Report of Geotechnical Exploration
Former Post Office/Proposed Museum – Elevator Addition
Newberry, South Carolina
S&ME Project No. 1461-17-091

PREPARED FOR:

Newberry County
1301 College Street
Newberry, South Carolina 29108

PREPARED BY:

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

December 26, 2017

December 26, 2017

Newberry County
1301 College Street
Newberry, South Carolina 29108

Attention: Mr. Ervin West

Reference: **Report of Geotechnical Exploration**
Former Post Office/Proposed Museum – Elevator Addition
1300 Friend Street
Newberry, South Carolina
S&ME Project No. 1461-17-091

Dear Mr. West:

As requested, S&ME, Inc. has completed field testing for the Former Post Office/Proposed Museum – Elevator Addition site, in Newberry, South Carolina. Our work was performed in general accordance with our proposal No. 14-1700823, dated December 18, 2017.

This report provides information on the exploration and testing procedures used, our boring records, and our recommendations regarding site conditions, excavation considerations, dewatering considerations, suitability of on-site soils for use as structural fill, fill placement and compaction, lateral earth pressures, shallow foundation design values, estimated settlements and slab-on-grade design values.

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 12-22-17 ROBERT C. BRUORTON

Robert C. Bruorton, P.E.
Senior Engineer/Project Manager

Matthew F. Cooke, P.G.
Geotechnical Group Leader

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

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Appendices

1.0 Project Information

Initial information about the project was provided during a teleconference held between members of Newberry County and Messrs. Owen Astwood and Chad Bruorton, P.E. of S&ME on December 14, 2017. Representatives of Newberry County and Mr. Astwood were meeting regarding the UST on-site and Mr. Bruorton was teleconferenced in to discuss the elevator addition at the site. Additional information was provided via email correspondence from Mr. Michael Kohn of Michael Kohn Architects to Mr. Bruorton on December 19 and 22, 2017. Additional information included the *Existing Conditions and Demolition Plan* and the *Site Layout Plan*, both prepared by Michael Kohn Architects, dated November 20, 2017 as well as a detail of the elevator shaft.

From our review of the provided information, it is understood that the former Post Office building is located at 1300 Friend Street in Newberry, South Carolina, as shown on the Site Location Plan, Figure 1 in Appendix I. The planned development at the site includes the renovation of the former Post office building into a new Museum building for Newberry County. Specific to the renovation is the addition of an elevator to make the building ADA accessible. The new elevator will provide access between the ground floor and the existing basement of the building, and is to be located along the southeastern side of the existing building. This area is currently part of the existing brick paved parking lot for the existing building.

The elevator shaft is understood to be three-stories in height and is to be constructed of reinforced masonry walls with brick veneer above grade and reinforced concrete walls below grade, with a wood truss roof system. The elevator shaft and pit are understood to be supported by a single mat foundation with approximate plan dimensions of 9 foot-4 inches by 11 foot-4 inches. From our telephone conversation with Mr. Chuck Knobeloch, P.E., of K&P Engineering, the structural engineer of record for the project, the elevator shaft and pit will apply a uniform vertical load of approximately 1,500 pounds per square foot (psf).

The elevator pit will extend roughly 5 feet below the basement floor elevation of 492.22 feet, to approximate elevation 487 feet, to allow access to the basement level and space for the elevator equipment.

2.0 Exploration Procedures

The subsurface exploration of this project included two (2) Standard Penetration Test (SPT) soil borings. The approximate locations of each of the borings are shown in the *Boring Location Plan* attached as Figure 2 in Appendix I.

2.1 Reconnaissance of Project Area

On December 21, 2017, a representative of S&ME visited the site to observe current site conditions and lay out the proposed soil test boring locations. Soil test boring locations were marked in the field with orange spray paint. Soil test boring locations were laid out using our sub-meter GPS equipment. Boring locations were directed by Mr. Ervin West of Newberry County. The boring locations indicated on the attached *Boring Location Plan* must be considered as approximate. No formal survey of boring locations or elevations was conducted by S&ME.

2.2 Field Testing and Sampling

The following sections detail our field and sampling activities at the site. A summary of our exploration procedures is included in Appendix II.

2.2.1 Concrete Coring

Two (2) concrete cores were performed at the boring locations by Advanced Concrete under subcontract to S&ME. The cores were drilled in the vertical orientation using a 6-inch, outer diameter, diamond-tipped concrete core barrel to allow access to the underlying subsurface soils for soil boring activities.

2.2.2 Soil Test (SPT) Borings

Two (2) soil test borings with SPT sampling and testing were performed on December 21, 2017. The SPT soil test borings were performed by Southern Drill, Inc. under subcontract to S&ME using a truck-mounted CME 55 drill rig. The borings were advanced using 2¼-inch inside diameter hollow-stem augers to termination depths of roughly 15 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.

Ground water measurements were attempted in the borings shortly after drilling was completed. Once drilling was completed, the boreholes were left open to allow for delayed ground water measurements after a period of approximately 24 hours. After ground water measurements, the boreholes were backfilled with auger cuttings and a plastic hole plug was placed within the boreholes that exceeded 5 feet in depth, followed by surface patching of the pavement materials with commercially available asphalt cold patch materials.

