REPORT OF SUBSURFACE EXPLORATION

York County - Coroner's Office

Rock Hill, South Carolina
ESP Project Number: E4-KW23.300

Prepared For:
York County Engineering
6 South Congress Street
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December 21, 2022

ESP

December 21, 2022

Mr. Ron Pompey, PE
York County Engineering
6 South Congress Street
York, South Carolina 28745

Reference: **REPORT OF SUBSURFACE EXPLORATION**
York County - Coroner's Office
Rock Hill, South Carolina
ESP Project No. - E4-KW23.300

Ladies and Gentlemen:

ESP Associates, Inc. (ESP) has completed a subsurface exploration for the proposed Coroner's Office in Rock Hill, South Carolina. This exploration was performed in general accordance with our Proposal No. E4-221016, Revision No. 1, dated November 2, 2022, authorized by Ron Pompey, PE with York County Engineering.

ESP appreciates the opportunity to assist you during this phase of the project. If you should have any questions concerning this report, or if we may be of further assistance, please contact us.

Sincerely,

ESP Associates, Inc.

Valentina Ayala

Valentina Ayala, EI
Project Manager

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SOUTH CAROLINA ESP ASSOCIATES, INC. No. 5839 CERTIFICATE OF AUTHORIZATION

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APPENDIX

1.0 INTRODUCTION

1.1 Purpose of Services

The purpose of the exploration was to evaluate the general subsurface conditions within the proposed building and pavement areas. This report contains a brief description of the field and laboratory testing procedures performed for this study and a discussion of the soil conditions encountered at the site. Our findings, conclusions and recommendations for foundation and pavement design, as well as construction considerations for the proposed foundations and paved areas are presented within this report.

1.2 Site Description

The subject property is a portion of the land belonging to the Family Court building in Rock Hill, York County, South Carolina. The site consists of the open field area located in the rear of the existing building, in between the west parking lot and the access road off of West Main Street. Relief on the site is on the order of 5-10 feet, sloping downward from north to south.

1.3 Project Description

Based on the drawing titled "York County Family Court," dated March 13, 2018, and prepared by Moseley Architects, plans are to develop the site with a new Coroner's Office building located to the rear of the existing Family Court building. Due to the preliminary nature of this project, additional information regarding the building plans and structural loading information have not been provided to us at the time of this report. For the purpose of this report, ESP anticipates the new construction will be a single-story structural masonry building supported upon shallow foundations with a concrete slab-on-grade.

The selection of test boring locations and depths is based on our experience with similar projects in the area, as well as the previously referenced plans. For more detailed information, reference the attached "Test Location Plan with Site Vicinity Map - Figure 1" located in the Appendix.

2.0 EXPLORATION PROCEDURES

2.1 Field

The following methods were used to evaluate the subsurface conditions of the site. Additional descriptions of the field exploration procedures are also presented in the Appendix. The test locations were located in the field by a representative from our office using a handheld GPS. While in the field and where applicable, a representative of the geotechnical engineer visually examined the samples obtained or subsurface material encountered to evaluate the type of soil, soil plasticity, moisture condition, organic content, presence of lenses and seams, colors and apparent geological origin using general guidance from "ASTM D 2488 Standard Practice for Description and Identification of Soils (Visual Manual Procedures)." Test locations are shown on the attached "Test Location Plan," Figure 1.

2.1.1 Soil Test Borings

Three soil test borings (Borings B-01 through B-03) were extended to depths ranging between 20 and 37 feet below the existing ground surface using a Geoprobe 7822 GT drill rig. Hollow-stem, continuous flight augers were used to advance the borings into the ground. Standard Penetration Tests were performed within the soil test borings using an automatic hammer. The Standard Penetration Test provides the Standard Penetration Resistances (N-values) reported in blows per foot (bpf) as outlined in the Field Exploration Procedures section located in the Appendix. Water level measurements were attempted at the termination of drilling.

The results of the visual soil classifications for the borings, as well as field test results and N-values, are presented on the individual "Test Boring Record," included in the Appendix. Similar soils were grouped into strata on the records. The strata lines represent approximate boundaries between the soil types; however, the actual transition between soil types in the field may be gradual in both the horizontal and vertical directions.

2.2 Laboratory

Select samples of the on-site soils obtained during the field testing program were tested in the laboratory. Tests performed included:

- Atterberg limits
- Grain size distribution

The results of the laboratory tests performed for this study are attached in the Appendix. A brief description of the procedures used are also presented in the Appendix.

3.0 SUBSURFACE CONDITION

3.1 Site Geology

The referenced property is located in Rock Hill, South Carolina which is in the Piedmont Physiographic Province. The Piedmont Province generally consists of hills and ridges which are intertwined with an established system of draws and streams. The Piedmont Province is predominately underlain by igneous rock (formed from molten material) and metamorphic rock (formed by heat, pressure and/or chemical action), which were initially formed during the Precambrian and Paleozoic eras.

The residual soils encountered in this area are the product of in-place chemical weathering of rock which was similar to the rock presently underlying the site. In areas not altered by erosion or disturbed by the activities of man, the typical residual soil profile consists of clayey soils near the surface, where soil weathering is more advanced, underlain by sandy silts and silty sands. The boundary between soil and rock is not sharply defined. This transitional zone termed “partially weathered rock” is normally found overlying the parent bedrock. Partially weathered rock is defined, for engineering purposes, as residual material with Standard Penetration Resistances in excess of 100 blows per foot (bpf). Weathering is facilitated by fractures, joints and by the presence of less resistant rock types. Consequently, the profile of the partially weathered rock and hard rock is quite irregular and erratic, even over short horizontal distances. Also, it is common to find lenses and boulders of hard rock and zones of partially weathered rock within the soil mantle, well above the general bedrock level.

3.2 Subsurface Findings

Subsurface conditions as indicated by the borings generally consist of topsoil and fill underlain by residual soils and partially weathered rock. The generalized subsurface conditions at the site are described below and are graphically depicted in the Appendix. For more detailed soil descriptions and stratifications at a particular test location, the attached “Test Boring Record” should be reviewed.

3.2.1 Surface

A topsoil layer approximately 6 to 7 inches thick was encountered at Borings B-01 through B-03. The thickness of topsoil or similar organic-laden surface materials may be greater or lower in thickness between the relatively widely spaced boring locations.

3.2.2 Fill

Fill soils are either site soils or imported soils that were manipulated and placed on the site previous to this exploration. Underlying the topsoil in Borings B-01 through B-03, fill soils were encountered. The fill consists of very soft to stiff sandy elastic silts to sandy high plasticity clays. N-values in the fill ranged from WOH to 10 blows per foot (bpf). The fill extended to depths ranging between 7 and 18.5 feet below the existing ground surface.

3.2.3 Residuum

Residual soils are mineral material accumulated by the in-place chemical weathering of the underlying parent rock. Beneath the fill in Borings B-01 through B-03, residual soils were encountered. The residuum generally consists of firm to very stiff sandy clays and silts, as well as medium dense clayey sands. N-values in the residuum varied between 6 and 58 bpf. The residuum extends to depths ranging between 20 and 33.5 feet below the existing ground surface. Soil Test Borings B-01 and B-03 were terminated in the residual soils at a depth of approximately feet below the existing ground surface.

3.2.4 Partially Weathered Rock (PWR)

Partially weathered rock is defined, for engineering purposes, as residual material with Standard Penetration Resistances in excess of 100 blows per foot. Underlying the residuum in Boring B-02, partially weathered rock (PWR) was encountered. When sampled, the PWR generally breaks down into silty sands with rock fragments. Boring B-02 was terminated in the PWR at a depth of approximately 37 feet below the existing ground surface.

