Report of Geotechnical Exploration
Miller Lane Bridge Replacement
Johnson City, Tennessee
S&ME Project No. 1281-20-005

PREPARED FOR:

Vaughn & Melton
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March 11, 2020

March 11, 2020

Vaughn & Melton
127 Bob Fitz Road
Gray, Tennessee 37615

Attention: Mr. Dean Helstrom, PE

Reference: **Report of Geotechnical Exploration**
Miller Lane Bridge Replacement
Johnson City, Tennessee
S&ME Project No. 1281-20-005

Dear Mr. Helstrom:

This report presents the results of the geotechnical exploration for the bridge replacement project at Miller Lane Bridge over Sinking Creek in Johnson City, Tennessee. Our work was performed in general accordance with S&ME Proposal No. 41-1900580 dated January 28, 2020.

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.

James Andrew Reed
No. 117056
3/11/2020

Drew Reed, 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 two soil test borings. Rock was cored in the test borings to evaluate the continuity, composition, and load resistance of the refusal materials. The samples collected during our exploration were returned to our laboratory where they were further evaluated by a professional engineer.

2. A laboratory testing program was performed on selected soil and rock core samples to provide additional data to better define engineering properties of the subsurface materials and evaluate foundation alternatives. The laboratory testing program consisted of natural moisture content, Atterberg limits and grain size testing on selected soil samples. Unconfined compressive strength testing was performed on two rock core samples.

3. Undifferentiated bedrock of the Knox 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. Based on the nature of this project, it is our opinion that the risk for sinkhole development is not increased due to the construction of the replacement bridge.

4. Subsurface conditions consisted of fill and alluvial soils to auger refusal. Fill was encountered in boring B-1 drilled on the west side of the bridge and was composed of brown and orange brown silty sand. Alluvium was encountered in both borings beneath the fill or beneath the topsoil. The alluvium was composed of brown or yellow-brown clay, silt, gravel and cobbles. A thin interval of residual soil was encountered in boring B-2 beneath the alluvium. The residuum was composed or orange-brown silty clay.

5. Auger refusal was encountered in the borings at depths of about 13½ and 12 feet below the existing ground surface. Rock coring was performed in each of the borings. Hard, continuous limestone bedrock was encountered from the auger refusal depths to the boring termination depths of about 20 feet.

6. Groundwater was encountered in the borings at depths that approximate the water level in Sinking Creek.

7. Pre-drilled H piles may be used to support the abutments. Foundations should be designed in accordance with TDOT standard design criteria. Subsequent report sections provide recommendations for the recommended foundation system.

1.0 Introduction

S&ME, Inc. has completed the geotechnical exploration at the Miller Lane Bridge Replacement site in Washington County, Tennessee. Our work was performed in general accordance with S&ME Proposal Number 41-1900580 dated January 28, 2020. Our services were authorized by Mr. Danl Hall, PE of Vaughn & Melton on February 5, 2020.

The purpose of our work is to explore the subsurface soil and rock conditions and groundwater level, provide feasible foundation recommendations, and provide applicable earthwork recommendations for the replacement bridge. 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. Design of retaining walls and a detailed slope stability analysis was also outside the scope of our services.

A Site Location Plan and a Boring Location Plan are included in Appendix I. A discussion of the field investigative procedures, a legend of soil classification and symbols, the Test Boring Records and photographs of the rock core recovered 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 project information provided to us by Mr. Dean Helstrom of Vaughn & Melton in the form of a request for proposal (RFQ #2019-47). The request for proposal outlines plans to replace the Miller Lane Bridge over Sinking Creek. A Site Location Plan, Figure 1, showing the general project site location is provided in Appendix I.

The existing bridge is constructed from a combination of structural steel beams and reinforced concrete. The bridge is constructed at a slight skew to Miller Lane with a width of about 16 feet and an overall length of about 18 feet. Wingwalls are present at each corner of the bridge and six-inch curbs run along either side of the bridge deck. The bridge is located within five to six feet of the travel edge of Sinking Creek Road to the west.

The bridge is currently supported on shallow foundations with a north and a south abutment. Both abutment foundations are undermined below the water line due to scour and the south abutment has significant cracking due to differential settlement. During our subsurface investigation, it appears that half of the south abutment has dropped about 10 inches more than is apparent in photographs presented in the provided request for proposal, indicating recent distress likely associated with heavy creek flow. Spalling of the concrete is also apparent.

We understand the new bridge will likely be a bottomless precast culvert. We have not been provided with design specific information relative to the type, size or abutment configuration of the replacement bridge. We understand this project will be designed using load and resistance factor design (LRFD).

We request the project information and any assumptions listed herein be reviewed and confirmed by the appropriate team members. Modifications to our recommendations may be required if the planned bridge differs from our stated information and/or assumptions.

3.0 Regional Geology

Johnson City, 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 the Johnson City Quadrangle, dated 1997, undifferentiated bedrock of the Knox Group underlies the site. The Knox Group is composed of various dolomite and siliceous limestone members. The rock is generally gray to blue-gray, fine to very fine-grained rock. Residual soils derived from the Knox Group are typically red-brown to yellow-brown clays with locally heavy amounts of chert fragments. The strata of the Knox formations weather to form an overburden typically in excess of 40 feet thick.

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, excavated at this site did not encounter open voids or other signs of incipient sinkhole conditions. 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, it is our opinion the proposed construction will not increase the risk of sinkhole development.

4.0 Subsurface Conditions

4.1 Field Exploration Procedures

The procedures used by S&ME, Inc. for field sampling and testing are in general accordance with ASTM and AASHTO 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 two soil test borings, B-1 and B-2, to obtain subsurface information at the project site. Rock coring was performed in the borings to evaluate the continuity and composition of refusal materials. Members of our engineering staff established the boring locations in the field by measuring distances and estimating right angles relative to on-site landmarks. The boring elevations were estimated from available GIS data. Therefore, 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.

Our field representative packaged the soil samples in sealed containers and boxed the rock core samples, labeled them for identification, and returned them to our 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 and rock descriptions are shown on the Test Boring Records in Appendix II. Samples were then selected for laboratory testing.

4.2 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.