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 various available topographic and geologic maps for relevant information.

3.1 Surface Conditions

As previously mentioned, the site currently consisted of the former Post Office building and associated infrastructure. The area of the proposed elevator addition at the site was observed to be located within the southeastern portion of the site, along the southern side of the existing building. Ground cover in this area was observed to consist of brick pavement, underlain by concrete rigid pavement. From our visual observations, the area appeared to be relatively flat.

3.2 Subsurface Conditions

Recovered field samples and field boring logs were reviewed in the laboratory by a member of our geotechnical staff. Soil test boring records and other field data are assembled in Appendix II.

3.2.1 *Site Geology*

The site lies within the Piedmont Physiographic Province of South Carolina, an area underlain by soils weathered in place from the parent crystalline bedrock material. Residual soils of the Carolina Piedmont consist of stiff or very stiff micaceous silts and clays, grading to firm sands with depth. These soils have been completely weathered in place from the parent bedrock material, but below depths of a few feet retain most of the relict rock structure. Soil strength derives largely from relict intermolecular bonding and remolded materials generally less exhibit lower shear strength than do undisturbed samples. Piedmont soils are normally consolidated to slightly over consolidated.

3.2.2 *Interpreted Subsurface Profile*

The generalized subsurface conditions at the site are described below. The discussed subsurface description is of a generalized nature to highlight the major subsurface stratification features and material characteristics. The boring records included in Appendix II should be reviewed for specific information at each boring location. The depth and thickness of the subsurface strata indicated on the boring records was estimated based on the drill cuttings and the samples recovered. The transition between materials may be more gradual than indicated on the boring records. Information on actual subsurface conditions exists only at the specific boring locations and is relevant to the time the exploration was performed. Variations may occur and should be expected at locations remote from the boring. 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. Soil test boring records are attached in Appendix II.

3.2.2.1 Surface Materials

Surface materials were roughly 17 to 18 inches in thickness and consisted of approximately 4 inches of brick paver underlain by roughly 2 inches of bedding sand, followed by approximately 7 to 8 inches of rigid concrete pavement underlain by roughly 4 inches of graded aggregate base course.

3.2.2.2 Possible/"Undocumented" Fill Materials

Beneath the surface materials, possible/"undocumented" fill materials consisting of low plasticity silts with little fine sands (ML) were encountered to a depth of approximately 8½ feet below the existing ground surface. These materials are assumed to have been placed during construction of the existing basement for the structure.

Recovered samples of these materials were red and brown in color and were moist to the touch. Standard Penetration Test (SPT) N-values ranged from 2 to 10 blows per foot (bpf), indicating very soft to stiff consistencies.

3.2.2.3 Piedmont Residuum

Below the possible/"undocumented" fill materials, residual Piedmont soils were encountered to the termination depth of our borings at roughly 15 feet below the existing ground surface. The Piedmont residuum consisted of low plasticity silts with some fine sands (ML).

Recovered samples of the Piedmont residuum were red, brown and orange in color and were moist to the touch. SPT N-values ranged from 8 to 10 bpf, indicating firm to stiff consistencies.

Based on our understanding of the planned grade for the proposed elevator pit, the Piedmont residual soils encountered in our borings will serve as the bearing stratum for the elevator pit foundation.

3.2.3 *Ground Water*

Ground water was not encountered in our borings at the time of drilling. As previously mentioned, the boreholes were left open overnight and ground water measurements were attempted after at least 24 hours. Ground water was not encountered in the borings after at least 24 hours. Therefore, it appears that ground water will not affect excavation and construction activities on-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 Appendix II.

4.0 Conclusions and Recommendations

The following paragraphs include our conclusions and recommendations regarding excavation considerations, dewatering considerations, suitability of on-site soils for use as structural fill, fill placement and compaction, lateral earth pressures, shallow foundation design values, estimated settlements and slab-on-grade design values.

The soil profile encountered at this site would appear generally suitable for the proposed development. However, it is important to note that several aspects of construction at this site could adversely affect the adjacent utilities and structures. Therefore, proper design and special care during construction will be needed to protect the adjoining properties.

4.1 Pre-Construction Survey

Perform a preconstruction survey on existing structures prior to construction in the immediate vicinity. This should include the adjacent, existing structure to the North, at a minimum. This should include a detailed inventory of the structural condition, including existing cracks or other damage. A structural engineer should assist in assessing building sensitivity to settlement. Obtain loads on existing foundations from as-built plans. On older structures, it may be necessary to analyze the structure to approximate these loads. This survey is recommended for all existing structures that are to remain after development of the site.

4.2 Excavations Adjacent to Structures

Proposed excavations will be either adjacent to or within 5 feet of the foundations supporting the adjacent, existing structure North of the proposed construction.

Excessive horizontal or vertical displacement of the structural elements may occur due to loss of support to the bearing soils afforded by removal of the excavated soils, unless steps are taken to support these soils. New footings adjacent to old footings are unlikely to cause additional settlement of the existing structure provided that bearing elevations are similar.