3.2.5 Auger Refusal

Auger refusal is defined as material that could not be penetrated with the drill rig equipment used on the project. Auger refusal material may consist of large boulders, rock ledges, lenses, seams or the top of parent bedrock. Core drilling techniques would be required to evaluate the character and continuity of the refusal material. Boring B-02 was terminated in the PWR upon auger refusal at a depth of approximately 37 feet below the existing ground surface.

3.3 Subsurface Water

Subsurface water level measurements were attempted at the completion of drilling. Test Locations B-01 through B-03 were backfilled upon completion of water level readings at time of drilling due to safety concerns. Hole cave-in depths ranged from 16.3 to 32.7 feet below the existing ground surface, and may be an indication of the presence of water. Subsurface water levels tend to fluctuate with seasonal and climatic variations, as well as with some types of construction operations. Therefore, water may be encountered during construction at depths not indicated during this study. The generalized subsurface water conditions encountered during our exploration are described below. For more detailed information, the attached individual “Test Boring Record” should be reviewed.

Test Location	Water Depth at Time of Testing	Cave-In Depth at Time of Testing
B-01	Dry	16.5
B-02	Dry	32.7
B-03	Dry	16.3

3.4 Laboratory Test Results

Laboratory tests were performed on select samples obtained from bulk and split spoon samples. Laboratory testing consisted of Atterberg Limits and Grain Size Distribution Testing. The laboratory testing results are summarized below.

Sample Location	Depth (feet)	USCS Classification	Percent Fines (%)	Liquid Limit	Plasticity Index	Maximum Dry Unit Weight (lb/ft^3)	Optimum Water Content (%)
B-02	3.5-5	CH	65.5	56	32	-	-

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 Geotechnical Considerations

Based on the project information previously discussed, the data obtained from the field and laboratory testing program and our analysis, the following conditions should be considered and addressed in the proposed development and are further discussed in the following sections of this report.

- Existing Fill
- High Plasticity Clay/Elastic Silts
- Low-Consistency Soils
- Difficult Excavation

Our conclusions and recommendations are based on the project information previously discussed and on the data obtained from the field and laboratory testing program. If the structural loading, geometry or proposed building locations are changed or significantly differ from those discussed, or if conditions are encountered during construction that differ from those encountered by the borings, ESP requests the opportunity to review our recommendations based on the new information and make any necessary changes.

4.2 Site Development

4.2.1 Existing Fill

The exploration and evaluation of the subsurface conditions indicate that fill soils are present in Borings B-01 through B-03. The fill extends to depths ranging between 7 and 18.5 feet below the existing ground surface. N-values obtained in the fill ranged from WOH to 10 bpf. Based on our visual observations of the split-spoon samples recovered and our field observations, the fill encountered in the soil test borings appeared clean of concentrated organics, debris and other deleterious materials.

Concentrated organics, debris and other deleterious materials were not observed in the soil test borings performed by ESP. However, due to the limited testing performed and the wide spacing of the borings, the possibility of deleterious inclusions and variable density material in or under the existing fill cannot be completely ruled out. If the fill contains wood fragments, trash, organics, voids or soft lenses, excessive settlement could result causing building and slab-on-grade distress. Also, the presence of the existing fill beneath pavement areas present the risk of increased settlement and subsequently possible increased long term maintenance of the pavement areas. If the owner is not willing to accept the risk, then several options may be considered. These options may include:

1) remove the existing fill and replace with compacted, suitable structural fill.
2) extend the foundations through the existing fill to bear on competent residual soils.

To reduce the risk imposed by the existing fill, additional evaluations including test pit excavations, hand auger borings with Dynamic Cone Penetrometer tests, proofrolling and additional soil test borings could be performed to further evaluate the character and continuity of the fill. If the owner chooses to allow the existing fill to remain in place beneath the buildings, a thorough field evaluation should be performed by a representative of the geotechnical engineer while monitoring construction activity.

4.2.2 High Plasticity Clay/Elastic Silt

Laboratory tests were performed on select samples obtained from the split spoon samples. Laboratory testing consisted of Atterberg Limits and grain size testing. Typically, soils with a Plasticity Index (PI) less than 30 are considered to be low to moderate plasticity material. A summary of the laboratory test results are presented in the table below:

Sample Location	Depth (feet)	USCS Classification	Percent Fines (%)	Liquid Limit (%)	Plasticity Index
B-02	3.5-5	CH	65.5	56	32

In addition to the laboratory testing, manual manipulation of recovered samples in the field indicates that sandy high plasticity clays were encountered in Borings B-02 and B-03 to depths ranging from approximately 8.5 to 18.5 feet below the existing ground surface and elastic silts were encountered in Borings B-01 and B-03 to depths ranging from approximately 7 to 14.7 feet below the existing ground surface. Our experience indicates that these soils can undergo significant change in volume (shrink/swell) with changes in their moisture content. If high plasticity clay and/or elastic silt soils such as those encountered on the site become wet during or after construction, there may be an increase in their volume (swelling) and/or a reduction in their strength. In addition, if these materials are in-place during construction and subsequently dry out, there may be a decrease in their volume (shrinking) resulting in settlement. While swell testing was beyond the scope of our services, the presence of soils with plasticity indices greater than 30 within the near surface (upper 2 to 3 feet) soil profile may present an increased risk of distress to the proposed foundations, slabs-on-grade or pavements due to swell or shrinkage of these materials with variations in moisture content.

Foundations, slabs and/or pavements may not be sufficiently weighted to reduce the potential for swell and/or heave, if bearing directly on high plasticity clays or elastic silts. In addition, our past experience indicates that high plasticity clays and elastic silts may exhibit reduced long-term stability for support of flexible pavements. If encountered during construction, ESP recommends removing high plasticity clay and elastic silt soils in the upper 3 feet of the proposed subgrade or bearing elevations, whichever is deeper, and replacing them with properly compacted, low plasticity/elasticity fill soils.

A more detailed exploration and laboratory testing should be performed, once site layout and grading plans are developed, to evaluate the potential for swell of the high plasticity clay and elastic silt soils and to provide detailed recommendations for remediation.

A thorough field evaluation should be performed by a representative of the geotechnical engineer at the time of construction to further determine the presence of high plasticity clay or elastic silt soils that may adversely affect the performance of the proposed structures and pavements.

In addition, it should be noted that, based on our previous experience, high plasticity clays and elastic silts are typically very sensitive to moisture variations and tend to break down under construction traffic when left exposed at proposed subgrades. Therefore, we recommend providing and maintaining proper site drainage during and after construction and limiting construction traffic in areas where these materials are present at or near the proposed subgrade elevations. Excessive construction traffic on these soils prior to construction of the proposed structures or pavements may result in damage to the subgrade and the need for undercutting and/or repairs. We also recommend that grading operations take place during the typically drier, warmer periods of the year, if practical.

4.2.3 Low-Consistency Soils

Results from the soil test borings performed at the site indicate that lower consistency (N-values less than 7 bpf) residual soils are present in Borings B-01 through B-03. The lower consistency soils were encountered at various depths below the existing ground surface. N-values obtained in the lower consistency soils ranged from weight of hammer (WOH) to 6 bpf. Depending on the final design grades, if the lower consistency residual soils are present in the near-surface, some undercutting, re-working, or stabilization may be required. Remediation recommendations can typically be developed at the time of construction through routine engineering evaluations.

The presence of lower consistency soils may lead to excessive settlement and long term structure, slab-on-grade, and/or pavement distress. The presence and depth of the lower consistency soils were considered in the development of recommendations provided in subsequent sections of this report.