4.2.1 Surface Materials

Asphalt underlain by basestone was encountered in boring B-1. About six inches of topsoil was encountered at the ground surface in boring B-2.

4.2.2 Fill

Fill was encountered below the asphalt and basestone in boring B-1 to a depth of about five feet. Fill is material that has been transported to its present location by man. The fill was generally composed of brown and orange-brown silty sand. Standard Penetration Test (SPT) N values in the fill ranged from 8 to 20 blows per foot, indicating loose to firm relative density. We have not been provided with documentation regarding how or when the fill was placed.

4.2.3 Alluvium

Alluvial soil was encountered in borings B-1 and B-2 to respective depths of about 13 and 11 feet below the ground surface. Alluvial soil is soil that has been transported to its present location by flowing water. The alluvial soil encountered at the site was composed of brown or yellow-brown clay, silt, gravel and cobbles. SPT N values in the alluvium ranged from 6 to 31 blows per foot, indicating a very stiff soil consistency in the clay and silt alluvial soils and a loose to dense relative density in the sandy or gravelly alluvial soils.

4.2.4 *Residuum*

A thin interval of residuum was encountered in boring B-2, below the alluvium and just above the bedrock. The residuum was composed of orange-brown silty clay with an SPT N value of 6, indicating a firm soil consistency.

4.2.5 *Auger Refusal*

Auger refusal was encountered in both borings B-1 and B-2 at respective depths of about 13½ and 12 feet below the ground surface.

4.2.6 *Bedrock*

Rock was cored in both borings. The rock core was generally composed of gray, very hard, continuous limestone.

Core recovery and rock quality designation (RQD) were measured for each core run. The rock core recovery is a measure of the length of core drilled to that recovered expressed as a percentage. The RQD is a measure of the rock quality based on the percentage of the rock core run containing pieces greater than 4 inches in length. Table 4-1 summarizes the core data.

Table 4-1 – Rock Core Summary

Boring No.	Depth to Bedrock Surface (feet)	Core Run Depth Intervals (feet)	RQD (%)	Core Recovery (%)	Coring Termination Depth (feet)	Bedrock Description
B-1	13.3	14.3* – 15.3	100	100	20.3	**13.3 ft. – 20.3 ft.:** LIMESTONE, light gray and gray, continuous, excellent quality, fine grained, 30° to 45° bedding, fresh, very hard
		14.3 – 20.3	94	100		
B-2	12.0	12.1* – 15.6	94	100	20.6	**12.0 ft. – 20.6 ft.:** LIMESTONE, light gray and gray, continuous, excellent quality, fine grained, 30° to 45° bedding, fresh, very hard
		15.6 – 20.6	100	100		

*Core run start depths differ from the depth to bedrock due to a casing advancer utilized to set casing prior to the start of NQ rock coring.

4.3 Water Levels

Groundwater was encountered in the borings at the approximate elevation of Sinking Creek at the time of drilling. At the time of drilling, recent heavy rainfall had resulted in elevated creek levels. We backfilled the boreholes shortly after completion due to safety concerns and delayed groundwater level measurements were not obtained. We expect groundwater levels at the site typically approximate the creek level.

4.4 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 and to evaluate the relative volume change potential of on-site soils. The resulting soil descriptions are shown on the Test Boring Records in Appendix II.

In addition to index property testing performed on soil samples, unconfined compression testing was performed on two samples of limestone rock core. The testing resulted in an unconfined compressive strength of about 30.7 kips per square inch (ksi) in a sample taken at a depth of about 16½ feet in boring B-1, and 24.8 ksi in a sample taken at a depth of about 12½ feet in boring B-2. The average unit weight of the rock core samples tested was about 170 pounds per cubic foot (pcf).

The laboratory test results and a brief description of the laboratory test procedures are presented in Appendix III.

5.0 Bridge Foundation Recommendations

The conclusions and recommendations presented in this report are based on the preceding project information and the results of this exploration. Variations in subsurface conditions may occur between the boring locations and could occur in small lateral distances. If it becomes apparent during construction the encountered conditions vary substantially from those presented herein, we should be notified. S&ME should be retained to evaluate and modify the recommendations of this report, if necessary. Also, if the scope of this project should change significantly from that described herein, S&ME should be contacted as the recommendations may need to be reevaluated.

5.1 Pre-drilled Steel H-Piles

Steel H-piles socketed in hard, competent bedrock may be used to support the proposed bridge abutments. The pile locations are typically drilled, with the diameter of the hole slightly larger than the pile. We recommend that the holes be extended 4 feet into bedrock. After the piles are seated in the hole by hammer, the rock socket of each pile should be backfilled with high strength, non-shrink grout. After the grout has set, the void space around the remainder of the pile may be backfilled with open graded washed stone or approved clean sand, as is TDOT's standard procedure. We expect the pile caps will be constructed about 5 feet below grade. TDOT requires a minimum length for steel H-piles of 10 feet as measured from the bottom of the bridge substructure.

5.1.1 *Pile Tip Resistance*

We understand TDOT typically uses HP10X42 or HP12X53 piles for steel H pile foundations. The design of pre-drilled steel H-piles end bearing on bedrock will be controlled by the structural capacity of the pile and should be determined in accordance with AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017 (AASHTO 2017) Article 6.9.4.1 with the resistance factors specified in Article 6.5.4.2 and Article 6.15. We calculated the nominal and factored axial compressive pile resistance for HP10X42, HP12X53 and HP14X73 piles as shown in Table 5-1. The resistances in Table 5-1 assume the H-piles have a yield strength of 50 kips per square inch (ksi), negligible moment and are fully braced along their lengths. Since the piles will be supported in competent bedrock, settlement should not be a project concern. We anticipate pile settlements of ½ inch or less. The axial compressive capacity of pile groups will be the sum of the individual pile capacities. No pile axial compression group reduction is required for end bearing piles supported in bedrock.

