Report of Geotechnical Exploration
Former Harriet Tubman Homes Site
Chattanooga, Tennessee
S&ME Project No. 4181-18-046

PREPARED FOR:

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December 14, 2018

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Barge Design Solutions, Inc.
1110 Market Street, Suite 200
Chattanooga, Tennessee 37402

Attention: Mr. Russell Moorehead, PE

Reference: **Report of Geotechnical Exploration**
Former Harriet Tubman Homes Site
Chattanooga, Tennessee
S&ME Project No. 4181-18-046

Dear Mr. Moorehead:

This report presents the results of the geotechnical exploration for the Former Harriet Tubman Homes site in Chattanooga, Tennessee. Our work was performed in general accordance with S&ME Proposal No. 41-18-00380 dated June 29, 2018.

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

Sincerely,

S&ME, Inc.

David Grass, PE
Project Engineer

James P. McGirl, PE
Principal Engineer

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Appendices

♦ Executive Summary

This summary is presented for the convenience of the reader. The full report text should be studied and understood before preparing an estimation of quantities or preparing designs based on this report, as it contains important information and recommendations that are not included in this brief summary.

1. The geotechnical exploration included drilling and sampling of seven soil test borings. The samples collected during our exploration were returned to our Chattanooga laboratory where they were further evaluated by a professional engineer.

2. Natural moisture content and Atterberg limits laboratory tests were performed on selected samples to aid our soil classification and to evaluate the on-site soil's volume change potential. Unconfined compressive strength testing was performed on selected Shelby tube samples to determine the soil's undrained shear strength.

3. The Chickamauga Group is mapped to underlie the site. There is always some risk of sinkhole development at any site underlain by limestone bedrock. However, the test borings drilled at this site did not encounter open voids or other signs of incipient sinkhole conditions. Further, the geophysical testing did not identify areas of concern. It is our opinion that the risk for sinkhole development is not increased due to the nature of this project.

4. Subsurface conditions generally consisted of fill or alluvial soils overlying residual soils to boring termination or auger refusal. Fill soils were typically composed of firm to very stiff silty lean or fat clays with chert fragments to depths of about 12 feet below the existing ground surface. Fill soils were encountered along the sites north perimeter. Alluvial soils were typically composed of firm to very stiff silty clays to depths of about 7 to 12 ½ feet. The underlying residual soils were typically composed of firm to hard silty lean and fat clays to boring termination or auger refusal depths.

5. Auger refusal was encountered in two of the seven borings at depths ranging from about 7 to 12 ½ feet below the existing ground surface. The remaining borings were terminated at a predetermined depth of about 20 feet.

6. Groundwater was encountered in boring numbers B-102, B-103, and B-104 at the time of drilling. We expect groundwater control will be necessary during construction specifically along the northern boundary of the site. Groundwater control can typically be achieved by pumping from sumps during construction.

7. The existing sewer line will likely be abandoned in place by plugging both ends. Future development of the site may require the removal or filling of this pipe depending on the location of proposed structures.

8. Maximum excavation slopes should be assigned based on OSHA regulations for Class B soils. However, we expect that trench boxes or shoring will be required due to the proposed trench depth and site constraints.

9. We expect material requiring difficult excavation techniques will be encountered during utility construction along Roanoke Avenue. The depth to rock along the north and western boundaries of the property appears to be deeper than the proposed pipe depth.

1.0 Introduction

S&ME, Inc. has completed the geotechnical exploration at the Former Harriet Tubman Homes site in Chattanooga, Tennessee. Our work was performed in general accordance with S&ME Proposal Number 41-1800380 dated June 29, 2018. Our services were authorized by Mr. Russell Moorehead of Barge Design Solutions, Inc. on November 2, 2018.

The purpose of our work was to explore the subsurface soil conditions and groundwater level, identify approximate bedrock elevation, if encountered, and provide excavation benching/shoring recommendations for the construction of a new sewer line. This report describes our understanding of the project, presents the results of the field exploration and laboratory testing, and discusses our conclusions and recommendations relative to the above considerations.

The scope of our geotechnical services did not include an environmental assessment for evaluating the presence or absence of wetlands, or hazardous or toxic materials.

A Site Location Plan, a Boring/Seismic Profile Location Plan, and Geophysical Data Profiles are included in Appendix I. A discussion of the field investigative procedures, a legend of soil classification and symbols, and the Test Boring Records are included in Appendix II. Appendix III contains a discussion of the laboratory test procedures and the laboratory test results. Appendix IV contains a document titled “Important Information About Your Geotechnical Engineering Report”.

2.0 Site and Project Description

Our understanding of the project is based on our discussions with Mr. Moorehead, Ms. Lindsay Hiatt of the Chattanooga Area Chamber of Commerce, and Ms. Charita Allen with the City of Chattanooga. We have also been provided preliminary civil plans, undated, as prepared by Barge Design Solutions.

The 42-acre site is located north of the intersection of Southern Street and Roanoke Avenue in Chattanooga, Tennessee. A Site Location Plan, Figure 1, showing the general project site location is provided in Appendix I. The Harriet Tubman Homes complex previously occupied the site. The former multi-tenant residential structures were demolished in 2015/2016. The majority of the property is currently grass-covered expect for the two-lane asphalt roads located within the site.

We understand an unreinforced concrete gravity sewer line bisects the site from the northeast corner to the southwest corner near Sholar Avenue. This sewer line exists at a depth of about 15 feet below the existing ground surface. We understand that the City is considering abandoning this sewer line in order for the site to be more marketable to potential industrial tenants. The sewer line will be re-routed around the perimeter of the site with two new sections. The primary section will be located around the northern and western boundaries of the site from existing manhole A-10 to manhole A-1 near the intersection of Southern Street and Sholar Avenue. The project will also include a new line along the west side of Roanoke Avenue from existing manhole B-1 to manhole B-5. Based on the preliminary civil drawings, we understand the proposed sewer lines will vary in depth from about 10 to 25 feet below the existing ground surface.

3.0 Regional Geology

Chattanooga, Tennessee is located in the Valley and Ridge Physiographic Province. Elongated ridges that trend in a northeast-southwest direction characterize this province. The ridges are typically formed on highly resistant sandstones and shales, while the valleys and rolling hills are formed on less resistant limestone, dolomite, and shales.

Based on our review of the Geologic Map of Tennessee, dated 1966, bedrock of the lower member of the Chickamauga Group underlies the site. The lower member of the Chickamauga is composed of light gray to gray, fine to coarse grained limestone. Residual soils derived from this geology are typically composed of silts and clays with overburden thicknesses less than 15 feet.

Carbonate rock, such as the strata underlying this site, is of great geologic age and has been subject to solution weathering over geologic time. Rainwater falling onto the surface and percolating downward through the soil and into cracks and fissures gradually dissolves the rock, producing insoluble impurities such as chert and clay. Since carbonate rock varies greatly in its resistance to weathering, the soil/bedrock contact may be extremely irregular. More soluble bedrock develops a thicker soil cover and a more irregular bedrock surface with pinnacles and slots, and less soluble bedrock usually develops a thinner soil cover and a less irregular soil-bedrock surface.

These large variations in bedrock depth are greatly enhanced by the presence of fractures, bedding planes, and faults, which provide an increased opportunity for a greater influx of percolating water. The weaknesses may form clay-filled cavities or enlarge into caves and may be connected by a network of passageways. If a cave forms close to the bedrock surface, its roof may collapse and the overlying soils may erode into the cave. Once the weight of the overlying soil exceeds the soil's arching strength, the soil collapses and an open hole or depression may appear at the ground surface. Such a feature is termed a sinkhole.

There is always some risk associated with developing any site underlain by carbonate bedrock. However, the test borings drilled at this site did not encounter open voids or other signs of incipient sinkhole conditions. Further, the geophysical test data obtained near the approximate sewer alignment did not indicate the presence of significant Karst features. We have reviewed the USGS quadrangle map for this area. The map does not show a pattern of closed depressions that would indicate past sinkhole activity in near proximity to the site. We also observed successful development in the surrounding area. Therefore, we believe the risk of sinkhole development for this to be low.

4.0 Geophysical Services

4.1 Geophysical Methodology, Field Efforts, and Data Processing

On November 7 and 8, 2018, S&ME completed a Seismic Refraction Tomography (SRT) survey along the accessible portions of the northern and western edges of the property. SRT measures travel times of seismic compression waves (P-waves) at receivers (geophones) located along a linear array. The velocity at which the seismic waves propagate along the array can be determined from the slope of arrival times. Waves in soil and highly weathered bedrock (low-density) will travel slower than waves in more competent bedrock (high density). Where increases in elastic material properties occur, the seismic waves are refracted much like light in a prism.

Depths to higher velocity strata such as rock can be determined from the location of a slope change in the first arrival time vs. distance plots.

S&ME performed the SRT survey in general accordance with ASTM D5777-00 (2011) "*Standard Guide for Using the Seismic Refraction Method for Subsurface Investigation*" using a Geometrics ES3000 seismograph equipped with twenty-four (24) 10 Hz vertical geophones. A total of five SRT profiles ranging from about 150 feet to 920 feet in length were collected (Lines SRT-1 through SRT-5; Figure 2). Geophones were spaced at 10 feet intervals along the profiles. Data from several shots (off end, end, quarter, and mid-point) were acquired, where accessible, for each survey profile and a sledgehammer was used as the energy source. The SRT data was interpreted and processed using the OYO Corporation's SeisImager^{TM} software (Pickwin^{TM} and Plotrefa^{TM} modules), and Golden Software's Surfer® program was used to produce two-dimensional cross sections of P-wave velocity (Figure 3). Elevations were derived from 2.5 foot DEM data (TNGIS.org) and not from actual field survey measurements. As such, presented elevations should be considered approximate.