4.2.4 Difficult Excavation

Based on the results of the soil test borings, it appears that the majority of the general excavation will be in very soft to stiff and medium dense residual soil. We anticipate that the residual soils can be excavated using pans, scrapers, backhoes and front end loaders. Boring B-02 indicated that PWR was encountered at a depth of approximately 33.5 feet below the existing ground surface. In addition, auger refusal was encountered in Boring B-02 at a depth of approximately 37 feet below existing ground surface. Due to the depth of the partially weathered rock, intermittent rock lenses, bedrock and/or boulders are not likely to be encountered during general site grading and excavation for the installation of footings and utilities.

The depth to, and thickness of, PWR and rock lenses or seams, can vary dramatically in short distances and between boring locations; therefore, PWR or bedrock may be encountered during construction at locations or depths between boring locations, not encountered during this exploration. Additional information regarding excavation conditions and definitions are included in subsequent sections of this report.

4.2.5 Site Preparation

The entire building and pavement areas should be stripped of all topsoil, high plasticity near surface soils, trash, debris and other organic materials to a minimum of 10 feet and 5 feet beyond the structural and pavement limits, respectively. It has been our experience that stripping depths of topsoil may vary from the depths recorded on the Test Boring Records due to variability between boring locations. Deeper stripping may be required to adequately remove rootmat and stumps from wooded sites and may be dependent on surface conditions at the time of grading, such as wetter conditions during winter months. It is often desired by project owners to place topsoil/strippings in non-structural areas of the site, such as in over-built slopes or buried in on-site borrow pits. If on-site topsoil disposal is considered, the geotechnical engineer should be consulted to provide additional analysis and recommendations, as needed in this regard.

Upon completion of the stripping operations, the exposed subgrade in areas to receive fill should be proofrolled with a loaded dump truck or similar pneumatic tired vehicle (minimum loaded weight of 20 tons) under the observation of a representative of the geotechnical engineer. The proofrolling procedures should consist of complete passes of the exposed areas, with half of the passes being in a direction perpendicular to the preceding ones. After excavation of the site has been completed, the exposed subgrade in cut areas should also be proofrolled as previously described. Any areas which deflect, rut or pump excessively during proofrolling or fail to improve sufficiently after successive passes should be undercut to suitable soils and replaced with structural fill.

Unsuitable soils may be encountered between the borings during site grading or excavation for foundations. Some undercutting of the soft near surface soils in various portions of the site, as well as the areas where high plasticity clay or elastic silt soils are present within the upper 3 feet of subgrade or the bearing surface should be anticipated. The extent of the undercut required should be evaluated in the field by an experienced representative of the geotechnical engineer while monitoring construction activity. The evaluation should consist of a comprehensive proofrolling program and thorough field evaluation during construction. After the proofrolling operation has been completed and approved, final site grading should proceed immediately. If construction progresses during wet weather, the proofrolling operation should be repeated with at least one pass in each direction immediately prior to proceeding with site grading. If unstable conditions are exposed during this operation, then undercutting should be performed.

4.2.6 Fill Material and Placement

All fill used for site grading operations should consist of a clean (free of organics and debris), low plasticity soil (Plasticity Index less than 30). The proposed fill should have a maximum dry density of at least 90 pounds per cubic foot as determined by a Standard Proctor Moisture-Density Relationship test, ASTM D 698. All fill should be placed in loose lifts not exceeding 8 inches in thickness and compacted to a minimum of 95 percent of its Standard Proctor maximum dry density, with at least 100 percent achieved in the upper 12 inches. We recommend that field density tests, including one-point Proctor verification tests, be performed on the fill as it is being placed at a frequency determined by an experienced geotechnical engineer to verify the compaction criteria. Any fills that may be constructed greater than 10 feet in height should be evaluated with regard to long term settlement, consolidation and slope stability. This analysis should be requested of the geotechnical engineer once grading plans are complete and available.

Based on the results of the soil test borings and our past experience with similar type materials, the residual soils encountered, except for the high plasticity clay and elastic silt soils, appear suitable for re-use as structural fill. High plasticity clay and elastic silt soils may be used in deep fill areas (more than 5 feet of fill) or in landscaped areas provided they can be manipulated and properly compacted. As with any grading operation, moisture conditioning of the fill soils may be required.

4.2.7 Cut and Fill Slopes

For landscaping and mowing concerns, final project slopes should be designed to be 3 horizontal to 1 vertical or flatter. Slopes can be designed as steep as 2 horizontal to 1 vertical; however, soil erosion, slope sloughing and slope maintenance should be expected. If designing slopes steeper than 3 horizontal to 1 vertical, a slope stability analysis should be performed to verify stability of the slope. The tops and bases of all slopes should be located a minimum of 10 feet from structural and 5 feet from pavement limits. The fill slopes should be adequately compacted as outlined within this report, and all slopes should be seeded and maintained after construction.

4.2.8 Temporary Excavations

Excavations greater than four feet in depth should be sloped or shored in accordance with local, state, and federal regulations, including OSHA “Construction Standard for Excavations” (29 CFR Part 1926.650-652).

The contractor is usually solely responsible for site safety. This information is provided only as a service and under no circumstances should ESP be assumed to be responsible for construction site safety.

4.3 Foundation Support

The foundation for any structure must satisfy two inter-dependent design criteria for satisfactory performance. First, it must have an acceptable factor of safety against bearing failure of the foundation soils under the maximum design loads. Second, the settlement of the foundations due to consolidation of the underlying soils should be within tolerable limits for the structures.

The exploration indicates the existing, erratically-compacted fill soils are not suitable for shallow foundation support without experiencing excessive settlements. Therefore, we recommend the fill be undercut and replaced with well-compacted structural fill or ground improvement with stone columns be used to support the structure. Undercutting and replacement would require removal and replacement of a large volume of material. Ground improvement with stone columns is typically designed and installed by specialty contractors, and this option is discussed if further detail below.

4.3.1 Shallow Foundations

The exploration indicates the proposed structure can be supported with shallow foundations if the existing fill soils are undercut and replaced with suitable structure fill or if ground improvement is performed. If the existing fill is removed and replaced structure fill compacted to at least 95 percent of the soil's standard Proctor maximum dry density, a net allowable bearing pressure of up to 2,500 pounds per square foot (psf) can be used for design of the foundations. If ground improvements with stone columns is used, bearing pressures of 4,000 to 5,000 psf will likely be available, but this must be determined by the ground improvement design. Regardless of which method is used, minimum wall and column footing dimensions of 18 and 24 inches, respectively, should be maintained to reduce the possibility of a localized, punching-type shear failure. Exterior foundations and foundations in unheated areas should be designed to bear at least 18 inches below finished grade for frost protection.

Based on the general stratigraphy in the building area, past experience with similar projects and the anticipated magnitude of the building loads, it is our opinion that the total and differential settlement potentials for the building should be on the order of 1 inch and ½ inch, respectively. This conclusion is contingent upon the undercut and replacement of the existing fill in general compliance with the site preparation and fill placement recommendations outlined in this report.

We recommend the undercut of the unsuitable fill soils and placement of new suitable structural fill be observed by a representative of the geotechnical engineer prior to foundation installation. This is to assess their suitability for foundation support and confirm their consistency with the conditions upon which our recommendations are based.

The subgrade materials can be sensitive to moisture variations; therefore, foundation excavations should be opened for a minimum amount of time, particularly during inclement weather. Soils exposed to moisture variations may become highly disturbed and require undercutting prior to placing foundations.

4.3.2 Stone Columns

It may be more economical to leave the existing fill in place and performed ground improvement with stone columns to support the proposed structure. Stone columns can improve and stiffen the foundation bearing soils within the improved depth where the implied foundation stresses are predominate allowing the structure to be supported on conventional shallow spread footings. These systems provide intermediate foundation support by increasing the soil support strength and reducing settlement potentials within the upper improved soil profile. Typically, this ground improvement technique can improve the bearing soils to provide net allowable bearing pressures ranging from 4,000 to 5,000 psf at the project site where stone columns are used to reinforce the subsurface conditions and limit total and differential settlements to 1 inch and ½ inch, respectively.