Table 5-1 – Pre-Drilled Steel H Pile Compressive Resistance

Pile Section	Nominal Pile Compressive Resistance (kip)	LRFD Resistance Factor*	Factored Pile Compressive Resistance (kip)
HP 10 X 42	300	0.6	180
HP 12 X 53	450		270
HP 14 X 73	700		420

*Resistance Factor from AASHTO LRFD Bridge Design Manual, 8th Ed. (2015), Section 6.5.4.2

5.1.2 *Lateral Load Analysis*

The lateral load capacity of the piles should be determined in accordance with AASHTO 2017 Article 10.7.3.12. For lateral load analyses, using a software program such as LPile, we recommend input parameters provided in Table 5-2 and Table 5-3. Group effects will need to be accounted for as discussed in AASHTO 2017 Article 10.7.2.4-1 and Table 10.7.2.4-1. Bedrock was encountered at an elevation of about 1780 feet in the borings. The following input parameters are recommended for use in lateral load analyses:

Overburden Soil – Sandy silt and clay, gravel and cobbles

Unit weight:	0.115 kcf
Buoyant unit weight:	0.053 kcf
Friction angle:	24 degrees
Static soil modulus, k:	100 pci

Bedrock – Limestone

Unit weight:	0.170 kcf
Buoyant unit weight:	0.108 kcf
Intact Uniaxial Compressive Strength:	25 ksi
Rock Mass Modulus, E_m:	8,000 ksi
Strain Factor, k_{rm}:	0.00015
Rock Quality Designation, RQD:	95%

5.1.3 Uplift

For uplift resistance at the abutments, if needed, we recommend the side friction of the overburden soils be neglected and the pullout resistance of the pile socketed and grouted into bedrock be used. The predrilled hole diameter should be used to calculate the side friction area. Once a nominal resistance is calculated based on the nominal unit side resistance value provided in Table 5-2, the resistance factor provided below should be used to factor the resulting resistance value.

The resistance values provided assume piles socketed into hard limestone bedrock and non-shrink grout with a minimum compressive strength of 4,000 psi. For pile group effects, uplift resistance should be evaluated in accordance with AASHTO 2017 Article 10.7.3.11 with the nominal uplift resistance of a pile group taken as the lesser of the sum of the individual piles or the uplift resistance of the pile group considered as a block.

Table 5-2 – Unit Side Resistance for Grouted Piles

Rock Type	Nominal Unit Side Resistance (ksf)	Resistance Factor*
Bedrock - Limestone	50	0.4

*Resistance Factor from AASHTO LRFD Bridge Design Manual, 8th Ed. (2015), Table 10.5.5.2.4-1

5.2 Scour

The bridge will bear on pre-drilled steel H-piles socketed into bedrock. We recommend that any below grade bridge or retaining wall elements associated with the construction which will be constructed in scour prone areas be extended to a depth of 2 feet below the potential scour depth. Based on laboratory testing performed on material sourced from the creek bottom, we recommend a D_{50} value of 1.0 inch be used to calculate the scour depth.

5.3 Seismic Considerations

Based on the drilling data, we recommend Seismic Site Class D for the proposed bridge (reference Table 3.10.3.1-1 – Site Class Definitions, AASHTO 2017) bearing on steel H piles founded on hard limestone bedrock. From AASHTO 2017 Article 3.10 and the seismic maps from the USGS website we obtained the following peak ground acceleration (PGA), short- and long-period spectral accelerations (S_S and S_1, respectively) and five-percent-damped-design response spectrum accelerations (A_S, S_{DS}, and S_{D1}, respectively) for the site:

- PGA = 0.112 g
- S_s = 0.219 g
- S_1 = 0.062 g
- A_S = 0.176 g
- S_{DS} = 0.351 g
- S_{D1} = 0.148 g

With an S_{D1} value of 0.148, the bridge is assigned to Seismic Zone 1 (AASHTO 2017, Article 3.10.6). Given the bridge is assigned to Seismic Zone 1, a liquefaction assessment is generally not required because the sustained ground acceleration is typically not large enough or does not act over a long enough period of time for liquefaction to occur (AASHTO 2017, Article C10.5.4.2).

5.4 Site Preparation Considerations

5.4.1 Demolition

The existing bridge structure will be demolished prior to construction. This work should include the removal of all existing site concrete and foundations that will interfere with the new construction. Abandoned utilities should be removed and replaced with compacted fill. Active utilities should be relocated outside of the construction area.

5.4.2 Stripping

The site should be cleared and grubbed in accordance with Section 201.03 of TDOT's 2015 Standard Specifications.

5.4.3 General

In areas less than 5 feet below subgrade where soil fill will be placed, proofrolling of the exposed surface of the subgrade soils should be performed after completion of stripping in areas to receive fill and once grade is achieved in cut areas. The purpose of proofrolling is to locate pockets of soft or unstable soils. Proofrolling should be performed using a fully loaded dump truck or other heavy equipment approved by the construction engineer. The proofrolling operation should traffic the construction area with parallel passes of the vehicle starting at one side of the area and continuing to the other. Each pass should overlap the preceding pass to ensure complete coverage. We recommend two complete passes. The construction engineer should be present to observe the proofrolling operations and to provide recommendations should unstable soils be encountered.

In accordance with Section 205.03 of TDOT's 2015 Standard Specifications, in areas where fill is placed less than 3 feet below subgrade, the cleared ground surface should be completely broken up by plowing, scarifying or stripping to a minimum depth of 6 inches and re-compacted. In cut areas, the top one foot of subgrade soils should be scarified, moisture conditioned, and re-compacted.

5.4.4 Compacted Fill

We anticipate the soils generated through excavation as part of the project will likely be used where fill is required. Borrow soils may be required if the project excavation does not provide enough soil for the required fills. Soil used for compacted fill should meet the requirements for AASHTO M 145, classification A-6 or better if reasonably available. If classification A-6 is not reasonably available, the borrow soil should be no worse than the predominant soil type in the roadway excavation based on AASHTO classification.

As stipulated in Section 205.04 of TDOT's 2015 Standard Specifications, fill should be placed in thin, horizontal lifts with a maximum loose thickness of 10 inches, then compacted to 95 percent of the standard Proctor maximum dry density. Typically, the moisture content should have a maximum range within 3 percent of the optimum moisture content, but the moisture content range that will be acceptable in order to obtain adequate compaction will depend on the shape of the Proctor curve. The top 6 inches of the roadbed in both cut as well as in fill areas where compacted soil fill will be placed should be compacted to 100 percent of the standard Proctor maximum dry density. The compacted fill should extend to the limits and grades shown on the roadway design plans.