Where basement and foundation excavation is performed adjacent to the existing structure, care must be taken not to undermine the existing footings. Excavation for any purpose shall not extend closer than one foot to a surface drawn at 45 degrees to the horizontal through the lower edge of any adjacent existing footing or foundation, unless such footing or foundation is first properly underpinned or otherwise protected against movement.

4.3 Excavation Considerations

It is understood that the planned pit for the elevator addition will be excavated to a depth of roughly 5 feet below the existing finished floor elevation of the existing basement level of the adjacent structure, or roughly 12 feet from existing grade.

Piedmont residuum consisting of firm to very stiff cohesive soils, similar to those encountered in our soil test borings, can typically be excavated using backhoes. The degree of difficulty that mobile equipment will encounter rises dramatically in materials exceeding about 70 to 80 blows per foot. These conditions were not encountered in our soil borings.

4.4 Slope Considerations

The cut excavations will result in a sloped excavation of roughly 12 feet deep below existing grade. Once construction of the elevator pit is complete these excavation slopes will be backfilled to finished grade.

4.4.1 Temporary Excavation Stability

All excavations shall be sloped or shored in accordance with local, state, and federal regulations, including OSHA (29 CFR Part 1926) excavation trench safety standards. The contractor is usually solely responsible for site safety. This information is provided only as a service, and under no circumstances shall S&ME be assumed to be responsible for construction site safety.

4.4.2 Excavation Slopes

As previously mentioned, the planned excavation at the site will be advanced through mostly Piedmont residual soils and may approach upwards of 12 feet. Slope stability analysis is outside of our current scope of work; however, based upon our experience and information obtained by borings at the site, we recommend the excavated cut slopes not exceed a maximum inclination of 2H:1V (horizontal:vertical). These values are for planning purposes and will need to be confirmed during construction by direct observation of the excavated slopes, and inclinations modified, if necessary, based on the observed conditions. If these slopes are to be exceeded, then temporary/permanent retainage will be necessary.

4.4.3 *General Slope Recommendations*

Recommended slopes are preliminary and assume that ground water is controlled at the lowest level of the excavation continuously while the excavation is open. Ground water should not be permitted to flow or emerge from soil excavation slopes. Surface water should be captured by appropriate drainage measures above the slope crest and not allowed to drain down the slope. If perched groundwater is observed emerging from the face of the slope or if surface water is adversely affecting the slope, S&ME should be contacted immediately.

Cut slope profiles should be made relatively uniform such that local slopes do not significantly exceed the recommended slopes. Finally, the recommended slope inclination requires that slopes are monitored for indications of instability and that slopes are flattened or other measures taken if appropriate. Monitoring of the slopes during construction is presently not part of our contracted scope of services for this project.

Stability can be reduced by a number of additional factors including excessive erosion, non-uniform sloping resulting in areas of steeper grades, loose seams in the cut face, and/or ground water emerging from the cut slopes. As a result, proper channeling of surface water is critical. Surface runoff shall be directed away from the slopes via the use of berms, swales, or slope drains. If loose seams are encountered within cut faces during excavation or ground water is encountered, an in-depth analysis of slope stability should be performed.

4.5 Temporary/Permanent Retaining Structures

It is assumed that temporary/permanent retaining structures will be required to protect the existing, adjacent building to the North during excavation and construction of the planned elevator pit addition. Design of temporary/permanent retaining structures for vertical or near-vertical excavations, was beyond our scope. Typically, such designs are done by specialty contractors working directly for the general contractor. Contractor's and designer's responsibilities for design and construction of temporary bracing need to be clearly defined in the contract documents.

Temporary/permanent retaining structures typical for a scenario similar to this project may include "top-down" or "bottom-up" construction of the retainage system. "Top-down" techniques result in the retainage structure being constructed as the excavation is advanced, while "bottom-up" techniques are constructed to the full depth of planned excavation, then the soils are removed, creating the excavation. Due to the proximity of the elevator pit excavation to the existing structure to the North, the limited space on-site, and the age of the existing structure to the North, a "top-down" technique appears to be more favorable.

Some options of temporary/permanent retaining structures, utilizing the "top-down" technique, typical for a scenario similar to this project may include, but are not limited to, stabilization by hydro-insensitive, high-density, polyurethane injection, soldier piles with lagging and soils nails with shotcrete/concrete facing. The contractor would select the more economical method used based on his computation of costs involved. Lateral earth pressures regarding backfill materials for elevator pit walls are provided below.

4.6 Use of On-site Soils as Structural Fill

Fine grained low plasticity silts (ML) containing varying amounts of fine sands similar to those encountered in cut areas of the exploration are typically suitable to marginally suitable for use as structural fill. Suitability of these soils for use depends a great deal on the moisture content of the material at time of placement. Before beginning to place fill, sample and test each proposed fill material to determine its maximum dry density, optimum moisture content, natural moisture content, and suitability as a structural fill material.