The following recommendations should be considered prior to design and construction.

- The specialty contractor should consult the structural engineer to determine the target bearing pressure to be achieved and acceptable total and differential settlement limits.
- The specialty contractor should design the stone columns with proper depth, spacing, and other details based on the soil conditions and project requirements and prepare specifications for installation. The design and specification should be submitted to the structural engineer and geotechnical engineer for review and approval.
- At least one demonstration stone column should be installed using the contractor's proposed procedures and then load-tested to determine the composite modulus of the improved ground. The demonstration column should be installed at the foundation grade level. The geotechnical engineer should participate in the testing program.
- An engineer working for the specialty contractor should perform calculations to show the design assumptions, including soil modulus, have been verified through the test program. Additional stone columns should be installed and tested if the test column fails to meet the design requirements.
- The geotechnical engineer should be retained to monitor the installation of all production piers to maintain continuity.

After the foundation soils have been improved with stones columns, adequate protection of the reinforced ground is required. This includes proper drainage to eliminate ponding water and maintaining excavation distance from the installed columns. Prior to foundation installations, the reinforced ground surface should be cleared and cleaned to the satisfaction of the geotechnical engineer, and shallow foundations installed at design bearing levels.

Stone columns 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 stone columns in building area to develop bearing pressures and limit settlements to levels acceptable to the structural engineer. 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.

We recommend that the ground improvement installer's quality control (QC) program be monitored full-time by the project geotechnical consultant. The QC program includes observing the installation of the system to verify the installation process is in accordance with the designer's installation procedures.

4.4 Seismic Considerations

South Carolina adopted the 2018 edition of the International Building Code (IBC 2018) on January 1, 2020. The IBC 2018 references ASCE 7-16, which includes revised seismic hazard provisions, for determination of design loads.

4.4.1 Site Class

The seismic site classification was determined pursuant to Section 1613.2.2 of the IBC 2018 and Chapter 20 of ASCE 7 using the soil test boring data and our experience with Piedmont residual soils. Based on our review of this information, it is our opinion the seismic Site Class is D.

4.4.2 Ground Motions

Table 1 presents ground motions for this site.

Table 1 – Ground Motion Parameters

Site Class	S_S	S_1	F_a	F_v	PGA_M	S_{DS}	S_{D1}
D	0..237g	0.089g	1.6	2.4	0.194g	0.252g	0.142g

4.4.3 Geotechnical Seismic Hazards

Geotechnical seismic hazards for commercial structures generally include soil strength lose (i.e., liquefaction), settlements, and lateral ground movement. Based on the soils encountered at this site and our past experience with the site soils, liquefaction and lateral spreading are not expected to occur at this site as a result of the design IBC seismic event.

4.5 Slab-On-Grade

The slab-on-grade should be completely isolated from the structural components to allow independent movements between the slab and the foundations of the structure. The slab-on-grade floor system can be adequately supported on the low-plasticity residual/native soils or newly compacted fill, provided the site preparation and fill placement procedures outlined in this report are implemented.

The need for a base material between the soil subgrade and the slab-on-grade is dependent on subgrade soil strength characteristics, variability of subgrade soil constituents and the free draining characteristics of the subgrade soils. The inclusion of a water vapor retarder beneath the floor slab is a design element based on the subgrade constituents and design use of the structure and floor covering systems. For design guidance, refer to ACI 360R Design of Slabs on Grade, ACI 302.1R-15 Guide for Concrete Floor and Slab Construction and ASTM E1643 Standard Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs.

Immediately prior to constructing the floor slabs, we recommend that the areas be proofrolled or otherwise evaluated to detect unstable, low consistency/relative density areas or areas that may have been exposed to wet weather or construction traffic. Areas that are found to be unstable or indicate low consistency/relative density during the evaluation should be undercut and replaced with adequately compacted structural fill. The evaluation should be performed by a representative of the geotechnical engineer.

4.6 Pavements

We recommend that special care be given to providing adequate drainage away from pavement areas to reduce infiltration of surface water to the base course and subgrade materials in these areas. This is very important on this site due to the presence of high plasticity clay/elastic silt soils that have a high shrink/swell potential. If these materials are allowed to become saturated during the life of the pavement section, then there will be a strength reduction of the materials that could result in a reduced life of the pavement section. All water should be routed away from the pavement areas and adequate slopes provided to maintain drainage off site. Pavement areas should be proofrolled prior to placing structural fill and/or base course. Proofrolling procedures are outlined in subsequent sections of this report.

4.7 Site Retaining Walls

At the time of this report, the information provided to us did not include site retaining walls. Therefore, the scope of services and the information contained within this report are not intended, nor sufficient, for the design of retaining walls. If retaining walls are included in the proposed construction at this site, additional subsurface exploration is required. In addition, design of the retaining walls, including global stability analyses and analyses of other design criteria must be performed by the wall designer.

5.0 OTHER CONSIDERATIONS

5.1 Drainage

Soil strength and settlement potential is highly dependent upon the moisture condition of the supportive soil. Soil characteristics can change dramatically when moisture conditions change. As such, building pads, roadways, structures and surrounding grades should be properly designed and constructed to properly control water (surface and subsurface). Building pads should be designed to shed surface water prior to building construction. Grades surrounding structures should be adequately sloped away from the structure to promote positive drainage and prevent water from ponding near or against the structure. Swales and/or storm drainage structures should be constructed to collect and remove all surface water run-off. All roof drain downspouts should be connected to drain leaders that are properly daylighted or connected to storm drainage structures such that water is removed from structural areas. Foundation drains should be designed and constructed to properly protect foundations from changing moisture conditions. Foundation drains constructed should be properly daylighted or connected to storm drain structures to remove all water from foundation areas. Roof drain lines and foundation drain lines should always remain independent of each other. Any subsurface water that may rise near structural grades should be controlled by adequately constructed subsurface drainage mechanisms.

5.2 Effects of Construction Methods

Several aspects of construction at this site could adversely affect the adjacent streets, utilities and nearby facilities. Therefore, proper design and special care during construction will be needed to protect the adjoining properties. These items are discussed below.

Jackhammering, blasting, pile driving and other construction activities can generate vibrations that travel off-site. These vibrations can cause damage to adjacent structures if not properly controlled. Care must be taken to prevent damage of newly placed structures, especially fresh concrete. Any blasting charges that are used must be properly sized and timed to prevent structural damage. We recommend that vibration monitoring be performed for structures located nearby during the construction activities that generate a large amount of vibration. This will reduce the potential for large magnitude vibrations and subsequent damage claims.

General site dewatering can sometimes cause settlement of adjacent structures due to an increase in effective stresses which can consolidate soils. Based on the available data, we anticipate that this will generally not be a problem at this site. However, pumping of fine soil particles due to improper dewatering techniques can result in unwanted subsidence. Therefore, proper dewatering systems, if required, should be implemented to reduce these effects.

5.3 Temporary Base Stability

Based on our experience with similar subsurface conditions and construction activities, we anticipate that soils at the bottom of the deeper excavations may soften or loosen prior to completing foundation construction. Hydrostatic pressures, construction equipment and construction traffic among other factors, can be contributing factors to reducing the allowable bearing capacity at the exposed bearing elevation. We recommend that construction traffic be kept off the base of the excavations. Upon completion of the excavations for the coroner’s office building, we recommend placing a “working mat” at the bottom of the excavation on soils suitable for foundation support. The “working mat” should consist of washed stone such as NCDOT #57, wrapped and overlapped in a non-woven needle punched geotextile such as Tencate Mirafi® 140N or approved equivalent. Overlapping the fabric from the exterior edges of the stone mat to beneath the foundations should be adequate. The working mat should extend a minimum of 2 feet beyond the edge of the foundation area. The working mat should be at least 12 inches thick and is dependent on the exposed conditions.