An experienced soils technician should test the density and moisture content of each lift before placing additional lifts in order to evaluate that the specified degree of compaction is being achieved. The actual testing frequency should be determined by the geotechnical engineer based on the type of soil being placed, the equipment being used, and the time of year the fill is being placed. More frequent testing should be performed in confined areas. Areas that do not meet the compaction specification should be scarified, moisture conditioned if necessary, and re-compacted to achieve compliance.

Positive surface drainage should be maintained during grading operations to prevent water from ponding on the surface. The surface should be rolled smooth to enhance drainage if precipitation is expected. The geotechnical engineer should provide recommendations for treatment if the soils become excessively wet, dry, or frozen.

5.4.5 Ground Water/Water Considerations

We expect groundwater levels at this site will approximate the water elevation of Sinking Creek. Considering the creek flow and the time of year the construction is performed, sandbags may be a suitable means to reduce water infiltration into the excavations during pile cap construction. However, diverting the creek and/or "pumping around" the foundation excavations may be required. Groundwater will also likely be encountered in the pre-drilled H-pile holes. The contractor should be prepared to tremie grout to the rock socket.

6.0 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. S&ME is not responsible for the conclusions, opinions, or recommendations of others based on this data.

Our conclusions and recommendations are based on the design information furnished to us, the data obtained during the geotechnical exploration, the laboratory test results, and our past experience. They do not reflect variations in the subsurface conditions that are likely to exist between our borings and in unexplored areas of the site due to the inherent variability of the subsurface conditions in this geologic region and past land use. If such variations are found during construction, re-evaluating our conclusions and recommendations will be necessary.

If changes are made in the location of the planned bridge or elevations of the tops of the planned foundations, the recommendations contained in this report will not be considered valid unless our firm has reviewed the changes and modified or verified our recommendations in writing. You should retain us and give us the opportunity to review the final plans and the applicable portions of the project specifications after design completion. This review will allow us to check whether these documents are consistent with the intent of our recommendations.

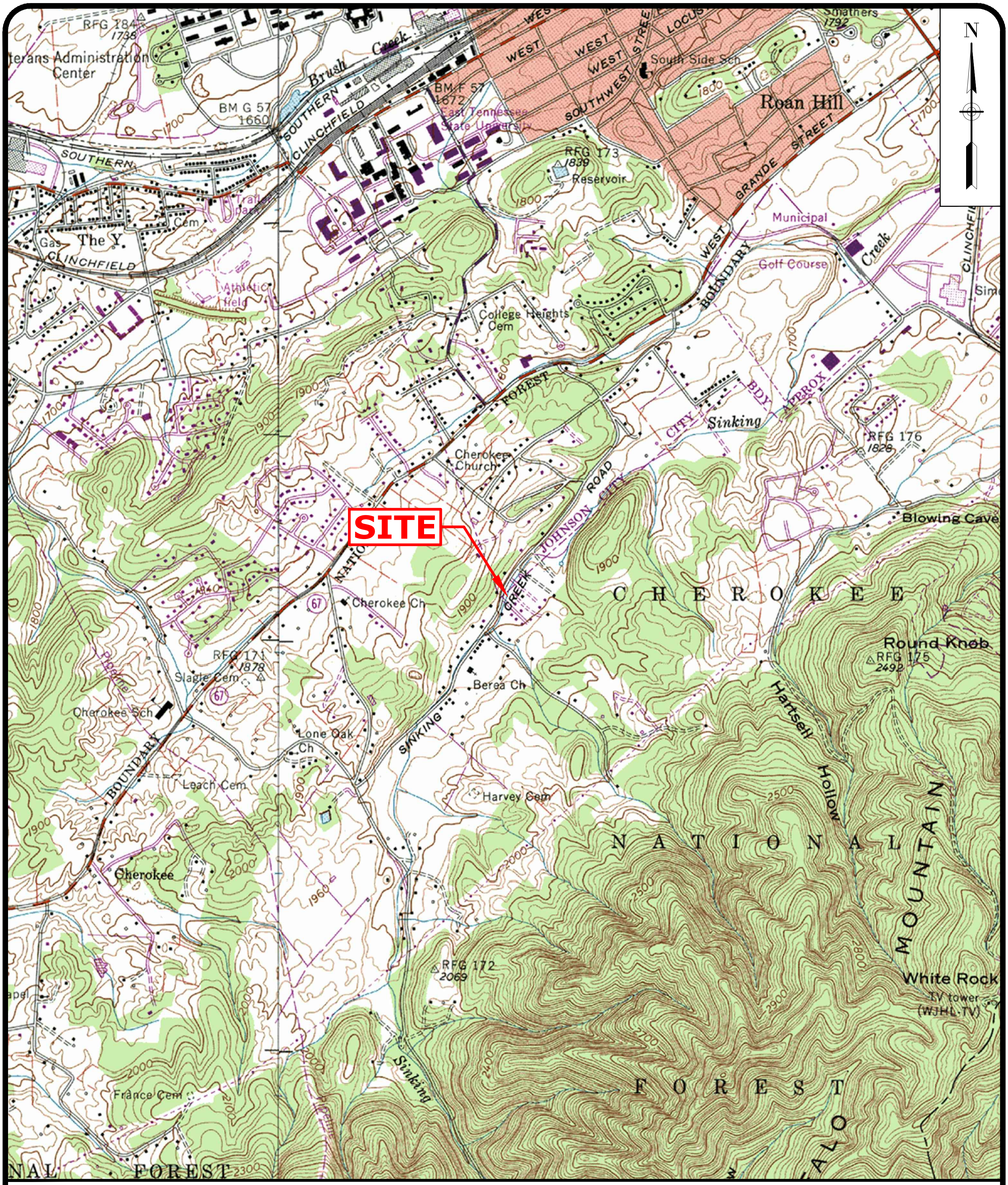
For more information on the use and limitations of this report, please read the ASFE document included in Appendix V.