4.2 Geophysical Results

Lines SRT-1 through SRT-5 indicate seismic P-wave velocities ranging from approximately 1,000 feet per second (ft/s) to about 9,000 ft/s (Figure 3). Rock was not encountered in the adjacent borings (B-102, B-103, B-104, and B-106) as they were terminated at 20 feet below ground surface. However, our experience suggests that rock is typically greater than about 6,000 ft/s. As such, it appears that the SILTY CLAY (CL) and CLAY (CH) overburden identified in the borings are generally less than about 6,000 ft/s, which is shown as yellow in the presented SRT profiles, and range between about 7 feet to over 20 feet in thickness. The shallowest interpreted rock appears to be located along line SRT-1. Highly weathered rock could however exhibit velocities similar to soils. In addition, B-102, B-103, and B-104 encountered a water table between about 12 and 14 feet below ground surface which could produce similar P-wave velocities as rock (6,000 ft/s).

5.0 Subsurface Conditions

5.1 Field Exploration Procedures

The procedures used by S&ME, Inc. for field sampling and testing are in general accordance with ASTM procedures and established engineering practice in the State of Tennessee. Appendix II contains brief descriptions of the procedures used in this exploration.

S&ME, Inc. drilled seven soil test borings to obtain subsurface information at the project site. Members of our engineering staff established the actual boring locations in the field by measuring distances and estimating right angles relative to on-site landmarks. Boring elevations were obtained by superimposing boring locations onto the provided topographic information and interpolating between contours. Therefore, both the boring locations shown on Figure 2 – Boring Location Plan in Appendix I, and the elevations shown on the Test Boring Records in Appendix II, should be considered approximate.

Shelby tube soil samples were collected from selected depths and locations in conjunction with the drilling for subsequent laboratory testing. After each boring was completed, we measured the groundwater level, if present. The borings were then backfilled with auger cuttings before leaving the site.

Our field representative packaged the soil samples in sealed containers, labeled them for identification, and returned them to the Chattanooga office where a geotechnical engineer further examined them. We visually classified the soils according to the Unified Soil Classification System (ASTM D 2488). The resulting soil descriptions are shown on the Test Boring Records in Appendix II. Samples were then selected for laboratory testing.

5.2 Soil Stratification

The results of our field testing program are summarized in the following paragraphs, and are shown on the Test Boring Records in Appendix II. These records present our interpretation of the subsurface conditions at specific boring locations at the time of our exploration. The stratification lines represent the approximate boundary between soil types. The actual transitions may be more gradual than implied.

SURFACE MATERIALS

Surface material consisting of topsoil was encountered from the ground surface to a depth of about 4 inches in boring B-103. Although not documented by our field personnel, we expect a similar depth of topsoil is present in the general vicinity of the remaining borings.

FILL

Fill was encountered in borings B-102, B-103, and B-104 to a depth of about 12 feet below the existing ground surface. Fill is material that has been transported to its present location by man. The fill was generally composed of red-brown or yellow-brown silty clay or fat clay with either chert or gravel fragments. Standard Penetration Test (SPT) N values in the fill ranged from 5 to 19 blows per foot, indicating firm to very stiff soil consistencies. The SPT data indicates the fill was placed with some compactive effort. However, the compactive effort appears to have been inconsistent.

ALLUVIUM

Alluvial soils were encountered in borings B-101, B-105, B-106, and B-107 from the ground surface to depths ranging from about 7 to 12 ½ feet below the existing ground surface. The alluvial soil interval was not penetrated above auger refusal in borings B-101 and B-105. Alluvial soil is soil that has been transported to its present location by flowing water. The alluvial soils encountered at the site were typically composed of brown and gray or yellow-brown and red-brown silty lean and fat clay. SPT N values in the alluvium ranged from 5 to 16 blows per foot, indicating firm to very stiff soil consistencies.

RESIDUUM

Residual soils were encountered in each of the test borings except B-101 and B-105 below the fill or alluvial soils to auger refusal or boring termination depths. Residual soil forms from the in-place weathering of the underlying bedrock. The residual soils encountered at the site were typically composed of gray or yellow-brown and brown silty lean and fat clay. SPT N values in the residuum ranged from 6 to over 50 blows per foot, indicating firm to hard soil consistencies.

AUGER REFUSAL / BORING TERMINATION

Auger refusal was encountered in borings B-101 and B-105 at depths of about 7 and 12 ½ feet, respectively. The remaining borings were terminated at a predetermined depth of about 20 feet below the existing ground surface.

5.3 Water Levels

The boreholes were observed for the presence of groundwater at the termination of boring. Groundwater was encountered in test borings B-102, B-103, and B-104 at depths ranging from about 12 to 14 ½ feet (elevations ranging from about 644 to 648 feet) below the ground surface at the time of drilling. We backfilled the boreholes shortly after completion due to safety concerns, and therefore delayed groundwater level measurements were not obtained. It should be noted that groundwater levels can fluctuate with seasonal, climatic, and environmental changes. Further, groundwater may be encountered at depths different from those identified in our borings in the future.

6.0 Laboratory Testing

Laboratory tests were performed on representative split-spoon samples obtained during the field exploration phase of this project. We conducted moisture content and Atterberg limits tests on selected samples to aid our soil classification. The resulting soil descriptions are shown on the Test Boring Records in Appendix II.

In addition to the index property testing, unconfined compression testing was performed on selected Shelby tube samples to evaluate the soil's undrained shear strength for use in developing excavation inclination/shoring requirements. The laboratory test results and a brief description of the laboratory test procedures are presented in Appendix III.

7.0 Assessment

On the basis of this geotechnical exploration, we conclude that this site is adaptable for the proposed construction. In order to develop and adapt this site, a few items should be addressed during the planning, design, and construction phases of the project.

We understand the existing sewer line will be abandoned in place by plugging both ends of the pipe which is out of service. Future development of the site may include structures over this abandoned pipe's location. We expect that excavation of the pipe for removal will be costly given the pipe's depth. As an alternative, the owner may consider filling the pipe in place with pumpable grout or concrete such as flowable fill or lightweight celluar concrete. If the proposed structure has subsurface pits in the general vicinity of the existing sewer line, removal may be required in such areas. Loss of soil into an abandoned pipe can cause settlement and subsequent damage to the structures above the pipe.

Based on the test boring results, we expect that groundwater will be encountered during excavation of the proposed sewer line specifically along the northern boundary of the site, especially during times of heavy rain. Based on our experience with similar projects, pumping from sumps constructed within the excavation, will be required to prevent the accumulation of water in the excavation.

Based on the boring data and geophysical test results obtained during field exploration activities, we expect material requiring difficult excavation techniques will be encountered during utility construction along Roanoke Avenue. The rock surface along the north and west boundaries of the site appears to be below the invert elevations of the proposed sewer line. However, isolated pinnacles of rock may be encountered, likely near the bottom of the proposed excavation.

8.0 Design and Construction Recommendations

8.1 Limitations of Report

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

The analyses and recommendations submitted herein are based, in part, on the data obtained from the subsurface exploration. The nature and the extent of variations between the widely-spaced borings will not become evident until the time of construction. If variations appear evident, then we will re-evaluate the recommendations of this report. In the event any changes in the nature or location of the proposed sewer lines are planned, the conclusions and recommendations contained in this report will not be considered valid unless the changes are reviewed and the conclusions verified or modified in writing.

The geophysical method used for this survey has inherent limitations and active site activity (e.g. heavy equipment, trains, etc.) can cause noise/interference in the data sets. The SR method is limited to geologic conditions in which P-wave velocities increase with depth, and as such, a lower velocity layer beneath a high velocity layer would not be identified. Because SR data averages the conditions over the length of the profile, individual variations are not often detected. In addition, predicting the presence of isolated or relatively small areas of nested boulders is very unlikely, as is differentiating thin or discontinuous rock layers. Water in the subsurface can mask the SR results and be interpreted as the top of rock as saturated soil typically has a P-wave velocity in the range of 6,000 ft/s. Depth restrictions are associated with the SR method and the energy source.

We recommend S&ME be provided the opportunity to review the final design plans and specifications in order that earthwork and other recommendations are properly interpreted and implemented. The recommendations in this report are contingent on S&ME, Inc.'s observation and monitoring of grading and construction activities.

8.2 Groundwater

Based on the test boring results, we expect groundwater will be encountered during excavation of the proposed sewer line specifically along the northern boundary of the site. Groundwater was encountered in borings B-102, B-103, and B-104 at elevations of approximately 644 to 648 feet. Based on our experience with similar projects, pumping from sumps constructed within the excavation will be required to prevent the accumulation of water in the excavation. Water from pumps should be discharged beyond the construction boundaries.

8.3 Excavation Shoring and Bracing

We recommend an Occupational Safety & Health Administration (OSHA) soil class of B be assigned for determining the maximum slope for subsurface excavations. According to OSHA regulation, excavations made in soil type B in which workers will be entering are required to have side slopes no steeper than 1 Horizontal to 1 Vertical (1H:1V). If the excavation is extended into competent rock, OSHA states that excavations may have vertical sides.

