Geotechnical Exploration Report
Town of Summerville
Fire Station #6 Conceptual
Summerville, South Carolina
S&ME Project No. 211087

PREPARED FOR:
Town of Summerville Fire & Rescue
300 W. Second North Street
Summerville, South Carolina 29483

PREPARED BY:
S&ME, Inc.
620 Wando Park Boulevard
Mount Pleasant, South Carolina 29464

March 4, 2021

March 4, 2021

Town of Summerville Fire & Rescue
300 W. Second North Street
Summerville, South Carolina 29483

Attention: Chief Richard G. Waring, IV

Reference: **Geotechnical Exploration**
Fire Station # 6 Conceptual
Summerville, South Carolina
S&ME Project No. 211087

Dear Chief Waring:

We have completed the geotechnical exploration for the conceptual plans of the proposed Fire Station #6 in Summerville, South Carolina. The purpose of our services was to explore subsurface conditions at the site, evaluate those conditions, and provide recommendations for site preparation, and foundation and pavement support. Our services were performed pursuant to S&ME Proposal No. 211087 dated January 29, 2021.

Project Information

We understand conceptual plans have been developed for the proposed Fire Station #6 on the approximately 2.16-acre site along Miles Jamison Road in Summerville, South Carolina. The site is identified by Charleston County GIS as a portion of TMS#156-00-00-073.

Design-build procurement is planned for the facility, and therefore, specific structural and grading information is unknown. We assume maximum column and wall loads will be 40 kips and 4 k/ft, respectively. We assume site grading will be such that fill heights of 2 ft or less will be required.

The request for this proposal and project information were provided by Chief Richard Waring of Summerville Fire & Rescue to Mr. Melvin Williams of our firm via e-mail on January 25, 2021. Attached to the email was a conceptual site plan of Fire Station # 6 dated November 4, 2020.

The project information and assumptions presented herein should be reviewed and confirmed by the appropriate team members. Modifications to our recommendations and conclusions may be required if the actual conditions vary substantially from the project information and assumptions stated herein.

Methods of Exploration

Field Testing

Our field exploration included a site reconnaissance by a geotechnical professional; two, 20 ft-deep Cone Penetration Test (CPT) soundings; and two, 4-ft deep hand-auger borings.

In a CPT sounding (ASTM D 5778), an electronically instrumented cone penetrometer is hydraulically pushed through the soil to measure point stress, pore water pressure, and sleeve friction. The CPT data is used to determine soil stratigraphy and may be used to estimate soil parameters such as preconsolidation stress, friction angle, and undrained shear strength.

The hand-auger borings were drilled by manually turning a steel auger into the ground, and the soils encountered were visually classified in the field using the Unified Soil Classification System (USCS). Upon completion of the borings, each bore hole was backfilled with soil cuttings.

The test locations were located in the field by S&ME personnel referencing existing site features, measuring distances, and approximating angles. The approximate test locations are shown on the Test Location Plan (Figure 1) in the Appendix. A more detailed description of our field-testing procedures, the CPT sounding logs, and the Hand-Auger Boring Logs are also included in the Appendix.

Site and Subsurface Conditions

Site Conditions

The 2.16-acre site is located near the intersection of Miles Jamison Road and Chandler Creek Road in Summerville, South Carolina. The entire site is undeveloped and heavily wooded. During the exploration, we observed low areas with standing water due to recent rainfall.

Subsurface Conditions

Details of the subsurface conditions encountered by the soundings and borings are shown on the logs in the Appendix. These logs represent our interpretation of the subsurface conditions based upon field data. Stratification lines on the sounding logs represent approximate boundaries between soil behavior types[1] ; however, the actual transition may be gradual. The general subsurface conditions and their pertinent characteristics are discussed in the following paragraphs.

[1] Soil Behavior Type is calculated based on empirical correlations with tip resistance, sleeve friction, and pore pressure. A CPT may define a soil based on its behavior as one type while its grain size and plasticity, the traditional basis for soil classification, may define it as a different type.

The exploration initially encountered 4 in. of organic-laden topsoil underlain by natural Coastal Plain soils. The Coastal Plain soils consist of very loose sands with varying fines contents (i.e. silts and clays) and interbedded layers of very stiff silts and clays and loose to medium dense sands and sandy silts that extend to the deepest explored depth of 20 ft.