Appendices

Appendix I

Figure 1 - Site Location Plan

Figure 2 - Boring Location Plan



SOURCE: USGS 7.5 Minute Topographic Map -- JOHNSON CITY, TENNESSEE (1968)
DRAWING FOR ILLUSTRATION PURPOSES ONLY

SITE LOCATION PLAN

MILLER LANE BRIDGE REPLACEMENT
JOHNSON CITY, TENNESSEE

SCALE:
1"=2,000'

DATE:
2/24/2020

PROJECT NUMBER
1281-20-005

FIGURE NO.
1

NOTES:

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

LEGEND:

⊕ - APPROXIMATE BORING LOCATION

BORING LOCATION PLAN

MILLER LANE BRIDGE REPLACEMENT
JOHNSON CITY, TENNESSEE

SCALE: 1"=20'
DATE: 2/24/2020
PROJECT NUMBER: 1281-20-005
FIGURE NO. 2

Appendix II

Field Exploration Procedures

Test Boring Record Legend

Test Boring Records

Rock Core Photographs

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.

ROCK CORING PROCEDURES

ASTM D 2113

Refusal materials were explored using a diamond-studded bit fastened to a double tube core barrel. An NQ-size bit was used during this exploration, which obtains core samples approximately 2 inches in diameter. The materials recovered were placed in a sample box. Our engineer classified the type and hardness of the rock, core recovery, and Rock Quality Designation (RQD). Core recovery is the sample length recovered divided by the length drilled, and RQD is the sample length recovered in pieces 4 inches or longer divided by the length drilled. Both core recovery and RQD are expressed as percentages. The Test Boring Record Legend contains the criteria for these classifications.

TEST BORING/PIT RECORD LEGEND

FINE AND COARSE GRAINED SOIL INFORMATION

COARSE GRAINED SOILS (SANDS & GRAVELS)

N	Relative Density
0-4	Very Loose
5-10	Loose
11-20	Firm
21-30	Very Firm
31-50	Dense
Over 50	Very Dense

FINE GRAINED SOILS (SILTS & CLAYS)

N	Consistency	Qu, KSF Estimated
0-1	Very Soft	0-0.5
2-4	Soft	0.5-1
5-8	Firm	1-2
9-15	Stiff	2-4
16-30	Very Stiff	4-8
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.

$$\text{RQD} = \frac{\text{Sum of 4 in. and longer Rock Pieces Recovered}}{\text{Length of Core Run}} \text{ X100}$$

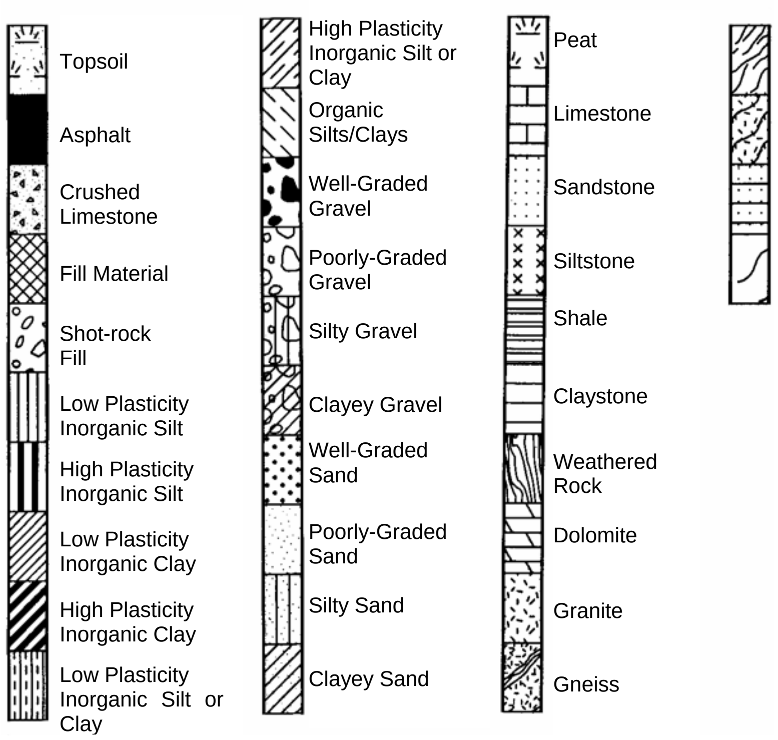
$$\text{Recovery} = \frac{\text{Length of Rock Core Recovered}}{\text{Length of Core Run}} \text{ X100}$$

