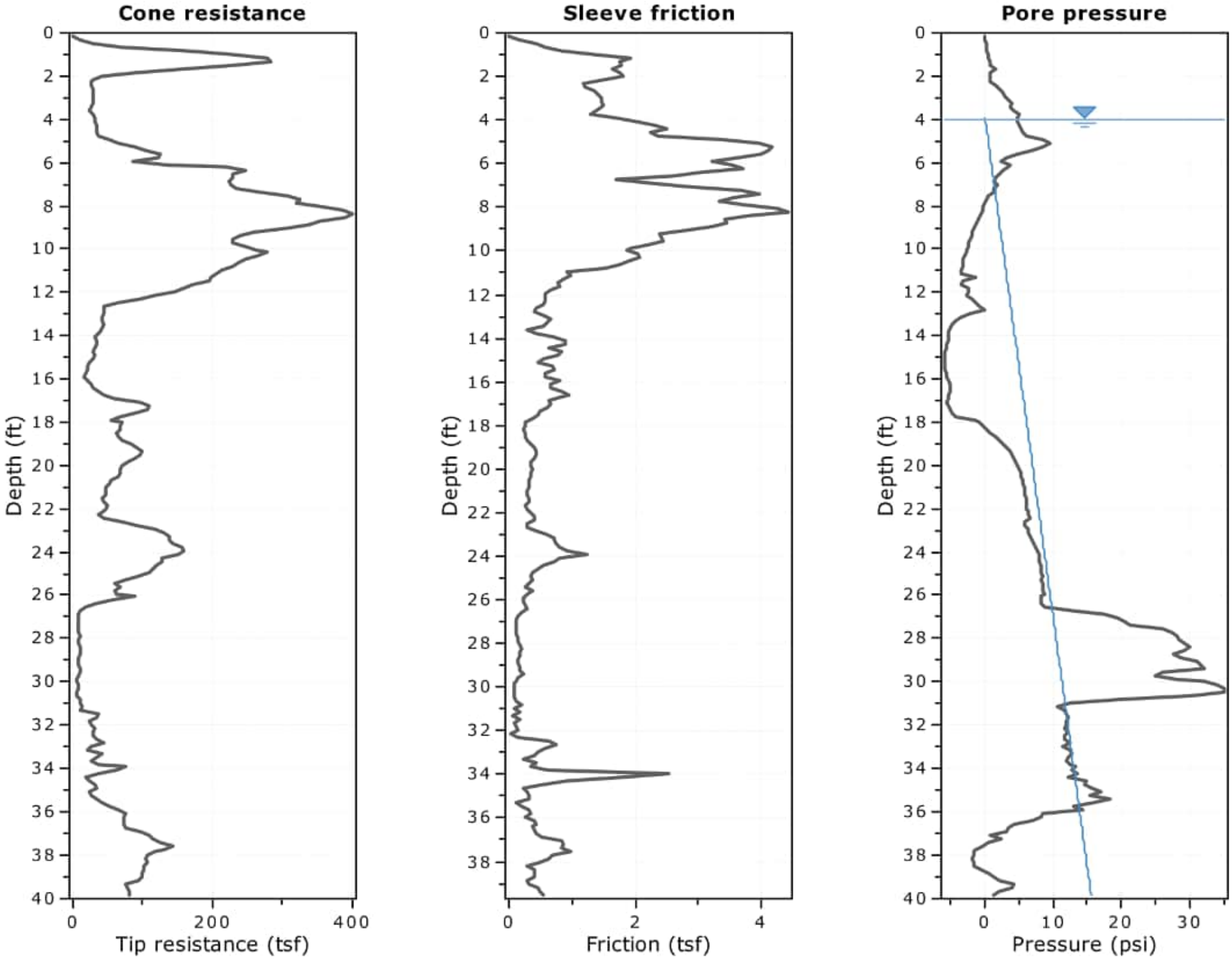
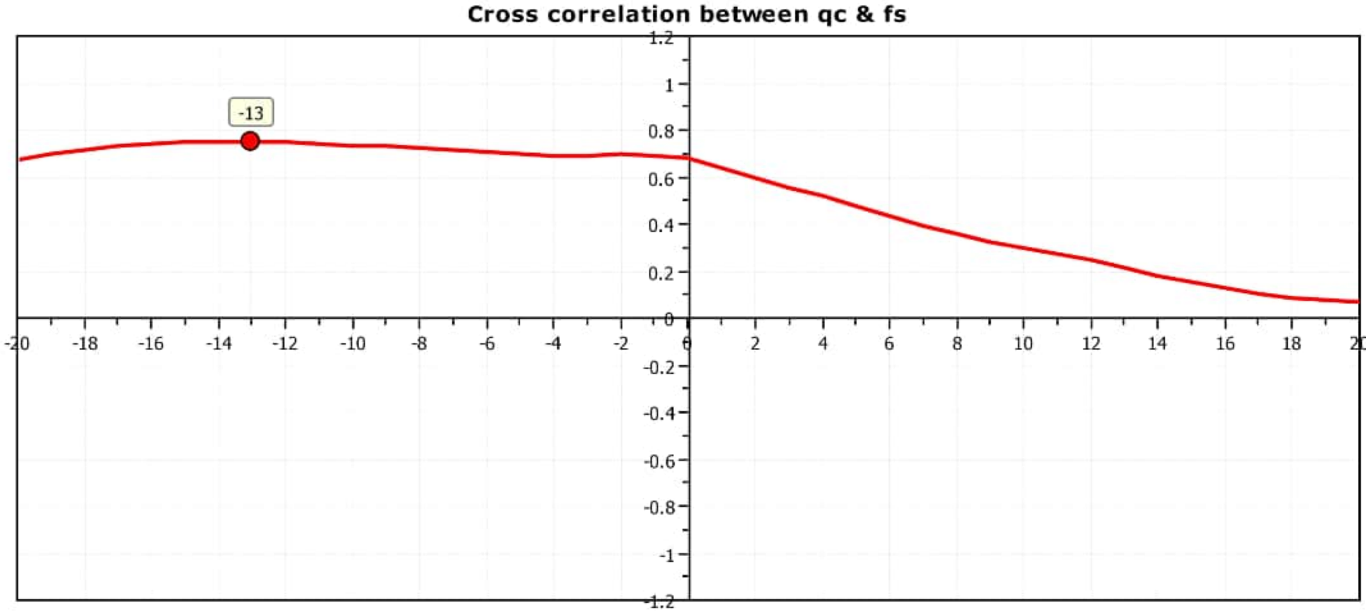
CPT-11
Total depth: 39.86 ft, Date: 9/8/2024
Surface Elevation: 98.00 ft
Cone Operator: BM