Groundwater

Groundwater was measured in the CPT soundings and hand auger borings at 3 feet below ground surface. Groundwater levels at the site will fluctuate during the year due to such things as seasonal and climatic variations and the construction activity in the area.

Conclusions and Recommendations

The exploration indicates the site is adaptable for the proposed development. The proposed building may be supported on shallow foundations provided the risk associated with liquefaction is accepted or mitigated and our site preparation and controlled fill placement and compaction recommendations are followed.

The following presents our geotechnical recommendations regarding site preparation and structural support. During review of these recommendations, it should be kept in mind that unexpected subsurface conditions may be encountered. The unexpected conditions can normally be handled during construction by on-site engineering evaluation.

Site Preparation

Site work should begin with installing gravity ditches to help drain ponded surface waters, lower groundwater levels, and direct stormwater flows. The ditches should be installed as deep and as far in advance of site work as possible. If ditches cannot drain to an outfall by gravity, they should be tied to sumps and pumped. Since the near-surface soils are moisture-sensitive, improving site drainage will be very important to improving subgrade stability and reducing potential stabilization measures (e.g., undercutting and replacement).

Site preparation should continue with clearing and grubbing vegetation and roots, stripping organic-laden topsoil, removal of unsuitable surface materials, and removal of tap root systems. Voids should be cleaned of any unsuitable materials and backfilled with well-compacted controlled fill. We expect the initial site preparations to disturb the surface soils. Therefore, the disturbed subgrade should be recompacted by making sufficient perpendicular passes with a large compactor (vibratory turned off) to recompact the subgrade.

The exposed subgrade should then be evaluated by proofrolling with a heavily loaded, tandem-axle dump truck or similar rubber-tired equipment under the observation of the Geotechnical Engineer. Proofrolling will not only densify the subgrade prior to new fill placement, but it will expose unstable subgrade areas. Areas that pump or rut excessively should be densified in place or undercut and replaced with well-compacted controlled fill or crushed stone as recommended by the Geotechnical Engineer. Undercutting should be observed by the Geotechnical Engineer to determine that all unsuitable materials are removed and to prevent removal of suitable materials.

The exploration encountered clayey sands below the topsoil. These soils are moisture sensitive and difficult to work when wet. The stability of these soils will be heavily dependent on final grades; the climatic conditions during construction; the aggressiveness of the earthwork schedule; site and excavated soil drainage; and the grading contractor's experience, equipment, means, and methods.

Stabilization measures may include undercutting and replacement, bridging, or chemical (i.e., cement) stabilization and are best determined at the time of construction by joint consultation of the grading contractor and Geotechnical Engineer. Undercutting should be observed by the Geotechnical Engineer to determine that all unsuitable materials are removed and to prevent removal of suitable materials.

Controlled Fill

Controlled fill material should be cohesionless soil containing no more than 15% fines (material passing the No. 200 sieve) by weight and having a maximum dry density of at least 100 pcf as determined by a laboratory modified Proctor compaction test (ASTM D 1557). The soil should be relatively free of organics, deleterious matter, and elongated or flat particles susceptible to degradation. All fill should be placed in uniform lifts of 10 in. or less (loose measure) and compacted to at least 95% of the modified Proctor maximum dry density.

Fill placement should be observed by a qualified engineering 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 the contractor's equipment and methods can achieve the required degree of compaction.

Seismic Considerations

We performed a liquefaction analysis based on the design earthquake prescribed by the 2018 edition of the International Building Code (IBC 2018).[2] An age correction factor, which increases the liquefaction resistance of older sand deposits of the type that were encountered at this site, was applied.[3] Our analysis indicates that liquefiable sands are present between depths of approximately 3 ft and 15 ft. Additionally, some of the liquefiable sands are located within a few feet of the ground surface which could lead to a loss of bearing capacity for shallow foundations.

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.[4] The LPI considers the factor

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[2] Liquefaction, the loss of a soil's shear strength due to the increase in porewater pressure resulting from seismic vibrations, is always a potential concern in coastal South Carolina. Analysis was performed using the "simplified procedure" presented by Youd et al. (2001).

The IBC design earthquake has a hazard equal to 2% probability of exceedance in 50 years. This is statistically equivalent to an event that occurs about once every 2,500 years. The design ground motions incorporate a target risk of structural collapse equal to 1% in 50 years. Our liquefaction analysis was based on an earthquake with a magnitude of 7.3 and ground surface acceleration of 1.22g.

