Report of Geotechnical Exploration
Georgetown Wet Well
Georgetown, South Carolina
S&ME Project No. 213382

PREPARED FOR:

The Wooten Company
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PREPARED BY:

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May 11, 2021

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The Wooten Company
1830 Marion Street, Suite A
Columbia, South Carolina 29201

Attention: Aaron Marshall

Reference: **Report of Geotechnical Exploration**
Georgetown Wet Well
Georgetown, South Carolina
S&ME Project No. 213382

Dear Mr. Marshall:

S&ME, Inc. has completed the subsurface exploration for the referenced project after receiving signed authorization to proceed from you on April 15, 2021. Our exploration was conducted in general accordance with our Proposal No. 213382, dated April 5, 2021.

The purpose of this study was to characterize the surface and subsurface soils on the proposed site, and to provide recommendations for site preparation, earthwork, and foundation support for the proposed structures.

This report describes our understanding of the project, presents the results of the field exploration, laboratory testing, and engineering analysis and discusses our conclusions and recommendations based on these considerations. S&ME, Inc. appreciates this opportunity to be of service to you. Please call if you have questions concerning this report or any of our services.

Respectfully submitted,

S&ME, Inc.

SOUTH CAROLINA CERTIFICATE OF AUTHORIZATION S&ME, INC. No. C00473

Kara Fugate, E.I.T.
Geotechnical Staff Professional

SOUTH CAROLINA REGISTERED PROFESSIONAL ENGINEER No. 21248 RON FOREST JR. 5/11/21

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♦ Introduction

The purpose of this exploration was to obtain subsurface information to allow us to characterize the subsurface conditions at the site and to develop recommendations concerning grading, foundation design, and other related construction issues. This report describes our understanding of the project, presents the results of the field exploration and laboratory testing, and discusses our conclusions and recommendations.

A site plan showing the approximate test locations is included in Appendix I. The boring logs, a discussion of the field exploration procedures, and a legend to soil classification and symbols in included in Appendix II.

Project Information

Project information was provided in a telephone conversation between Mr. Aaron Marshall (The Wooten Company) and Ron Forest, Jr. (S&ME) on April 1, 2021. Additional information was provided by Mr. Marshall in a follow-up email on April 1, 2021. Attached to the email was an undated and untitled sheet drawing by The Wooten Company.

The drawing provided by Mr. Marshall in an email showed the layout, plan and sections of a proposed wet well pump to be installed on a lot located on Asbury Street in Georgetown, South Carolina (33°20'26.3"N 79°17'27.7"W), directly between the existing fenced-in pump station and the bay. A site vicinity map is included as Figure 1 in Appendix I.

Based on the information provided we understand that the new wet well will have a 12-ft diameter foundation with a 6 inch or 1 foot extension footer placed approximately 18 feet below the existing ground surface. We understand that this new well is needed because the existing wet well is not sufficient. The old wet well will remain in place and pass contents to the new well via pipes.

♦ Exploration Program

Field Exploration

On April 27, 2021 and April 30, 2021, representatives of S&ME, Inc. visited the site. Using the information provided, we performed the following tasks.

1. We performed a site walkover, observing general features of topography, existing structures, ground cover, and surface soils at the project site.
2. We established the location of one (1) discrete test (B-1) at which a standard penetration test boring was advanced. The approximate test location is shown on the test location sketch included in Figure 2 of Appendix I. The boring was advanced to approximately 40 feet below the surface.
3. Drilling was conducted using the mud rotary method and at standard spacing samples were collected using the split spoon method.
4. Groundwater levels were measured after a period of 24 hours had passed from drilling the boring. The boring was then backfilled to the original ground surface with soil cuttings.

A brief description of the field exploration procedures performed, as well as a soil classification legend, and the boring logs are attached in Appendix II.

♦ Site and Surface Conditions

The area we explored was grassed, and located just outside of the enclosure for the existing wet well system. There is a pile of gravel toward the site entrance and trees surrounding the site measuring over 30 feet tall. Ponded water was not observed on the site surface at the time of our exploration.

Topography

It was not within the scope of work to conduct surveying on site and we were not provided with a topographic survey of the site. For the purposes of our boring logs and interpreted subsurface soil profile the ground surface elevation has been set to 6.0 feet as interpreted from the schematic provided by the client.

Local Geology

The site is located in the Coastal Plain Physiographic Region of South Carolina. This area is dominated topographically by a series of relic beach terraces, which progressively increase in surface altitude as they proceed inland. These terraces have been extensively mapped and correlated over wide areas. The terraces have been exposed by uplifting of the local area over the last 250,000 years. Since the terraces are relatively young features, they exhibit only minor surface erosion and can be traced large distances on the basis of surface elevation. The soil profile typically consists of a thin veneer of terrace deposited sediments of the Socastee Formation. Beneath this upper, relatively young formation, geologic mapping and soils encountered within borings indicate the Penholoway Formation, which is characterized by sands, silts and clays often with shell fragments and cemented materials, laid down during the Lower Pleistocene Epoch approximately 760,000 years ago.

Below the Penholoway Formation, soils are mapped as sands and silts of the Black Mingo Formation. These are Paleocene-age (early Tertiary) materials that were laid down approximately 55 to 65 million years ago. Our boring did not penetrate the Black Mingo Formation.

♦ Subsurface Conditions

The generalized subsurface conditions at the site are described below. For more detailed descriptions and stratifications at the test location, the respective boring log should be reviewed in Appendix II.

Interpreted Subsurface Profile

An interpreted subsurface cross-sectional profile of the site soils is attached as Figure 3 in Appendix I to illustrate a general representation of the subsurface conditions within the proposed construction adjacent to the proposed wet well.

The strata indicated in the profile are characterized in the following section. Note that the profile is not to scale and was prepared for illustrative purposes only. Subsurface stratifications may be more gradual than indicated, and conditions may vary between test locations.

Soils presented on the profile were grouped into several general strata and substrata based on estimated physical properties derived from the borings and the recovered samples. The strata encountered are labeled I through IV on the soil profile to allow their properties to be systematically described.

Description of Subsurface Soils

This section describes subsurface soil conditions observed at the site.

Topsoil

At the ground surface, our boring encountered approximately 3 inches topsoil. Topsoil thickness may vary across the site.

Stratum I: Upper Clayey Sands

Underlying the existing topsoil, approximately 3 feet of clayey sand (USCS Classification "SC") was encountered. This sand was orange and grey and wet upon recovery. The SPT N-values within the clayey sand was measured to be 4 blows per foot (bpf). This indicates a very loose relative density.

