Report of Geotechnical Exploration
Mountain Home VA Hospital Elevated Walkway
Johnson City, Tennessee
S&ME Project No. 1243-19-046

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October 18, 2019

October 18, 2019

AKEA Design Inc.
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Attention: Mr. Mike Tootle, P.E.

Reference: **Report of Geotechnical Exploration**
Mountain Home VA Hospital Elevated Walkway
Johnson City, Tennessee
S&ME Project No. 1243-19-046

Dear Mr. Tootle:

The following report presents the results of our geotechnical exploration performed for the proposed elevated walkway at the Mountain Hove Veterans Administration (VA) Hospital in Johnson City, Tennessee. The work was performed in general accordance with S&ME Proposal No. 12-1900429, Rev. 1, dated October 17, 2019, as authorized by you.

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

Sincerely,

S&ME, Inc.

Tim Lawrence
Oct 18 2019 11:09 AM

Timothy S. Lawrence, P.E.
Senior Geotechnical Engineer
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1.0 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 not included in this brief summary.

The geotechnical exploration included drilling and sampling of four soil test borings. Refusal materials were cored in one of the borings. The soil and rock core samples collected during our exploration were returned to our laboratory and visually classified and logged by a member of our engineering staff. Additionally, two relatively undisturbed thin-walled tube samples were obtained at selected depths in selected borings. Natural moisture content, Atterberg limits, grain size analysis (without hydrometer), and unconfined compression testing were performed on selected samples. Unconfined compression testing was also performed on a sample of rock core.

Three of the four borings encountered an interval of fat clay fill to depths of about 3 to 7 feet overlying fat clay residual soils to auger refusal. The remaining boring encountered fat clay residual soils beneath a thin interval of topsoil to auger refusal. The soil consistencies ranged from very soft to very hard. A thick interval of soft and very soft soil was encountered in Boring B-2-1 from a depth of about 8 feet to just above auger refusal at a depth of about 45 feet. Boring B-2-1 encountered auger refusal at a depth of about 45.7 feet. The remaining borings encountered auger refusal at depths ranging from about 7 to 17 feet. Boring B-2-2 was extended just under 16 feet into refusal materials using rock coring techniques. Refusal materials consisted of dolomite bedrock. Based on the boring data, it appears Boring B-2-1 was likely drilled in a slot/solution feature in the bedrock. Subsurface water was not observed in the borings at completion of drilling. Recovered soil samples from Borings B-2-1 and B-2-3 were wet below depths of about 8 and 13 feet, respectively.

Based on this geotechnical exploration, the expected structural loads discussed previously, and our engineering evaluation of the subsurface conditions at this site, we believe the site is adaptable for the proposed project; however, items identified in the exploration will likely affect construction costs and schedules. The borings encountered variable consistency fill and residual soils. Boring B-2-1 encountered soft and very soft soils and was likely drilled in a slot/solution feature. Given the moderate foundation loads (pier loads up to 350 kips), settlement and sinkhole potential of the site, we do not recommend supporting the elevated walkway on shallow soil-supported foundations. We recommend supporting the elevated walkway on bedrock supported deep foundations. Foundation recommends are provided in the body of this report.

2.0 Introduction

The purpose of this geotechnical exploration was to explore subsurface conditions pursuant to developing foundations recommendations for the subject project. This geotechnical exploration involved field exploration, laboratory testing, and engineering analysis. Following the text of this report; a Boring Location Plan, the boring logs, and laboratory test results are provided in the appendices. This report provides the following:

- A summary of the project and provided information;
- A summary of current site conditions and topography;

- A summary of area geologic conditions, including common geologic hazards, as applicable;
- A summary of the field exploration methods;
- A Boring Location Plan and individual boring logs for each boring;
- A summary of subsurface conditions encountered in the borings;
- A summary of laboratory test methods and results;
- Conclusions and site assessment;
- Recommended Seismic Site Class using the general procedure in general accordance with the 2018 International Building Code (IBC);
- Recommendations for foundation design and construction guidelines;

