REPORT OF
GEOTECHNICAL EXPLORATION
PROPOSED DORN VAMC POLICE STATION

US Highway 378/Garners Ferry Road
Columbia, South Carolina
S&ME Project No. 1461-14-067

<u>Prepared For:</u>

BES Design/Build, LLC
1941 Savage Road, Suite 300-E
Charleston, South Carolina 29407

<u>Prepared By:</u>

S&ME, Inc.
134 Suber Road
Columbia, South Carolina 29210

December 15, 2014

December 15, 2014

BES Design/Building, LLC
1941 Savage Road, Suite 300-E
Charleston, South Carolina 29407

Attention: Mr. Tom Lodge

Reference: **REPORT OF GEOTECHNICAL EXPLORATION**
Proposed Dorn VAMC Police Station
US Highway 378/Garners Ferry Road
Columbia, South Carolina
S&ME Project No. 1461-14-067

Dear Mr. Lodge:

As requested, S&ME, Inc. (S&ME) has completed field testing for the proposed VAMC Police Station project site in Columbia, South Carolina. Our work was performed in general accordance with our proposal No. 14-1400851R, dated November 21, 2014. This report provides information on the exploration and testing procedures used, our boring records, and our recommendations regarding site preparation, suitability of on-site soils for use as structural fill, fill placement, foundation type, foundation design values, estimated settlement and IBC 2012 Seismic Site Class.

S&ME appreciates this opportunity to work with you as your geotechnical engineering consultant on this project. Please contact us at (803) 561-9024 if you have any questions or need any additional information regarding this report.

Sincerely,
S&ME, Inc.

SOUTH CAROLINA LICENSED PROFESSIONAL ENGINEER No. 26144 12-15-14 ROBERT C. BRUORTON

Robert C. Bruorton, P.E.
Senior Engineer

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Vice President/Technical Principal

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

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1.0 PROJECT INFORMATION

Information about the project was provided in a phone conversation and email from Tom Lodge with BES Design/Build, LLC to Trapp Harris with S&ME, Inc. on November 12, 2014. Included in the email was an undated CAD drawing showing the proposed building location and various site features.

The project consists of the construction of a new, approximately 7,000 square foot, single-story, police station on the Dorn VA Medical Center campus. The proposed structure is assumed to be constructed utilizing a wood framed wall and roof system with a brick exterior veneer and be supported by a shallow foundation system. Structural loads have not been provided as of the writing of this report. Based on previous experience with similar structures, we have assumed maximum column and wall loads of 50 kips and 3 kips per linear foot, respectively.

Proposed and existing site grade information was not available at the time of this report; therefore, we have assumed that cut and fill across the site will be on the order of 1 to 2 feet.

2.0 EXPLORATION PROCEDURES

The subsurface exploration of this project generally included Standard Penetration Test (SPT) borings. The approximate locations of each of the field tests detailed below are shown in the *Boring Location Plan* attached as Figure 2 in the Appendix.

2.1 Reconnaissance of the Project Area

On December 1, 2014, a representative from S&ME visited the site to observe current site conditions and lay out the proposed boring locations. Boring locations were marked in the field with orange pin flags, with the boring numbers inscribed. The boring locations indicated on the attached *Boring Location Plan* must be considered as approximate.

2.2 Field Testing and Sampling

Three soil test borings with SPT sampling and testing were performed on December 8, 2014.

The SPT soil test borings were performed using a truck-mounted drill rig. The borings were advanced using 2¼-inch inside diameter hollow-stem augers to depths of 20 feet below the existing ground surface. Split-spoon samples and Standard Penetration Test Resistance N-values were obtained at selected intervals in general accordance with ASTM D-1586. Representative samples of the soils obtained by the split-spoon sampler were collected and placed in suitably identified, sealed glass jars and transported to our laboratory.

A summary of our exploration procedures is included in the Appendix. All of the boreholes were backfilled prior to leaving the site.

3.0 SITE CONDITIONS

S&ME's assessment of the geotechnical conditions began with a reconnaissance of the topography and physical features of the site. We also consulted available topographic and geologic maps, as well as results of nearby explorations, for relevant information.

3.1 Surface Conditions

At the time of our exploration, the site of the proposed development consisted of a landscaped area within the northeastern portion of the Dorn VA Medical Center campus, between the existing asphalt paved parking lot/drives and U.S. Highway 378/Garners Ferry Road, between its intersections with Veterans Road and Dorn Drive.

The site was landscaped and is generally bordered by the existing asphalt paved parking/drives to the south, landscaped areas to the east and west and by U.S. 378/Garners Ferry Road to the north. Ground cover generally consisted of grass. Vehicular access to the site was available from Garners Ferry Road via the existing asphalt paved access drives.

Site topography appeared to be slightly sloping across the site from west to east by roughly 4 to 5 feet. Site drainage across the site appeared to be surface infiltration and sheet flow to the existing adjacent infrastructure and eventually into Reeder Point.