Marginal suitability refers to the fact that fine grained soils are moisture sensitive to some degree and can be difficult to work if allowed to become wet. These difficulties can include softening of exposed subgrade soils, excessive rutting or deflection under construction traffic, and the difficulty associated with adequately drying and compacting wet soil. Moisture-related earthwork difficulties can be reduced by performing the earthwork during the typically drier months of the year (May through October).

4.7 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. Structural soil fill material should have less than 5 percent organic matter, a standard Proctor maximum dry density of 90 pcf or greater and a plasticity index (PI) of 30 percent or less. We recommend that any off-site borrow meet the organic content, PI and density requirements of this section. Testing will be required before fill placement begins to determine the optimum moisture-density condition for the fill materials. Material to be used as soil fill should be tested and approved by the geotechnical engineer before being placed.

4.7.1 Density and Moisture Requirements

Place new fill in maximum 8-inch loose lifts and compact to at least 95 percent of maximum dry density (ASTM D-698 Standard Proctor). This level of compaction can be practically achieved with area soils, and has been found to provide adequate support for foundations and pavements. 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.

4.7.2 Compaction of Cohesive Soils

The compaction characteristics of silty soils (ML) with plastic properties encountered at this site will be highly dependent on the soil moisture content at the time of construction. 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.

The water content of these soils is usually very difficult to modify in the field. Above or below the optimum moisture content, the soils become progressively more difficult to manipulate and compact. Soils excavated above the water table are usually close enough to optimum moisture content to place and compact efficiently with little moisture conditioning required. Soils that are initially too wet or are allowed to become wet due to rainfall are more difficult to use. Drying wet silty soils usually requires favorable weather conditions and often requires repeated disking and rolling with sheeps-foot rollers to lower the moisture content.

4.7.3 *Monitoring and Testing*

Fill placement should be witnessed by an experienced soils technician working under the guidance of the geotechnical engineer. We recommend full time observation by a qualified soils technician with testing at random intervals to confirm compaction is being achieved.

4.8 Lateral Earth Pressures

The equivalent fluid pressures given below are recommended for backfill behind elevator pit walls. We recommend that the "At rest" lateral earth pressure coefficient be used since the walls will be restrained from rotation and that the "Active" lateral earth pressure coefficient be used if rotation is not restrained. In addition to the lateral loads exerted by the soil against the walls, allowance should be included for lateral stresses imposed by any temporary or long-term surcharge loads, such as loads from adjacent foundations, construction equipment/materials, or other vehicular traffic adjacent to the walls.

The values given in the following table are representative of in-situ soils encountered within the elevator pit area and properly placed and compacted backfill in accordance with recommendations elsewhere in this report. The static values in Table 4-1 are based on a moist unit weight of 120 pcf, friction angle of 15 degrees, and 500 psf cohesion for the in-situ soils and a moist unit weight of 125 pcf, friction angle of 28 degrees and 50 psf cohesion for the assumed fill soils along the height of the wall.

Table 4-1 – Equivalent Fluid Pressures

Support Condition (All conditions assume level backfill)	Equivalent Fluid Pressure In-situ	 Fill
Active Condition (Wall deflects laterally, as with a conventional cantilevered retaining wall)	70 pcf	45 pcf
At-rest Condition (Wall restrained from movement, as with a basement wall)	90 pcf	65 pcf
Passive Condition (Wall movement toward retained soil, as with a thrust block)	200 pcf	340 pcf
Coefficient of Sliding Friction (Tan δ)	0.30	

- The above values represent a fully-drained soil condition, as defined in IBC 2015 Section 1807.2.2. Where backfill soils are not drained using an appropriately designed toe drainage system, the lateral soil pressure must consider hydrostatic forces below the surface, and submerged soil unit weight.
- Grade walls lower than 8 feet high are not considered relatively rigid or restrained walls.
- Organic silts (OL or OH), inorganic elastic silts (MH), or inorganic highly plastic clays (CH) soils may not be used as backfill.
- Compact backfill directly behind walls with light, hand-held compactors. Heavy compactors and grading equipment should not be allowed to operate within 10 feet of the walls during backfilling to avoid developing excessive temporary or long-term lateral soil pressures.

Provide positive gravity drainage of the backfill using a permanent toe drain to limit buildup of hydrostatic pressures in the backfill. Gravity drainage may consist of a minimum two foot wide blanket of clean crushed stone or washed sand, separated from the backfill by a properly graded filter or approved filter fabric, or a specially designed geotextile material such as Enka-drain, or equivalent. Vertical drains should be tied into a permanent "toe" drain installed at the base of the wall. Where gravity drainage of retaining walls is not feasible, design walls to resist hydrostatic forces in addition to lateral earth pressure.

4.9 Shallow Foundation Design and Construction

Based on our soil test boring data and experience in the area, shallow mat foundation appears suitable for support of the planned elevator pit. We estimated bearing capacities for the mat footing configuration and dimension 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.

4.9.1 Allowable Bearing Pressure

Assuming proper design and construction, an allowable bearing pressure of 3,000 pounds per square foot (psf) or less is recommended for a mat foundation bearing on native soils placed and compacted as recommended in previous sections 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.

4.9.2 Anticipated Settlement

We estimated compression of the bearing soils under the assumed applied 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 data and our experience with similar soils.