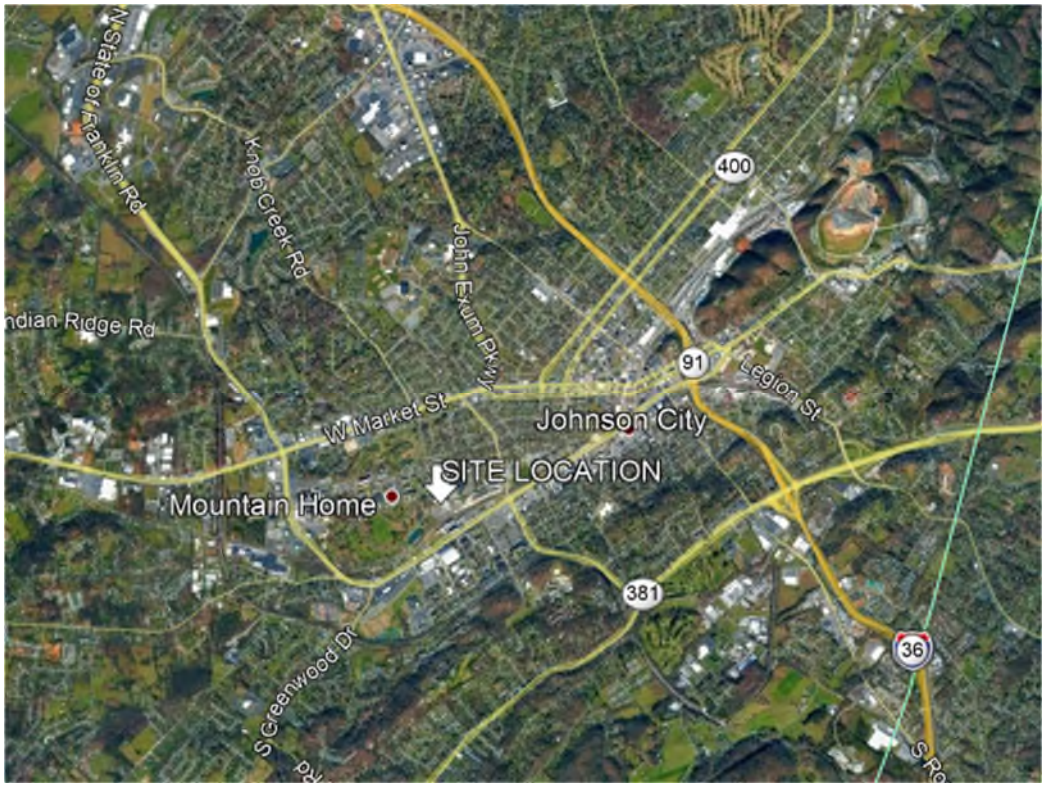
The scope of services for this project did not include an environmental assessment for determining the presence or absence of hazardous or toxic materials in the soil, bedrock, surface water, ground water, or air, on, below, or around this site. Any statements in this report or on the boring logs regarding odors, colors, and unusual or suspicious items or conditions are strictly for informational purposes.

3.0 Project Information and Site Conditions

Project information was provided via email correspondence and telephone conversations between Mr. Mike Tootle, P.E. of AKEA Design, Inc. (AKEA) and Mr. Dan Boles, P.E. with S&ME on August 28, 2019. Included in the email correspondence was a 25% Schematic Design Drawings Not Approved for Construction, no date, by AKEA. In addition, S&ME reviewed the *Final Report of Geotechnical Exploration New Parking Garage VA Mountain Home*, S&ME Project No, 1240-14-024, dated August 20, 2014. The parking garage has since been built and is adjacent to the present project area as discussed below.

The proposed project site is located along Bradford Pear Lane on the Mountain Home Veterans Administration (VA) Hospital Campus in Johnson City, Tennessee (see Figure 3-1). An elevated walkway is planned to extend from the existing parking garage (the existing parking garage site was the subject of our 2014 exploration referenced above) over Bradford Pear Lane to Building 160. The walkway will be approximately 200 feet long and enter Building 160 on one of the floors above exterior grades. The walkway is expected to be four or five spans and three to four piers are anticipated to support the walkway. No foundation loads have been provided. We anticipate pier loads will not exceed 350 kips.

Figure 3-1 Site Location



Base map from Google Earth Pro accessed on October 12, 2019

The site presently contains covered walkways and lawn and landscaped areas with scattered trees. The project area is relatively level. Borings B-7 and B-9 were drilled during the 2014 exploration at the south end of the proposed project area to the east and west of the proposed walkway location. These borings encountered and interval of clayey fill to depths of about 5 to 7 feet, underlain by clayey residual soils to auger refusal. Auger refusal was encountered in Borings B-7 and B-9 at depths of about 8 and 17 feet, respectively. Boring B-7 was extended about 16 feet below auger refusal using rock coring techniques. Refusal materials consisted of limestone bedrock. A void and soil seam where encountered within the bedrock mass at depths of about 14.5 to 14.7 and 18 to 19.1 feet, respectively. The approximate locations of Borings B-7 and B-9 are shown on Figure 1 in the Appendix I.