Due to the anticipated depth of the new sewer line and physical site constraints in certain areas, we expect that achieving this maximum slope of excavation walls will not be feasible in some areas. In such cases, shoring of the excavation walls or trench boxes should be used to protect workers from cave-ins. We recommend the general contractor's responsibility for the design and construction of the trench excavation be clearly defined prior to beginning excavation at the site.

9.0 Construction Considerations

9.1 Pipe Abandonment

We understand the existing sewer line will be abandoned in place by plugging both ends of the pipe. Future development of the site may include structures over this abandoned pipe. The excavation, removal and backfilling of the pipe trench will be costly given the pipe's depth. As an alternative, we recommend the owner consider filling the pipe in place with pumpable grout, flowable fill or lightweight celluar concrete. If the proposed structure has subsurface pits in the general vicinity of the existing sewer line, removal of the abandoned pipe may be required in this area.

9.2 Fill Placement

Soils proposed for use as trench backfill should consist of low to moderately plastic clay or silt with a plasticity index of less than thirty (PI<30) and a standard Proctor maximum dry density greater than 95 pounds per cubic foot. The fill should contain no rock fragments larger than 4 inches in any dimension, and no organic matter.

Backfill operations should not begin until representative samples of proposed fill soils are collected and tested. The test results will be used to assess whether the proposed fill material meets the previously discussed plasticity and density criteria, and for quality control during backfilling. Please allow at least 3 to 5 days for testing before the fill operations begin.

We recommend compacted aggregate such as ASTM D 448 No. 57 or No. 67 stone be used to backfill the excavation to the top of the pipe. We recommend this particular aggregate as backfill, because it is relatively easy to compact, is durable, and it can be placed during inclement weather. We recommend observation of compacted aggregate placement by our engineering technician to determine the maximum lift thickness and compaction method necessary to obtain suitable compaction.

Soil fill should be placed in thin lifts with a maximum loose thickness of 4 inches, then compacted to 95 percent of the standard Proctor maximum dry density. Wetting or drying of these soils may be required, depending on the time of year site grading is performed. A representative of S&ME should test the density and moisture content of each lift before placing additional lifts.

9.3 Difficult Excavation

Based on the boring data and geophysical test results obtained during field exploration activities, we expect material requiring difficult excavation techniques will be encountered during utility construction along Roanoke Avenue. The rock surface along the north and west boundaries of the site appears to be below the invert elevations of the proposed sewer line. However, isolated pinnacles of rock may be encountered, likely near the bottom of the proposed excavation.

In confined excavations such as utility trenches, removal of weathered rock typically requires the use of large backhoes, or a hoe ram. The difficulty of excavation will depend on the composition of the rock, the location and orientation of discontinuities and bedding, and the skill of the equipment operator. Should mass rock be encountered along Roanoke Avenue, blasting may be required.

10.0 Follow-Up Services

Our services should not end with the submission of this geotechnical report. S&ME should be kept involved throughout the design and construction process to maintain continuity and to determine if our recommendations are properly interpreted and implemented. To achieve this, we should review project plans and specifications with the designers to see that our recommendations are fully incorporated and have not been misinterpreted. We also should be retained by the owner to monitor construction.

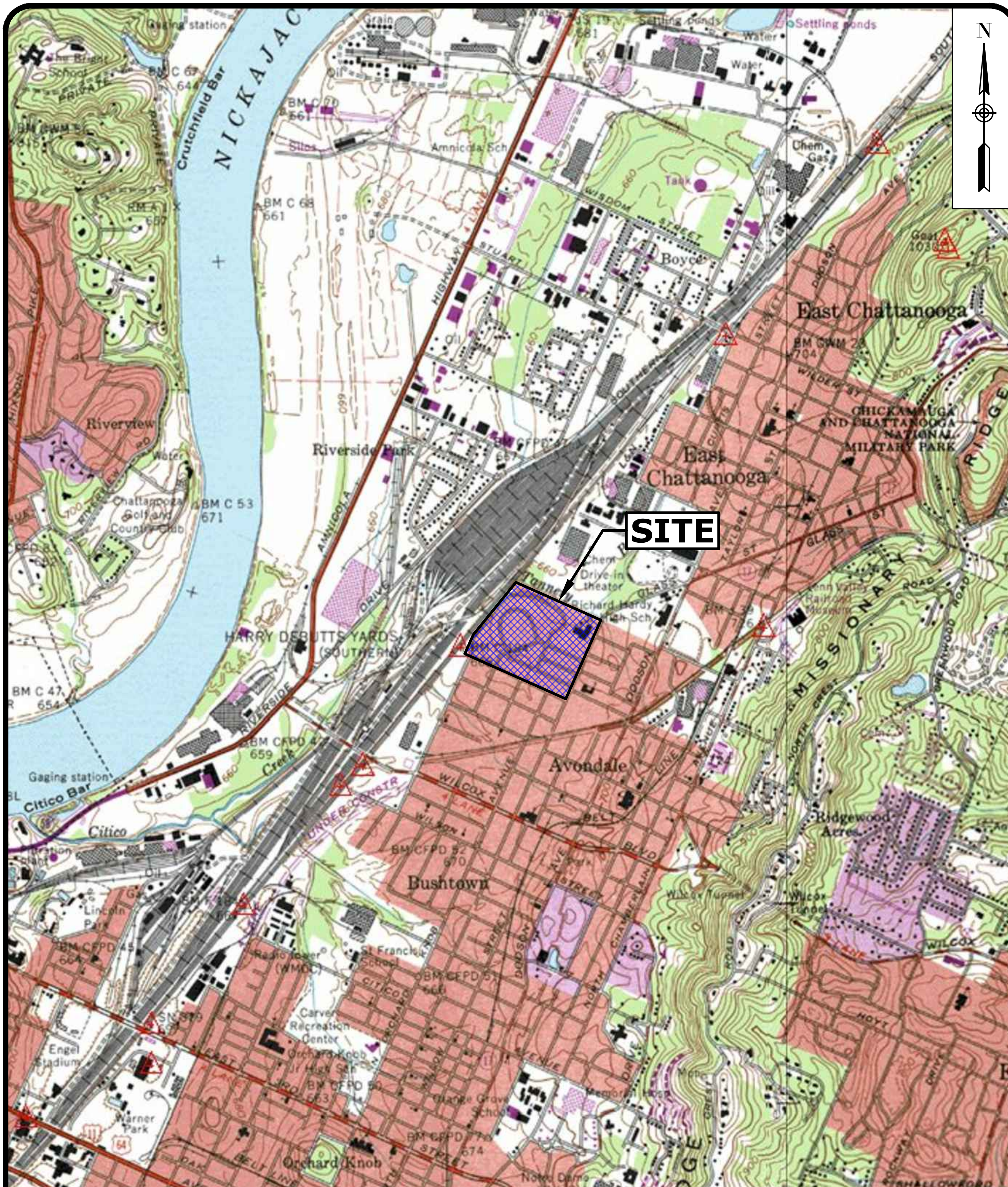
Appendices

Appendix I

Figure 1 - Site Location Plan

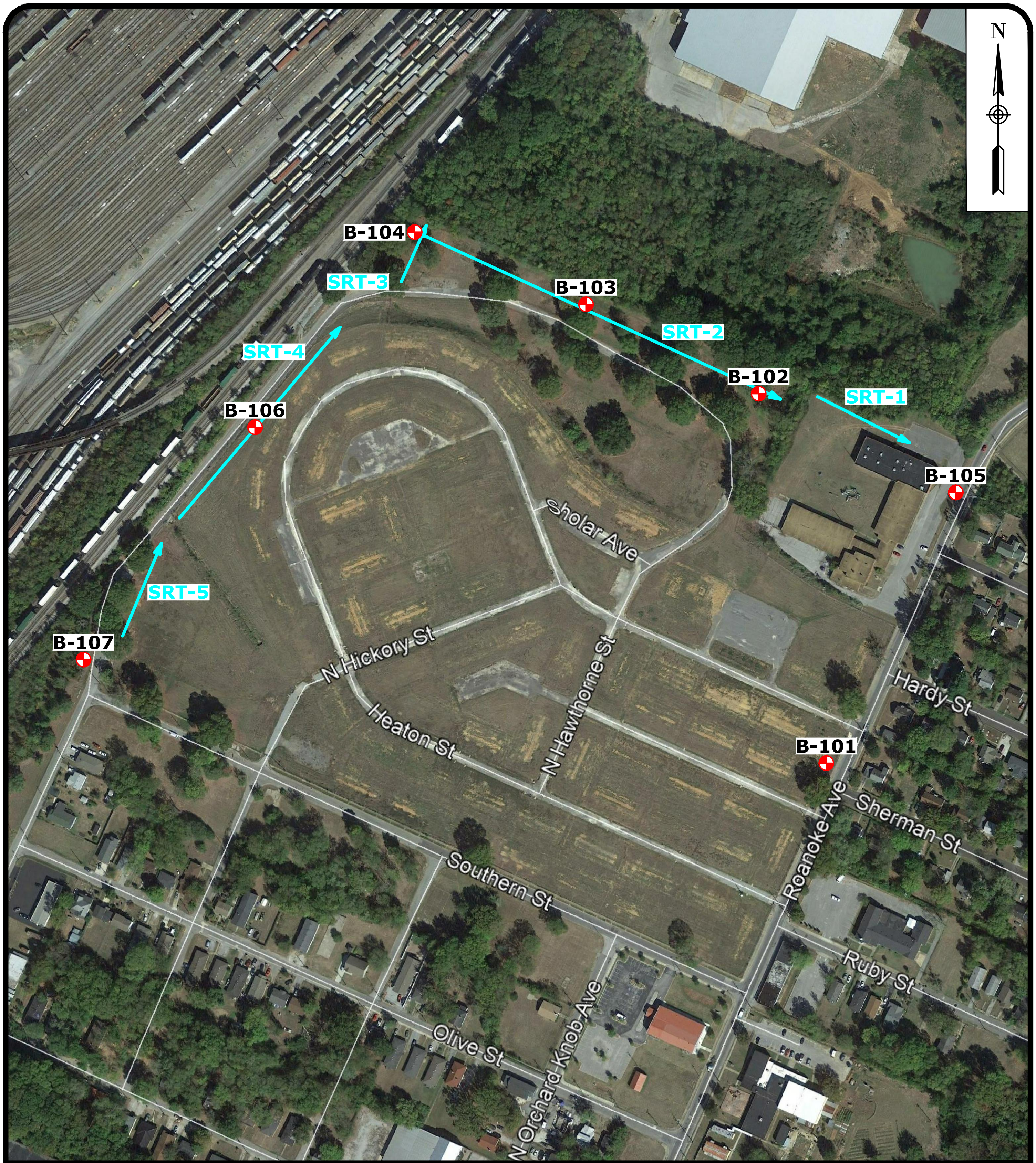
Figure 2 – Boring/Seismic Profile Location Plan