From our computations utilizing the provided maximum loading conditions and dimensions presented in this report, total anticipated settlement for a properly constructed mat foundation will be less than one-half inch for the understood 9 foot-4 inch x 11 foot-4 inch mat foundation and 1,500 psf maximum contact pressure. Differential settlement across the mat foundation carrying similar loads is estimated as less than one half of the total settlement. These estimated settlements account for an adjustment in the net applied stresses caused by excavating the elevator pit area to the planned elevation of roughly 487 feet.

4.9.3 Settlement Time Rate

We estimated time rate of settlement using our general experience in similar soils in the Piedmont region. A large portion of soil compression will occur elastically upon placement of structural loads. Since the soils loaded by the mat foundation lie above the water table, time for primary consolidation to occur is typically rapid and settlements associated with secondary compression are negligible. We estimate that approximately 70 percent of total settlements estimated above will occur with load placement. Remaining settlements are expected to largely occur within the next 2 to 3 weeks.

4.9.4 Foundation Lateral Capacity

Lateral capacity of foundations includes a soil lateral pressure and coefficient of friction as described in IBC Section 1806. Where bearing in natural soils, foundations will be embedded in material similar to those described as Class 5 in Table 1806.2. Where foundations are cast neat against the sides of excavations in natural soils, an allowable lateral bearing pressure of 100 psf per foot depth below natural grade may be used in computations.

A cohesion of 130 psf multiplied by the contact area may be used to calculate lateral sliding resistance for Class 5 soils. 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.

4.9.5 Construction and Observation of Footings

It is preferable for foundation concrete to be placed in the excavation the same day the foundation is excavated, if possible. However, due to the large size of the mat, we expect this may not be feasible. We therefore recommend placing a 2- to 3-inch "mud mat" of lean (2000 psi) concrete in the bottom of the excavation to protect the bearing soils and provide a stable working surface for rebar installation. (The bearing grade will need to be slightly over-excavated a few inches to make sure the mud mat does not extend higher than the plan bottom-of-mat elevation.) This will help limit the potential for additional excavation of wet, softened (or frozen) soils which often results when foundation excavations are exposed to inclement weather.

The geotechnical engineer or a representative of the geotechnical engineer should evaluate the foundation bearing grades 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 mat support. As previously noted, undercut areas below the design bearing elevation should be backfilled with an open-graded stone such as No. 57 stone, or flowable fill (lean concrete). If an open-graded stone is used, the stone should be tamped into place.

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

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

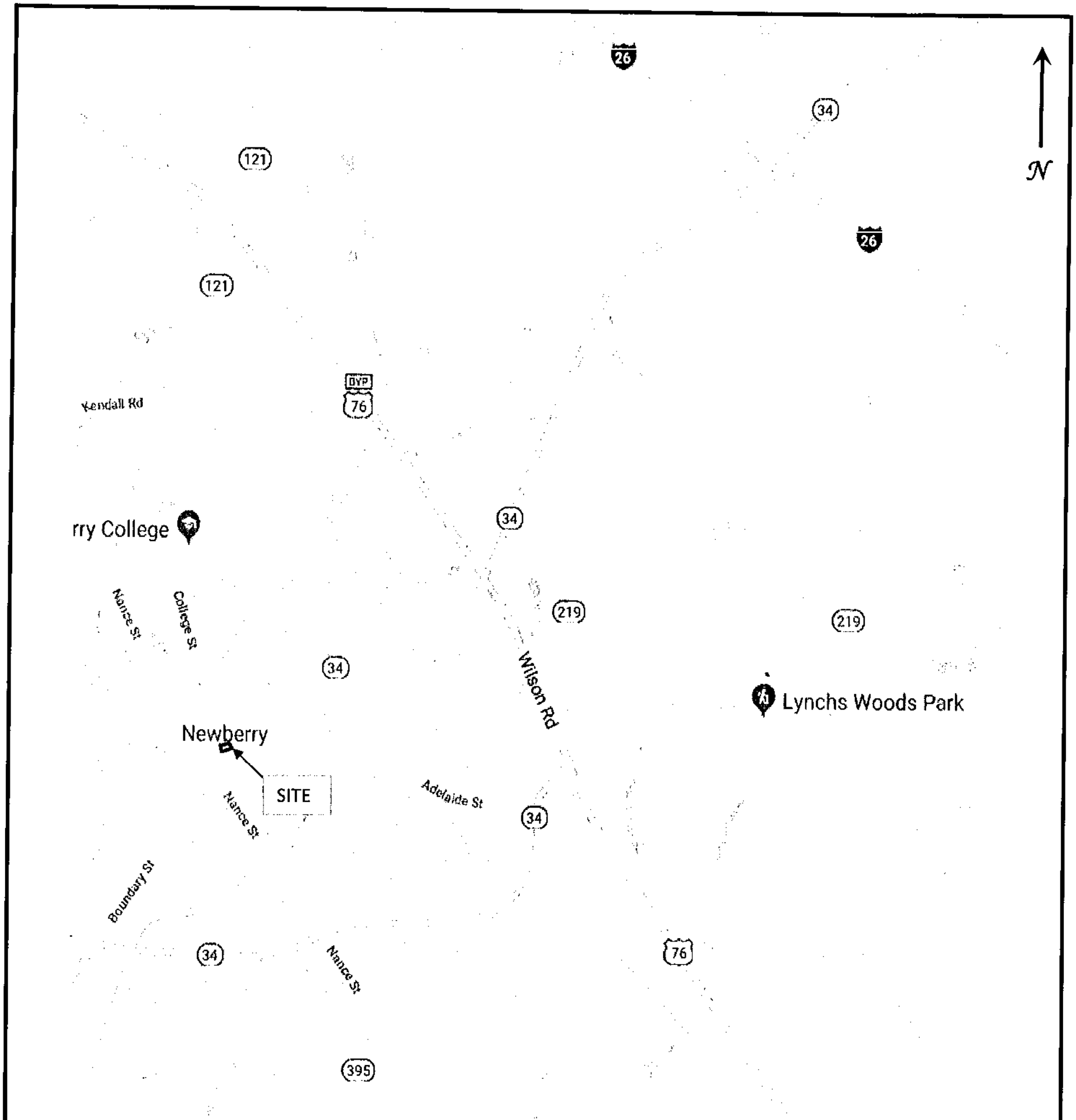
Our conclusions and recommendations are based on limited data from a field exploration program. Subsurface conditions can vary widely between explored areas. Some variations may not become evident until construction. If conditions are encountered which appear different than those described in our report,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