Stratum II: Intermediate Sands

Beneath the upper clayey sands of Stratum I, a zone of loose, saturated, poorly graded sand with silt (SP-SM) was encountered to a depth of approximately 17 feet below the surface. The SPT N-value within these soils ranged from 6 to 8 bpf, indicating a loose relative density. Upon recovery these soils were grey and wet.

Stratum III: Lower Sands

Beneath the poorly graded sands with silt of Stratum II, the boring encountered a poorly graded sand (SP) to a depth of approximately 24 feet below the surface. These soils were observed to be wet and dark grey. An SPT N-value of 4 bpf was measured, indicating a very loose relative density within the stratum.

Stratum IV: Cemented Cohesive Soils

Beneath the poorly graded sands of Stratum III, a stratum of cemented sandy silts (ML) was encountered to the maximum exploration depth of 40 feet below the surface. Strongly cemented zones were first encountered at a depth of approximately 24 feet below the surface. The SPT N-values in this cemented soil zone ranged from 41 bpf to greater than 100 bpf, indicating a hard to very hard consistency within the silts. The soils within this stratum were shades of gray, and were saturated upon recovery. These silts were also strongly reactive to dilute hydrochloric acid when tested in the laboratory. This strength of reactivity indicates significant quantities of calcium-carbonate marine materials resulting in the natural cementation of particles.

Groundwater

At the time of drilling the encountered water level was measured at a depth of 2 feet. After a period of approximately 24-hours the water level was measured to be 3 feet below the surface and the boring caved at a depth of 5 ½ feet below the surface. Groundwater levels can fluctuate seasonally and can be influenced by site construction activities. Groundwater levels may fluctuate seasonally at the site, being influenced by rainfall variation and other factors. Site construction activities can also influence groundwater elevations.

♦ Seismic Site Class and Design Parameters

As of January 1, 2020, the 2018 edition of the International Building Code (IBC) has been adopted for use in South Carolina. We classified the site as one of the Site Classes listed in IBC Section 1613.3, using the procedures described in Chapter 20 of ASCE 7-16.

Evaluation of the Potential for Site Class F Conditions

Seismic-induced ground shaking at the foundation is the effect taken into account by seismic-resistant design provisions of the International Building Code (IBC). Other effects, including landslides and soil liquefaction, must also be considered.

The initial step in site class definition is to check for the four conditions described for Site Class F, which would require a site specific evaluation to determine site coefficients F_A and F_V. Soils vulnerable to potential failure include the following: 1) quick and highly sensitive clays or collapsible weakly cemented soils, 2) peats and highly organic clays, 3) very high plasticity clays, and 4) very thick soft/medium stiff clays. These soils were not evident in the borings.

One other determining characteristic, liquefaction potential under seismic conditions, was assessed. Soils were assessed qualitatively for liquefaction susceptibility based on their age, stratum, mode of deposition, degree of cementation, and size composition. This assessment considered observed liquefaction behavior in various soils in areas of previous seismic activity.

Our analysis, which is more fully described below, indicates that some liquefaction of subsoils appears likely to occur at this site in the event of the design magnitude earthquake. Testing indicates that some of the sands between depths of about 3 feet to 24 feet lie beneath the water table, appear to contain relatively few fines, and exhibit relatively low density characteristics.

- The wet well is proposed to bear at a depth of 18 feet below the surface, which means that if liquefaction of the soil between depths of 18 feet and 24 feet below the surface occurs during seismic shaking, it could result in the loss of direct bearing support for the well foundation (e.g. an effective soil bearing capacity of zero).

We consider the soil conditions within this site to be liquefaction prone; and therefore, seismic Site Class F conditions apply to this site.

Selection of Seismic Site Class

Based upon the weighted SPT N-values, this site would typically be categorized as Site Class D if the liquefaction potential in the subsoils were not significant. This recommendation is provided based on the recorded N-values, which average greater than 15 bpf when measured to a depth of 40 feet and extrapolated to a depth of 100 feet. However, Site Class D cannot be used for design if Site Class F conditions apply *unless* the structure meets the requirements of the exception described in ASCE 7-16 Chapter 20.3.1(1), which requires that the fundamental period of vibration of the structure be less than 0.5 seconds. Most subterranean structures typically do have a fundamental period of vibration of less than 0.5 seconds and would therefore qualify under this exception. This can be confirmed by the design structural engineer. Our analysis of the potential for Site Class F conditions is further discussed below. .

Liquefaction of Bearing Soils

Liquefaction of saturated, loose, cohesionless soils occurs when they are subjected to earthquake loading that causes the pore pressures to increase and the effective overburden stresses to decrease, to the point where large soil deformation or even transformation from a solid to a liquid state results. Earthquake-induced ground surface acceleration at the site was assumed from the building code design peak ground acceleration of 0.36g.

Liquefaction Potential Index (LPI)

To evaluate liquefaction potential, we performed analyses using the data obtained in the borings, considering the characteristics of the soil and water levels observed in the boring. The liquefaction analysis was performed based on the design earthquake prescribed by the 2015 edition of the International Building Code, the "simplified procedure" as presented in Youd et al. (2001), and recent research concerning the liquefaction resistance of aged sands (Hayati & Andrus, 2008; Andrus et al. 2009; Hayati & Andrus, 2009).

To help evaluate the consequences of liquefaction, we have computed the Liquefaction Potential Index (LPI), which is an empirical tool used to evaluate the potential for liquefaction to cause damage. The LPI considers the factor of safety against liquefaction, the depth to the liquefiable soils, and the thickness of the liquefiable soils to compute an index that ranges from 0 to 100. An LPI of 0 means there is no risk of liquefaction; an LPI of 100 means the entire profile is expected to liquefy. The level of risk is generally defined below.

- ***LPI < 5*** – surface manifestation and liquefaction-induced damage not expected.
- ***5 ≤ LPI ≤ 15*** – moderate liquefaction with some surface manifestation possible.
- ***LPI > 15*** – severe liquefaction and foundation damage is likely.

The average LPI for this site was calculated to be greater than 15, which indicates a severe risk of damage due to liquefaction, and indicates that foundation damage is likely to occur during the ground shaking associated with the design magnitude earthquake. Therefore, Site Class F applies to this site.