Figure 3 – Geophysical Data Profiles – Lines SRT-1 through SRT-5



SOURCE: USGS 7.5 Minute Topographic Map -- CHATTANOOGA, TENNESSEE (1976)
DRAWING FOR ILLUSTRATION PURPOSES ONLY

SITE LOCATION PLAN	SCALE: 1"=2,000'	FIGURE NO.
FORMER HARRIETT TUBMAN HOMES SITE CHATTANOOGE, TENNESSEE	DATE: 12/11/2018 PROJECT NUMBER 4181-18-046	**1**



NOTES:

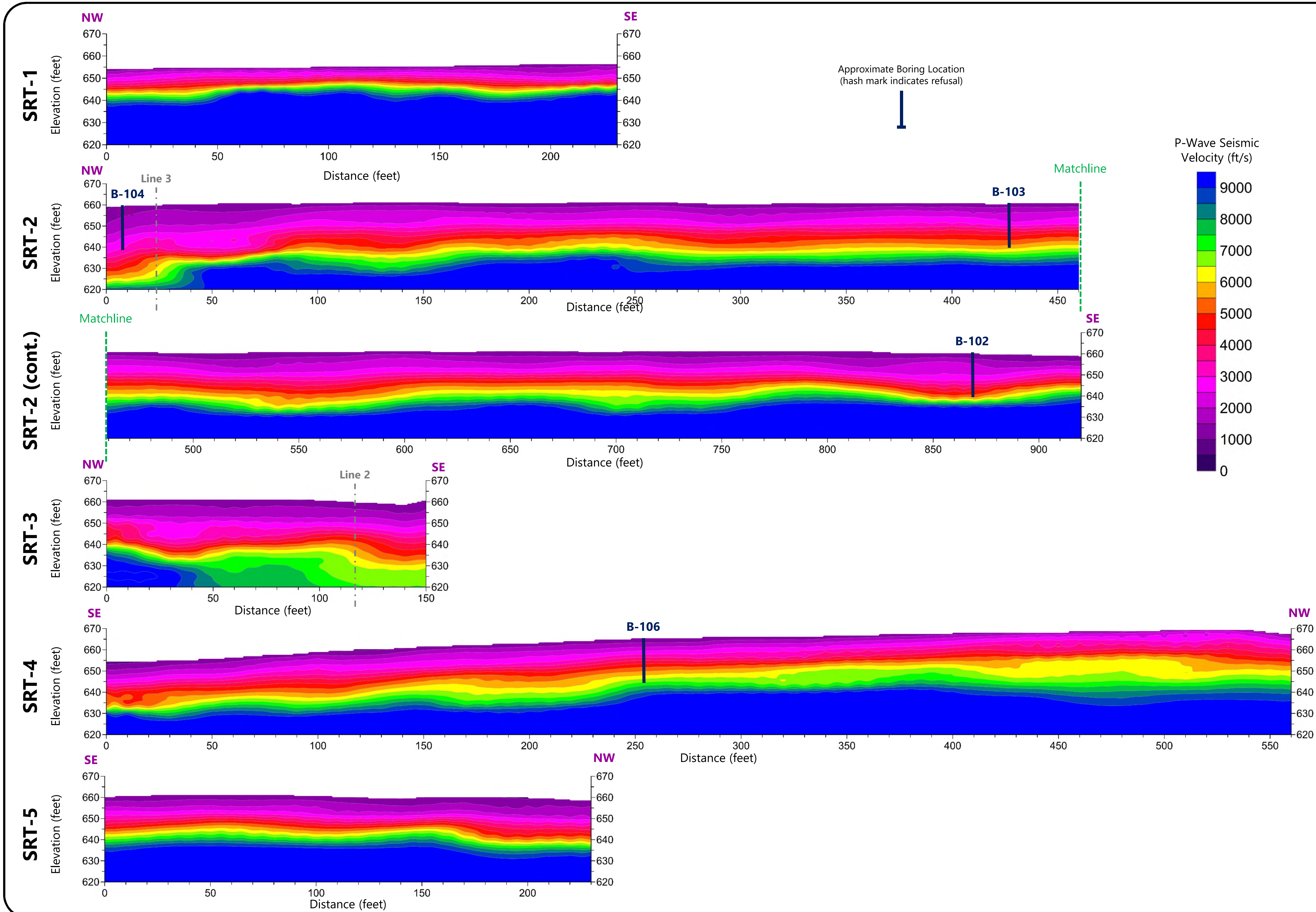
- DRAWING FOR ILLUSTRATIVE PURPOSES ONLY
- BASE IMAGE OBTAINED FROM GOOGLE EARTH

LEGEND:

- APPROXIMATE BORING LOCATION

- APPROXIMATE LOCATION OF SEISMIC PROFILES (SRT)

S&ME	BORING/SEISMIC PROFILE LOCATION PLAN	SCALE: 1"=300'	FIGURE NO.
		DATE: 12/11/2018	2
	FORMER HARRIETT TUBMAN HOMES SITE CHATTANOOGE, TENNESSEE	PROJECT NUMBER 4181-18-046	



GEOPHYSICAL DATA PROFILES – LINES SRT-1 THROUGH SRT-5

FORMER HARRIET TUBMAN HOMES SITE
CHATTANOOGA, TENNESSEE

SCALE:
AS SHOWN

DATE:
12/13/2018

PROJECT NUMBER
4181-18-046

FIGURE NO.
3

Appendix II

Field Exploration Procedures

Test Boring Record Legend

Test Boring Records

HOLLOW STEM AUGERING PROCEDURES
WITH STANDARD PENETRATION RESISTANCE TESTING
ASTM D 1586

The borings were advanced using auger drilling techniques. At regular intervals, soil samples were obtained with a standard 1.4-inch I.D., 2.0-inch O.D., split-tube sampler. The sampler was initially seated 6 inches to penetrate any loose cuttings and then driven an additional foot with blows of a 140-pound hammer falling 30 inches. The number of hammer blows required to drive the sampler the final foot is the standard penetration resistance. Standard penetration resistance, when properly evaluated, is an index to the soil's strength and density. The criteria used during this exploration are presented on the Test Boring Record Legend.

Representative portions of the soil samples, thus obtained, were placed in sealed containers and transported to the laboratory. The engineer selected samples for laboratory testing. The Test Boring Records in this Appendix provide the soil descriptions and penetration resistances.

Soil drilling and sampling equipment may not be capable of penetrating hard cemented soils, thin rock seams, large boulders, waste materials, weathered rock, or sound continuous rock. Refusal is the term applied to materials that cannot be penetrated with soil drilling equipment or where the standard penetration resistance exceeds 100 blows per foot. Core drilling is needed to determine the character and continuity of the refusal materials.

SHELBY TUBE SAMPLING PROCEDURES
ASTM D 1587

Shelby tube samples were obtained for laboratory testing. A 3-inch O.D., 16-gauge, steel tube was slowly and uniformly pushed into the soil at the desired sampling level. The tube was then removed from the ground and the encased soil was sealed at the ends to prevent loss of moisture. The depth at which Shelby tube samples were taken is indicated on the Test Boring Records.