Project: **Wastewater Treatment Plant**
Location: **Pembroke, Georgia**



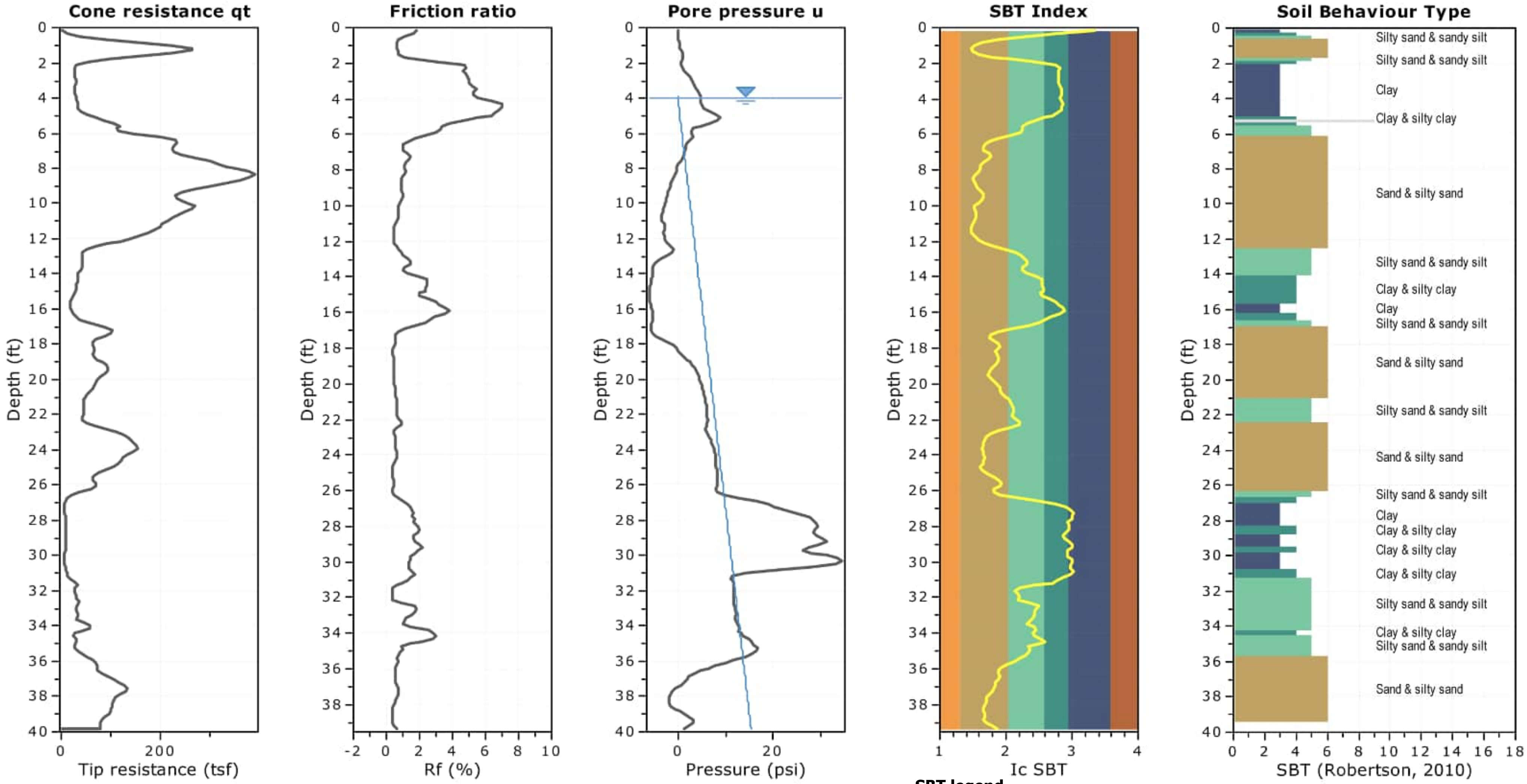
The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