[3] Hayati & Andrus (2008), Andrus et al (2009), Hayati & Andrus (2009).

[4] Iwasaki et al. 1982, Toprak & Holzer (2003).

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 as:

- ***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 LPI for this site between 10 and 20, which indicates the risk of adverse effects from liquefaction is likely.

We assume the adverse effects from liquefaction (i.e., settlement, loss of bearing capacity) are not acceptable for a fire station and liquefaction mitigation will therefore be necessary. A ground improvement option is presented below.

Section 1613.2.2 of the IBC 2018 classifies sites with the potential for liquefaction as Seismic Site Class F. However, the IBC 2018 allows the design spectral response accelerations for a site to be determined without regard to liquefaction provided the structure has a fundamental period of less than or equal to 0.5 seconds and the risks of liquefaction are considered in design. We assume the proposed structures will meet these criteria; however, this must be confirmed by the Structural Engineer. Provided the above criteria are met, the design accelerations may be calculated using Site Class D site coefficients as shown in Table 1.

Table 1 – Ground Motion Parameters

Site Class	S_s	S_1	F_a	F_v	PGA_M	S_{DS}	S_{D1}
F	1.93g	0.58g	1.0	1.72*	1.4g	1.28g	0.66g*

* The acceleration parameters should only be used when calculating the Seismic Response Coefficient (C_s) per the exception to the site-specific ground motion procedures requirement detailed in section 11.4.8 of ASCE 7-16.

The 1-second spectral acceleration (S_1) for this site is 0.58g. IBC 2018 requires site-specific ground motion procedures to be followed when S_1 is greater than or equal to 0.2g (see ASCE 7-16 section 11.4.8). However, the code provides an exception to this requirement in ASCE 7-16 section 11.4.8 if certain conditions are met when determining the Seismic Response Coefficient (C_s). As with the liquefaction-related exception, Site Class D site coefficients and corresponding spectral accelerations, as presented in Table 1, may be used for purposes of computing C_s in accordance with the S_1 exception.

Ground Improvement

Liquefaction Mitigation

We assume the consequences of liquefaction are not acceptable for a fire station and mitigation will therefore be required. We recommend earthquake (EQ) drains be used to reduce liquefaction risks and allow for shallow foundation support of the building.

Earthquake (EQ) Drains are a common ground improvement technique used in this area. EQ drains can be used to mitigate liquefaction within structural areas. The drains allow for the rapid dissipation of excess soil porewater pressures generated during a seismic event thus reducing liquefaction. The drains are composed of corrugated, perforated plastic pipe encased in a filter fabric which prevents migration of fines into the pipe. Pipe diameters and spacings vary according to the anticipated liquefaction risk and may be supplemented with man-made gravel reservoirs for additional water storage. The drains are installed by vibrating a steel casing into the ground which helps densify the surrounding loose sands and allows insertion of the drain pipe.

Earthquake drains are typically provided in a design-build contract by a specialty contractor experienced with the design and installation of the system. We recommend a request be submitted to qualified contractors to prepare a proposal to furnish all necessary labor, equipment, and materials to design and install EQ drains to reduce liquefaction-induced deformations and strength loss for the IBC 2018 design earthquake in building areas to a magnitude acceptable to the structural engineer. The IBC 2018 design event comprises a 7.3 magnitude earthquake with a peak ground acceleration (PGA) of 1.3g. A copy of this report should be submitted with the request to provide the necessary subsurface data to perform the design. The proposals should be evaluated by the project Geotechnical and Structural Engineers, and then a contractor should be selected based on technical approach, experience, and cost. It will be important that the EQ drain design is coordinated with the civil design.

EQ drain installation should be observed by a representative of the Geotechnical Engineer to confirm that 1) EQ drains are installed in all locations, 2) EQ drains are installed to the design depth, and 3) note any non-conformance with the EQ drain design.

The EQ drain system is a common ground improvement technique used in this area; however, there are other ground improvement techniques that may have application to this site. These techniques are typically proprietary systems of specialty contractors. Our presentation of earthquake drains should not exclude evaluation of other options that may be presented by ground improvement contractors. However, other options should be evaluated by the Geotechnical Engineer to confirm their feasibility to this project.