The project information and assumptions detailed above should be reviewed and confirmed by the appropriate team members. Modifications to our recommendations may be required if the actual conditions vary from the project information and assumptions described herein.

4.0 Exploration and Testing Program

Subsurface conditions were explored by drilling four soil test borings (designated B-2-1 through B-2-4). Please see Appendix I, Figure 1 for the approximate locations of the test borings. All depths in this report reference the

ground surface that existed at the time of this exploration. After completion of the field drilling and sampling phase of this project, the soil samples were returned to our laboratory where they were visually classified in general accordance with the *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*. The laboratory test results are discussed in the following sections of the report and individual test reports are provided in Appendix III.

4.1 Field Exploration

The borings were advanced in the overburden soil to refusal using hollow stem augers with an inside-diameter of 3¼ inches with a Diedrich D-50 drill rig on an all-terrain carrier. One boring was extended below auger refusal using rock coring techniques. The drill crew worked in general accordance with the American Society for Testing and Materials International (ASTM) D6151 method for soil drilling and sampling using hollow stem augers. Sampling and standard penetration testing of overburden soils was performed using a standard split spoon sampler (ASTM D1586). Additionally, two relatively undisturbed thin-walled tube samples (ASTM D1587) were obtained at selected locations. Rock coring was performed in general accordance with ASTM D2113 using double tube wireline coring techniques and an NQ2 size diamond impregnated core bit.

S&ME personnel selected the boring locations and depths with the assistance of AKEA and VA personnel based on site plans showing existing utilities. The boring locations were marked in the field comparing locations to existing site features. Topographic information was not provided to us at the time of this exploration and therefore, boring ground surface elevations are unknown. The test boring locations should be considered approximate based on the methods used to establish them.

Standard penetration resistance tests (SPT) were generally performed in the borings at intervals of 2½ feet to depths of about 10 feet and on 5-foot intervals thereafter, to the boring termination depth. The sampler was first seated 6 inches and then driven an additional foot with blows of the 140-pound hammer falling 30 inches. The number of hammer blows required to drive the sampler the final foot was recorded and is designated the *standard penetration resistance* (N-value) with units of blows per foot (bpf). An automatic trip drop hammer was used for the standard penetration testing. The automatic hammer generally has a higher efficiency than a manual hammer and may yield lower N-values. The N-values reported on our boring logs are the field values without any adjustments or corrections.

The drill crew documented if subsurface water was encountered in the borings during drilling and upon completion of drilling process. Upon completion of drilling and sampling of each boring, the boring was backfilled with soil cuttings, a hole plug was set just below the ground surface and borings in pavement areas were patched with quick set concrete before departing the site. Detailed information pertaining to the boring locations can be found on the boring logs provided in Appendix II of this report.

4.2 Laboratory Test Program

Representative soil specimens were then tested for moisture content (ASTM D2216), grain size analysis (ASTM D6913), Atterberg Limits (ASTM D4318), and unconfined compressive strength (ASTM D2166). Unconfined compressive strength testing (ASTM D7012, Method C) was also performed on a selected rock core sample. The laboratory test results are summarized in a subsequent section of this report and the test reports are provided in Appendix III.

5.0 Subsurface Conditions

5.1 Site Geology

The project site lies within the Appalachian Valley and Ridge Physiographic Province of East Tennessee. This Province is characterized by elongated, northeasterly-trending ridges formed on highly resistant sandstone and shale. Between ridges, broad valleys and rolling hills are formed primarily on less resistant limestone, dolomite, and shale.

The *Geologic Map of the Johnson City Quadrangle* (Jewell, J.W., 1997, Tennessee Division of Geology) indicates the project site is underlain by bedrock of the Knox Group (Undifferentiated). In this area the Knox Group is primarily composed of gray to blue-gray, fine- to very fine-grained limestone with some dolomite. The soils weathered from these formations are generally red and tan clays and silts.

Since the bedrock underlying this site contains carbonate rock (i.e. limestone/dolomite), it is susceptible to the hazards of irregular weathering, cave and cavern conditions, and overburden sinkholes. Carbonate rock, while appearing very hard and resistant, is soluble in slightly acidic water. This characteristic, plus differential weathering of the bedrock mass is responsible for these hazards. Of these hazards, the occurrence of sinkholes is potentially the most damaging to overlying soil-supported structures. Sinkholes occur primarily due to differential weathering of the bedrock and flushing or raveling of overburden soil into the cavities within the bedrock. This loss of solids creates a cavity, or dome, in the overburden. Growth of the cavity over time, or excavation over the dome, can create a condition in which rapid subsidence, or collapse, of the roof of the dome occurs.