WSP USA
2000 Buisness Center Drive, Suite 235
Savannah, Georgia
www.wsp.com

CPT-11
Total depth: 39.86 ft, Date: 9/8/2024
Surface Elevation: 98.00 ft
Cone Operator: BM

Project: **Wastewater Treatment Plant**
Location: **Pembroke, Georgia**



SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

DEPTH (ft)	SOIL CLASSIFICATION AND REMARKS SEE KEY SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS USED BELOW.	LEGEND	ELEV (ft)
0	**TOPSOIL LAYER:** 11"		105
	Gray to Orangish Brown Fine Clayey Sand (SC)		
	Reddish Orange Fine Silty Sand (SM)		
	Reddish Orange Fine Clayey Sand (SC)		
	Gray		
5	Boring was terminated at approximately 5 feet.		100
10			95

SOIL TEST BORING PEMBROKE WASTEWATER TREATMENT PLANT.GPJ GINT WSP STD.GDT 9/25/24

DRILLER:
EQUIPMENT:
METHOD: Hand Auger
HOLE DIA.: 4 inches

REMARKS: Groundwater not encountered.

PREPARED BY: GS REVIEWED BY: YH

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BEWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

SOIL TEST BORING RECORD

BORING NO.: HA-1
PROJECT: City of Pembroke Wastewater Treatment Expansion
LOCATION: Pembroke, Georgia
DRILLED: September 9, 2024
PROJECT NO.: US0039300.2246

wsp

DEPTH (ft)	SOIL CLASSIFICATION AND REMARKS SEE KEY SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS USED BELOW.	LEGEND	ELEV (ft)
0	**TOPSOIL LAYER:** 11"		98
	Gray Fine Silty Sand (SM)		
	Light Brown to Gray		
5			93
10			88

SOIL TEST BORING PEMBROKE WASTEWATER TREATMENT PLANT.GPJ GINT WSP STD.GDT 9/25/24

DRILLER:
EQUIPMENT:
METHOD: Hand Auger
HOLE DIA.: 4 inches

REMARKS: Groundwater not encountered.

PREPARED BY: GS REVIEWED BY: YH

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BEWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

SOIL TEST BORING RECORD

BORING NO.: HA-2
PROJECT: City of Pembroke Wastewater Treatment Expansion
LOCATION: Pembroke, Georgia
DRILLED: September 9, 2024
PROJECT NO.: US0039300.2246

wsp

DEPTH (ft)	SOIL CLASSIFICATION AND REMARKS SEE KEY SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS USED BELOW.	LEGEND	ELEV (ft)
0	**TOPSOIL LAYER:** 4"		98
	Orangish Brown to Gray Fine Clayey Sand (SC)		
5			93
10			88

DRILLER:
EQUIPMENT:
METHOD: Hand Auger
HOLE DIA.: 4 inches

REMARKS: Groundwater encountered at 4.8 feet.