3.2 Site Geology

The site is located on one of several Tertiary-age river terraces that flank the Congaree River Valley both within and immediately south of the downtown Columbia area. Up to six distinct river terraces have been mapped in the Congaree Valley. The upper terraces, including areas adjacent to Columbia and West Columbia, formed when the Congaree River valley stood at higher elevations during the Miocene to Pliocene epochs. These relatively old terraces have been locally severely eroded and often have little surface expression.

Materials comprising the terraces typically consist of medium dense, coarse-grained oxidized red-brown clayey sands or stiff reddish sandy silts or clays with numerous rounded quartz pebbles embedded in the soil binder. The fine grained soil binder is typically well desiccated and oxidized and generally moderately preconsolidated. Under these conditions the soils are not highly compressible under light to moderate structural loads.

Beneath the terrace sediments, native Coastal Plain soils similar to the Red Sand Hills geographic province were encountered. The Red Sand Hills lie seaward and stratigraphically above the White Sand Hills to the northwest. It denotes an area underlain by Tertiary and Eocene age sediments of the Orangeburg or Black Mingo groups. Impervious interbedded red sandy clays or clayey sands, with multiple horizons of gravel or rounded pebbles and occasional thick, very tough iron-oxide cemented sands, claystones, or fullers earth seams, occur within a very complex stratigraphy locally termed the McBean Formation. Groundwater perched on top of these layers commonly occurs at shallow depth. The ground surface is gently rolling with few deeply incised stream channels and supports heavy forest cover. Major stream channels eroded through the strata typically create very steep, near vertical side slopes characteristic of cohesive, deeply weathered strata.

3.3 Subsurface Conditions

Recovered field samples and field boring records were reviewed in the laboratory by the geotechnical engineer. Soil test boring records and other field data are assembled in the Appendix.

The depth and thickness of the subsurface strata indicated on the soil test boring records and on other field records were generalized from and interpolated between test locations. The transition between materials will be more or less gradual than indicated and may be abrupt. Information on actual subsurface conditions exists only at the specific boring locations and is relevant to the time the exploration was performed. The stratification lines were used for our analytical purposes and, unless specifically stated otherwise, should not be used as the basis for design or construction cost estimates.

Top-of-ground elevations shown on the boring records were estimated from Google Earth. Boring locations and elevations shown on the attached drawings and elevations indicated in this report were not surveyed and should be considered approximate.

3.3.1 Surface Materials

Up to approximately 6 inches of topsoil were encountered at our boring locations. While these measurements are likely representative of the existing topsoil that will be encountered during construction, the potential exists that greater thicknesses may be encountered at other locations on the site.

3.3.2 Terrace Sediments

The borings across the site encountered roughly 10 feet of terrace sediments consisting of clayey sands (SC). Samples examined by hand consisted of fine to medium sands with little to some low plasticity fines. Standard penetration resistance values varied from 11 to 83 blows per foot (bpf) indicating medium dense to very dense relative densities. Most samples were moist to dry.

3.3.3 Coastal Plain Deposits

Below the terrace sediments, the borings across the site encountered Coastal Plain deposits generally consisting of clayey sands (SC) to termination depths. The exception was Boring B-3, which encountered poorly graded sands with silt (SP-SM) at a depth of roughly 15 feet that continued to the termination depth.

The clayey sand samples examined by hand consisted of fine to medium sands, with little to some low to medium plasticity fines. Standard penetration resistance values varied from 9 to 26 bpf indicating loose to medium dense relative densities. Most samples were moist to dry.

The sample examined by hand of the deep layer of sands with silt (SP-SM) encountered in Boring B-3 consisted of fine sands and few low plasticity fines. The standard penetration resistance value was on the order of 16 bpf indicating a medium dense relative density. The sample was moist to dry.

3.3.4 Ground Water

Ground water was not encountered in the borings at the time of drilling. However, at the completion of drilling, the borings had dry caved at depths of 15½ to 17 feet below the existing ground surface. Borings typically cave in poorly cohesive to cohesionless soils within 2 to 3 feet of the ground water table. Due to safety concerns with leaving the boreholes open overnight; our borings were backfilled at completion of drilling. Due to the lack of encountered ground water and the depth at which cave-in was encountered during exploration; it will likely not significantly impact proposed construction across the site.

We note that ground water levels are influenced by precipitation, long term climatic variations, and nearby construction. Measurements of ground water made at different times than our exploration may indicate ground water levels substantially different than indicated on the boring records in the Appendix.

4.0 BUILDING CODE SEISMIC PROVISIONS

Seismic induced ground shaking at the foundation is the effect taken into account by building code seismic-resistant design provisions. Other effects, such as soil liquefaction, are not addressed in building codes but must also be considered.

4.1 IBC Site Class

As of July 1, 2013, the 2012 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 7-10.

The initial step in site class definition is a 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 under item 1) including quick and highly sensitive clays or collapsible weakly cemented soils were not observed in the borings. Three other conditions, 2) peats and highly organic clays; 3) very high plasticity clays (H>25 feet); and 4) very thick soft/medium stiff clays were also not evident in the borings performed.

The remaining vulnerability, liquefaction, appears unlikely at this site due to the age, density and fines content of the soils encountered.