Unless specifically noted otherwise, our field exploration program did not include an assessment of regulatory compliance, environmental conditions or pollutants or presence of any biological materials (mold, fungi, bacteria). If there is a concern about these items, other studies should be performed. S&ME can provide a proposal and perform these services if requested.

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

Appendices

Appendix I – Figures



SOURCE: Google Maps

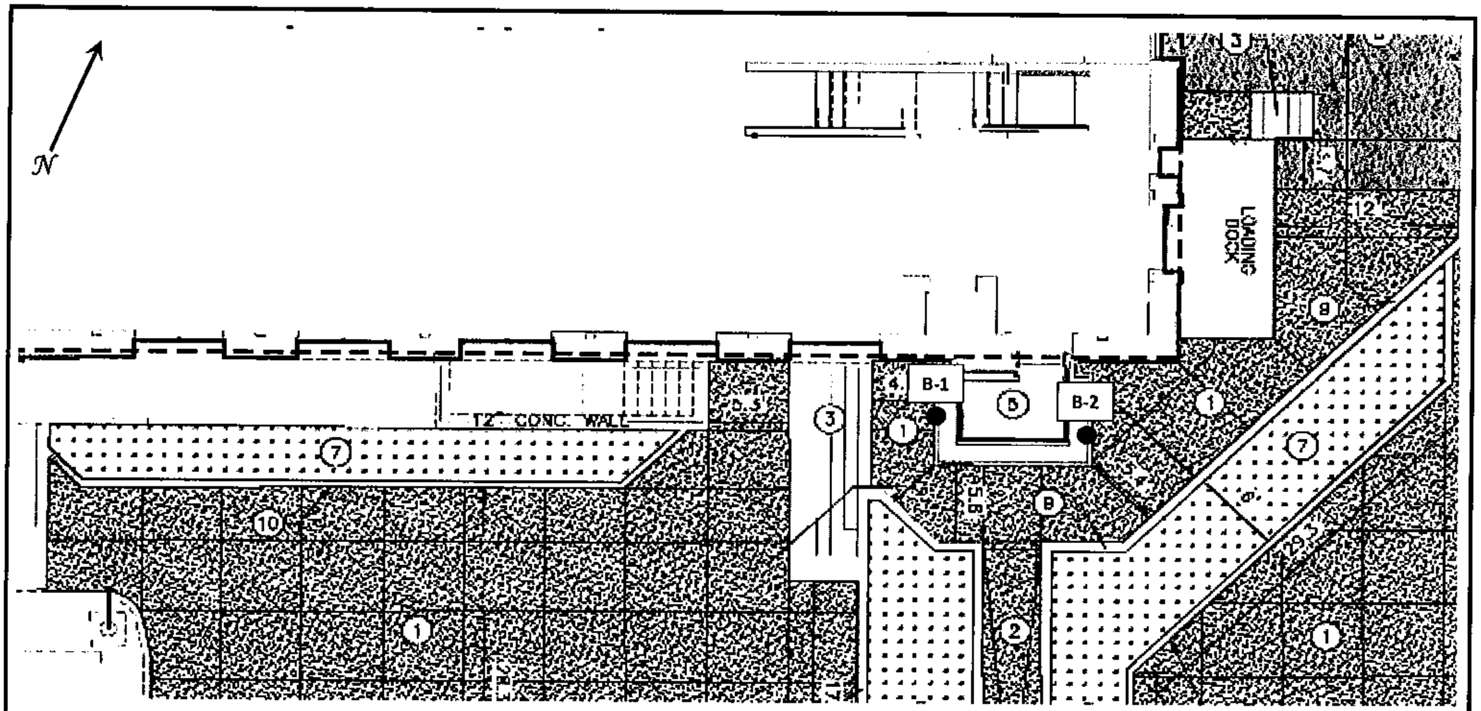
SITE LOCATION PLAN

JOB NAME: Newberry Old PO/New Museum - Elevator Add.
LOCATION: 1300 Friend Street
CITY, STATE: Newberry, South Carolina
JOB NO.: 1461-17-091

SCALE:	NTS	CHECKED BY:	MFC
DATE:	12/26/2017	DRAWN BY:	RCB

FIGURE NO.