5.4 Excavation Conditions and Definitions

It has been our past experience in this geologic area that materials having Standard Penetration Resistances of less than 50 blows per 0.4 foot can generally be excavated using pans and scrapers by first loosening with a ripper attached to a suitable sized dozer such as a Caterpillar D-8. On earthwork projects requiring ripping, questions sometimes develop as to whether the materials can be removed by ripping or whether blasting is required. It should be noted that ripping is dependent upon finding the right combination of equipment and techniques used, as well as the operator’s skill and experience. The success of the ripping operation is dependent on finding the proper combinations for the conditions encountered. Excavation of the weathered rock is typically much more difficult in confined excavations. Jackhammering or blasting is anticipated to be required for materials having Standard Penetration Resistances in excess of 50 blows per 0.2 foot.

We recommend that materials requiring blasting or hammering to remove be well defined in the project specifications and/or construction contract documents. Below are recommended definitions for “rock.” Please note the definition below for boulders regarding difficult excavation is different to the USCS definition of boulders regarding soil classification.

> Mass Rock: Material that cannot be dislodged by a Caterpillar D-8 Bulldozer, or equivalent, equipped with a single tooth ripper.
>
> Trench Rock: Material that cannot be dislodged by a Caterpillar 320 hydraulic backhoe, or equivalent, equipped with a rock bucket.
>
> Boulders: Masses of rock exceeding 1 cubic yard in volume for mass excavations and ½ cubic yard in volume for trench excavations shall also be considered mass or trench rock, respectively, during excavation.

These classifications are for information purposes only and are not considered contractual definitions unless referenced as such by the project plans and/or contract documents. The classifications do not include materials such as loose rock, concrete, or other materials that can be removed by means other than impact hammering, but which for any reason, such as economic reasons, the contractor chooses to remove by impact hammering.

We also recommend that quantification guidelines for payment purposes be established prior to removal of materials defined above. These guidelines should include the following measurements to be used during quantity calculations:

- The depth below proposed subgrade for mass rock.
- The depth below proposed utility design depth for trench rock.
- The width on each side of the utility for trench rock.

These guidelines should establish a base line for payment and should be completely independent of the means and methods of the contractor.

6.0 LIMITATIONS of REPORT

This report has been prepared in accordance with generally accepted geotechnical engineering practice with regard to the specific conditions and requirements of this site. The conclusions and recommendations contained in this report were 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 analysis and recommendations submitted herein are based, in part, upon the data obtained from the subsurface exploration. The nature and extent of variations between the borings will not be known until construction is underway. If variations appear evident, then we request the 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 will not be considered valid unless the changes are reviewed and conclusions modified or verified in writing by ESP.

In order to verify that earthwork and foundation recommendations are properly interpreted and implemented, we recommend that ESP be provided the opportunity to review the final plans and specifications. Any concerns observed will be brought to our client’s attention in writing.

Our conclusions and recommendations are based on the project information previously discussed and on the data obtained from the field and laboratory testing program. If the structural loading, geometry or proposed building locations are changed or significantly differ from those discussed, or if conditions are encountered during construction that differ from those encountered by the borings, ESP requests the opportunity to review our recommendations based on the new information and make any necessary changes.

FIELD EXPLORATION

Soil Test Boring: Three (3) soil test borings were drilled at the approximate locations shown on the attached Boring Location Plan, Figure 1. Soil sampling and penetration testing were performed using general guidance from ASTM D 1586.

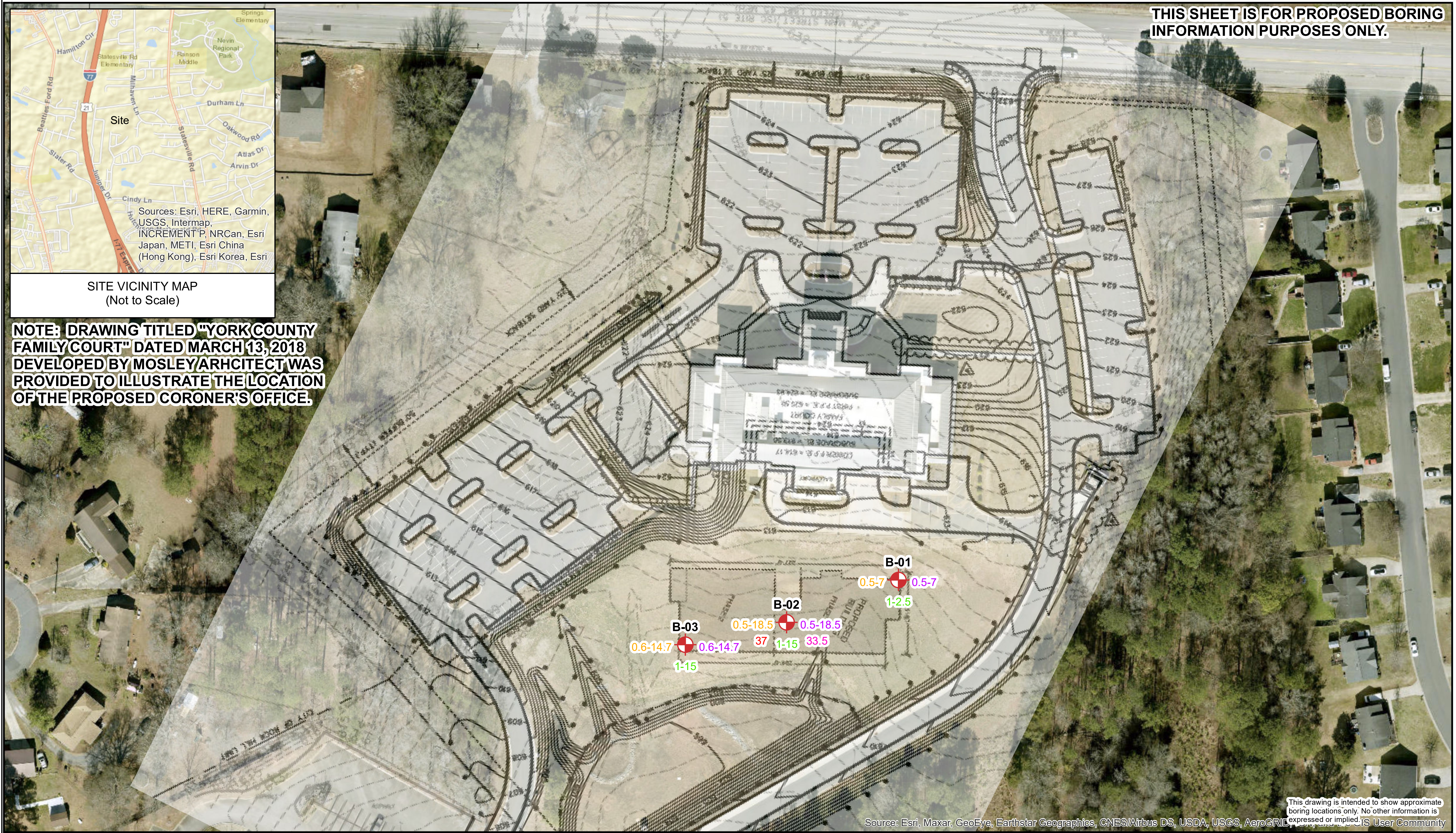
The borings were advanced with hollow-stem augers and, at standard intervals, soil samples were obtained with a standard 1.4-inch I.D., 2-inch O.D., split-tube sampler. The sampler was first seated six (6) inches to penetrate any loose cuttings, then driven an additional foot with blows of a 140-pound hammer falling 30 inches with the exception of penetration restrictions. The sum of the last foot of hammer blows is designated the “Standard Penetration Resistance.” Standard Penetration Tests were performed within the soil test borings utilizing an automatic hammer attached to the referenced drill rig(s) utilized in this exploration. The Standard Penetration Test values shown on the “Test Boring Records” have not been corrected for theoretical energy or depths adjustments. When properly evaluated, the Standard Penetration Resistances provide an index to soil strength, relative density, and ability to support foundations.