PREPARED BY: GS REVIEWED BY: YH

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BEWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

SOIL TEST BORING RECORD

BORING NO.: HA-3
PROJECT: City of Pembroke Wastewater Treatment Expansion
LOCATION: Pembroke, Georgia
DRILLED: September 9, 2024
PROJECT NO.: US0039300.2246

wsp

DEPTH (ft)	SOIL CLASSIFICATION AND REMARKS SEE KEY SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS USED BELOW.	LEGEND	ELEV (ft)
0	**TOPSOIL LAYER:** 10"		105
	Orangish Brown to Gray Fine Clayey Sand (SC)		
	Gray Fine Silty Sand (SM)		
	Gray to Yellowish Brown Fine Silty Clayey Sand (SC-SM)		
5			100
10			95

SOIL TEST BORING PEMBROKE WASTEWATER TREATMENT PLANT.GPJ GINT WSP STD.GDT 9/25/24

DRILLER:
EQUIPMENT:
METHOD: Hand Auger
HOLE DIA.: 4 inches

REMARKS: Groundwater not encountered.

PREPARED BY: GS REVIEWED BY: YH

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BEWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

SOIL TEST BORING RECORD

BORING NO.: HA-4
PROJECT: City of Pembroke Wastewater Treatment Expansion
LOCATION: Pembroke, Georgia
DRILLED: September 9, 2024
PROJECT NO.: US0039300.2246

wsp

DEPTH (ft)	SOIL CLASSIFICATION AND REMARKS SEE KEY SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS USED BELOW.	LEGEND	ELEV (ft)
0	**TOPSOIL LAYER:** 11"		98
	Light Brown to Gray Fine Silty Sand (SM)		
	Gray to Yellowish Brown Fine Silty Clayey Sand (SC-SM)		
5			93
10			88

SOIL TEST BORING PEMBROKE WASTEWATER TREATMENT PLANT.GPJ GINT WSP STD.GDT 9/25/24

DRILLER:
EQUIPMENT:
METHOD: Hand Auger
HOLE DIA.: 4 inches

REMARKS: Groundwater encountered at 0.3 feet.

PREPARED BY: GS REVIEWED BY: YH

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BEWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

SOIL TEST BORING RECORD

BORING NO.: HA-5
PROJECT: City of Pembroke Wastewater Treatment Expansion
LOCATION: Pembroke, Georgia
DRILLED: September 9, 2024
PROJECT NO.: US0039300.2246

wsp

DEPTH (ft)	SOIL CLASSIFICATION AND REMARKS SEE KEY SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS USED BELOW.	LEGEND	ELEV (ft)
0	Gray to Yellowish Brown Fine Clayey Sand (SC)		97
5			92
10			87

SOIL TEST BORING PEMBROKE WASTEWATER TREATMENT PLANT.GPJ GINT WSP STD.GDT 9/25/24

DRILLER:
EQUIPMENT:
METHOD: Hand Auger
HOLE DIA.: 4 inches

REMARKS: No topsoil. Groundwater encountered at 2.7 feet.

PREPARED BY:__GS__ REVIEWED BY:__YH__

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BEWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

SOIL TEST BORING RECORD

BORING NO.: HA-6
PROJECT: City of Pembroke Wastewater Treatment Expansion
LOCATION: Pembroke, Georgia
DRILLED: September 9, 2024
PROJECT NO.: US0039300.2246

wsp

DEPTH (ft)	SOIL CLASSIFICATION AND REMARKS SEE KEY SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS USED BELOW.	LEGEND	ELEV (ft)
0	**TOPSOIL LAYER:** 10"		100
	Brown Fine Silty Sand (SM)		
	Gray to Orangish Brown Fine Silty Clayey Sand (SC-SM)		
5			95
10			90

SOIL TEST BORING PEMBROKE WASTEWATER TREATMENT PLANT.GPJ GINT WSP STD.GDT 9/25/24

DRILLER:
EQUIPMENT:
METHOD: Hand Auger
HOLE DIA.: 4 inches

REMARKS: Groundwater not encountered.