1



SOURCE: Site Layout Plan, produced by Michael Kohn Architect, dated November 20, 2017

BORING LOCATION PLAN

JOB NAME:	Newberry Old PO/New Museum - Elevator Add.		FIGURE NO.
LOCATION:	1300 Friend Street		2
CITY, STATE:	Newberry, South Carolina		
JOB NO.:	1461-17-091		
SCALE: NTS		CHECKED BY: MFC	
DATE: 12/26/2017		DRAWN BY: RCB	

Appendix II – Field Data

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%.

PROJECT: **Former Post Office/Proposed Museum - Elevator Addition**
Newberry, South Carolina
S&ME Project No. 1461-17-091

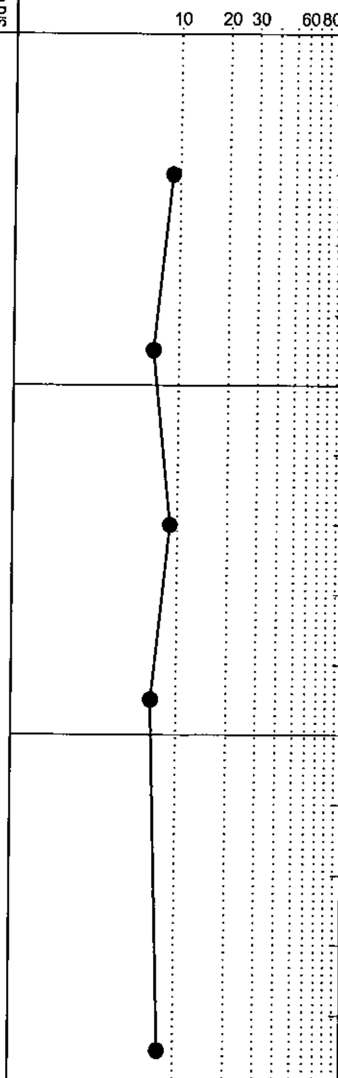
BORING LOG B-1

DATE DRILLED: **12/21/17**	ELEVATION: **499.0 ft**
DRILL RIG: **CME 550X**	BORING DEPTH: **15.0 ft**
DRILLER: **H. Wessinger**	WATER LEVEL: **Not Encountered**
HAMMER TYPE: **Auto**	LOGGED BY: **HGM**
SAMPLING METHOD: **Split Spoon**	NORTHING: **888558**
DRILLING METHOD: **3¼" H.S.A.**	EASTING: **1813278**

NOTES: Northing and Easting recorded on-site using sub meter GPS equipment. Elevation estimated from Google Earth. No formal survey performed by S&ME.

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 BRICK PAVER.							
	2 inches of BEDDING SAND.							
	8 inches of CONCRETE PAVEMENT.							
	4 inches of GRADED AGGREGATE BASE COURSE.			SS-1	3	3	6	9
	POSSIBLE FILL - SILT WITH SAND (ML) - mostly low to medium plasticity fines, little fine sands, moist, red, stiff. --- @ 3 feet - firm.			SS-2	2	3	4	7
5			494.0					
	--- @ 6 feet - stiff.			SS-3	2	4	5	9
	PIEDMONT - SANDY SILT (ML) - mostly low plasticity fines, some fine sands, moist, reddish-brown and orangish-brown with black staining, firm.			SS-4	2	3	4	7
10			489.0					
		HC						
				SS-5	3	4	4	8
15	Boring terminated at 15 ft		484.0					

STANDARD PENETRATION TEST DATA (blows/ft) / REMARKS



NOTES:

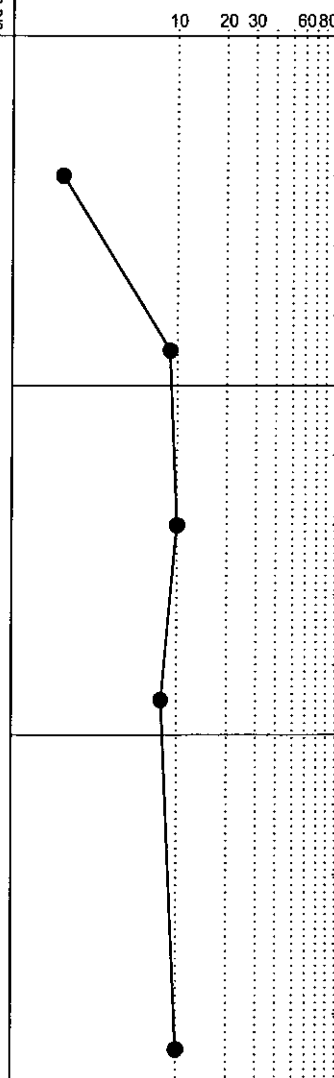
1. *THIS LOG IS ONLY A PORTION OF A REPORT PREPARED FOR THE NAMED PROJECT AND MUST ONLY BE USED TOGETHER WITH THAT REPORT.*
2. *BORING, SAMPLING AND PENETRATION TEST DATA IN GENERAL ACCORDANCE WITH ASTM D-1586.*
3. *STRATIFICATION AND GROUNDWATER DEPTHS ARE NOT EXACT.*
4. *WATER LEVEL IS AT TIME OF EXPLORATION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