Soil test boring data within the limits of the proposed structure on-site extends to a depth of about 20 feet. Based on the soil test boring data and our knowledge of the general geologic profile of this area, **Site Class D** appears to generally represent conditions and may be assumed for design of the facility as planned.

4.2 Design Spectral Values

S&ME determined the spectral response parameters for the site using the general procedures outlined under the 2012 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 2012 IBC seismic provisions of Section 1613 use the 2008

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 in IBC Section 1613.3.3, tables 1613.3.3(1) and 1613.3.3(2). 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 2012 criteria.

The 2012 IBC specifically references ASCE 7-10 for determination of peak ground acceleration value for computation of seismic hazard. Peak ground acceleration is separately mapped in ASCE 7-10 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.

Table 1: Spectral Design Values

	2012 IBC (2008 Seismic Hazard Maps)
S_{DS}	0.416 g
S_{D1}	0.215 g
PGA_M	0.31 g

Under the 2012 IBC, for a structure having a Seismic Use Group classification of I, II, or III, spectral response acceleration factors given above correspond to **Seismic Design Category D**.

5.0 RECOMMENDATIONS

The soil profile encountered at this site appears generally suitable for the proposed development. The following paragraphs include our conclusions and recommendations for site preparation, suitability of on-site soils for use as structural fill, fill placement and design and construction of foundations.

5.1 Site Preparation

Site preparation should include removal of all unsuitable surface materials within the building footprint. This should include surface vegetation, organic laden topsoil, stumps, root bulbs and any unstable/unsuitable surface or subsurface soils encountered during site grading activities.

5.1.1 Existing Utilities

Remove or plug any existing utilities to be abandoned prior to construction. If not removed or plugged, pipes may serve as conduits for subsurface erosion resulting in formation of voids below buildings or pavements. Where existing utilities are left in place and plugged in the building footprint, it may be necessary to undercut poorly compacted backfill to provide adequate support for footings or slabs.

5.1.2 Surface Preparation/Proofrolling

After clearing and stripping is complete, all areas at grade or to receive fill should be evaluated by the geotechnical engineer or a representative of the geotechnical engineer to confirm that the exposed subgrade is suitable for support of slabs. To aid in evaluation of the exposed soils, the area should be proofrolled using a loaded dump truck or similarly loaded piece of equipment. Areas that rut, pump, or move excessively under movement of the equipment should be undercut to firm materials prior to placement of new fill soil, concrete, or base course stone. If left in place, soft or wet soils will exhibit substantially lower bearing for foundations.

5.2 Fill Placement and Compaction

Before beginning to place fill, sample and test each proposed fill material to determine maximum dry density, optimum moisture content, natural moisture content, gradation and plasticity of the soil.

The existing terrace sediments, where excavated on the site, appear generally suitable for reuse as structural fill. These soils may be placed in lifts and compacted as described below. However, any organic or debris laden soils must not be incorporated into structural fill, but should be segregated by the contractor and wasted from construction areas. The underlying Coastal Plain soils, if encountered, also appear acceptable for use as compacted fill based on visual examination; however, classification, moisture content and compaction tests should be performed to verify this assessment.

5.2.1 Density and Moisture Requirements

Place new fill in maximum 8-inch loose lifts and compact to at least 98 percent of maximum dry density (ASTM D-698 Standard Proctor). Fill moisture content should be maintained within +/- 3 percent of the optimum moisture content. Contractor should be prepared to wet or dry soils as necessary to achieve compaction. In addition to meeting the compaction requirement, fill material should be stable under movement of the construction equipment and should not exhibit rutting or pumping.

5.2.2 Compaction of Granular Soils (SC)

A vibratory sheeps-foot roller will likely be effective for compaction of the clayey sandy soils (SC) encountered at the site. Sheeps-foot compactors will likely be preferable because the pads better penetrate the soil and they tend to break down the natural cohesive bonds between the particles. Pneumatic tire compactors can also be used but will likely be better suited only where the soils have a low to medium plasticity index.

5.2.3 Monitoring and Testing

Fill placement should be witnessed by an experienced soils technician working under the guidance of the geotechnical engineer. Conduct at least one field density test every 2,500 square feet for each 1 foot of fill lift in mass grading and for each 50 cubic feet of fill placed in confined areas such as wall backfill or trenches.

5.2.4 Wet Weather Grading

Based on our experience, low to medium plasticity clayey sands similar to those encountered in our borings can be difficult to work if allowed to become wet and may also require extended drying times. The grading contractor should take measures so that periodic rain does not significantly affect grading. This includes diverting rainwater runoff away from the construction area and sealing the ground surface with a smooth drum roller to help prevent rainwater from migrating below the surface soils.

Our experience indicates that allowing heavy equipment to run on the existing ground surface will result in heavy rutting. Running heavy equipment on previously placed fill during rain events or where water is ponded will result in degradation of the fill. If these conditions are evident or persist and routinely cause issues, then during construction, gravity-drained surface ditches should be installed around the site to promote surface runoff. Ditches should have at least 6 inches of relief per 100 feet of length to facilitate flow.