Shallow Foundation Support

The proposed building may be supported with conventional shallow foundations bearing in suitable natural soils or well-compacted fill provided our site preparation, fill recommendations, and foundation evaluation recommendations are followed. A maximum allowable bearing pressure of 2,000 psf may be used for sizing footings. Wall and column footings should be a minimum of 18 and 24 in. wide, respectively.

All foundation excavation bottoms must be evaluated by a representative of the Geotechnical Engineer prior to steel and concrete placement. This evaluation should include probing, hand-auger borings, and dynamic cone penetrometer (DCP) testing. Any loose material should be properly compacted or undercut and replaced with well-compacted controlled fill or crushed aggregate such as No. 57 stone. This evaluation will help determine if individual footings are directly underlain by suitable bearing material. If practical, concrete placement should be completed the same day as the footing excavation. Slag is not recommended as backfill due to its potentially expansive nature

Our analysis indicates post-construction, static (not seismically-induced) settlement due to the given column and wall loads of 40 kips and 4 kips/ft will be up to 1 in. Differential settlement is typically approximately half of the total settlement.

Floor Slabs

Building floor slabs can be soil supported provided our site preparation and fill placement and compaction recommendations are followed. A subgrade modulus (k) of 180 pci is available for floor slab design. This recommended modulus is representative of a 30-in. diameter plate load test and must be reduced for wide area loads.

Based on the results of our exploration, the floor slab will not be subjected to hydrostatic pressure from groundwater. However, water vapor transmission through the slab is still a design consideration. Evaluating the need for and design of a vapor retarder or vapor barrier for moisture control is outside our scope of services and should be determined by the project architect/structural engineer based on the planned floor coverings and the corresponding design constraints as outlined in ACI 302.1R-04 Guide for Concrete Floor and Slab Construction. Details regarding proper backfill of utility trenches below the building floor slab should be planned. Suitable granular material should be used as backfill materials. The backfill should be placed and compacted in accordance with the Controlled Fill recommendations discussed previously.

Pavement Recommendations

We have evaluated new flexible (asphalt) using the *SCDOT Pavement Design Guide* and associated literature. Traffic loading data was not provided. Based on our experience with similar projects, we assume light-duty pavement will be subjected primarily to passenger cars and light truck traffic and heavy-duty pavements will be subjected to heavy truck traffic.

Pavement section recommendations are based on our experience with sites that have similar soils and subgrades consisting of at least 24 in. of well-compacted controlled fill. Table 2 presents our recommendations for minimum pavement sections.

Table 2 – Minimum Recommended Pavement Sections

Material	Flexible Pavement		Rigid Pavement	
	HeavyDuty	Standard Duty	Heavy Duty	Standard Duty
Asphaltic Concrete Surface Course	3 in.	2 in.	-	-
Graded Aggregate Base Course	8 in.	6 in.	-	-
Portland Cement Concrete	-	-	8 in.	5 in.
Compacted/Proofrolled Subgrade	24 in.			

Rigid pavement should be considered for high truck traffic areas, truck turning areas, and any areas supporting the heavier fire truck traffic. Concrete pavement thickness will generally vary from 5 inches thick in light duty areas to at least 8 inches for heavy duty areas. A rigid pavement section is recommended in dumpster pad areas to support, at a minimum, the front wheels of the truck. Rigid pavements should also be used in areas subjected to repeated lateral loading (turning, stopping, starting) such as any truck loading and turning areas. Based on our experience, this should be adequate for the assumed traffic and a typical 15-year pavement life.

Construction traffic has not been included in our analysis, and construction traffic should be restricted from prepared subgrades and new pavements. If pavements must support construction traffic, staged construction or a thicker asphalt section will be required.

All materials and workmanship should be in accordance with the South Carolina Department of Transportation's *Standard Specifications for Highway Construction*, 2007 Edition.

A stable subgrade is very important to pavement performance. Immediately prior to paving, the subgrade should be proofrolled, and any unstable areas should be repaired. The base course should be compacted to at least 98% of the maximum dry density as determined by the modified Proctor compaction test (ASTM D 1557). In-place field density tests should be performed by a qualified Materials Technician, and the area should be methodically proofrolled under their evaluation to confirm that the base course has been uniformly compacted. The thickness should not be deficient in any area by more than ½ in. The asphalt pavement thickness should not be deficient by more than ¼ in. in any area.