Seismic Design Coefficients for Site Class D

Selection of the base shear values for structural design for earthquake loading is the responsibility of the structural engineer. However, for the purpose of evaluating seismic hazards at this site, S&ME has evaluated the spectral response parameters for the site using the general procedures outlined under the 2018 International Building Code Section 1613.3. This approach utilizes a mapped acceleration response spectrum reflecting a targeted risk of structural collapse equal to 1 percent in 50 years to determine the spectral response acceleration at the top of seismic bedrock for any period. The 2018 IBC seismic provisions of Section 1613 use the Seismic Hazard Maps published by the National Earthquake Hazard Reduction Program (NEHRP) to define the base rock motion spectra.

The Site Class is used in conjunction with mapped spectral accelerations S_S and S_1 to determine Site Amplification Coefficients F_A and F_V from tables 11.4-1 and 11.4-2 in section 11.4.7 of ASCE 7-16. For purposes of computation, the Code includes probabilistic mapped acceleration parameters at periods of 0.2 seconds (S_S) and 1.0 seconds (S_1), which are then used to derive the remainder of the response spectra at all other periods. The mapped S_S and S_1 values represent motion at the top of seismic bedrock, defined as the Site Class B-C boundary. The surface ground motion response spectrum, accounting for inertial effects within the soil column overlying rock, is then determined for the design earthquake using spectral coefficients F_A and F_V for the appropriate Site Class.

The design ground motion at any period is taken as 2/3 of the smoothed spectral acceleration as allowed in section 1613.3.4. The design spectral response acceleration values at short periods, S_{DS}, and at one second periods, S_{D1}, are tabulated below for the unimproved soil profile using the IBC 2018 criteria.

The 2018 IBC specifically references ASCE 7-16 for determination of peak ground acceleration value for computation of seismic hazard. Peak ground acceleration is separately mapped in ASCE 7-16 and corresponds to the geometric mean Maximum Credible Earthquake (MCE_G). The mapped PGA value is adjusted for site class effects to arrive at a design peak ground acceleration value, designated as PGA_M. The seismic design coefficients for this site based on our findings are shown in Table 1 below.

Table 1 – Seismic Design Coefficients

Criteria	Seismic Site Class	S_S	S_1	S_{DS}	S_{D1}	PGA_M	Seismic Design Category
2018 IBC	F*	0.49	0.16	0.46	0.24	0.36	D

* Use Site Class D based on the "exception" listed in ASCE 7-16, Section 20.3.1 (1.)

Seismic Design Category

For a structure having a Risk Category classification of I, II, or III, the S_{DS} and S_{D1} values obtained are consistent with "Seismic Design Category D" as defined in section 1613.3.5 of the IBC.

♦ Conclusions and Recommendations

The conclusions and recommendations included in this section are based on the project information outlined previously and the data obtained during our exploration. If the construction scope is altered, the locations of the structures changed, or if conditions are encountered during construction that differ from those encountered in the borings, then S&ME, Inc. should be retained to review the following recommendations based upon the new information and make any necessary changes.

Based upon the results of our exploration and our past experience with similar soils in the site vicinity, the site appears generally adaptable for the proposed development. Based on the assumed loading and settlement tolerances, it appears that shallow foundation support is feasible for this area at a depth of 18 feet if the wet well is considered a sacrificial structure in the event of an earthquake. Due to the potential total loss of bearing capacity of the soils beneath the well footing and the down drag of the soils that will potentially liquefy above the bearing zone, the wet well is likely to experience an excessive amount of settlement due to liquefaction in the event of the design magnitude earthquake.

Seismic Considerations

We have estimated that up to about 5 inches of total settlement under the proposed bottom of wet well, and up to 9.5 inches of total settlement from the surface may occur in the event of the design magnitude earthquake. Based on discussions at the time of our proposal we understand that the structure may not fall within the seismic design requirements of the Building Code. This is likely a Risk Category I, II, or III (non-essential facilities) structure, so the 2018 IBC requires that the design account for (or mitigate) the effects of liquefaction in order to prevent structural collapse and the potential for loss of life.

If the new wet well is not considered a sacrificial structure in the event of an earthquake, then densification of the soils using a vibroprobe may be considered as one possible means of compacting the soils beneath the proposed footing at a depth of 18 feet to mitigate the liquefaction risk. However, using a vibroprobe this close to the existing wet well could potentially damage the existing well, so those risks would need to be further considered and evaluated.

Surface Preparation

The following recommendations are provided regarding site preparation and earthwork:

The site should be stripped of organic topsoil. After the stripping operation is complete, densify the bearing surface with the largest available roller that will fit in the space. Following densification, the densified subgrade surface should be proofrolled by the contractor under the observation of the Geotechnical Engineer (S&ME) by making repeated passes with a fully-loaded dump truck or equipment with similar weight and tire pressures. The proofrolling should be conducted only during dry weather. Areas of rutting or pumping soils indicated by the proofroll may require selective undercutting or further stabilization prior to any new fill placement or slab construction, as determined by the Geotechnical Engineer. This should be done prior to construction of the wet well.

Excavation Considerations

1. Subsurface water was observed and estimated to be approximately 3 feet below the existing ground surface at the time of our exploration. Where subsurface water is encountered during excavation, the water level should be maintained at least 2 to 3 feet below excavation bottoms to help maintain bottom stability.

   A. Soils at depths of about 18 feet, at the likely excavation bottom, consist of saturated, low fines content sands. Since these soils are relatively cohesionless, and are located about 15 feet below the estimated subsurface water level, it is likely that these soils would experience hydraulic uplift failure (heave) in the bottom of the excavation unless the water table is temporarily lowered by some means.
   B. If interlocked sheet piles are used to brace the excavation and are socketed into the hard sandy silts encountered between depths of about 24 and 40 feet, and the exterior water sources are completely cut off from the excavation, then it may be feasible to construct the excavation without an exterior

dewatering system. In such an approach, one possible method of temporarily drawing down the water level during construction may be to excavate a temporary sump hole within the sheet pile enclosed area, and use sump pumping to maintain the water level inside the excavation at least 2 to 3 feet below the bottom of the excavation to help reduce hydraulic uplift. This may require the installation of several "whistles", or slotted pipes embedded in gravel below grade.