5.3 Foundation Design and Construction

Based on our boring data and experience in the area, assuming the site preparation recommendations provided above have been followed, shallow foundations appear suitable for support of the proposed structure. We estimated bearing capacities for typical spread footings and wall footing configurations and dimensions using our boring data and our experience with similar soils under similar loading conditions. Estimated ultimate bearing capacity exceeds recommended allowable bearing pressures by a safety factor of at least 3 on level ground, provided that footings are designed and constructed as outlined in this report. The following represents our geotechnical recommendations regarding structural support.

5.3.1 Allowable Bearing Pressure

Assuming proper design and construction of the proposed footings, a net bearing pressure of 3,000 pounds per square foot (psf) or less is recommended for individual spread footings or wall footings bearing on natural soils similar to those encountered in our borings or well-compacted fill soils placed and compacted as recommended in Section 5.2 of this report. Excavated footings should be examined by the geotechnical engineer or representative of the geotechnical engineer prior to placement of concrete to determine that variations in the soil do not lower the allowable bearing capacity. It may be necessary to redesign footings in the field (e.g. widen or deepen footings) based on observed conditions.

5.3.2 Bearing Depth and Dimension

Minimum individual spread footing and wall footing widths should be at least 24 and 18 inches, respectively, with a minimum embedment depth of 12 inches below final grade.

This recommendation is made to help prevent a "localized" or "punching" shear failure condition that could exist with very narrow footings.

5.3.3 Settlement

We estimated compression of the bearing soils under the assumed applied column and wall loads, assuming the Westergaard distribution of stresses below the center and at the corners of an infinitely flexible surface load, and then averaging estimated settlements to account for the effect of rigidity of a reinforced footing. Soil compression under imposed loads was estimated based on boring and laboratory data and our experience with similar soils. Differential settlement between similarly loaded footings was assumed to be one-half of the average total settlement.

Assuming surface and site preparation as recommended in Section 5.1 and fill placement as recommended in Section 5.2, total settlement of properly constructed footings will be one inch or less for a typical 4 ft. by 4 ft. spread footing and a 1-1/2 foot wide continuous wall footing using an assumed 3,000 psf allowable bearing pressure. Differential settlement between adjacent footings carrying similar loads is estimated as one-half of the total settlement or less than one-half inch.

5.3.4 Settlement Time Rate

We estimated time rate of settlement using our general experience in similar soils in the Coastal Plain region. A large portion of soil compression will occur elastically upon placement of fill or building loads. Since the soils loaded by the footings lie above the water table, time for primary consolidation to occur is likewise very short and settlements associated with secondary compression are negligible. We estimate that approximately 90 percent of total settlements estimated above will occur with load placement. Remaining settlements are expected to largely occur within the next 5 to 7 days.

5.3.5 Foundation Lateral Capacity

Lateral capacity of footings includes a soil lateral pressure and coefficient of friction as described in IBC Section 1806. Where bearing in natural soils, footings will be embedded in material similar to those described as Class 4 in Table 1806.2. Where footings are cast neat against the sides of excavations in natural soils, an allowable lateral bearing pressure of 150 psf per foot depth below natural grade may be used in computations.

Lateral sliding resistance can be calculated by multiplying a coefficient of friction value of 0.25 by the dead load. An increase of one-third in the allowable lateral capacity may be considered for load combinations, including wind or earthquake, unless otherwise restricted by design code provisions.

5.3.6 Construction and Observation of Footings

When possible, concrete should be placed the same day footings are excavated to the planned bearing elevations. Remove soils softened by water intrusion or exposure before placing concrete. The geotechnical engineer or a representative of the geotechnical engineer should observe cleaned footing excavations prior to concrete placement. S&ME should also observe undercut areas prior to backfilling to confirm that poor soils have been removed and that the exposed subgrade is suitable for support of footings. Footings designed for a bearing pressure of 3,000 psf which are required to be undercut below the design bearing elevation should be backfilled with an open-graded stone such as No. 57 stone, flowable fill or well compacted soil fill. If an open-graded stone is used, the stone should be tamped into place.