The performance of asphalt pavements will be dependent upon a number of factors including subgrade conditions at the time of paving, drainage, and traffic. The geometric design should provide positive drainage for the pavement surface and subgrades. This is very important and may require the use of underdrains in sags and low areas.

Pavement design typically has relatively low factors of safety; therefore, it will be very important that the specifications are followed closely during pavement construction. Our experienced-based recommendations are intended to be consistent with a 15-year design life; however, some isolated areas could require repair in a shorter period of time.

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 contained in this report are based upon applicable standards of our practice in this geographic area at the time this report was prepared. No other representation or warranty either express or implied, is made.

We relied on project information given to us to develop our conclusions and recommendations. If project information described in this report is not accurate, or if it changes during project development, we should be notified of the changes so that we can modify our recommendations based on this additional information if necessary.

Our conclusions and recommendations are based on limited data from a field exploration program. Subsurface conditions can vary widely between explored areas. Some variations may not become evident until construction. If conditions are encountered which appear different than those described in our report, we should be notified. This report should not be construed to represent subsurface conditions for the entire site.

S&ME should be retained to review the final plans and specifications to confirm that earthwork, foundation, and other recommendations are properly interpreted and implemented. The recommendations in this report are contingent on S&ME's review of final plans and specifications followed by our observation and monitoring of earthwork and foundation construction activities.

Closure

We appreciate the opportunity to provide our services to this project. If you have any questions concerning this report, please call.

Sincerely,

S&ME, Inc.

SOUTH CAROLINA PROFESSIONAL ENGINEER REGISTERED No. 24562 WILLIAM M. CAMP, III

SOUTH CAROLINA CERTIFICATE OF AUTHORIZATION S&ME, INC. C00473

William Camp III
Mar 4 2021 12:21 PM

Justin H. Cox, PG
Geotechnical Project Manager

William M. Camp, III, PE, D.GE
Technical Principal/Vice President

Appendix

Test Location Plan

CPT Soundings Logs

Hand Auger Boring Logs

Field Testing Procedures

Test Location Plan

	TEST LOCATION PLAN	SCALE: Not to Scale	FIGURE NO.
		DATE: 03/05/2021	**1**
	Town of Summerville Fire Station 6 – Conceptual Plan Summerville, South Carolina	PROJECT NUMBER 211087	