C. If the water volume is too great for sumps or whistles to control, or if the pile interlocks are not sufficiently water tight, it may become necessary to treat the soil zone around the sheetpiles in an attempt to prevent water from infiltrating the excavation. Injection of polyurethane into the ground is one method that has been used in the past to seal the soil around sheetpiled excavations. Another possibility may be to to install a vacuum-assisted pumping system, also known as vacuum wellpoints. It was beyond the scope of this analysis to analyze the effectiveness of or to perform the design of specific dewatering systems.

D. Managing the effects of dewatering on nearby structures should be evaluated and are the responsibility of the designers and constructors of any dewatering system. There have been past events within Georgetown County where dewatering has jeopardized the integrity of adjacent structures due to generalized water table drawdown resulting in the collapse of subsurface voids in the soil profile.

2. All excavations should be sloped or shored in accordance with local, state, and federal regulations, including OSHA (29 CFR Part 1926) excavation trench safety standards for Type C soils. The contractor is solely responsible for site safety. This information is provided only as a service, and under no circumstances should S&ME be assumed to be responsible for construction site or excavation safety.
3. All of the sandy soils encountered within our boring below a depth of 3 feet are subject to collapse due to saturated conditions below the water table, exhibit little to no apparent cohesion, and relatively low SPT penetration resistance N-values indicating loose to very loose density materials. Excavations will have to be temporarily stabilized (braced or shored) in order to remain open, because these soils are not expected to hold an open cut face on their own. These upper sandy soils are also estimated not to be competent end-bearing materials for sheet piling. Sheet piles would need to toe into the cemented silty sands of Stratum IV.
4. Based on the provided drawing, we understand that at least 1 foot of washed, clean, crushed gravel, such as SCDOT No. 57 stone may provide a stable working surface upon which to compact backfill. More than 1 foot of gravel may be needed if the excavation bottom is particularly soft or unstable, or if the groundwater is not adequately controlled.
5. Design of the specific excavation bracing systems was beyond the scope of our work. Additional driving depth or internal bracing may be needed to provide global stability of the excavation bracing system, which was not analyzed as part of this work.

Uplift Resistance and Anchorage

Based on previous projects of similar nature we have assumed that the precast wet well will likely have an inside diameter of 10.67 ft with minimum 8 in.-thick walls and a base that is a minimum of approximately 14 ft x 14 ft x 0.5 ft. Considering these preliminary wet well dimensions and an assumed groundwater depth of approximately 3 ft based on the exploration, we conclude that the weight of the wet well structure is sufficient to resist the estimated buoyancy without the need for additional foundation anchors. If the wet well geometry and dimensions change from these assumptions, or if the design groundwater level is shallower than 3 ft, we should be given the opportunity to confirm that this conclusion is still applicable based on final wet well dimensions.

Excavation Bracing Design Parameters

The following sections provide our geotechnical recommendations for temporary sheet pile design parameters and other earth retaining structures, including soil strength parameters for use during design of earth-retaining systems. Based on the subsurface conditions at the site and the anticipated excavation depth, we assume interlocking steel sheet piles, possibly coupled with horizontal anchors or internal bracing, may be used during the performance of the work. The following sections provide our geotechnical recommendations for use in designing these systems.

Soil Strength Parameters

The recommended soil parameters for use in designing piles are presented in Table 2 below. Due to the sandy, cohesionless characteristics of these soils, the “undrained” or short-term conditions and “drained” or long-term soil strength parameters are similar, although the unit weight of the material inside the excavation may vary as the water table is drawn down. The values presented in Table 2 are based on the SPT borings performed during our geotechnical exploration, and our experience with the soils in this area.

Table 2 – Soil Strength Parameters

Stratum	Depth (feet)	USCS Soil Classification	Effective Soil Internal Friction Angle (deg.)	Effective Cohesion (psf)	Moist Unit Weight (pcf)	Buoyant Unit Weight (pcf)	Soil/Steel Wall Interface Friction Angle (deg.)
I	0 – 3	SC	26	0	115	53	14
II	3 – 17	SP-SM	28	0	115	53	17
III	17 – 24	SP	25	0	115	53	17
IV	24 – 40	ML	28	1,100	120	58	11

Earth Pressure Coefficients

Earth-retaining structures must be capable of resisting any lateral earth pressures that will be imposed on them. If the walls are relatively rigid and structurally braced against rotation, they should be designed for a condition approaching the "at-rest" lateral earth pressure.

In the event that the wall is free to deflect, such as for walls that are not restrained or rigidly braced, the "active" earth pressure conditions would be applicable for design. Cantilevered retaining walls or sheet piles are normally designed to yield (rotate outward) under the influence of this pressure, which is termed the “active” case.

The lateral earth pressure coefficients in Table 3 are recommended for use during the design of earth-retaining systems at this site.

Table 3 – Static and Seismic Lateral Earth Pressure Coefficients

Stratum	Depth (feet)	USCS Soil Classification	Static Lateral Earth Pressure Coefficients			Seismic Lateral Earth Pressure Coefficients (PGA = 0.36g)		
			At-Rest Coefficient (K_o)	Active Coefficient (K_a)	Passive Coefficient (K_p)	At-Rest Coefficient (K_o)	Active Coefficient (K_a)	Passive Coefficient (K_p)
I	0 – 3	SC	0.56	0.39	2.56	0.78	0.52	2.26
II	3 – 17	SP-SM	0.53	0.36	2.77	0.73	0.48	2.46
III	17 – 24	SP	0.58	0.41	2.46	0.81	0.54	2.17
IV	24 – 40	Cemented ML	0.36	0.22	4.60	0.47	0.31	4.21

The given earth pressure coefficients (K_o, K_a, and K_p) in Table 3 assume a level backfill and a frictionless wall. In Table 2, we provided soil/steel wall interface friction values which may be used in conjunction with the earth pressure coefficients presented in Table 3 to evaluate soil-on-steel interface friction. We note that at the time of our boring, the water level was measured approximately 3 feet below the existing ground surface. However, this water level can fluctuate with construction in the area and with the adjacent bay levels and drawdown of the water table that is performed during construction.

Water Effects on Earth Pressures

Assuming a dewatering system is **not** used outside the excavation, the temporary excavation bracing system (sheet pile) walls should be designed to withstand hydrostatic pressure outside the wall. Additionally, the walls should be designed to support any applied surcharge or structural loads. Lateral earth pressures arising from surcharge loading (including construction equipment) and slopes above the walls should be added to the earth pressures given above in Table 3 to determine the total lateral pressure.