Select portions of each soil sample were placed in sealed containers and taken to our office. The samples were examined by a representative of the geotechnical engineer for classification. Test Boring Records are attached showing the soil descriptions and Standard Penetration Resistances.

LABORATORY PROCEDURES

Grain Size Test: Grain size tests were performed to determine the particle size and distribution of the samples tested. The grain size distribution of soils coarser than a No. 200 sieve was determined by passing the samples through a set of nested sieves. This test was conducted using general guidance from ASTM D 421 and 422. The results are presented on the attached Grain Size Distribution Sheets.

Soil Plasticity Tests (Atterberg Limits Test): Select samples were identified for Atterberg Limits testing to determine the soil's plasticity characteristics. This test was conducted using general guidance from ASTM D 4318. The Plasticity Index (PI) is representative of this characteristic and is determined utilizing the Liquid Limit (LL) and the Plastic Limit (PL). The Liquid Limit is the moisture content at which the soil will flow as a heavy viscous. The Plastic Limit is the moisture content at which the soil transitions between the plastic and semi-solid states. The data obtained is presented on the attached Atterberg Limits Results sheet.



Legend

Approximate Boring Location

Boring Labels

High Plasticity/Elasticity Soil Depth (ft)
Low Consistency Soil Depth (ft)
Partially Weathered Rock Depth (ft)
Auger Refusal Depth (ft)
Fill Soil Depth (ft)

SHEET TITLE:

Test Location Map with Site Vicinity Map
Figure 1

York County - Coroner's Office
Rock Hill, SC

The reproduction, alteration, copying, or other use of this drawing without consent is prohibited and any infringement will be subject to legal action.

N

DATE: *December 08, 2022*

PROPOSAL NO.: *E4-KW23.300*
SCALE: *NTS*
DRAWN BY: *VAD*
CHECKED BY: *CJB*

ESP
3475 Lakemont Blvd.
Fort Mill, SC 29708
www.espassociates.com
Phone: 803-802-2440

LEGEND TO SOIL CLASSIFICATION AND SYMBOLS

ABC Stone	Asphalt/Concrete
Concrete/Brick Debris	Coquina Shell Base Course
Topsoil/Rootmat	Topsoil/Grassmat
Topsoil	Wood and Roots
High Plasticity Clay	Moderate Plasticity Clay
Clay	Clayey Silt
Elastic Silt	Silt
Organic Clay	Organic Silt
Organic Silt and Clay	Peat
Poorly Graded Gravel	Poorly Graded Gravel with Silt
Poorly Graded Gravel with Clay	Clayey Gravel
Silty Gravel	Poorly Graded Gravel with Silt and Clay
Well Graded Gravel	Well Graded Gravel with Silt
Well Graded Gravel with Clay	Silty Sand
Poorly Graded Sand	Poorly Graded Sand with Clay
Poorly Graded Sand with Silt	Well Graded Sand
Well Graded Sand with Clay	Well Graded Sand with Silt
Partially Weathered Rock	Cored Rock

LEGEND TO SOIL CLASSIFICATION AND SYMBOLS

SAMPLER TYPES

(Shown in Samples Column)

Shelby Tube

Split Spoon

Rock Core

No Recovery

CONSISTENCY OF COHESIVE SOILS

CONSISTENCY	STANDARD 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

WATER LEVELS

= Water Level at Boring Termination

= Water Level at 1 Day

= Loss of Drilling Fluid

HC = Hole Cave

CONSISTENCY OF COHESIONLESS SOILS

CONSISTENCY	STANDARD 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

TERMS

Standard Penetration Resistance - The number of blows it takes a 140 lb. hammer falling 30 in. to drive a 1.4 in I.D. split spoon sampler 1 foot (N-Value) as specified in ASTM D-1586.

Dynamic Cone Penetrometer Test Data - The cone point is driven up to three 1 ¾ inch intervals using a 15-pound weight falling 20 inches. The penetrometer test result is the average number of blows per interval. The penetrometer test result is similar to the Standard Penetration Resistance (N-value), as defined by ASTM D 1586. When properly evaluated, the penetrometer test results provide an index for estimating soil strength and relative density.

Kessler Dynamic Cone Penetrometer Test Data – The cone point is driven using a 17.6-pound weight falling 22.6 inches. The total penetration for a given number of blows is measured and recorded in mm/blow as specified in ASTM D 6951. When properly evaluated, the penetrometer test result can be used to describe soil stiffness and estimate an in-situ CBR strength from an appropriate correlation chart.

REC - Total length of rock recovered in the core barrel divided by the total length of the core run times 100 (expressed as a percentage).

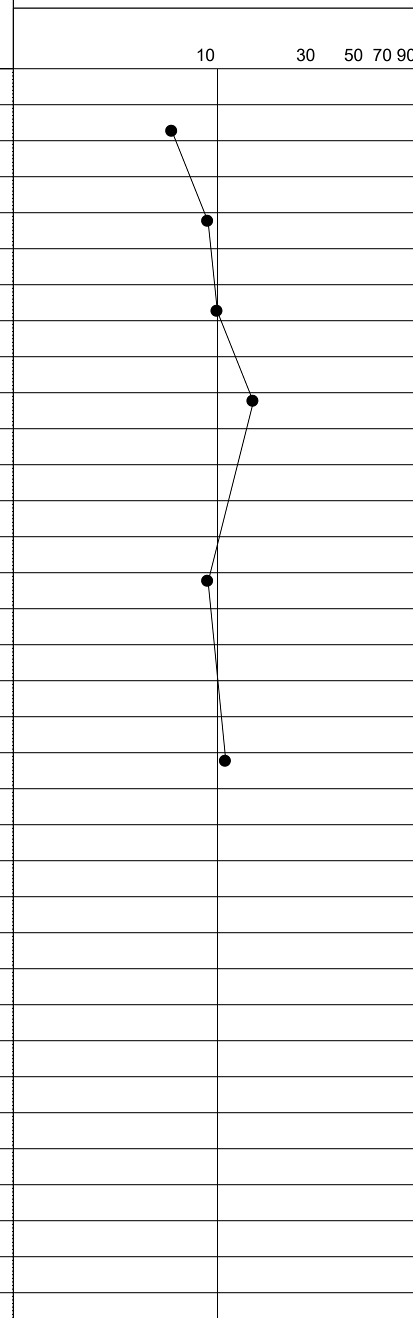
RQD - Total length of sound rock segments recovered that are longer than or equal to 4” (mechanical breaks included) divided by the total length of the core run times 100 (expressed as a percentage).