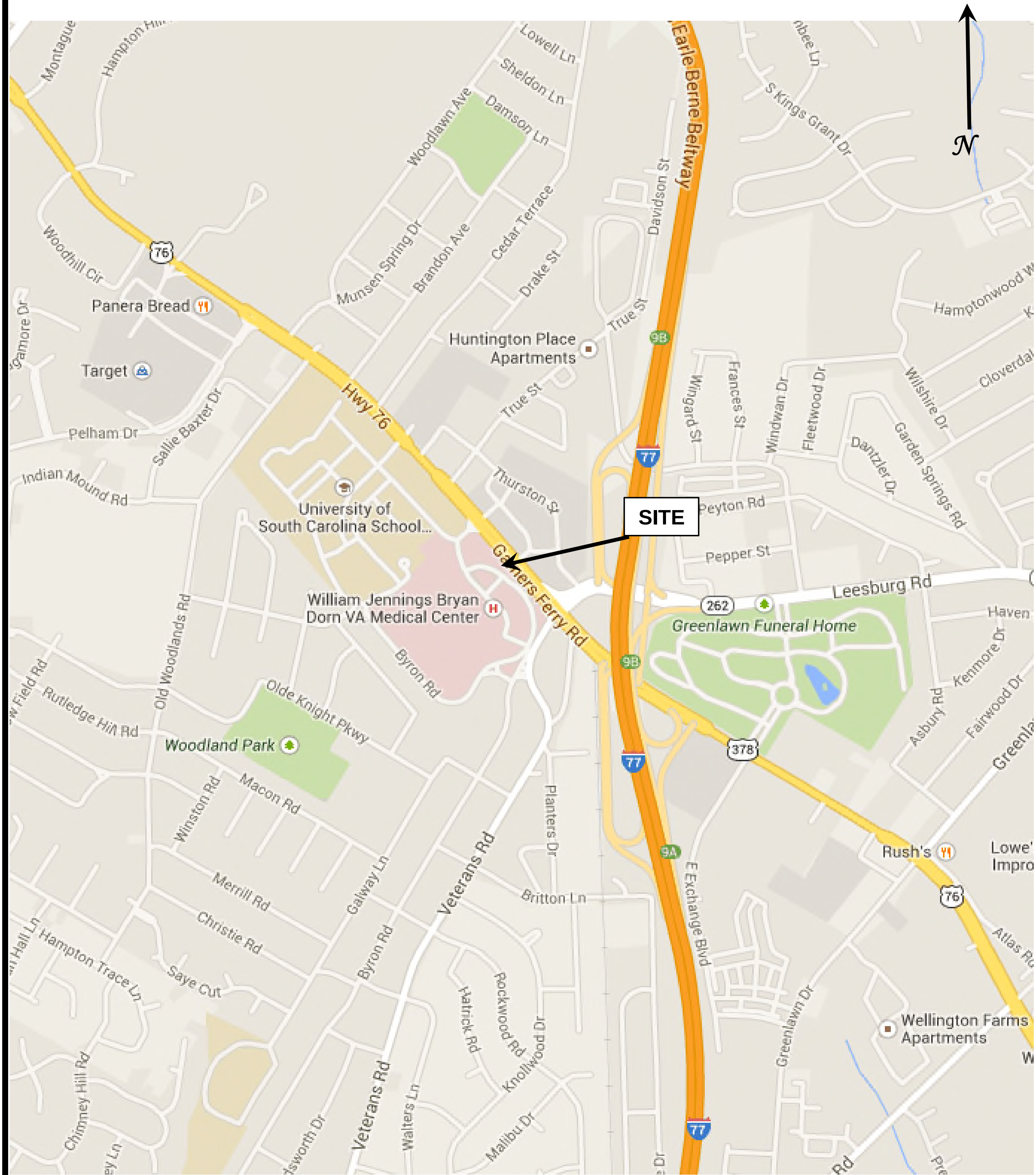
6.0 QUALIFICATIONS OF REPORT

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 were based on the applicable standards of our profession at the time this report was prepared. No other warranty, express 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 between the borings will not become evident until construction. If variations appear evident, then we will re-evaluate the recommendations of this report. In the event that any changes in the nature, design, or location of the proposed 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.

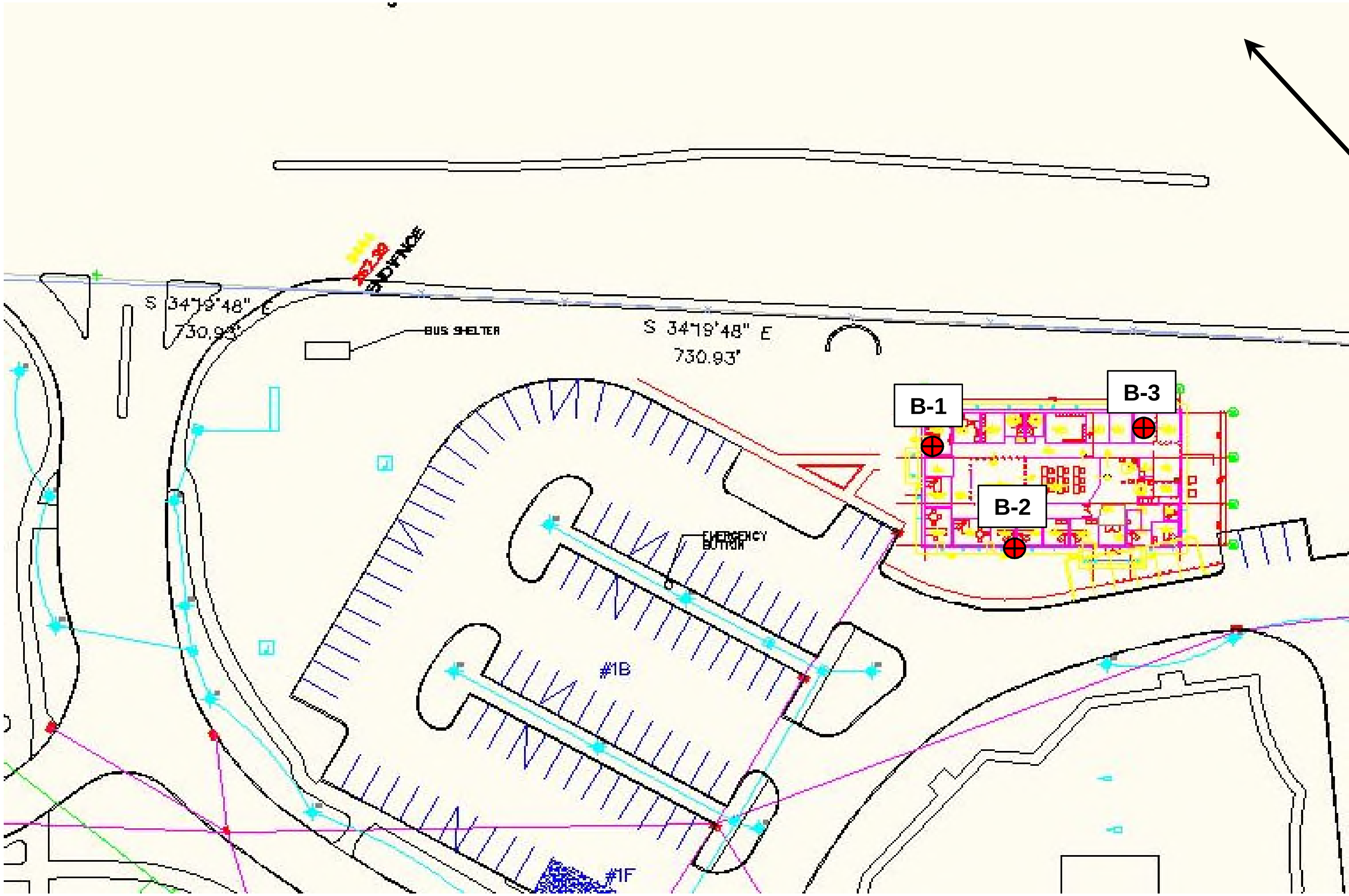
We recommend that S&ME, Inc. be provided the opportunity to review the final design plans and specifications in order to ensure that earthwork and foundation recommendations are properly interpreted and implemented.