A certain degree of risk with respect to sinkhole formation and subsidence should be considered with any site located within geologic areas underlain by potentially soluble rock units. A rigorous effort to assess the potential for sinkhole formation on this site was beyond the scope of this evaluation. However, Boring B-2-1 did encounter subsurface conditions indicative of the karst geology and solution activity. Boring B-2-1 may have been drilled in a slot/solution feature in the bedrock. Boring B-2-1 encountered a thick interval of soft and very soft soils below depths of about 8 feet and encountered auger refusal at a much greater depth than the other borings. The site has an apparent sinkhole risk. Foundation alternatives regarding the sinkhole risk are discussed further subsequently.

5.2 Field Exploration Information

5.2.1 Summary of Overburden Soil Conditions

Overburden soil conditions encountered in the borings are summarized in Table 5-1.

Table 5-1 Boring Overburden Soil Summary

Boring No.	Surface Cover (Inches)	Material	SPT N-Values (blows per foot)	Boring Termination Depth (feet)
B-2-1	Topsoil 2	Residuum to 45.7 feet: Fat Clay (CH) & Weathered Rock (weathered rock from 45 to 45.7 feet)	Residuum: 0 to 50+ (zero represents sampler advanced under the static weight of the hammer; 50+ near refusal)	45.7 (auger refusal)
B-2-2	Concrete 7	Fill to 3 feet: Fat Clay (CH) Residuum 3 to 7.3 feet: Fat Clay and Elastic Silt (CH & MH)	Fill: 6 Residuum: 6 to 50+ (50+ near refusal)	7.3 (auger refusal)
B-2-3	Topsoil 2	Fill to 3 feet: Fat Clay (CH) Residuum 3 to 16.9 feet: Fat Clay (CH)	Fill: 29 Residuum: 1 to 14	16.9 (auger refusal)
B-2-4	Topsoil 2	Fill to 5.5 feet: Fat Clay (CH) Residuum 5.5 to 12.5 feet: Fat Clay (CH)	Fill: 12 to 13 Residuum: 9 to 50+ (50+ near refusal)	12.5 (auger refusal)

NOTE: See the boring logs in Appendix II for specific information at individual boring locations.

5.2.2 Auger Refusal

Auger refusal was encountered in the borings at depths of about 7 to 46 feet as summarized in Table 5-1. Refusal is a designation applied to material that cannot be penetrated by the power of the drilling equipment. Refusal discussed herein is based on conditions impenetrable to the drilling equipment used in this exploration (i.e., Diedrich D-50 Rig) and does not necessarily suggest the conditions would be impenetrable to other equipment.

5.2.3 Refusal Materials/Bedrock

Boring B-2-2 was extended about 16 feet below auger refusal using rock coring techniques. Refusal materials consisted of weathered to competent dolomite. 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. A summary of the rock coring data along with photographs of the core recovered are provided in Figure 5-1.

Figure 5-1 Boring B-2-2 Rock Core Photos and Summary

BORING B-2-2-2 CORING SUMMARY				
RUN	DEPTH (feet)	RECOVERY (%)	RQD (%)	ROCK DESCRIPTION
1	8 to 10.5	100	96	LIMESTONE, gray, thinly bedded, competent high quality finely crystalline, slightly weathered
2	10.5 to 15.5	100	94	
3	15.5 to 20.5	99	96	
4	20.5 to 22.5	96	96	

5.2.4 Subsurface Water

Subsurface water was not observed in the borings at completion of drilling. Delayed water level readings were not obtained as the borings were backfilled shortly after drilling for safety. Recovered soil samples from Borings B-2-1 and B-2-3 were wet below depths of about 8 and 13 feet, respectively. Subsurface water levels may fluctuate due to seasonal changes in precipitation amounts, construction activities in the area, and/or other factors. The subsurface water information presented in this report is the information collected at the time of our field activities.

5.2.5 General

The subsurface conditions summarized above are of a generalized nature to highlight the major subsurface stratification features and material characteristics. The boring logs included in the Appendix should be reviewed for specific information at individual boring locations. The depth and thickness of the subsurface strata indicated on the boring logs were generalized from and estimated between boring locations. The transition between materials may be more gradual than indicated on the boring logs. Information on actual subsurface conditions exists only at the specific boring locations and is relevant to the time the exploration was performed. Variations may occur and should be expected between boring locations.