PROJECT: York County Coroner's Office, York, South Carolina	**TEST BORING RECORD B-1**

PROJECT No.:	**ELEVATION:**	**DRILLING METHOD:**	**AUGER I.D.:**	**DRILLING COMPANY:**
KW23.300	Existing Ground Surface	Hollow Stem Auger	2.25 in	CVET
LOGGED BY:	**BORING DEPTH:**	**DRILL RIG:**	**NOTES:**	
Nathan McLaren	20.0 Feet	Geoprobe 7822 DT	Backfilled at time of boring due to safety concerns.	
DATE DRILLED:	**WATER LEVEL:**			
11/21/22	Dry @ TOB	N/A		

DEPTH (ft)	SOIL DESCRIPTION	WATER LEVEL	ELEV. (ft)	BPF
0	Topsoil, Approximately 6 inches of topsoil.			
0.5	FILL: Firm To Stiff Grayish Brown to Orange Tan Sandy Elastic SILT, (moist)			6
5			-5.0	9
7	RESIDUUM: Stiff Tan and Orange Sandy SILT, (moist)			10
10			-10.0	15
15			-15.0	9
16.5		HC		
20	Boring was terminated at 20.0 feet. Cave-in depth at 16.5 feet. Backfilled with soil.		-20.0	11
25			-25.0	
30			-30.0	

STANDARD PENETRATION TEST DATA (Blows/ft)



DEPTH MEASUREMENTS ARE SHOWN TO ILLUSTRATE THE GENERAL ARRANGEMENTS OF THE SOIL TYPES ENCOUNTERED AT THE BOREHOLE LOCATIONS. DO NOT USE DEPTH MEASUREMENTS FOR DETERMINATION OF DISTANCES OR QUANTITIES.

ESP

PROJECT:	York County Coroner's Office York, South Carolina		**TEST BORING RECORD** **B-2**	
PROJECT No.: KW23.300	**ELEVATION:** Existing Ground Surface	**DRILLING METHOD:** Hollow Stem Auger	**AUGER I.D.:** 2.25 in	**DRILLING COMPANY:** CVET
LOGGED BY: Nathan McLaren	**BORING DEPTH:** 37.0 Feet	**DRILL RIG:** Geoprobe 7822 DT	**NOTES:** Backfilled at time of boring due to safety concerns.	
DATE DRILLED: 11/21/22	**WATER LEVEL:** Dry @ TOB / N/A			

DEPTH (ft)	GRAPHIC LOG	SOIL DESCRIPTION	WATER LEVEL	SAMPLE	ELEV. (ft)	STANDARD PENETRATION TEST DATA (Blows/ft) 10 30 50 70 90	BPF
		Topsoil, Approximately 6 inches of topsoil.					
		FILL: Very Soft To Firm Reddish Orange and Grayish Brown Sandy High Plasticity CLAY, few organics, (moist)					2
5					-5.0		7
							6
10					-10.0		WOH
15					-15.0		2
20		RESIDUUM: Stiff Gray and Orange Sandy CLAY, (moist)			-20.0		13
25		RESIDUUM: Very Stiff To Very Hard Tan and Gray Sandy SILT, (moist)			-25.0		26
30					-30.0		58
			HC				50/0.1
35		PARTIALLY WEATHERED ROCK: When Sampled Becomes Gray and Tan Silty SAND, fine to medium, (moist)			-35.0		
							50/0
		Auger refusal encountered at 37.0 feet. Boring was terminated at 37.0 feet. Cave-in depth at 32.7 feet. Backfilled with soil.					

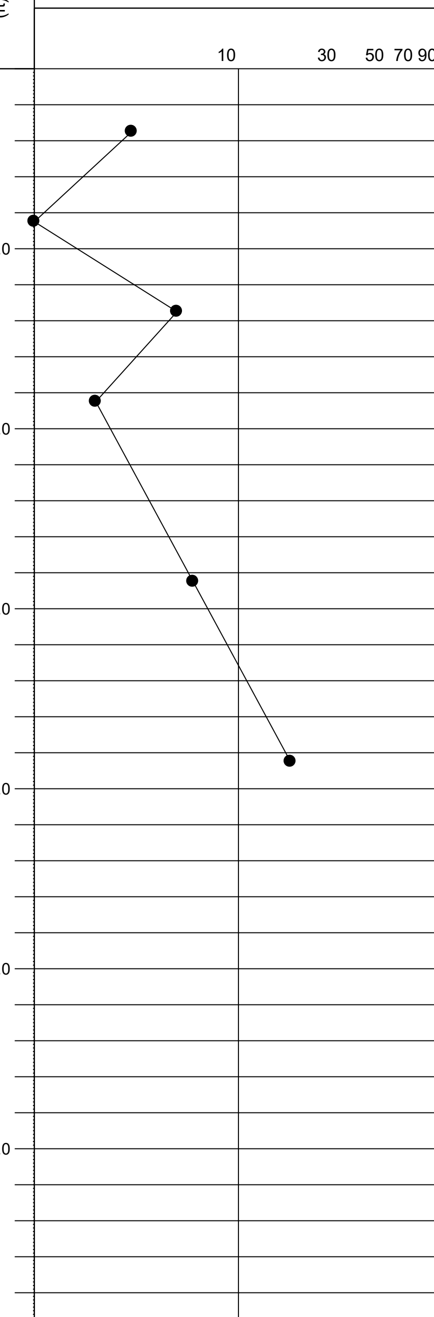
DEPTH MEASUREMENTS ARE SHOWN TO ILLUSTRATE THE GENERAL ARRANGEMENTS OF THE SOIL TYPES ENCOUNTERED AT THE BOREHOLE LOCATIONS. DO NOT USE DEPTH MEASUREMENTS FOR DETERMINATION OF DISTANCES OR QUANTITIES.

ESP

PROJECT:	York County Coroner's Office York, South Carolina			**TEST BORING RECORD** **B-3**	
PROJECT No.: KW23.300	**ELEVATION:** Existing Ground Surface	**DRILLING METHOD:** Hollow Stem Auger		**AUGER I.D.:** 2.25 in	**DRILLING COMPANY:** CVET
LOGGED BY: Nathan McLaren	**BORING DEPTH:** 20.0 Feet	**DRILL RIG:** Geoprobe 7822 DT		**NOTES:** Backfilled at time of boring due to safety concerns.	
DATE DRILLED: 11/21/22	**WATER LEVEL:** Dry @ TOB	N/A			