Preliminary Sheet Pile Design Considerations

The design engineer will need to check bending moments in the sheet piles against allowable values for the given sheet pile configuration chosen. The actual embedment depth should be determined by the wall designer after all variables are known. If the wall system is designed with internal bracing or external soil anchors, then wall deflections, moments, and the required penetration depths may vary from these assumptions.

Pile section size should be determined based on driving conditions. Typically, a penetration depth of 1.5 times the exposed wall portion is required to provide suitable resistance for a cantilever type system; however, internal bracing or an external tie-back system may need to be installed to sufficiently brace the excavation.

The designer and builder should also recognize that due to the very hard condition of the Stratum IV soils, the depth of penetration that can be achieved into the very hard silts may be somewhat limited, particularly if lightweight or vibratory driving equipment is used to install the sheet sections.

Again, this information is intended to be preliminary, and is only intended to be used for planning purposes and to provide some initial guidance regarding the geotechnical parameters that may need to be considered by the wall designer during wall design. Design of the excavation bracing system was not in our scope of services.

Construction Considerations

Sandy soils such as those penetrated by our borings may exhibit "moderate" to "high" infiltration rates (in the range of 2 to 7 inches per hour) and the flow of subsurface water into excavations may be relatively quick. Therefore, the excavation may require the use of sumps with pumps inside the perimeter of the excavation and/or other more aggressive methods such as a vacuum well point system at regularly spaced intervals to control water levels. The water level should be maintained at least 2 to 3 feet below the bottom of the excavation to mitigate bottom instability (heave). Water levels should be expected to fluctuate, and the possibility of groundwater fluctuations should be considered when developing the design and construction of the excavation support system.

Caution should be taken to avoid lowering the water level beneath any adjacent structures because this could result in new settlement that could lead to damage of these structures. The effects of dewatering on nearby structures should be addressed by the designer of the dewatering system, if applicable. It was beyond the scope of this exploration to analyze or design specific dewatering systems for this project.

Risk of Early Refusal to Driving

Based upon the dense SPT N-values that we measured in Stratum IV, refusal of the sheetpiles to advance to the desired penetration depth ("early refusal") is a risk. For this reason, we recommend that the piles be driven into place using a high energy impact hammer rather than vibrated into place. Also, installing steel sheeting through or into very hard silts such as those encountered during our subsurface exploration may cause deformation or damage to lighter (lower modulus) pile sections. Where the final design dictates that piles must penetrate into any material with a medium dense or greater relative density, consideration should be given to sizing the piles for drivability. In some cases, the piles may need to be over-designed with respect to support of the lateral earth pressures in order to facilitate driving through dense or very hard materials without compromising the structural integrity of the piles and their interlocks.

We recommend that several test piles be installed to verify that the specified pile section and hammer are sufficient.

Fill Placement and Compaction

Controlled fill material used at the surface should be cohesionless, non-plastic sandy soil containing no more than 15 percent fines (material passing the No. 200 sieve) by weight and having a maximum dry density of at least 100 pounds per cubic foot (pcf) as determined by a laboratory standard Proctor moisture density relationship test (ASTM D698). The soil should contain less than 5 percent organics or other deleterious matter. All fill should be placed in uniform lifts of 10 in. or less (loose measure) and compacted to at least 98 percent of the standard Proctor maximum dry density (ASTM D 698).

Fill placement should be observed by a qualified Materials Technician working under the direction of the Geotechnical Engineer. In addition to this visual evaluation, the Technician should perform a sufficient number of in-place field density tests to confirm that the required degree of compaction is being attained. At least one density test per each 5,000 square feet per lift of fill should be performed for large area fills.

Shallow Foundations

Since the structure will be embedded at a depth of 17 ½ feet below the surface with a concrete footer and a 1 foot thick gravel bed, the shallow foundation is expected to be constructed within a saturated very loose sand with very little fines, and close to the cemented zone of silts that begins at a depth of about 24 feet. The lower soil profile of the site appears generally suitable to support the proposed wet well with shallow foundations considering static loading conditions and the assumed maximum column. The design engineer needs to confirm that the assumed maximum loads are correct; if actual loads are higher, we should be notified and given a reasonable opportunity to reconsider these recommendations, because it could result in changes to the estimated available bearing capacity and static settlement magnitudes. It is important to note that without the compaction of these very loose soils using a vibroprobe the well should be considered a sacrificial structure in the event of an earthquake.

The following recommendations are provided for the design and construction of shallow foundations at this site for the proposed wet well:

1. Provided that the recommendations in this report are implemented, a net available bearing pressure of up to 2,500 psf may be used for design of individual spread footings.
2. The need for overexcavation in the footing excavations should be a field decision made in consultation with the Geotechnical Engineer at the time of construction based upon the conditions observed.

    A. We understand that the drawing calls for 12 inches of gravel base under the proposed footing. Due to the saturated sands encountered at the test location, it may become necessary to over-excavate and replace this saturated loose material where it is encountered beneath individual footings. We recommend that the over-excavations be backfilled with a clean, coarse gravel such as SCDOT No. 57 or No. 67 stone, because it is relatively insensitive to moisture and does not need to be compacted in thin lifts as a soil backfill would. Another option besides coarse gravel would be ready-mixed, flowable cementitious fill.

   B. Where overexcavation of footings is required, S&ME should be present at the site to observe conditions, confirm that poor soils have been removed, and observe that the overexcavated footings are properly backfilled.

3. The following discussion is provided regarding the estimated magnitude of settlements under static loading of the wet well footing embedded at a depth of 17 feet below existing grade, or 17 ½ feet below proposed grade.

   A. Based on an assumed maximum column load of 500 kips, and a 2,500 psf shallow foundation bearing pressure, the estimated total static settlement of an individual spread footing measuring roughly 14 feet by 14 feet in plan area will likely be 1 inch or less, with a differential settlement potential under static loading of ½ inch or less.