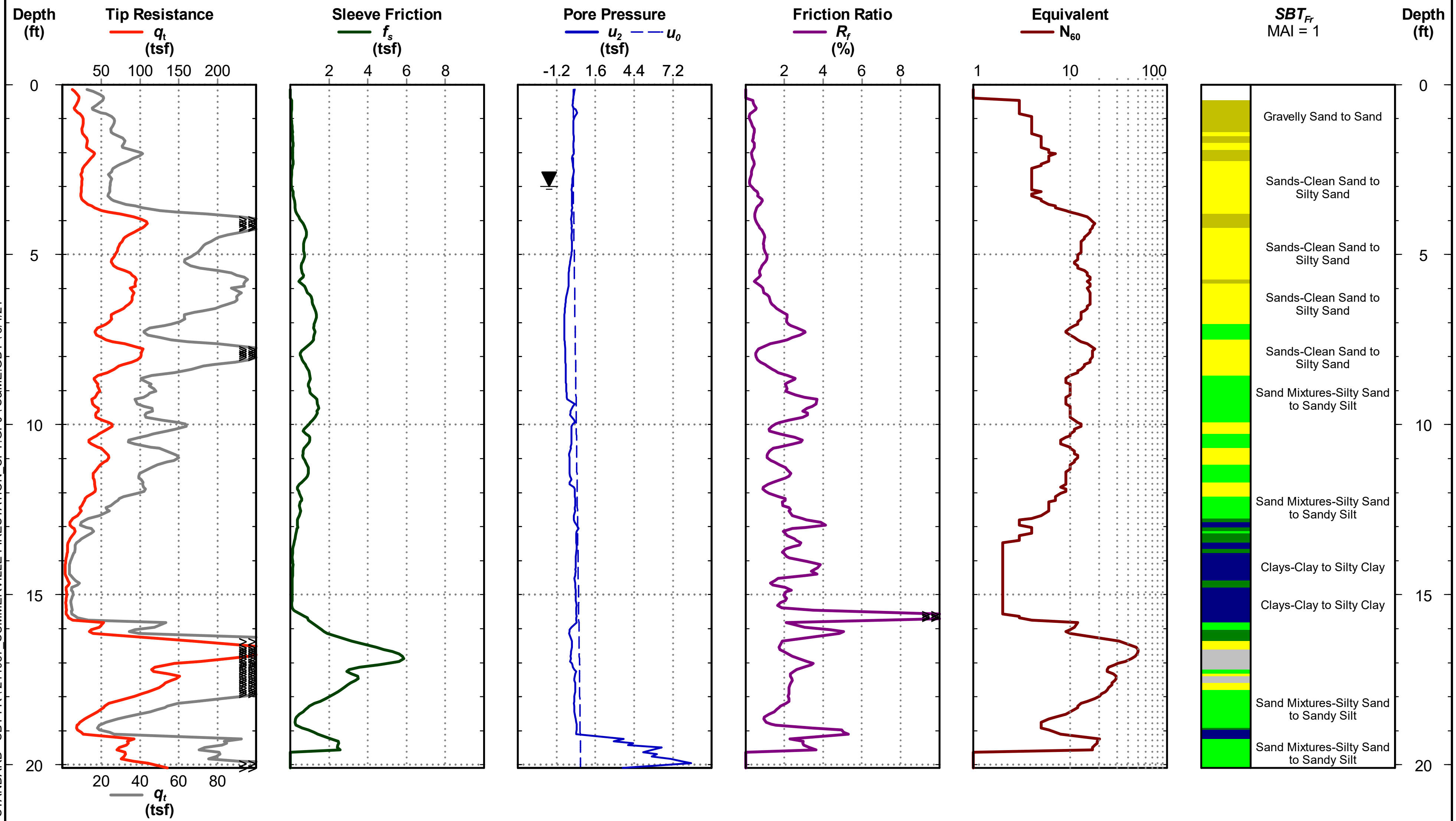
CPT Sounding Logs

Sounding ID: C-1

Summerville Fire Station
Summerville S.C.
S&ME Project No: 211087

Date: Feb. 22, 2021
Estimated Water Depth: 3 ft
Rig/Operator: Marooka/MW | TC

Total Depth: 20.1 ft
Termination Criteria: Target Depth
Cone Size: 1.75



Cone Penetration Test

Electronic Filename: C-1(002).DAT

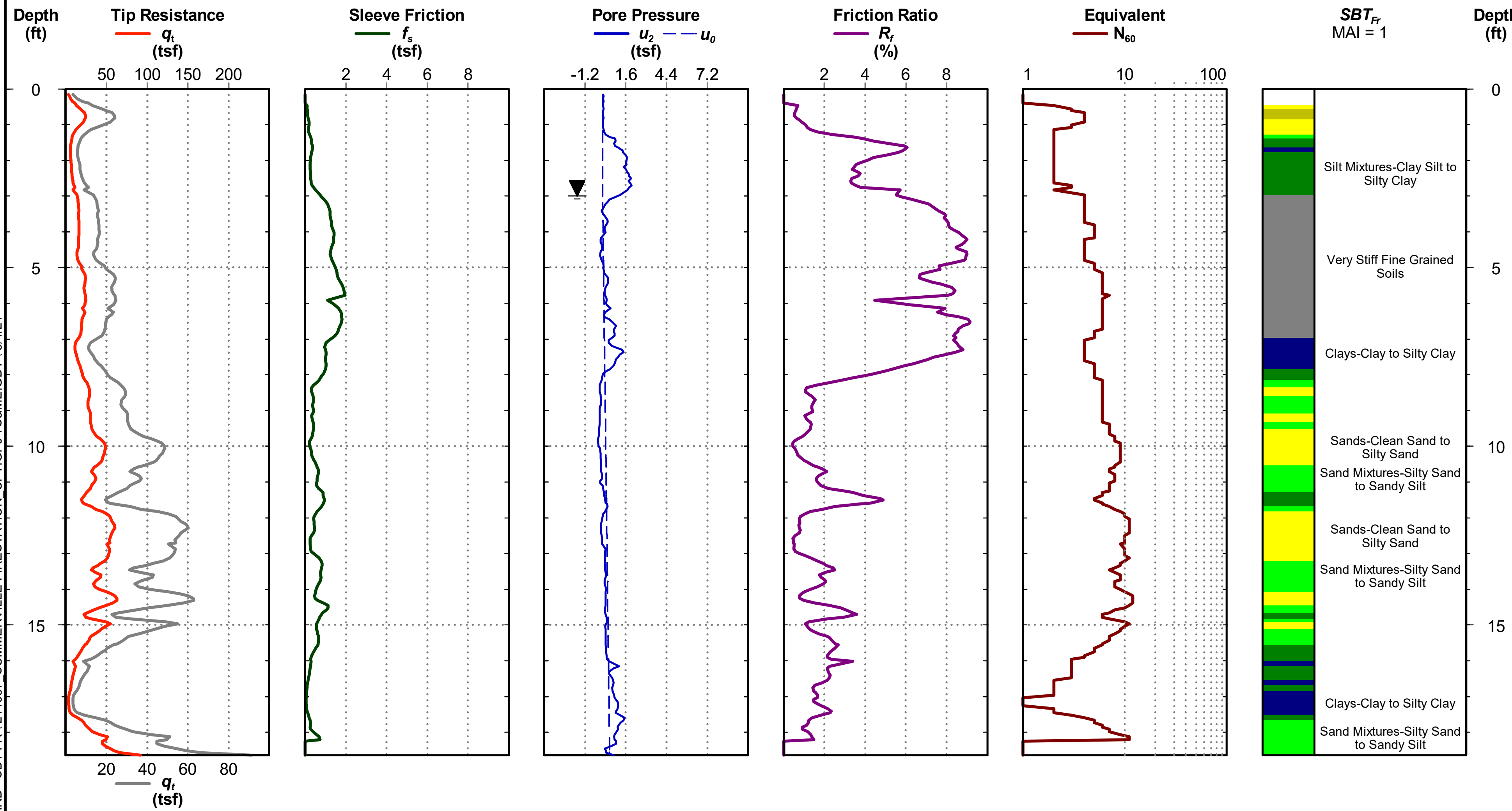
CPT REPORT - STANDARD - SBT FR \ 211087_SUMMERVILLE FIRESTATION_CPT.GPJ \ S&ME.GDT \ 3/4/21

Sounding ID: C-2

Summerville Fire Station
Summerville S.C.
S&ME Project No: 211087

Date: Feb. 22, 2021
Estimated Water Depth: 3 ft
Rig/Operator: Marooka/MW | TC

Total Depth: 18.6 ft
Termination Criteria: Refusal
Cone Size: 1.75



CPT REPORT - STANDARD - SBT FR \ 211087 SUMMERVILLE FIRESTATION CPT.GPJ \ S&ME.GDT \ 3/4/21

Cone Penetration Test

Electronic Filename: C-2(001).DAT

Hand Auger Boring Logs

PROJECT:	Town of Summerville Firestation #6 Summerville, South Carolina S&ME Project No. 211087	HAND AUGER BORING LOG: HA-1