43 RQD
NQ
63 REC

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

SYMBOLS

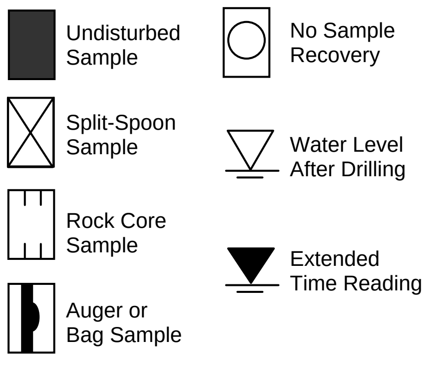
KEY TO MATERIAL TYPES



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-1

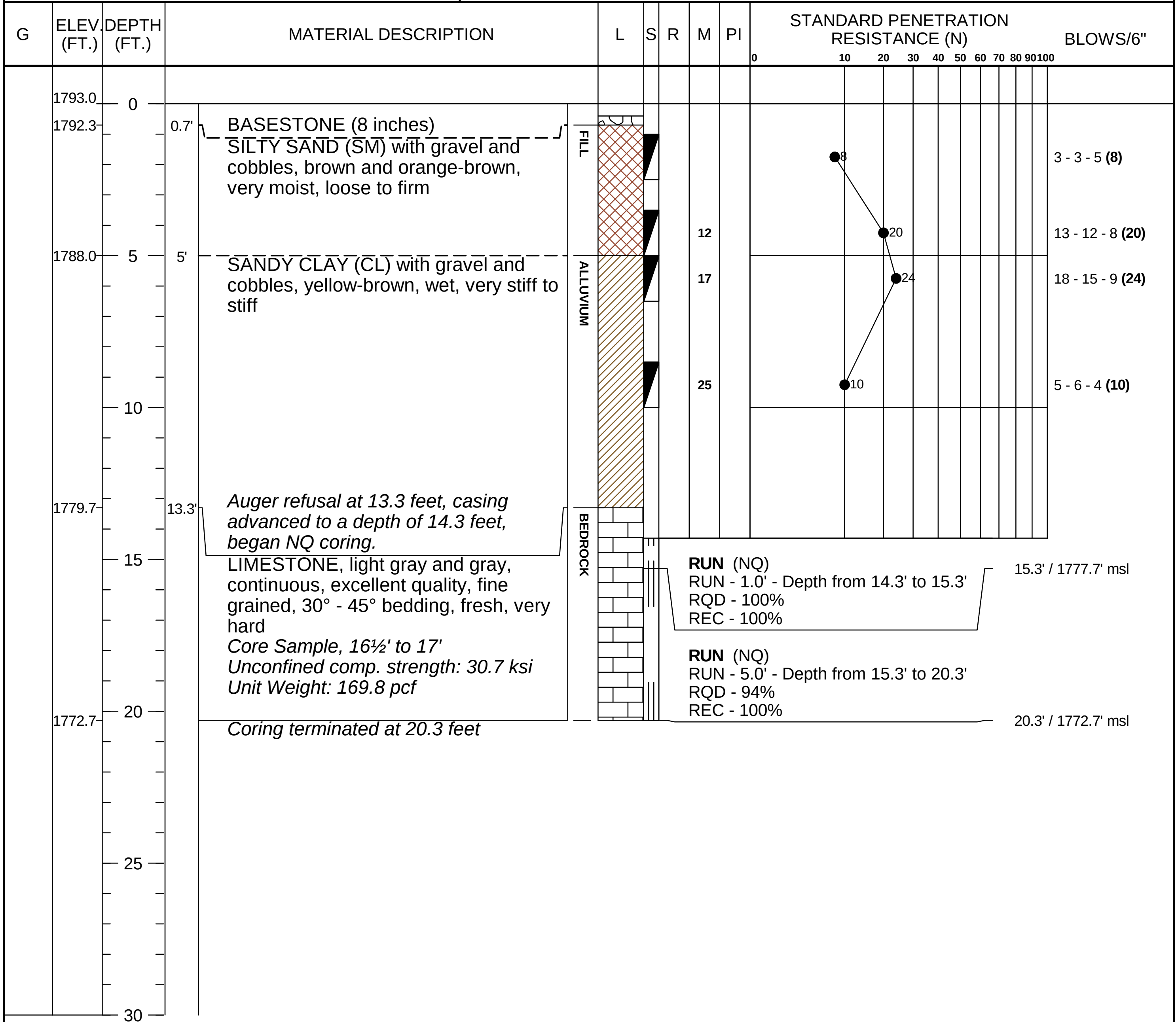
PROJECT: Miller Lane Bridge	JOB NO: 1281-20-005	SHEET 1 OF 1

PROJECT LOCATION: Johnson City, Tennessee

ELEVATION: 1,793 feet ±	BORING STARTED: 2/11/2020	RIG TYPE: CME-55	BORING DIA. (IN): 6¼
DRILLING METHOD: HSA, NQ Core	BORING COMPLETED: 2/11/2020	HAMMER: Automatic	CORE DIA.: NQ=1-7/8 in

GROUNDWATER:
Dry ATD

Remarks:



ELEV. (FT.)	DEPTH (FT.)	MATERIAL DESCRIPTION	M	BLOWS/6"
1793.0	0			
1792.3	0.7'	BASESTONE (8 inches)		
		SILTY SAND (SM) with gravel and cobbles, brown and orange-brown, very moist, loose to firm		3 - 3 - 5 **(8)**
			12	13 - 12 - 8 **(20)**
1788.0	5'	SANDY CLAY (CL) with gravel and cobbles, yellow-brown, wet, very stiff to stiff	17	18 - 15 - 9 **(24)**
			25	5 - 6 - 4 **(10)**
1779.7	13.3'	*Auger refusal at 13.3 feet, casing advanced to a depth of 14.3 feet, began NQ coring.*		
		LIMESTONE, light gray and gray, continuous, excellent quality, fine grained, 30° - 45° bedding, fresh, very hard *Core Sample, 16½' to 17'* *Unconfined comp. strength: 30.7 ksi* *Unit Weight: 169.8 pcf*		
1772.7		*Coring terminated at 20.3 feet*		