APPENDIX



SOURCE: Google Maps

SCALE: NTS	S&ME	**SITE LOCATION PLAN** Proposed Dorn VAMC Police Station Columbia, South Carolina	FIGURE NO.
CHECKED BY: JCL			1
DRAWN BY: RCB			
DATE: 12/15/2014		JOB NO. 1461-14-067	



SOURCE: CAD file provided by BES Design/Building, LLC

⊕ **APPROXIMATE BORING LOCATION**

SCALE:	AS SHOWN	S&ME	**BORING LOCATION PLAN**	FIGURE NO.
CHECKED BY:	JCL		Proposed Dorn VAMC Police Station	2
DRAWN BY:	RCB		Columbia, South Carolina	
DATE:	12/15/2014		JOB NO. 1461-14-067	

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-420-98, "Standard Guide to 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.

Boring and Sampling

Soil Test Boring with Hollow-Stem Auger - 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. Borings were made by mechanically twisting a continuous steel hollow stem auger into the soil. At regular intervals, 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.

Borehole Closure – Following collection of relevant geotechnical data, boreholes were filled by slowly pouring auger cuttings into the open hole such that minimal "bridging" of the material occurred in the hole. Backfilling of the upper two feet of each hole was tamped as heavily as possible with a shovel handle or other hand held equipment, and the backfill crowned to direct rainfall away on the surface.

Preservation and Transporting of Soil Samples without Moisture Control – Procedures for preserving soil samples obtained in the field and transportation of samples to the laboratory generally followed those given in ASTM D 4220, "Standard Practice for Preserving and Transporting Soil Samples" for Group A samples as defined in Section 4. Group A samples are those samples not suspected of being contaminated and for which only a general visual description will be performed. No attempt was made to maintain samples at the field moisture content value. Representative samples of the cuttings or split spoon samples, or representative bulk samples, were placed in suitably identified, non-sealed containers and transported to the laboratory. Sample identification numbers on the containers corresponded to sample numbers recorded on field boring records.

Field Tests of Earth Materials

The subsurface conditions encountered during drilling were reported on a field test boring record by the chief driller. The record contains information about the drilling method, samples attempted and sample recovery, indications of materials in the borings such as coarse gravel, cobbles, etc, and indications of materials encountered between sample intervals. Representative soil samples were placed in glass jars and transported to the laboratory along with the field boring records. Recovered samples not expended in laboratory tests are commonly retained in our laboratory for 60 days following completion of drilling. Field boring records are retained at our office.

Measurement of Static Water Levels – Water level readings were made in the open boreholes immediately after completing drilling and withdrawal of the tools. Where feasible, measurements were repeated after an elapsed period of 24 hours to gauge the stabilized water level. Procedures for measurement of liquid levels in open boreholes are described in ASTM D 4750, "Standard Test Method for Determining Subsurface Liquid Levels in a Borehole or Monitoring Well (Observation Well)." A weighted measuring tape was slowly lowered into each borehole until the liquid surface was penetrated by the weighted end. The reading on the tape was recorded at a reference point on the surface and compared to the reading at the demarcation of the wetted and unwetted portions of the tape. The difference between the two readings was recorded as the depth of the liquid surface below the reference point. Measurements made by this method were then repeated until approximately consistent values were obtained.