PREPARED BY: GS REVIEWED BY: YH

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BEWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

SOIL TEST BORING RECORD

BORING NO.: HA-7
PROJECT: City of Pembroke Wastewater Treatment Expansion
LOCATION: Pembroke, Georgia
DRILLED: September 9, 2024
PROJECT NO.: US0039300.2246

wsp

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of material is LARGER than No. 200 sieve size)	GRAVELS (More than 50% of coarse fraction is LARGER than the No. 4 sieve size)	CLEAN GRAVELS (Little or no fines)	GW	Well graded gravels, gravel - sand mixtures, little or no fines.
			GP	Poorly graded gravels or grave - sand mixtures, little or no fines.
		GRAVELS WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel - sand - silt mixtures.
			GC	Clayey gravels, gravel - sand - clay mixtures.
	SANDS (More than 50% of coarse fraction is SMALLER than the No. 4 Sieve Size)	CLEAN SANDS (Little or no fines)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand - silt mixtures
			SC	Clayey sands, sand - clay mixtures.
FINE GRAINED SOILS (More than 50% of material is SMALLER than No. 200 sieve size)	SILTS AND CLAYS (Liquid limit LESS than 50)		ML	Inorganic silts and very fine sands, rock flour, silty of clayey fine sands or clayey silts and with slight plasticity.
			CL	Inorganic lays 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 50)		MH	Inorganic silts, micaceous or diatomaceous 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.
FILL				Fill

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

SILT OR CLAY	SAND			GRAVEL		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

No.200 No.40 No.10 No.4 3/4" 3" 12"

U.S. STANDARD SIEVE SIZE

Reference: The Unified Soil Classification System, Corps of Engineers, U.S. Army Technical Memorandum No. 3-357, Vol. 1, March, 1953 (Revised April, 1960)

Undisturbed Sample	Auger Cuttings
Standard Penetration Test or Dynamic Cone Penetration Test	Bulk Sample
Rock Core	Crandall Sampler
Dilatometer	Pressure Meter
Packer	No Recovery
Water Table at time of boring	Water Table after 24 hours

Correlation of Standard Penetration Resistance with Relative Density and Consistency

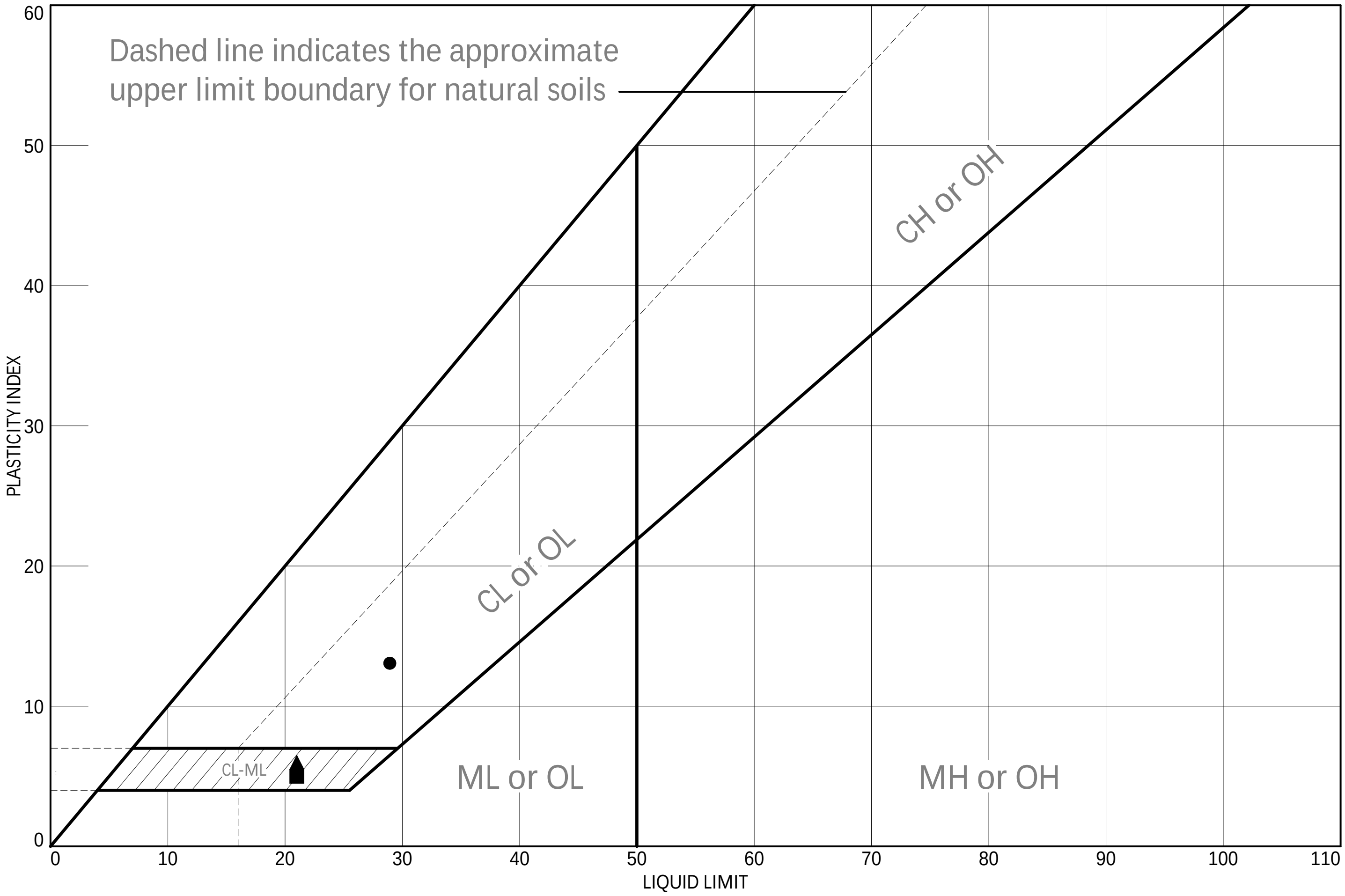
SAND & GRAVEL		SILT & CLAY	
No. of Blows	Relative Density	No. of Blows	Consistency
0 - 4	Very Loose	0 - 2	Very Soft
5 - 10	Loose	3 - 4	Soft
11 - 30	Medium Dense	5 - 8	Firm
31 - 50	Dense	9 - 15	Stiff
Over 50	Very Dense	16 - 30	Very Stiff
		31 - 50	Hard
		Over 50	Very Hard