RUN (NQ)
RUN - 1.0' - Depth from 14.3' to 15.3'
RQD - 100%
REC - 100%

15.3' / 1777.7' msl

RUN (NQ)
RUN - 5.0' - Depth from 15.3' to 20.3'
RQD - 94%
REC - 100%

20.3' / 1772.7' msl

Project Manager: D. Reed., PE

Logged by: D. Reed, PE

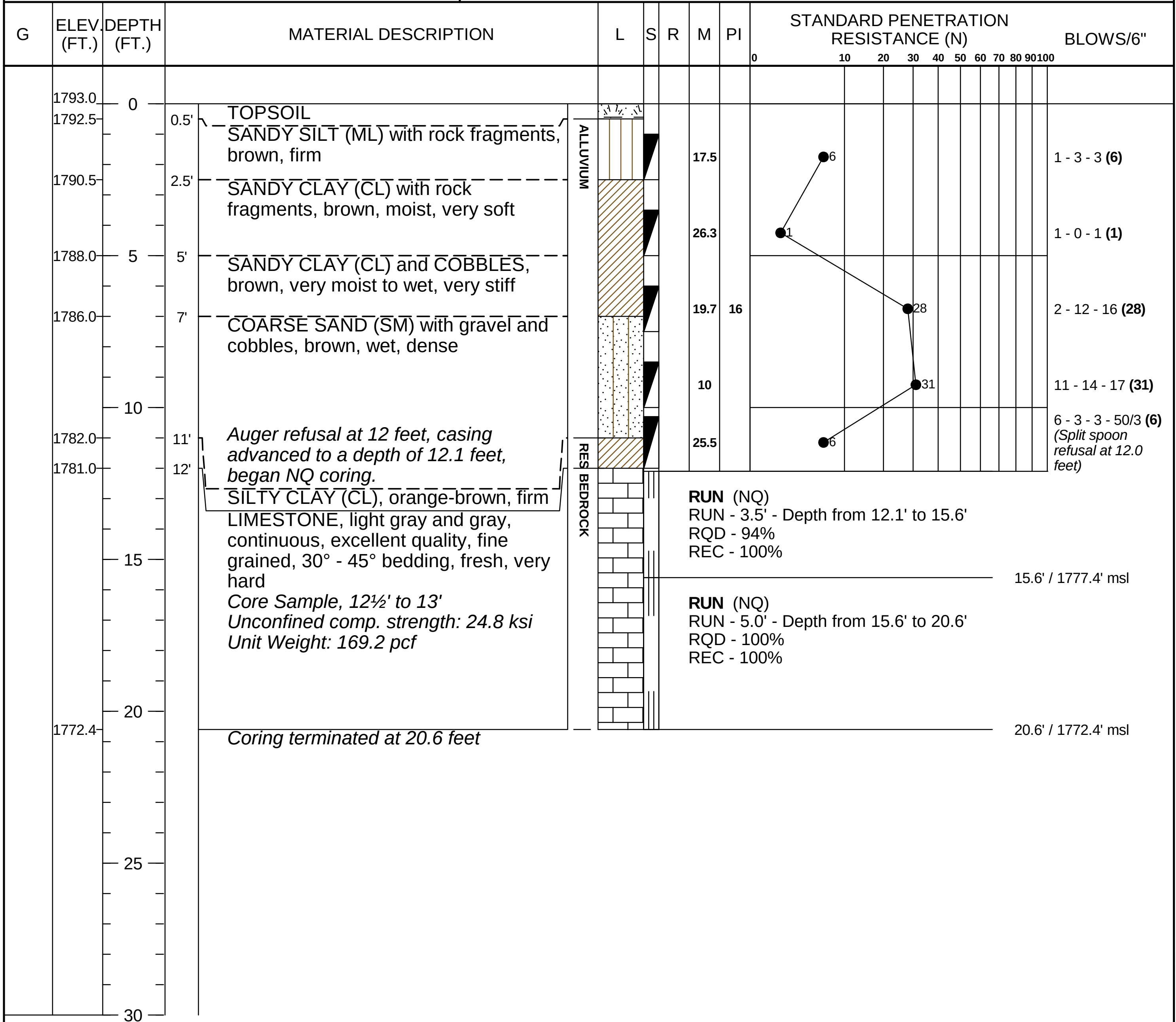
TEST BORING RECORD

BORING NO.: B-2

PROJECT: Miller Lane Bridge		JOB NO: 1281-20-005	SHEET 1 OF 1
PROJECT LOCATION: Johnson City, Tennessee			
ELEVATION: 1,793 feet ±	BORING STARTED: 2/10/2020	RIG TYPE: CME-55	BORING DIA. (IN): 6¼
DRILLING METHOD: HSA, NQ Core	BORING COMPLETED: 2/11/2020	HAMMER: Automatic	CORE DIA.: NQ=1-7/8 in

GROUNDWATER:
Dry ATD

Remarks:



ELEV. (FT.)	DEPTH (FT.)	MATERIAL DESCRIPTION
1793.0	0	TOPSOIL
1792.5	0.5'	SANDY SILT (ML) with rock fragments, brown, firm
1790.5	2.5'	SANDY CLAY (CL) with rock fragments, brown, moist, very soft
1788.0	5'	SANDY CLAY (CL) and COBBLES, brown, very moist to wet, very stiff
1786.0	7'	COARSE SAND (SM) with gravel and cobbles, brown, wet, dense
1782.0	11'	*Auger refusal at 12 feet, casing advanced to a depth of 12.1 feet, began NQ coring.* SILTY CLAY (CL), orange-brown, firm
1781.0	12'	LIMESTONE, light gray and gray, continuous, excellent quality, fine grained, 30° - 45° bedding, fresh, very hard *Core Sample, 12½' to 13'* *Unconfined comp. strength: 24.8 ksi* *Unit Weight: 169.2 pcf*
1772.4		*Coring terminated at 20.6 feet*

M	PI	N	BLOWS/6"
17.5		6	1 - 3 - 3 **(6)**
26.3		1	1 - 0 - 1 **(1)**
19.7	16	28	2 - 12 - 16 **(28)**
10		31	11 - 14 - 17 **(31)**
25.5		6	6 - 3 - 3 - 50/3 **(6)** *(Split spoon refusal at 12.0 feet)*

RUN (NQ)
RUN - 3.5' - Depth from 12.1' to 15.6'
RQD - 94%
REC - 100%

15.6' / 1777.4' msl

RUN (NQ)
RUN - 5.0' - Depth from 15.6' to 20.6'
RQD - 100%
REC - 100%

20.6' / 1772.4' msl

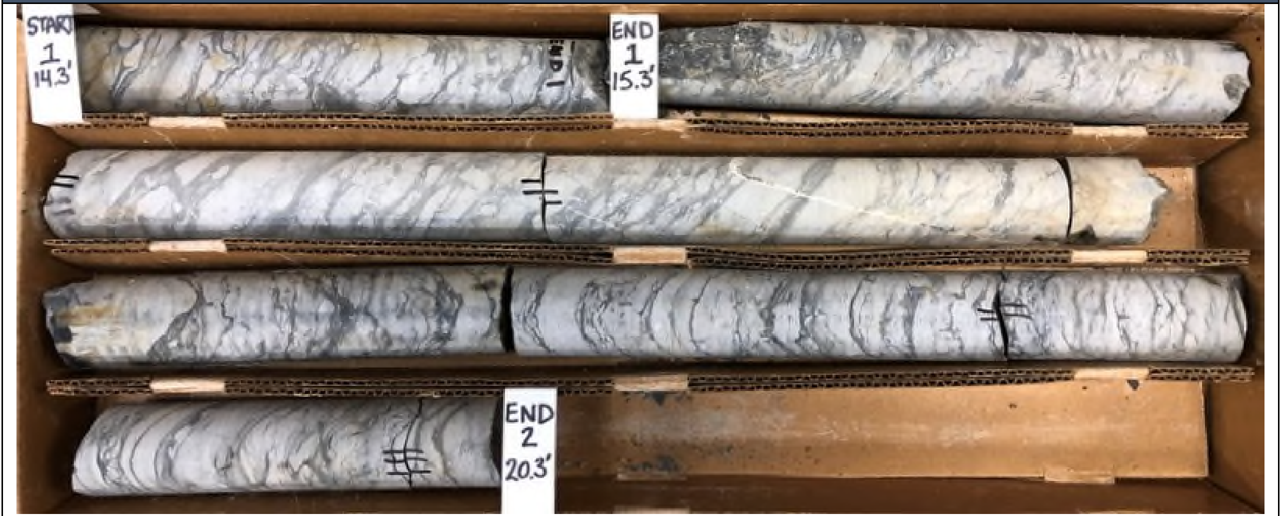
BORING RECORD S&ME 1281-20-005.GPJ 2016.GDT 3/11/20