Laboratory Tests of Soil and Rock

Recovered disturbed and undisturbed samples and the drillers' field logs were transported to the laboratory where they were examined by the geotechnical engineer. Selected samples representative of certain groups of soils were subjected to simple classification tests by hand or other simple means.

Examination of Split Spoon Soil Samples - Soil and rock samples and field boring records were reviewed in the laboratory by the geotechnical engineer. Soils were classified in general accordance with the visual-manual method described in ASTM D 2488, "Standard Practice for Description and Identification of Soils (Visual-Manual Method)". The geotechnical engineer also prepared the final boring records enclosed with this report.

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%.

S&ME

ENGINEERING • TESTING
ENVIRONMENTAL SERVICES

PROJECT: **Proposed Dorn VAMC Police Station**
Columbia, South Carolina
S&ME Project No. 1461-14-067

BORING LOG B-1

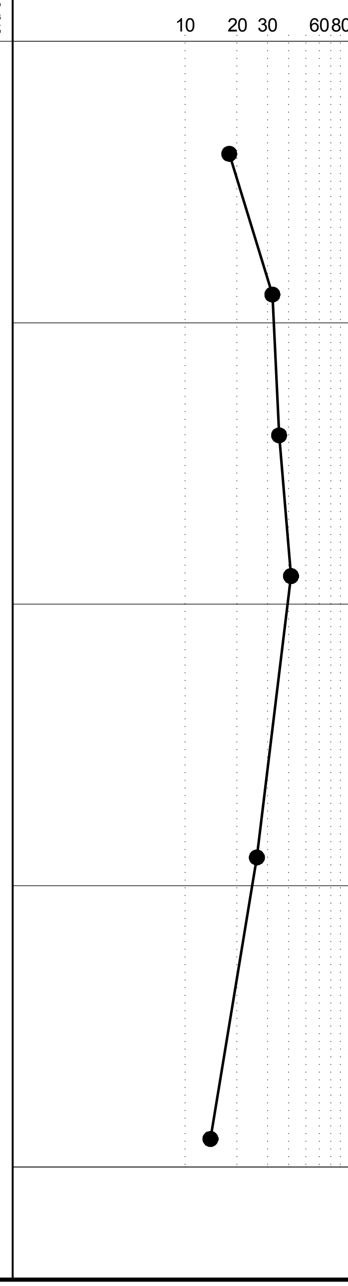
DATE DRILLED: **12/8/14**	ELEVATION: **250.0 ft**
DRILL RIG: **CME 550**	BORING DEPTH: **20.0 ft**
DRILLER: **H Wessinger**	WATER LEVEL: **Not Encountered**
HAMMER TYPE: **Auto**	LOGGED BY: **RCB**
SAMPLING METHOD: **Split spoon**	
DRILLING METHOD: **2¼" H.S.A.**	

NOTES: **Northings, Eastings and Elevations estimated from Google Earth.**

NORTHING: **779803** EASTING: **2012507**

DEPTH (feet)	MATERIAL DESCRIPTION	WATER LEVEL	ELEVATION (feet-MSL)	SAMPLE NO.	1st 6in / RUN #	2nd 6in / REC	3rd 6in / RQD	N VALUE
	SURFACE MATERIALS - 4 inches of TOPSOIL.							
	TERRACE SEDIMENTS - CLAYEY SAND (SC) - mostly fine to medium sands, some low plasticity fines, dry to moist, brown, medium dense.			SS-1	6	7	11	18
	- - - @ 3 feet - little low plasticity fines, mottled red and tan, dense.			SS-2	10	12	20	32
5			245.0					
				SS-3	13	16	19	35
				SS-4	16	18	23	41
10	**COASTAL PLAIN - CLAYEY SAND (SC)** - mostly fine to medium sands, some kaolinitic low plasticity fines, moist, mottled white and tan, medium dense.		240.0					
				SS-5	10	12	14	26
15			235.0					
	- - - @ 15 feet - mostly medium sands, little low to medium plasticity fines, orangish-tan.							
		HC						
				SS-6	5	7	7	14
20	Boring terminated at 20 ft		230.0					

STANDARD PENETRATION TEST DATA (blows/ft) / REMARKS



NOTES:

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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.