Correlation of Dynamic Cone Penetration Resistance with Relative Density and Consistency (Piedmont Residual Soils)

SAND & GRAVEL		SILT & CLAY	
No. of Blows	Relative Density	No. of Blows	Consistency
0 - 4	Very Loose	0 - 2	Very Soft
5 - 15	Loose	3 - 4	Soft
16 - 30	Medium Dense	5 - 10	Firm
		11 - 30	Stiff

KEY TO SYMBOLS AND DESCRIPTIONS

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Orangish Brown to Gray Clayey Sand	29	16	13		33.9	SC
■	Gray to Yellowish Brown Silty Clayey Sand	21	16	5		26.4	SC-SM
▲	Gray Clayey Sand	21	15	6		30.7	SC-SM

Project No. US0039300.2246 Client: M.E. Sack Engineering

Project: City of Pembroke WWTP

● Location: HA-3 Depth: 0'-2' Sample Number: 358

■ Location: HA-4 Depth: 3.2'-5' Sample Number: 359

▲ Location: HA-1 Depth: 4'-5' Sample Number: 360

Remarks:

Figure

Tested By: Richard Matjazic Checked By: Jonathan Konkel

MOISTURE CONTENT

ASTM D2216-19

Project Name: City of Pembroke WWTP
Project No. US0039300.2246
Tested By: Richard Matjazic
Reviewed By: Jonathan Konkel
Test Date: 9/18/2024
Review Date: 9/20/2024

Hand Auger No.	Sample No.	Depth (Ft)	Lab No.	Tare No.	Tare Wt. (grams)	Wet Soil + Tare (grams)	Dry Soil + Tare (grams)	Dry Soil Wt (grams)	Moisture Content (%)	% Passing #200 (%)
HA-3	358	0'-2'	6162	R-1	154.64	655.79	583.42	428.78	16.9	33.9
HA-4	359	3.2'-5'	6162	R-2	153.32	653.85	585.53	432.21	15.8	26.4
HA-1	360	4'-5'	6162	P-4	154.15	456.12	417.33	263.18	14.7	30.7
HA-7	361	1'-3'	6162	R-3	246.45	749.31	700.01	453.56	10.9	N/A
HA-6	362	0'-5'	6162	P-8	153.71	661.23	591.46	437.75	15.9	N/A

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

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