PROJECT: **Former Post Office/Proposed Museum - Elevator Addition** **Newberry, South Carolina** **S&ME Project No. 1461-17-091**	**BORING LOG B-2**

DATE DRILLED: **12/21/17**	ELEVATION: **499.0 ft**	**NOTES:** Northing and Easting recorded on-site using sub meter GPS equipment. Elevation estimated from Google Earth. No formal survey performed by S&ME.
DRILL RIG: **CME 550X**	BORING DEPTH: **15.0 ft**	
DRILLER: **H. Wessinger**	WATER LEVEL: **Not Encountered**	
HAMMER TYPE: **Auto**	LOGGED BY: **HGM**	
SAMPLING METHOD: **Split Spoon**		NORTHING: **888555** / EASTING: **1813268**
DRILLING METHOD: **3¼" H.S.A.**		

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 BRICK PAVER.							
	2 inches of BEDDING SAND.							
	7 inches of CONCRETE PAVEMENT.							
	4 inches of GRADED AGGREGATE BASE COURSE.			SS-1	1	1	1	2
	POSSIBLE FILL - SILT WITH SAND (ML) - mostly low plasticity fines, little fine sands, moist, red, very soft. --- @ 3 feet - stiff.			SS-2	3	4	5	9
5			494.0					
	--- @ 6 feet - reddish-brown.			SS-3	3	5	5	10
	PIEDMONT - SANDY SILT (ML) - mostly low plasticity fines, some fine sands, moist, reddish-brown and orangish-brown with black staining, firm.			SS-4	2	3	5	8
10			489.0					
		HC						
	--- @ 13.5 feet - stiff.			SS-5	2	4	6	10
15	Boring terminated at 15 ft		484.0					

STANDARD PENETRATION TEST DATA (blows/ft) / REMARKS



S&ME BORING LOG 1461-17-091 BORING LOGS.GPJ SME COLUMBIA GINT DATA TEMPLATE.GDT 12/26/17

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.*

Summary of Field Procedures

Boring and Sampling

Surface Coring of Concrete Pavement

Coring of concrete slabs or concrete pavement was performed in general accordance with ASTM C42, *Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete.* Samples were obtained for measuring approximate thickness only. Cores were drilled in the vertical orientation and, were used for thickness measurement, at least one foot from formed joints or obvious edges. Moisture conditioning and end surface preparation of recovered cores described in Section 7 of ASTM C42 was not performed.

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., 2-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. Where boreholes exceeded five feet in depth, a plastic hole plug was firmly tamped into place within the backfill at a depth of about two feet.

Patching of Asphalt Surface

Penetrations of asphalt surfaces made during the drilling process were patched using compacted asphalt cold patch material. Cold patch asphalt was placed to provide a surface flush with existing pavement adjacent to the boring. Cold patch asphalt was compacted by tamping it into the boring with a shovel handle or similar hand held equipment.

Preservation and Transporting of Soil Samples with Control of Field Moisture

Procedures for preserving soil samples obtained in the field and transportation of samples to the laboratory generally followed those given in ASTM D4220, *Standard Practice for Preserving and Transporting Soil Samples* for Group B samples as defined in Section 4. Group B samples are those samples not suspected of being contaminated and for which only water content and classification, proctor, relative density, or profile logging will be performed. Group B samples also include bulk samples that are intended to be remolded in the laboratory for compaction, swell pressure, percent swell, consolidation, permeability, CBR, or shear testing. Representative samples of the cuttings or split spoon samples, or representative bulk samples, were placed in suitably identified, sealed glass jars or plastic containers and transported to the laboratory. Sample identification numbers on the containers corresponded to sample numbers recorded on field boring records or test pit records. Thin-walled tube samples were sealed at the ends with paraffin and capped with plastic end caps.

◈ 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 D4750, *Standard Test Method for Determining Subsurface Liquid Levels in a Borehole or Monitoring Well (Observation Well)*. A calibrated cable with electrical wire encased, equipped with a weighted sensing tip at one end and an electric meter at the other, was slowly lowered into each borehole until the liquid surface was penetrated by the weighted end. Contact with the water closed an electric circuit and was recorded by the meter. The depth reading on the cable was then recorded relative to a reference point on the surface. Measurements made by this method were then repeated until approximately consistent values were obtained.

Summary of Laboratory Procedures

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.

Laboratory Tests of Soil

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.