TEST BORING/PIT RECORD LEGEND

FINE AND COARSE GRAINED SOIL INFORMATION

COARSE GRAINED SOILS (SANDS & GRAVELS) N	Relative Density	FINE GRAINED SOILS (SILTS & CLAYS) N	Consistency	Qu, KSF Estimated
0-4	Very Loose	0-1	Very Soft	0-0.5
5-10	Loose	2-4	Soft	0.5-1
11-20	Firm	5-8	Firm	1-2
21-30	Very Firm	9-15	Stiff	2-4
31-50	Dense	16-30	Very Stiff	4-8
Over 50	Very Dense	Over 31	Hard	8+

PARTICLE SIZE

Boulders	Greater than 300 mm (12 in)
Cobbles	75 mm to 300 mm (3 to 12 in)
Gravel	4.74 mm to 75 mm (3/16 to 3 in)
Coarse Sand	2 mm to 4.75 mm
Medium Sand	0.425 mm to 2 mm
Fine Sand	0.075 mm to 0.425 mm
Silts & Clays	Less than 0.075 mm

The **STANDARD PENETRATION TEST** as defined by ASTM D 1586 is a method to obtain a disturbed soil sample for examination and testing and to obtain relative density and consistency information. A standard 1.4-inch I.D./2-inch O.D. split-barrel sampler is driven three 6-inch increments with a 140 lb. hammer falling 30 inches. The hammer can either be of a trip, free-fall design, or actuated by a rope and cathead. The blow counts required to drive the sampler the final two increments are added together and designate the N-value defined in the above tables.

ROCK PROPERTIES

ROCK QUALITY DESIGNATION (RQD)

Percent RQD	Quality
0-25	Very Poor
25-50	Poor
50-75	Fair
75-90	Good
90-100	Excellent

ROCK HARDNESS

Very Hard:	Rock can be broken by heavy hammer blows
Hard:	Rock cannot be broken by thumb pressure, but can be broken by moderate hammer blows.
Moderately Hard:	Small pieces can be broken off along sharp edges by considerable hard thumb pressure; can be broken with light hammer blows.
Soft:	Rock is coherent but breaks very easily with thumb pressure at sharp edges and crumbles with firm hand pressure.
Very Soft:	Rock disintegrates or easily compresses when touched; can be hard to very hard soil.

$$\text{RQD} = \frac{\text{Sum of 4 in. and longer Rock Pieces Recovered}}{\text{Length of Core Run}} \times 100$$

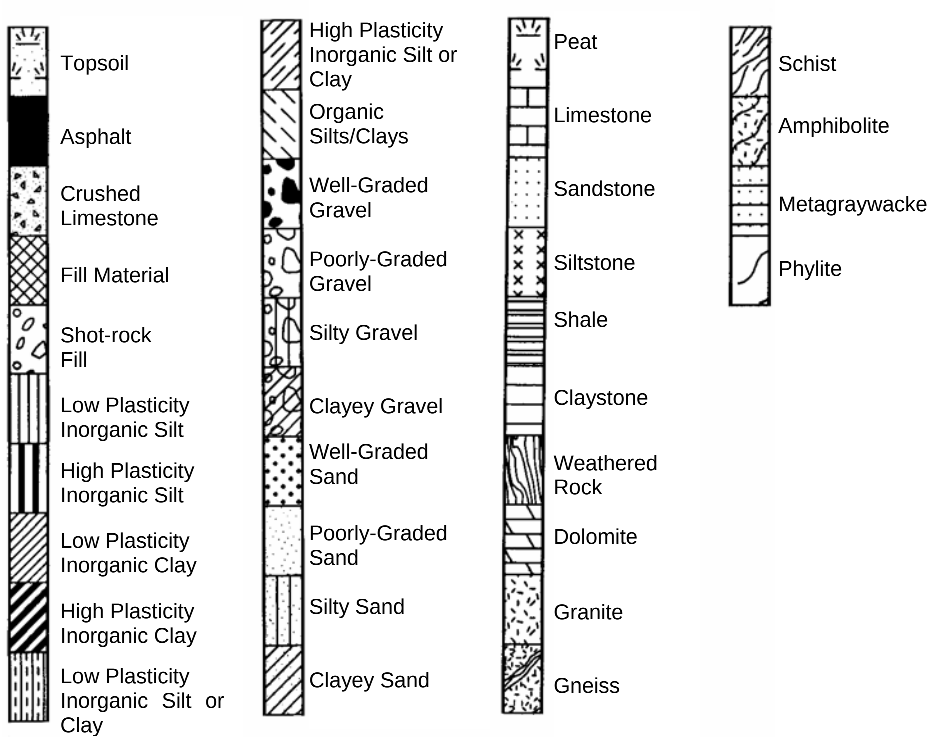
$$\text{Recovery} = \frac{\text{Length of Rock Core Recovered}}{\text{Length of Core Run}} \times 100$$

43 RQD
NQ
63 REC

Core Diameter	Inches
BQ	1-7/16
NQ	1-7/8
HQ	2-1/2

SYMBOLS

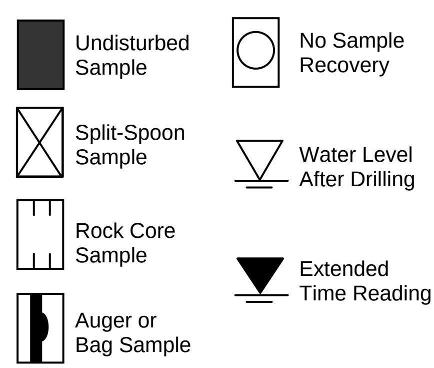
KEY TO MATERIAL TYPES



SOIL PROPERTY SYMBOLS

N:	Standard Penetration, BPF
M:	Moisture Content, %
LL:	Liquid Limit, %
PI:	Plasticity Index, %
Qp:	Pocket Penetrometer Value, TSF
Qu:	Unconfined Compressive Strength Estimated Qu, TSF
γ_D:	Dry Unit Weight, PCF
F:	Fines Content

SAMPLING SYMBOLS



TEST BORING RECORD

BORING NO.: B-101

PROJECT: Former Harriett Tubman Homes Site			JOB NO: 4181-18-046		SHEET 1 OF 1
PROJECT LOCATION: Chattanooga, Tennessee					
ELEVATION: 662 feet ±	BORING STARTED: 11/16/2018		RIG TYPE: CME-550		BORING DIA. (IN): 3.25
DRILLING METHOD: Hollow-Stem Augers	BORING COMPLETED: 11/16/2018		HAMMER: Automatic		

GROUNDWATER:
Dry ATD

Remarks:

G	ELEV. (FT.)	DEPTH (FT.)	MATERIAL DESCRIPTION	L	S	R	M	PI	STANDARD PENETRATION RESISTANCE (N)	BLOWS/6"
	662.0	0	SILTY CLAY (CL), brown and gray, stiff to firm — ALLUVIUM							
									10	5 - 5 - 5 **(10)**
									5	1 - 2 - 3 **(5)**
	657.0	5 (5')	SHELBY TUBE ATTEMPT (18 inches recovered)			75%	26.6	27		
	655.0	7 (7')	No Sample Recovered							
	654.9		*Auger refusal at 7.1 feet, boring terminated*							

BORING RECORD S&ME - SPLIT LITHOLOGY TUBMAN HOMES DUE DILIGENCE BORINGS.GPJ 2016.GDT 12/13/18

Borehole ID: 101

Project Manager: David Grass, PE

TEST BORING RECORD

BORING NO.: B-102

PROJECT: Former Harriett Tubman Homes Site	JOB NO: 4181-18-046	SHEET 1 OF 1
PROJECT LOCATION: Chattanooga, Tennessee		
ELEVATION: 660 feet ±	BORING STARTED: 11/16/2018	RIG TYPE: CME-550
BORING DIA. (IN): 3.25		
DRILLING METHOD: Hollow-Stem Augers	BORING COMPLETED: 11/16/2018	HAMMER: Automatic

GROUNDWATER:
14 feet ATD

Remarks:

G	ELEV. (FT.)	DEPTH (FT.)	MATERIAL DESCRIPTION	L	S	R	M	PI	STANDARD PENETRATION RESISTANCE (N)	BLOWS/6"
	660.0	0	SILTY CLAY (CL) with trace chert, red-brown, stiff to firm	FILL					10	3 - 5 - 5 **(10)**
									8	3 - 4 - 4 **(8)**
	655.0	5'	SHELBY TUBE ATTEMPT (24 inches recovered)			100%	23.0	40		
	653.0	7'	SILTY CLAY (CL) with chert fragments, yellow-brown, very stiff						19	6 - 13 - 6 **(19)**
	648.0	12'	SILTY CLAY (CL), gray, moist to wet, stiff	RESIDUUM						
▽									9	2 - 3 - 6 **(9)**
									11	3 - 4 - 7 **(11)**
	640.0	20	*Boring terminated at 20 feet*							