♦ Limitations of Report

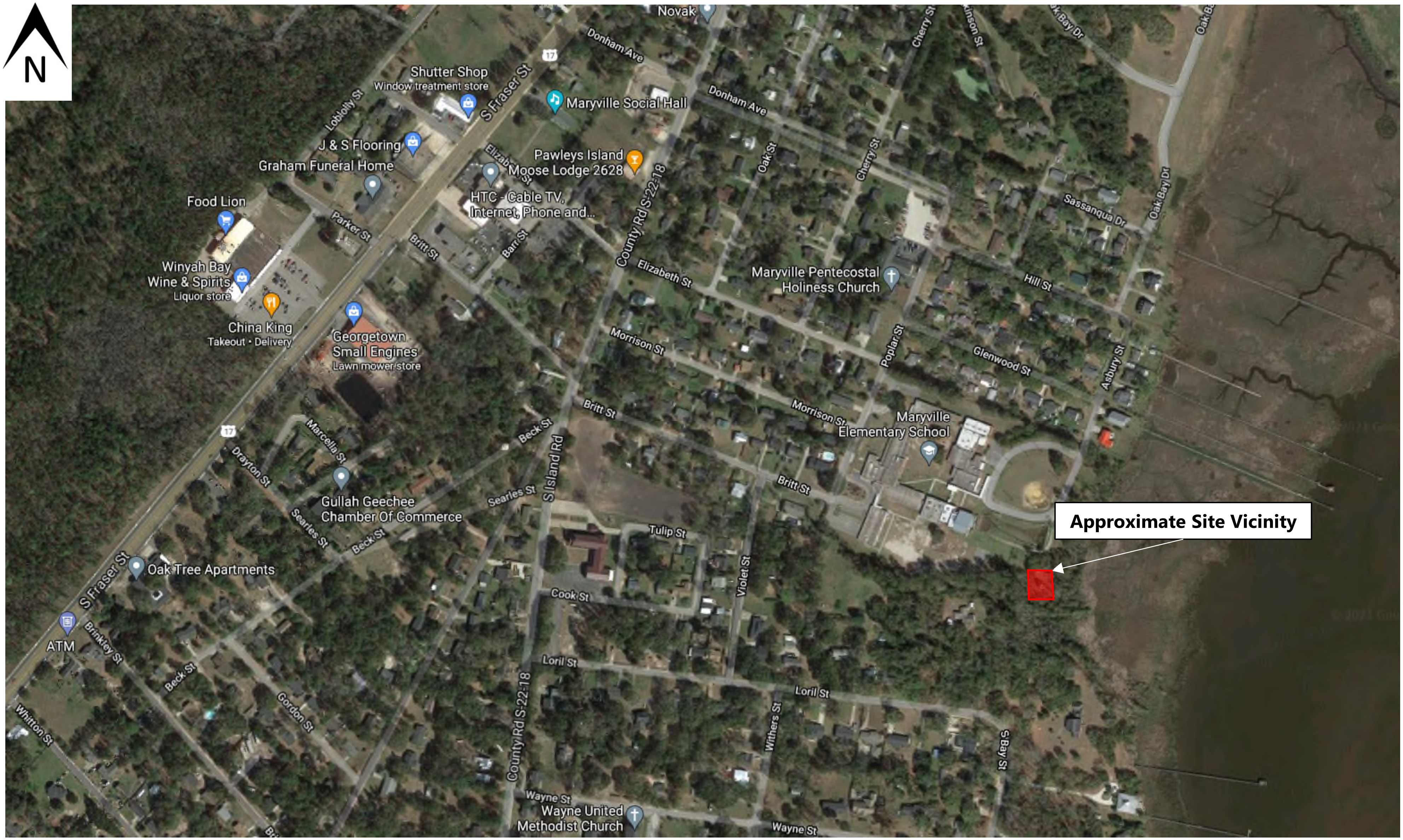
This report has been prepared in accordance with generally accepted geotechnical engineering practice for specific application to this project. The conclusions and recommendations in this report are based on the applicable standards of our practice in this geographic area at the time this report was prepared. No other warranty, expressed or implied, is made.

The analyses and recommendations submitted herein are based, in part, upon the data obtained from the subsurface exploration. The nature and extent of variations across the site may not become evident until construction. If variations appear evident, then we should be given a reasonable opportunity to re-evaluate the recommendations of this report. In the event that any changes in the nature, design, or location of the structures are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions modified or verified in writing by the submitting engineers.

Assessment of site environmental conditions; sampling of soils, ground water or other materials for environmental contaminants; identification of jurisdictional wetlands, rare or endangered species, geological hazards or potential air quality and noise impacts were beyond the scope of this geotechnical exploration.

Appendices

Appendix I – Figures



Site Vicinity Map

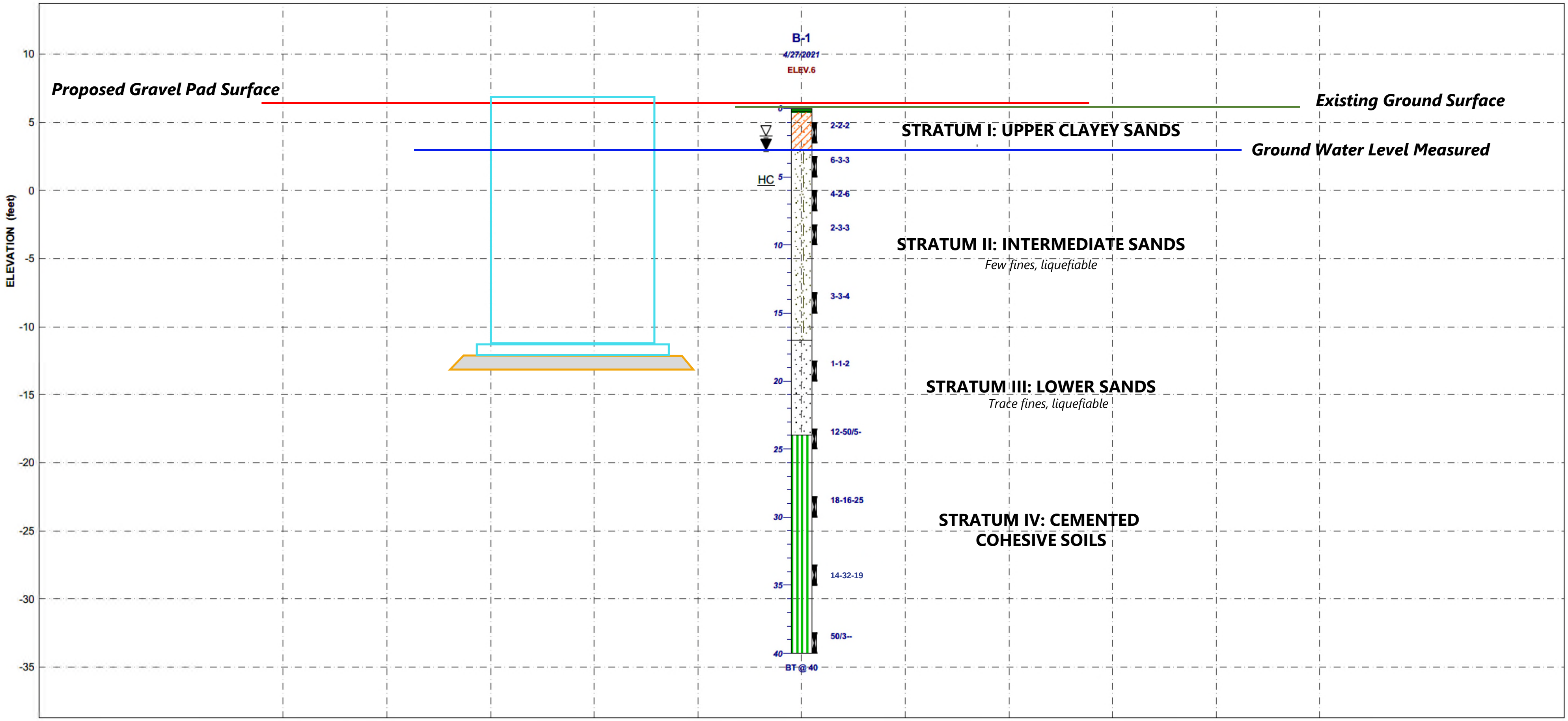
Georgetown Wet Well
Georgetown, South Carolina