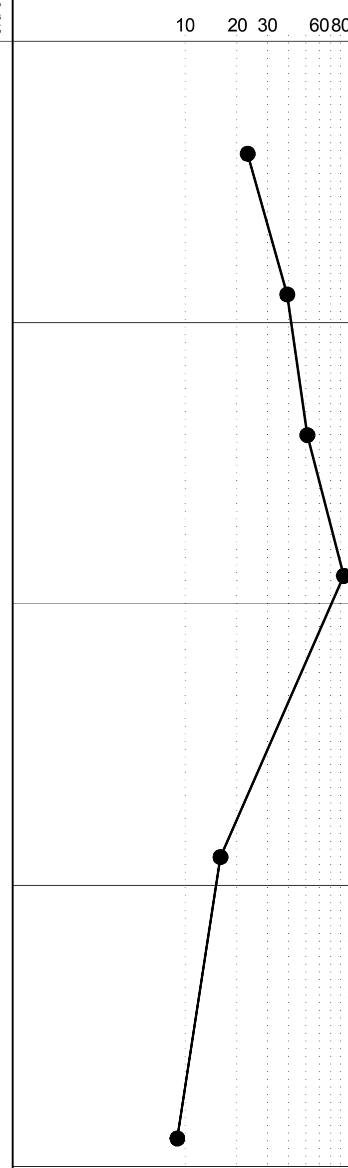
S&ME

PROJECT: **Proposed Dorn VAMC Police Station** **Columbia, South Carolina** **S&ME Project No. 1461-14-067**	**BORING LOG B-2**

- DATE DRILLED: **12/8/14**
- ELEVATION: **250.0 ft**
- DRILL RIG: **CME 550**
- BORING DEPTH: **20.0 ft**
- DRILLER: **H Wessinger**
- WATER LEVEL: **Not Encountered**
- HAMMER TYPE: **Auto**
- LOGGED BY: **RCB**
- SAMPLING METHOD: **Split spoon**
- DRILLING METHOD: **2¼" H.S.A.**
- NOTES: **Northings, Eastings and Elevations estimated from Google Earth.**
- NORTHING: **779748**
- EASTING: **2012497**

DEPTH (feet)	MATERIAL DESCRIPTION	WATER LEVEL	ELEVATION (feet-MSL)	SAMPLE NO.	1st 6in / RUN #	2nd 6in / REC	3rd 6in / RQD	N VALUE
0	**SURFACE MATERIALS** - 3 inches of TOPSOIL.							
	<u>TERRACE SEDIMENTS</u> - CLAYEY SAND (SC) - mostly fine to medium sands, some low plasticity fines, dry to moist, mottled red and tan, medium dense.			SS-1	7	10	13	23
	- - - @ 3 feet - little low plasticity fines, dense.			SS-2	11	16	23	39
5	- - - @ 5 feet - very dense.		245.0	SS-3	14	20	31	51
	- - - @ 8 feet - some kaolinitic low plasticity fines, mottled white, tan and red.			SS-4	20	36	47	83
10	**<u>COASTAL PLAIN</u> - CLAYEY SAND (SC)** - mostly medium sands, little kaolinitic low plasticity fines, dry to moist, mottled white and tan, medium dense.		240.0	SS-5	4	7	9	16
15	- - - @ 15 feet - little low to medium plasticity fines, moist, orangish-brown, loose.	HC	235.0	SS-6	4	4	5	9
20	Boring terminated at 20 ft		230.0					

STANDARD PENETRATION TEST DATA (blows/ft) / REMARKS



S&ME BORING LOG 1461-14-067 BORING LOGS.GPJ S&ME.GDT 12/15/14

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S&ME

PROJECT: **Proposed Dorn VAMC Police Station**
Columbia, South Carolina
S&ME Project No. 1461-14-067

BORING LOG B-3

DATE DRILLED: **12/8/14**	ELEVATION: **246.0 ft**
DRILL RIG: **CME 550**	BORING DEPTH: **20.0 ft**
DRILLER: **H Wessinger**	WATER LEVEL: **Not Encountered**
HAMMER TYPE: **Auto**	LOGGED BY: **RCB**
SAMPLING METHOD: **Split spoon**	NORTHING: **779747** / EASTING: **2012573**
DRILLING METHOD: **2¼" H.S.A.**	

NOTES: **Northings, Eastings and Elevations estimated from Google Earth.**

DEPTH (feet)	MATERIAL DESCRIPTION	WATER LEVEL	ELEVATION (feet-MSL)	SAMPLE NO.	1st 6in / RUN #	2nd 6in / REC	3rd 6in / RQD	N VALUE
0	**SURFACE MATERIALS** - 6 inches of TOPSOIL.							
	TERRACE SEDIMENTS - CLAYEY SAND (SC) - mostly fine to medium sands, some low to medium plasticity fines, trace finger roots, moist, brown, medium dense.			SS-1	3	4	7	11
	- - - @ 3 feet - minus finger roots, dry to moist, mottled red and tan, dense.			SS-2	12	17	30	47
5	- - - @ 5 feet - little low plasticity fines, mottled red, tan and white, very dense.		241.0	SS-3	16	21	34	55
				SS-4	21	36	42	78
10	**COASTAL PLAIN - CLAYEY SAND (SC)** - mostly fine sands, some kaolinitic low plasticity fines, moist, mottled red, orange, tan and white, medium dense.		236.0	SS-5	7	8	9	17
15	**POORLY GRADED SAND WITH SILT (SP-SM)** - mostly fine sands, trace medium sands, few low plasticity fines, dry to moist, yellowish-tan, medium dense.	HC	231.0	SS-6	7	8	8	16
20	Boring terminated at 20 ft		226.0					

STANDARD PENETRATION TEST DATA (blows/ft) / REMARKS: 10 20 30 60 80

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S&ME