BORING RECORD S&ME - SPLIT LITHOLOGY TUBMAN HOMES DUE DILIGENCE BORINGS.GPJ 2016.GDT 12/13/18

Borehole ID: 102

Project Manager: David Grass, PE

TEST BORING RECORD

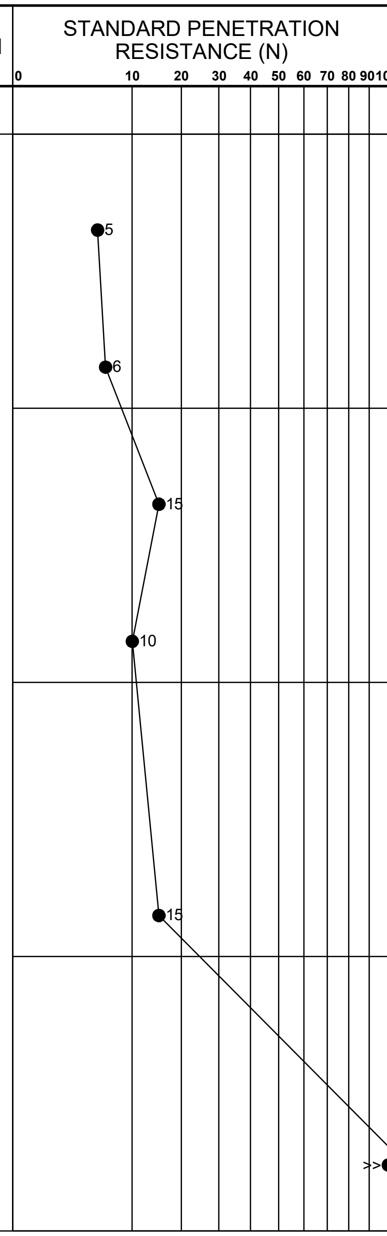
BORING NO.: B-103

PROJECT: Former Harriett Tubman Homes Site		JOB NO: 4181-18-046	SHEET 1 OF 1
PROJECT LOCATION: Chattanooga, Tennessee			
ELEVATION: 660 feet ±	BORING STARTED: 11/16/2018	RIG TYPE: CME-550	BORING DIA. (IN): 3.25
DRILLING METHOD: Hollow-Stem Augers	BORING COMPLETED: 11/16/2018	HAMMER: Automatic	

GROUNDWATER:
∇ 12 feet ATD

Remarks:

G	ELEV. (FT.)	DEPTH (FT.)	MATERIAL DESCRIPTION		L	S	R	M	PI	STANDARD PENETRATION RESISTANCE (N)	BLOWS/6"
	660.0	0									
	659.7	0.33'	TOPSOIL - 4 inches								
			SILTY CLAY (CL) with trace chert, red-brown, firm	FILL						5	2 - 2 - 3 **(5)**
										6	1 - 3 - 3 **(6)**
	654.5	5.5'	SILTY CLAY (CL), brown, yellow-brown, and gray, very stiff to stiff							15	4 - 6 - 9 **(15)**
										10	2 - 6 - 4 **(10)**
∇	648.0	12'	SILTY CLAY (CL), gray, stiff to hard	RESIDUUM						15	4 - 7 - 8 **(15)**
										>>	15 - 50/1 **(50+)**
	640.0	20	*Boring terminated at 20 feet*								



Project Manager: David Grass, PE

BORING RECORD S&ME - SPLIT LITHOLOGY TUBMAN HOMES DUE DILIGENCE BORINGS.GPJ 2016.GDT 12/13/18

Borehole ID: 103

TEST BORING RECORD

BORING NO.: B-104

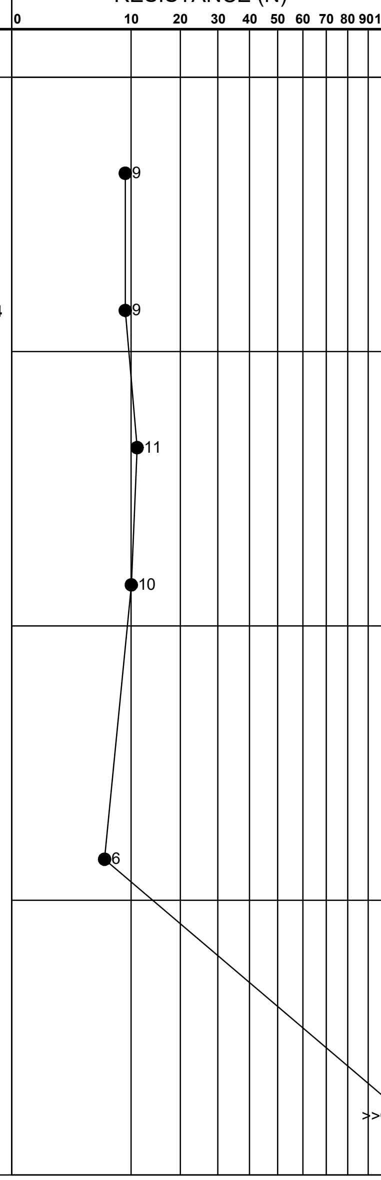
PROJECT: Former Harriett Tubman Homes Site	JOB NO: 4181-18-046	SHEET 1 OF 1
PROJECT LOCATION: Chattanooga, Tennessee		

ELEVATION: 658 feet ±	BORING STARTED: 11/16/2018	RIG TYPE:CME-550	BORING DIA. (IN): 3.25
DRILLING METHOD: Hollow-Stem Augers	BORING COMPLETED: 11/16/2018	HAMMER: Automatic	

GROUNDWATER: ∇ 14.4 feet ATD	Remarks:

ELEV. (FT.)	DEPTH (FT.)	MATERIAL DESCRIPTION	L	M	PI	BLOWS/6"
658.0	0	SILTY CLAY (CL) with trace chert, red-brown, stiff	FILL			5 - 3 - 6 **(9)**
				21.4	24	3 - 4 - 5 **(9)**
	5					5 - 5 - 6 **(11)**
650.0	8'	SILTY CLAY (CL) with gravel, brown, stiff				4 - 5 - 5 **(10)**
	10					
646.0	12'	SILTY CLAY (CL), gray, firm to hard	RESIDUUM			1 - 3 - 3 **(6)**
	15					
						6 - 50/4 **(50+)**
638.0	20	*Boring terminated at 20 feet*				

STANDARD PENETRATION RESISTANCE (N)



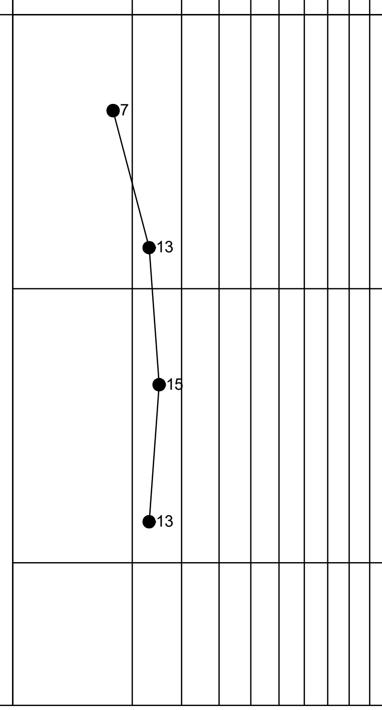
Project Manager: David Grass, PE

TEST BORING RECORD

BORING NO.: B-105

PROJECT: Former Harriett Tubman Homes Site		JOB NO: 4181-18-046	SHEET 1 OF 1
PROJECT LOCATION: Chattanooga, Tennessee			
ELEVATION: 660 feet ±	BORING STARTED: 11/16/2018	RIG TYPE: CME-550	BORING DIA. (IN): 3.25
DRILLING METHOD: Hollow-Stem Augers	BORING COMPLETED: 11/16/2018	HAMMER: Automatic	
GROUNDWATER: Dry ATD	Remarks:		

ELEV. (FT.)	DEPTH (FT.)	MATERIAL DESCRIPTION		STANDARD PENETRATION RESISTANCE (N)	BLOWS/6"
660.0	0	SILTY CLAY (CL), red-brown, firm	ALLUVIUM	7	2 - 3 - 4 **(7)**
657.0	3'	SILTY CLAY (CL), yellow-brown and red-brown, stiff		13	4 - 5 - 8 **(13)**
				15	4 - 7 - 8 **(15)**
652.0	8'	CLAY (CH), yellow-brown and gray, stiff		13	3 - 5 - 8 **(13)**
647.4		*Auger refusal at 12.6 feet, boring terminated*			



Project Manager: David Grass, PE

TEST BORING RECORD

BORING NO.: B-106

PROJECT: Former Harriett Tubman Homes Site	JOB NO: 4181-18-046	SHEET 1 OF 1
PROJECT LOCATION: Chattanooga, Tennessee		
ELEVATION: 665 feet ±	BORING STARTED: 11/16/2018	RIG TYPE: CME-550 / BORING DIA. (IN): 3.25
DRILLING METHOD: Hollow-Stem Augers	BORING COMPLETED: 11/16/2018	HAMMER: Automatic

GROUNDWATER:
Dry ATD

Remarks:

ELEV. (FT.)	DEPTH (FT.)	MATERIAL DESCRIPTION		R	M	PI	STANDARD PENETRATION RESISTANCE (N)	BLOWS/6"
665.0	0	SILTY CLAY (CL), red-brown, firm to stiff	ALLUVIUM				5	2 - 2 - 3 **(5)**
							10	2 - 4 - 6 **(10)**
660.0	5'	SHELBY TUBE ATTEMPT (21 inches recovered)		87%	25.9	22		
658.0	7'	SILTY CLAY (CL), yellow-brown and brown, very stiff					16	4 - 7 - 9 **(16)**
653.0	12'	CLAY (CH), brown and yellow-brown, stiff	RESIDUUM				12	3 - 5 - 7 **(12)**
							11	4 - 5 - 6 **(11)**
645.0	20	*Boring terminated at 20 feet*						