SCALE: AS SHOWN
DATE: 5-3-2021
PROJECT NO. 213382
FIGURE NO. 1

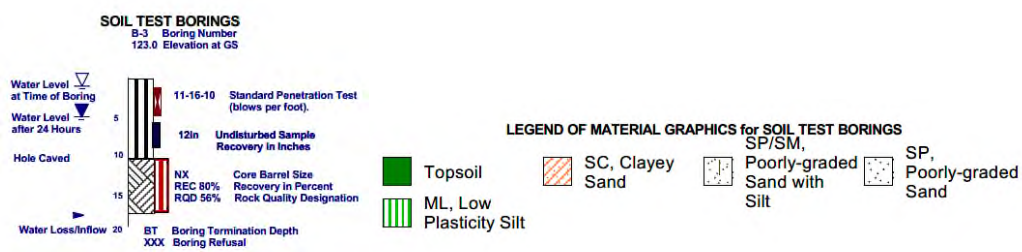
Test Location Sketch

Georgetown Wet Well
Georgetown, South Carolina

SCALE:	AS SHOWN
DATE:	5-3-2021
PROJECT NO.	213382
FIGURE NO.	2



Elevations were interpreted from the provided drawing from The Wooten Company



Subsurface Soil Profile	SCALE: AS SHOWN	FIGURE NO. 3
Georgetown Wet Well Georgetown, South Carolina	DATE: 5-3-2021 PROJECT NO. 213382	

Appendix II – Exploration Data

♦ Summary of Exploration Procedures

The American Society for Testing and Materials (ASTM) publishes standard methods to explore soil, rock and ground water conditions in Practice D-420-18, "*Standard Guide for Site Characterization for Engineering Design and Construction Purposes*." The boring and sampling plan must consider the geologic or topographic setting. It must consider the proposed construction. It must also allow for the background, training, and experience of the geotechnical engineer. While the scope and extent of the exploration may vary with the objectives of the client, each exploration includes the following key tasks:

- Reconnaissance of the Project Area
- Preparation of Exploration Plan
- Layout and Access to Field Sampling Locations
- Field Sampling and Testing of Earth Materials
- Laboratory Evaluation of Recovered Field Samples
- Evaluation of Subsurface Conditions

The standard methods do not apply to all conditions or to every site. Nor do they replace education and experience, which together make up engineering judgment. Finally, ASTM D 420 does not apply to environmental investigations.

Reconnaissance of the Project Area

We walked over the site to note land use, topography, ground cover, and surface drainage. We observed general access to proposed sampling points and noted any existing structures.

Checks for Hazardous Conditions - State law requires that we notify the South Carolina (SC 811) before we drill or excavate at any site. SC 811 is operated by the major water, sewer, electrical, telephone, CATV, and natural gas suppliers of South Carolina. SC 811 forwarded our location request to the participating utilities. Location crews then marked buried lines with colored flags within 72 hours. They did not mark utility lines beyond junction boxes or meters. We checked proposed sampling points for conflicts with marked utilities, overhead power lines, tree limbs, or man-made structures during the site walkover.

♦ Boring and Sampling

Soil Test Boring with Mud Rotary Drilling

Soil sampling and penetration testing were performed in general accordance with ASTM D1586, "Standard Test Method for Penetration Test and Split Barrel Sampling of Soils. Rotary drilling processes were used to advance the hole and a heavy drilling fluid was circulated in the bore holes to stabilize the sides and flush the cuttings. At regular intervals, drilling tools were removed and soil samples were obtained with a standard 1.4 inch I. D., two-inch O. D., split barrel sampler. The sampler was first seated six inches to penetrate any loose cuttings, then driven an additional 12 inches with blows of a 140-pound hammer falling 30 inches. The number of hammer blows required to drive the sampler through the two final six inch increments was recorded as the penetration resistance (SPT N) value. The N-value, when properly interpreted by qualified professional staff, is an index of the soil strength and foundation support capability.

Water Level Measurement

Subsurface water levels in the boreholes were measured during the onsite exploration and after a period of 24 hours by measuring depths from the existing grade to the current water level using a tape.

Backfilling of Borings

Once subsurface water levels were obtained, boring spoils were backfilled into the open bore holes. Bore holes were backfilled to the existing ground surface.

LEGEND TO SOIL CLASSIFICATION AND SYMBOLS

SOIL TYPES

(Shown in Graphic Log)

- Fill
- Asphalt
- Concrete
- Topsoil
- Gravel
- Sand
- Silt
- Clay
- Organic
- Silty Sand
- Clayey Sand
- Sandy Silt
- Clayey Silt
- Sandy Clay
- Silty Clay
- Partially Weathered Rock
- Cored Rock

WATER LEVELS

(Shown in Water Level Column)

▽ = Water Level At Termination of Boring
▼ = Water Level Taken After 24 Hours
◄ = Loss of Drilling Water
HC = Hole Cave

CONSISTENCY OF COHESIVE SOILS

CONSISTENCY	STD. PENETRATION RESISTANCE BLOWS/FOOT
Very Soft	0 to 2
Soft	3 to 4
Firm	5 to 8
Stiff	9 to 15
Very Stiff	16 to 30
Hard	31 to 50
Very Hard	Over 50

RELATIVE DENSITY OF COHESIONLESS SOILS

RELATIVE DENSITY	STD. PENETRATION RESISTANCE BLOWS/FOOT
Very Loose	0 to 4
Loose	5 to 10
Medium Dense	11 to 30
Dense	31 to 50
Very Dense	Over 50

SAMPLER TYPES

(Shown in Samples Column)

- Shelby Tube
- Split Spoon
- Rock Core
- No Recovery

TERMS

Standard Penetration Resistance - The Number of Blows of 140 lb. Hammer Falling 30 in. Required to Drive 1.4 in. I.D. Split Spoon Sampler 1 Foot. As Specified in ASTM D-1586.