DATE STARTED:	**2/17/21**	DATE FINISHED:	**2/17/21**	NOTES:
SAMPLING METHOD:	**Hand Auger**	PERFORMED BY:	**Justin Cox**	
WATER LEVEL:	**3' ATD**			

Depth (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (feet)	WATER LEVEL
		TOPSOIL = 4 in.		
		SAND WITH SILT (SP-SM) Light Reddish Brown, Fine Sand, Moist		
1		- - - Pale gray mottled with Pale Yellow and Reddish brown.		
2				
3		- - - Dark Gray. Wet		▽
4		Boring terminated at 4 ft Target Depth		

PROJECT:	Town of Summerville Firestation #6 Summerville, South Carolina S&ME Project No. 211087	HAND AUGER BORING LOG: HA-2

DATE STARTED:	2/17/21	DATE FINISHED:	2/17/21	NOTES:
SAMPLING METHOD:	Hand Auger	PERFORMED BY:	Justin Cox	
WATER LEVEL:	**3' ATD**			

Depth (feet)	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION (feet)	WATER LEVEL
		TOPSOIL = 4 in.		
		SAND WITH SILT (SP-SM) Light Reddish Brown, Fine Sand, Moist.		
1		**CLAYEY SAND (SC)** Pale brown mottled with Red. Fine sand, low plasticity, moist.		
2				
3		- - - Wet		▽
4		Boring terminated at 4 ft Target Depth		