Project Manager: David Grass, PE

BORING RECORD S&ME - SPLIT LITHOLOGY TUBMAN HOMES DUE DILIGENCE BORINGS.GPJ 2016.GDT 12/13/18

Borehole ID: 106

TEST BORING RECORD

BORING NO.: B-107

PROJECT: Former Harriett Tubman Homes Site		JOB NO: 4181-18-046	SHEET 1 OF 1
PROJECT LOCATION: Chattanooga, Tennessee			
ELEVATION: 662 feet ±	BORING STARTED: 11/16/2018	RIG TYPE: CME-550	BORING DIA. (IN): 3.25
DRILLING METHOD: Hollow-Stem Augers	BORING COMPLETED: 11/16/2018	HAMMER: Automatic	

GROUNDWATER:	Remarks:
Dry ATD	

ELEV. (FT.)	DEPTH (FT.)	MATERIAL DESCRIPTION		STANDARD PENETRATION RESISTANCE (N)	BLOWS/6"
662.0	0	SILTY CLAY (CL), red-brown, stiff	ALLUVIUM	10	3 - 4 - 6 **(10)**
				13	3 - 5 - 8 **(13)**
656.5	5.5'	SILTY CLAY (CL), brown, stiff		14	3 - 5 - 9 **(14)**
				12	3 - 5 - 7 **(12)**
650.0	12'	SILTY CLAY (CL), yellow-brown, stiff to hard	RESIDUUM	11	6 - 5 - 6 **(11)**
				>>	14 - 50/2 **(50+)**
642.0	20	*Boring terminated at 20 feet*			

Project Manager: David Grass, PE

Appendix III

Laboratory Test Procedures

Laboratory Test Results

NATURAL MOISTURE
ASTM D 2216, EM 1110-2-1906

The moisture content of soils is an indicator of various physical properties, including strength and compressibility. Selected samples obtained during exploratory drilling were taken from their sealed containers. Each sample was weighed and then placed in an oven heated to 110ºC ± 5ºC. The sample remained in the oven until the free moisture had evaporated. The dried sample was removed from the oven, allowed to cool, and re-weighed. The moisture content was computed by dividing the weight of evaporated water by the weight of the dry sample. The results, expressed as a percent, are shown on the attached Laboratory Test Results Summary.

ATTERBERG LIMITS DETERMINATION
ASTM D 4318/AASHTO T89/T90

Representative samples were subjected to Atterberg limits testing to determine the soil's plasticity characteristics. The plasticity index (PI) is the range of moisture content over which the soil deforms as a plastic material. The liquid limit (LL) marks the transition from the plastic state to the liquid state. The plastic limit (PL) marks the transition from the plastic state to the solid state.

To determine the liquid limit, a soil specimen is wetted until it is in a viscous fluid state. A portion of this soil is then placed in a brass cup of standardized dimensions, and a groove made through the middle of the soil specimen with a grooving tool of standardized dimensions. The cup is attached to a cam that lifts the cup 10 mm, and then allows the cup to fall and strike a rubber base of standardized hardness. The cam is rotated at about 2 drops per second until the two halves of the soil specimen come in contact at the bottom of the groove along a distance of 13 mm. The number of blows required to make this degree of contact is recorded, and a portion of the specimen is subjected to a moisture content determination. Additional water is added to the remainder of the specimen, and the grooving process and cam action process repeated. This testing sequence is repeated until the soil flows as a heavy viscous fluid. The number of blows vs. moisture content is then plotted on semi-logarithmic graph paper, and the moisture content corresponding to 25 blows is designated the liquid limit.

The plastic limit is the lowest moisture content at which the soil is sufficiently plastic to be manually rolled into threads 3 mm in diameter. It is determined by taking a pat of soil remaining from the liquid limit test, and repeatedly rolling, kneading, and air drying the specimen until the soil breaks into threads about 3 mm in diameter and 3 to 10 mm long. The moisture content of these soil threads is then determined, and is designated the plastic limit. The results of these tests are presented on the Laboratory Test Results Summary.

UNCONFINED COMPRESSIVE STRENGTH OF SOIL
ASTM D 2166/AASHTO T208-92

The unconfined compression test is an unconsolidated-undrained triaxial shear test with no lateral confining pressure. This test is used to determine the shear strength (cohesion) of clayey soils and rock. Shelby samples were prepared by cutting the ends perpendicular to the applied load. The sample was placed in a testing device and incrementally increasing vertical loads were applied until it failed. The test results are provided on the Unconfined Compression Test Reports.

Former Harriet Tubman Homes Site
Chattanooga, Tennessee
S&ME Project No. 4181-18-046

Laboratory Test Results Summary

Boring Number	Sample Type	Sample Depth (ft)	Moisture Content (%)	ATTERBERG LIMITS		
				Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
B-101	ST	5 – 7	26.6	42	15	27
B-102	ST	5 – 7	23.0	60	20	40
B-104	SPT	3.5 - 5	25.9	42	18	24
B-106	ST	5 - 7	21.4	44	22	22

SPT – Standard Penetration Test Sample
ST –Shelby Tube Sample

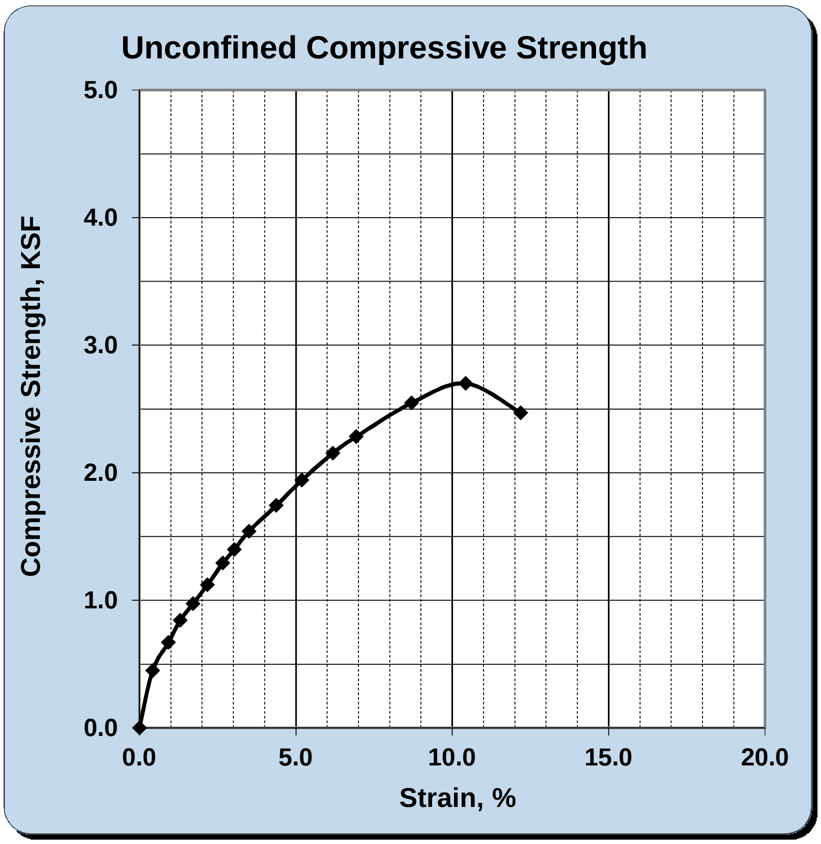
Form No. TR-D2166-01
Revision No. : 1
Revision Date: 08/16/17

UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS

ASTM D2166

S&ME, Inc. - Chattanooga: 4291 Highway 58, Suite 101, Chattanooga, TN 37416

Project No.:	4181-18-046 Log #: 18-243	Report Date:	12/11/2018
Project Name:	Former Harriett Tubman Homes Site	Test Date(s):	12/5/2018
Client Name:	Barge Design Solutions		
Client Address:	1110 Market Street, Suite 200		
Boring No.:	B-101 Sample No. UD	Sample Date:	11/16/2018
Location:	Onsite Boring	Depth:	5'-7'
Sample Description:	Dark Brown Silty Clay w/ Reddish Brown Streaking		CL



Failed Specimen

Type of Sample: Undisturbed
Source of Moisture Sample: Specimen

Liquid Limit: 42
Plasticity Index: 27
Height to Diameter Ratio: 2.0
Rate of Strain (%/min.): 1.4
Strain at Failure: 10.4

Initial Dry Unit Weight: 98.6 pcf Initial Water Content: 26.6%
Unconfined Compressive Strength, q_u: **2.701** KSF
Undrained Shear Strength, s_u: **1.350** KSF

References / Comments / Deviations:

David Grass, PE		Project Engineer	12/11/2018
Technical Responsibility	*Signature*	*Position*	*Date*