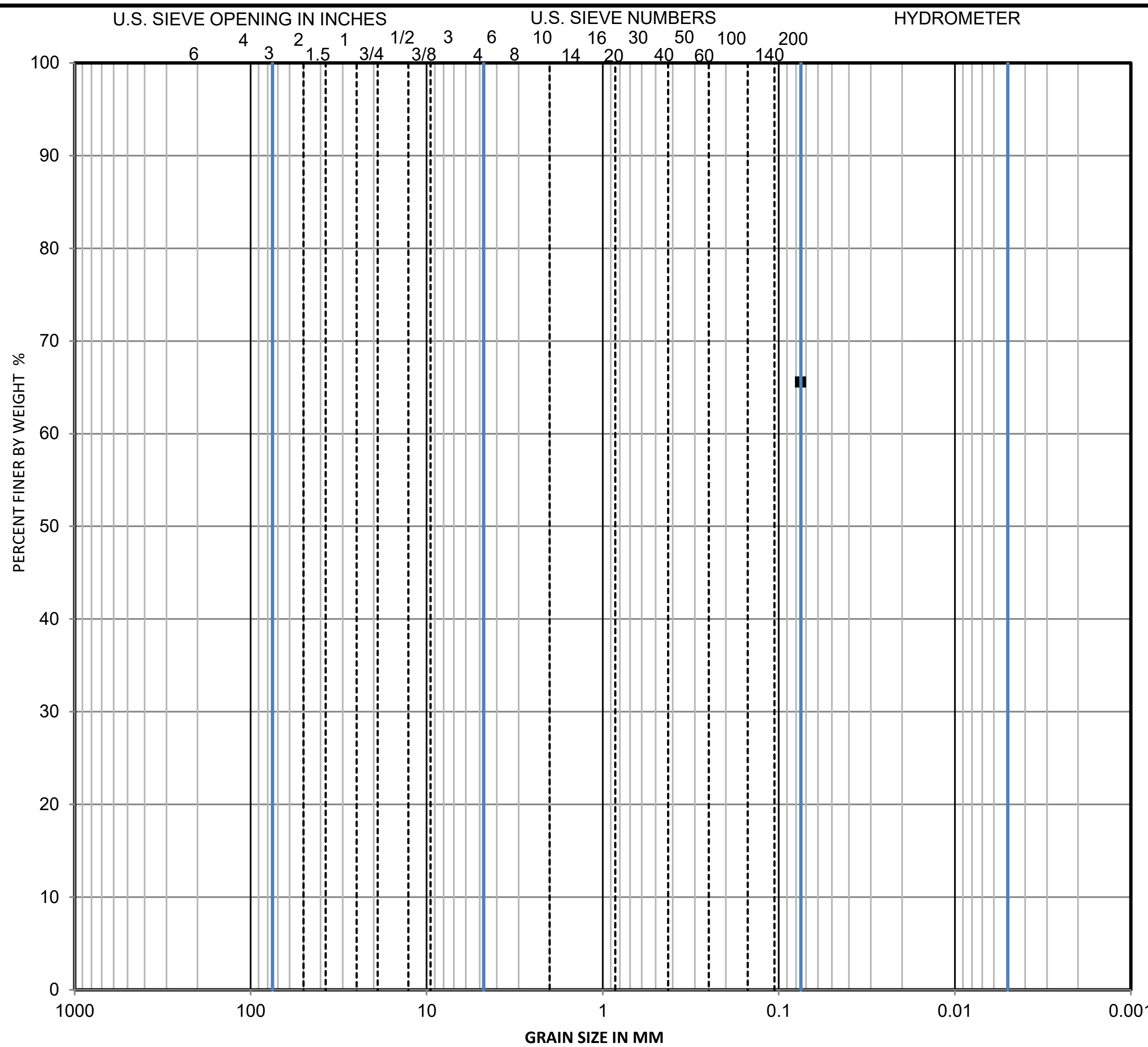
DEPTH (ft)	SOIL DESCRIPTION	WATER LEVEL	ELEV. (ft)	BPF
0	Topsoil, Approximately 7 inches of topsoil.			
0.6	FILL: Very Soft To Firm Red and Tan Sandy High Plasticity CLAY, (moist)			3
				WOH
5			-5.0	5
8.5	FILL: Very Soft To Firm Brown and Tan Sandy Elastic SILT, few organics, (moist)			2
10			-10.0	
				6
15	RESIDUUM: Firm To Very Stiff Gray and Orange Sandy CLAY, (moist)	HC	-15.0	
19	RESIDUUM: Medium Dense Gray and Tan Clayey SAND, fine to medium, (moist)			18
20	Boring was terminated at 20.0 feet. Cave-in depth at 16.3 feet. Backfilled with soil.		-20.0	
25			-25.0	
30			-30.0	

STANDARD PENETRATION TEST DATA (Blows/ft)



DEPTH MEASUREMENTS ARE SHOWN TO ILLUSTRATE THE GENERAL ARRANGEMENTS OF THE SOIL TYPES ENCOUNTERED AT THE BOREHOLE LOCATIONS. DO NOT USE DEPTH MEASUREMENTS FOR DETERMINATION OF DISTANCES OR QUANTITIES.

ESP



COBBLE	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification	Classification	LL	PL	PI	Cc	Cu
■	**B-2, S-1 (3.5' - 5')**	**Sandy fat clay**	**56**	**24**	**32**	**0**	**0**

	Specimen Identification	D100	D60	D30	D10	% Gravel	% Sand	% Silt	% Clay
■	**B-2, S-1 (3.5' - 5')**	**0.1**						**65.5**	

GRAIN SIZE DISTRIBUTION

ESP

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Telephone: 803-802-2440

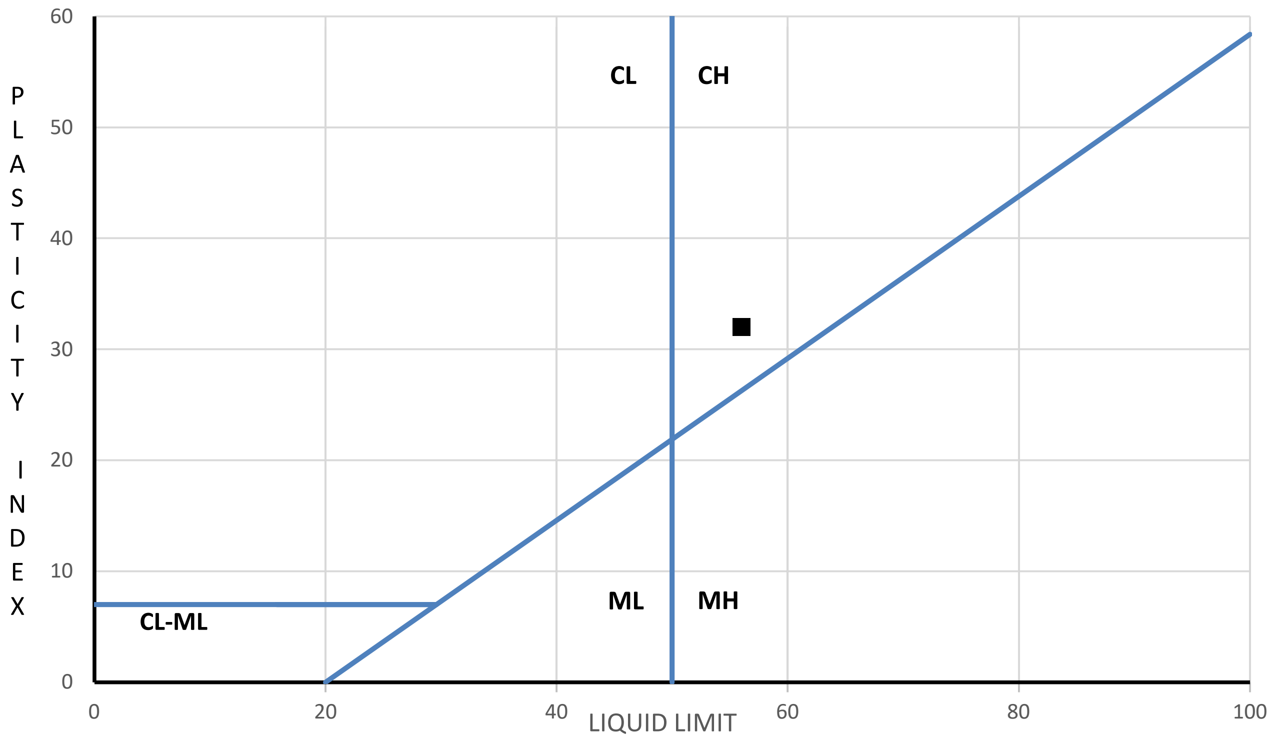
Project: York County - Coroner's Office

Number: KW23.300.000

Lab Technician tsummers

Project Manager: vayala

The test results shown are specific to the specimen/sample numbers tested, as noted above.



	Specimen Identification	LL	PL	PI	Fines	Classification
■	**B-2, S-1 (3.5' - 5')**	**56**	**24**	**32**	**65.5**	**Sandy fat clay CH**

Remarks:

ATTERBERG LIMITS' RESULTS

TEST METHOD: ASTM D4318

ESP

Address: 3475 Lakemont Boulevard
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Telephone: 803-802-2440

Project: **York County - Coroner's Office**

Number: **KW23.300.000**

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