Borehole ID: 2

Project Manager: D. Reed., PE

Logged by: D. Reed, PE

BORING B-1: BOX 1 OF 1



RUN	DEPTH (FT)	RQD (%)	RECOVERY (%)	ROCK DESCRIPTION
1	14.3 – 15.3	100	100	14.3 ft. – 20.3 ft.: LIMESTONE, light gray and gray, continuous, excellent quality, fresh, very hard
2	15.3 – 20.3	94	100	

BORING B-2: BOX 1 OF 1

RUN	DEPTH (FT)	RQD (%)	RECOVERY (%)	ROCK DESCRIPTION
1	12.1 – 15.6	94	100	12.1 ft. – 20.6 ft.: LIMESTONE, light gray and gray, continuous, excellent quality, fresh, very hard
2	15.6 – 20.6	100	100	

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.

UNIAXIAL COMPRESSIVE STRENGTH OF ROCK
ASTM D7012, Method C

A rock core specimen is cut to length and the ends are machined flat. The specimen is placed in a loading frame (with no confining). The axial load on the specimen is then increased and measured until the peak load and failure are obtained. The test results are provided on the Uniaxial Compression of Rock Test Report.

Miller Lane Bridge over Sinking Creek
Washington County, Tennessee
S&ME Project No. 1281—20-005
Laboratory Test Results Summary

Boring No.	Sample Type	Sample Depth (ft)	Visual USCS Classification	Natural Moisture Content (%)	Atterberg Limits			D_{50} (in)	Unconfined Compressive Strength (ksi)	Dry Density (pcf)
					LL	PL	PI			
B-1	SS	1 – 2½	SM							
		3½ - 5		12						
		5 – 6½	CL	17						
		8½ - 10		25						
	RC	16½ - 17	Limestone						30.7	169.8
B-2	SS	1 – 2½	ML	17.5						
		3½ - 5	CL	26.3						
		6 – 7½		19.7	35	19	16			
		8½ - 10	SM	10						
		10½ - 12	CL	25.5						
	RC	12½ - 13	Limestone						24.8	169.2
Creek Bottom Grab Sample			GM	-				1.0		

SPT – Standard Penetration Test Sample

RC – Rock Core Sample

Form No. TR-D7012C-01
Revision No. 0
Revision Date: 06/25/15

UNIAXIAL COMPRESSIVE STRENGTH OF ROCK

ASTM D 7012 Method C

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

Project No.:	1281-20-005	Log #: 20-023	Report Date:	02/20/20
Project Name:	Miller Lane Bridge Replacement		Test Date(s):	02/18/20
Client Name:	Vaughn & Melton		Sampled by:	Tri-State, LLC
Client Address:	127 Bob Fitz Road Gray, TN 37615		Received Date:	N/A
Boring ID:	B-1		Depth/Elev., ft:	16.5

Sample Description: Rock
Sample Location: B-1
Angle of load relative to lithology: Perpendicular

Test Results

Moisture Content ***0.1*** % ***Dry Unit Weight*** ***169.8*** pcf
Compressive Strength ***30730*** psi (148 MPa)

S&ME, Inc. - Chattanooga
Log #: 20-023
Project #: 1281-20-005
Project Name: Miller Lane Bridge Replacement
Boring # / Sample No. / Depth: B-1 / 16.5'
Sample Date: 2/10/2020
Sample Location:
Material Description: Rock
Tests:

BEFORE BREAK PHOTO

S&ME, Inc. - Chattanooga
Log #: 20-023
Project #: 1281-20-005
Project Name: Miller Lane Bridge Replacement
Boring # / Sample No. / Depth: B-1 / 16.5'

AFTER BREAK PHOTO

Stress rate: 0.63 MPa/sec.

Notes / Deviations / References: **Sample was capped with gypsum**

Drew Reed, PE		Project Engineer	2/20/2020
Technical Responsibility	*Signature*	*Position*	*Date*

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Form No. TR-D7012C-01
Revision No. 0
Revision Date: 06/25/15

UNIAXIAL COMPRESSIVE STRENGTH OF ROCK

ASTM D 7012 Method C

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

Project No.:	1281-20-005 Log #: 20-023	Report Date:	02/20/20
Project Name:	Miller Lane Bridge Replacement	Test Date(s):	02/18/20
Client Name:	Vaughn & Melton	Sampled by:	Tri-State, LLC
Client Address:	127 Bob Fitz Road Gray, TN 37615	Received Date:	N/A
Boring ID:	B-2	Depth/Elev., ft:	12.5
Sample Description:	Rock		
Sample Location:	B-2		

Angle of load relative to lithology: Perpendicular

Test Results

Moisture Content ***0.1*** **%** ***Dry Unit Weight*** ***169.2*** **pcf**

Compressive Strength ***24840*** **psi** **(148 MPa)**

BEFORE BREAK PHOTO

AFTER BREAK PHOTO

Stress rate: 0.63 MPa/sec.

Notes / Deviations / References: **Sample was capped with gypsum**

Drew Reed, PE		Project Engineer	2/20/2020
Technical Responsibility	*Signature*	*Position*	*Date*

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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.

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