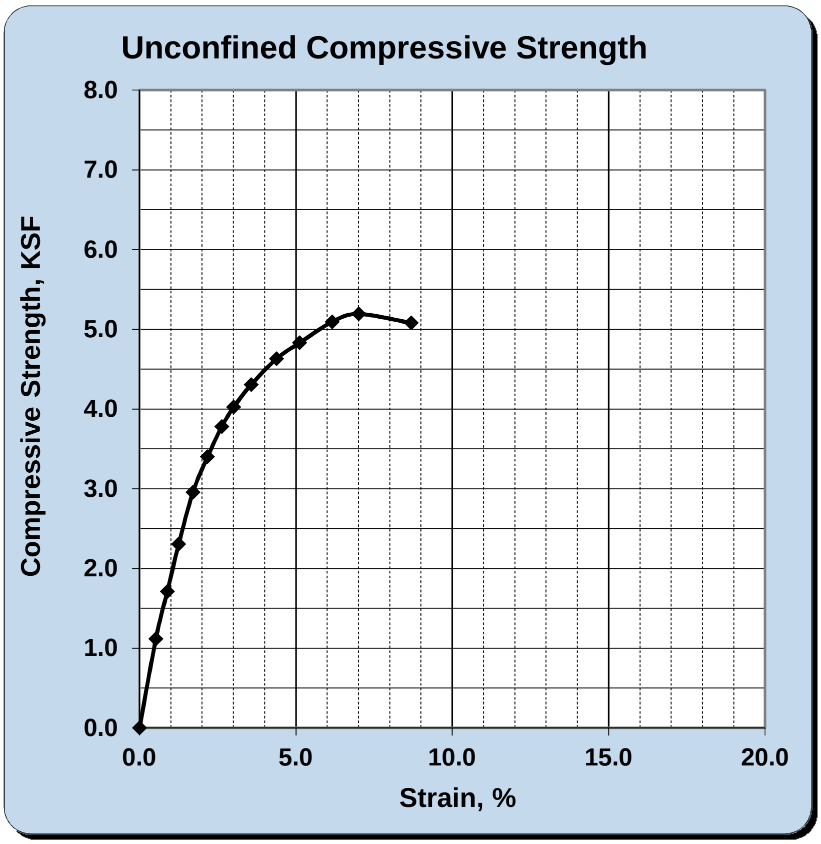
Form No. TR-D2166-01
Revision No. : 1
Revision Date: 08/16/17

UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS

ASTM D2166

S&ME, Inc. - Chattanooga: 4291 Highway 58, Suite 101, Chattanooga, TN 37416

Project No.:	4181-18-046 Log #: 18-243	Report Date:	12/11/2018
Project Name:	Former Harriet Tubman Homes Site	Test Date(s):	12/5/2018
Client Name:	Barge Design Solutions		
Client Address:	1110 Market Street, Suite 200		
Boring No.:	B-102 Sample No. UD	Sample Date:	11/16/2018
Location:	Onsite Boring	Depth:	5'-7'
Sample Description:	Reddish Brown Silty Clay		CH



Failed Specimen

Type of Sample:	Undisturbed
Source of Moisture Sample:	Specimen
Liquid Limit:	60
Plasticity Index:	40
Height to Diameter Ratio:	2.0
Rate of Strain (%/min.):	1.5
Strain at Failure:	7.0

Initial Dry Unit Weight: 104.3 pcf   Initial Water Content: 23.0%

Unconfined Compressive Strength, q_u: **5.192** KSF

Undrained Shear Strength, s_u: **2.596** KSF

References / Comments / Deviations:

David Grass, PE		Project Engineer	12/11/2018
Technical Responsibility	*Signature*	*Position*	*Date*

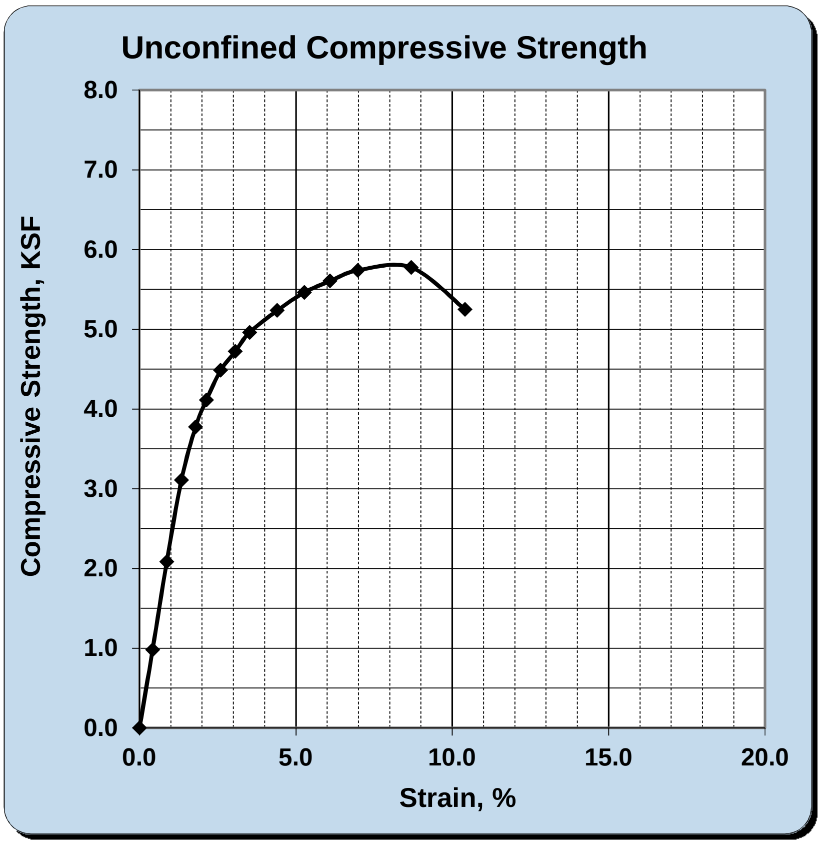
Form No. TR-D2166-01
Revision No. : 1
Revision Date: 08/16/17

UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS

ASTM D2166

S&ME, Inc. - Chattanooga: 4291 Highway 58, Suite 101, Chattanooga, TN 37416

Project No.:	4181-18-046 Log #: 18-243	Report Date:	12/11/2018
Project Name:	Former Harriet Tubman Homes Site	Test Date(s):	12/5/2018
Client Name:	Barge Design Solutions		
Client Address:	1110 Market Street, Suite 200		
Boring No.:	B-106 Sample No. UD	Sample Date:	11/16/2018
Location:	Onsite Boring	Depth:	5'-7'
Sample Description:	Reddish Brown Silty Clay		



Failed Specimen

Type of Sample: Undisturbed
Source of Moisture Sample: Specimen

Liquid Limit: TBD
Plasticity Index: TBD
Height to Diameter Ratio: 2.0
Rate of Strain (%/min.): 1.4
Strain at Failure: 8.3

Initial Dry Unit Weight: 100.1 pcf Initial Water Content: 25.9%
Unconfined Compressive Strength, q_u: **5.776** KSF
Undrained Shear Strength, s_u: **2.888** KSF

References / Comments / Deviations:

David Grass, PE		Project Engineer	12/11/2018
Technical Responsibility	*Signature*	*Position*	*Date*

Appendix IV

Important Information About Your Geotechnical Engineering Report

Important Information About Your
Geotechnical Engineering Report

Variations in subsurface conditions can be a principal cause of construction delays, cost overruns and claims. The following information is provided to assist you in understanding and managing the risk of these variations.

Geotechnical Findings Are Professional Opinions

Geotechnical engineers cannot specify material properties as other design engineers do. Geotechnical material properties have a far broader range on a given site than any manufactured construction material, and some geotechnical material properties may change over time because of exposure to air and water, or human activity.

Site exploration identifies subsurface conditions at the time of exploration and only at the points where subsurface tests are performed or samples obtained. Geotechnical engineers review field and laboratory data and then apply their judgment to render professional opinions about site subsurface conditions. Their recommendations rely upon these professional opinions. Variations in the vertical and lateral extent of subsurface materials may be encountered during construction that significantly impact construction schedules, methods and material volumes. While higher levels of subsurface exploration can mitigate the risk of encountering unanticipated subsurface conditions, no level of subsurface exploration can eliminate this risk.

Scope of Geotechnical Services

Professional geotechnical engineering judgment is required to develop a geotechnical exploration scope to obtain information necessary to support design and construction. A number of unique project factors are considered in developing the scope of geotechnical services, such as the exploration objective; the location, type, size and weight of the proposed structure; proposed site grades and improvements; the construction schedule and sequence; and the site geology.

Geotechnical engineers apply their experience with construction methods, subsurface conditions and exploration methods to develop the exploration scope. The scope of each exploration is unique based on available project and site information. Incomplete project information or constraints on the scope of exploration increases the risk of variations in subsurface conditions not being identified and addressed in the geotechnical report.

Services Are Performed for Specific Projects

Because the scope of each geotechnical exploration is unique, each geotechnical report is unique. Subsurface conditions are explored and recommendations are made for a specific project. Subsurface information and recommendations may not be adequate for other uses. Changes in a proposed structure location, foundation loads, grades, schedule, etc. may require additional geotechnical exploration, analyses, and consultation. The geotechnical engineer should be consulted to determine if additional services are required in response to changes in proposed construction, location, loads, grades, schedule, etc.

Geo-Environmental Issues

The equipment, techniques, and personnel used to perform a geo-environmental study differ significantly from those used for a geotechnical exploration. Indications of environmental contamination may be encountered incidental to performance of a geotechnical exploration but go unrecognized. Determination of the presence, type or extent of environmental contamination is beyond the scope of a geotechnical exploration.

Geotechnical Recommendations Are Not Final

Recommendations are developed based on the geotechnical engineer's understanding of the proposed construction and professional opinion of site subsurface conditions. Observations and tests must be performed during construction to confirm subsurface conditions exposed by construction excavations are consistent with those assumed in development of recommendations. It is advisable to retain the geotechnical engineer that performed the exploration and developed the geotechnical recommendations to conduct tests and observations during construction. This may reduce the risk that variations in subsurface conditions will not be addressed as recommended in the geotechnical report.