REC - Total Length of Rock Recovered in the Core Barrel Divided by the Total Length of the Core Run Times 100%.

RQD - Total Length of Sound Rock Segments Recovered that are Longer Than or Equal to 4" (mechanical breaks excluded) Divided by the Total Length of the Core Run Times 100%.

Georgetown Wet Well
Georgetown, South Carolina
S&ME Project No. 213382

BORING LOG B-1

DATE DRILLED: **4/27/21**	ELEVATION: **6.0 ft**
DRILL RIG: **CME 55**	BORING DEPTH: **40.0 ft**
DRILLER: **S. Hardee**	WATER LEVEL: **2' ATD, 3' 24 hr**
HAMMER TYPE: **Auto**	LOGGED BY: **K. Fugate**
SAMPLING METHOD: **Split Spoon**	
DRILLING METHOD: **Mud Rotary**	

NOTES: **Elevation approximated from provided The Wooten Company drawing.**

MATERIAL DESCRIPTION

TOPSOIL - Approximately 3 inches thick.

CLAYEY SAND (SC) - Mostly fine to medium sand, some low to medium plasticity fines, orange and grey, wet, very loose.

POORLY GRADED SAND WITH SILT (SP-SM) - Mostly fine to medium sand, few non plastic fines, grey, wet, loose.

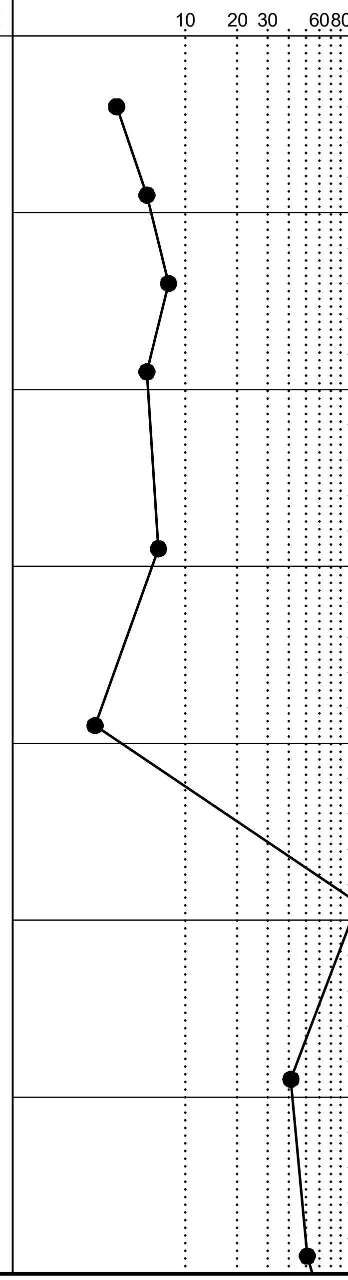
POORLY GRADED SAND (SP) - Mostly fine to medium sand, trace non plastic fines, dark grey, wet, very loose.

SANDY SILT (ML) - Mostly low plasticity fines, some fine sand, dark grey, wet, cemented, very hard.
- - - - Layer contains very thin cemented zones throughout.
- - - - Hard.
- - - - Very hard.

WATER LEVEL: HC

DEPTH (feet)	ELEVATION (feet)	SAMPLE NO.	1st 6in / RUN #	2nd 6in / REC	3rd 6in / RQD	N VALUE
		1	2	2	2	4
5	1.0	2	6	3	3	6
		3	4	2	6	8
10	-4.0	4	2	3	3	6
15	-9.0	5	3	3	4	7
20	-14.0	6	1	1	2	3
25	-19.0	7	12	50/5		50/5
30	-24.0	8	18	16	25	41
		9	14	32	19	51

STANDARD PENETRATION TEST DATA (blows/ft) / REMARKS



NOTES:

1. *THIS LOG IS ONLY A PORTION OF A REPORT PREPARED FOR THE NAMED PROJECT AND MUST ONLY BE USED TOGETHER WITH THAT REPORT.*
2. *BORING, SAMPLING AND PENETRATION TEST DATA IN GENERAL ACCORDANCE WITH ASTM D-1586.*
3. *STRATIFICATION AND GROUNDWATER DEPTHS ARE NOT EXACT.*
4. *WATER LEVEL IS AT TIME OF EXPLORATION AND WILL VARY.*

Georgetown Wet Well Georgetown, South Carolina S&ME Project No. 213382	BORING LOG **B-1**

DATE DRILLED: **4/27/21**	ELEVATION: **6.0 ft**	NOTES: **Elevation approximated from provided The Wooten Company drawing.**
DRILL RIG: **CME 55**	BORING DEPTH: **40.0 ft**	
DRILLER: **S. Hardee**	WATER LEVEL: **2' ATD, 3' 24 hr**	
HAMMER TYPE: **Auto**	LOGGED BY: **K. Fugate**	
SAMPLING METHOD: **Split Spoon**		
DRILLING METHOD: **Mud Rotary**		

DEPTH (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	WATER LEVEL	ELEVATION (feet)	SAMPLE NO.	SAMPLE TYPE	1st 6in / RUN #	2nd 6in / REC	3rd 6in / RQD	STANDARD PENETRATION TEST DATA (blows/ft) / REMARKS (10 20 30 60 80)	N VALUE
		SANDY SILT (ML) - Mostly low plasticity fines, some fine sand, dark grey, wet, cemented, very hard. *(continued)*			10		50/3				50/3
40		Boring terminated at 40 ft		-34.0							

S&ME BORING LOG \ LOGS.GPJ \ LIBRARY 2011_06_28.GDT \ 5/11/21

NOTES:

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