Field Testing Procedures

FIELD TESTING PROCEDURES

Cone Penetrometer Test (CPT) Sounding

The cone penetrometer test soundings (ASTM D 5778) were performed by hydraulically pushing an electronically instrumented cone penetrometer through the soil at a constant rate. As the cone penetrometer tip was advanced through the soil, nearly continuous readings of point stress, sleeve friction and pore water pressure were recorded and stored in the on-site computers. Using theoretical and empirical relationships, CPT data can be used to determine soil stratigraphy and estimate soil properties and parameters such as effective stress, friction angle, Young's Modulus and undrained shear strength.

The consistency and relative density designations, which are based on the cone tip resistance, q_t for sands and cohesive soils (silts and clays) are as follows:

SANDS		**SILTS AND CLAYS**	
Cone Tip Resistance, q_t (tsf)	**Relative Density**	**Cone Tip Resistance, q_t (tsf)**	**Consistency**
<20	Very Loose	<5	Very Soft
20 – 40	Loose	5 – 10	Soft
40 – 120	Medium Dense	10 – 15	Firm
		15 – 30	Stiff
120 – 200	Dense	30 –60	Very Stiff
>200	Very Dense	>60	Hard

CPT Correlations

References are in parenthesis next to the appropriate equation.

General

p_a = atmospheric pressure (for unit normalization)
q_t = corrected cone tip resistance (tsf)
f_s = friction sleeve resistance (tsf)
$R_f = 100\% * (f_s/q_t)$
u_2 = pore pressure behind cone tip (tsf)
u_0 = hydrostatic pressure
$B_q = (u_2-u_0)/(q_t-\sigma_{v0})$
$Q_t = (q_t-\sigma_{v0})/\ \sigma'_{v0}$
$F_r = 100\% * f_s/(q_t-\ \sigma_{v0})$
$I_c = ((3.47-\log Q_t)^2+(\log F_r+1.22)^2)^{0.5}$

N-Value
$N_{60} = (q_t/pa)/[8.5(1-I_c/4.6)]$ (6)

(6) Jefferies, M.G. and Davies, M.P., (1993), "Use of CPTu to estimate equivalent SPT N60", ASTM Geotechnical Testing Journal, Vol. 16, No. 4

CPT Soil Classification Legend

Zone	Q_t/N	Description
1	2	Sensitive, Fine Grained
2	1	Organic Soils-Peats
3	1.5	Clays-Clay to Silty Clay
4	2	Silt Mixtures-Clayey Silt to Silty Clay
5	3	Sand Mixtures-Silty Sand to Sandy Silt
6	4.5	Sands-Clean Sand to Silty Sand
7	6	Gravelly Sand to Sand
8	1	Very Stiff Clay to Clayey Sand*
9	2	Very Stiff, Fine Grained*

(*) Heavily Overconsolidated or Cemented

Robertson's Soil Behavior Type (SBT), 1990			
Group #	**Description**	**Ic**	
		Min	**Max**
1	Sensitive, fine grained	N/A	
2	Organic soils - peats	3.60	N/A
3	Clays - silty clay to clay	2.95	3.60
4	Silt mixtures - clayey silt to silty clay	2.60	2.95
5	Sand mixtures - silty sand to sandy silt	2.05	2.60
6	Sands - clean sand to silty sand	1.31	2.05
7	Gravelly sand to dense sand	N/A	1.31
8	Very stiff sand to clayey sand (High OCR or cemented)	N/A	
9	Very stiff, fine grained (High OCR or cemented)	N/A	

Soil behavior type is based on empirical data and may not be representative of soil classification based on plasticity and grain size distribution.

Relative Density and Consistency Table			
SANDS		SILTS and CLAYS	
Cone Tip Stress, qt (tsf)	Relative Density	Cone Tip Stress, qt (tsf)	Consistency
Less than 20	Very Loose	Less than 5	Very Soft
20 - 40	Loose	5 - 15	Soft to Firm
40 - 120	Medium Dense	15 - 30	Stiff
120 - 200	Dense	30 - 60	Very Stiff
Greater than 200	Very Dense	Greater than 60	Hard

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
- Marl

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%.