5.3 Laboratory Testing Results

The results of the laboratory testing are presented in Appendix III and summarized following. The moisture contents of the samples tested ranged from about 7 to 43 percent.

Atterberg limits, sieve analyses and unconfined compression testing were performed on the thin-walled tube sample taken from a depth of about 8 to 10 feet in Boring B-2-4. The Atterberg limits testing resulted in a liquid limit (LL) of 51, plastic limit (PL) of 17, and plasticity index (PI) of 34, respectively. The sample had a percent passing the No. 200 sieve of about 91.7 percent. The unconfined compression testing resulted in an unconfined compressive strength of about 4.27 kips per square foot (ksf). The sample had a dry unit weight of 102.7 pounds per cubic foot (pcf) and a moisture content of 22.6 percent.

6.0 Conclusions and 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, this office should be notified at once. 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 re-evaluated.

6.1 Site Assessment

Based on this geotechnical exploration, the expected structural loads discussed previously, and our engineering evaluation of the subsurface conditions at this site, we believe the site is adaptable for the proposed project; however, items identified in the exploration will likely affect construction costs and schedules. The borings

encountered variable consistency fill and residual soils. Boring B-2-1 encountered soft and very soft residual soils within about eight feet of the ground surface. In addition, it appears Boring B-2-1 was likely drilled in a slot/solution feature. Given the moderate foundation loads (pier loads up to 350 kips), settlement and sinkhole potential of the site, we do not recommend supporting the elevated walkway on shallow soil-supported foundations. We recommend supporting the elevated walkway on bedrock supported deep foundations.

An inherent disadvantage of a bedrock supported deep foundation system in this geologic setting is the discontinuous character of the bedrock. This makes it difficult to accurately estimate foundation lengths, bearing levels, and rock excavation quantities in advance of construction, resulting in an uncertain estimate of foundation costs. Therefore, once the project is further into design, the foundation loads are further defined, walkway pier locations are established, and rock socket requirements defined, we recommend drilling additional pilot holes (by consultant or contractor) with rock coring at each elevated walkway pier location to provide additional data for selecting shaft bottoms and/or socket zones for construction budgeting purposes.

6.2 Seismic Site Classification

Based on the International Building Code (IBC 2018) and the results of our exploration, we recommend a Seismic Site Class D for the project site. It may be possible to improve the site class if shear wave velocity testing is performed. However, such testing does not guarantee an improved site class.

6.3 Deep Foundations

The risk of settlement and differential settlement do to the variable and lower consistency overburden soils may be eliminated by extending foundations through the soil overburden and into bedrock. Additionally, a bedrock supported deep foundation system will reduce potential impacts on the structure in the event a sinkhole occurs in the overburden soils. Based on the project information provided and subsurface conditions encountered in the borings, we believe bedrock supported drilled shafts or drilled and grouted mini-piles are appropriate deep foundation alternatives.

The drilled shafts or mini-piles will need to be extended through the overburden soils and any weathered zones and cavities encountered within the bedrock mass to obtain support in the strong, slightly weathered to fresh, dolomite and limestone bedrock underlying the site. As previously stated, once the elevated walkway pier locations and foundation loads have been established, we recommend drilling additional pilot holes with rock coring at each elevated walkway pier location to provide additional data for selecting shaft bottoms and/or socket zones for construction budgeting purposes. Additional recommendations and considerations for drilled shaft and mini-pile foundations are provided following.

6.3.1 Drilled Shafts

6.3.1.1 Drilled Shaft Design

Drilled shafts can be designed as end-bearing or frictional elements or can utilize a combination of both. We recommend a maximum allowable end bearing capacity on strong, fresh to slightly weathered dolomite and limestone bedrock of 100 kips per square foot (ksf). For end bearing, the drilled shafts should be seated a minimum

of 2 feet into slightly weathered to continuous dolomite. We recommend side friction in the overburden soils be neglected in sizing the drilled shafts. We recommend allowable side friction within a continuous to slightly weathered dolomite rock socket of 5.5 ksf for computing both axial compression and uplift resistance. Artificial shaft roughing is not required to achieve this side friction capacity within the rock socket. Shaft bottoms and sockets should be clean for proper load transfer. We recommend a minimum drilled shaft diameter of 36 inches to provide reasonable entry space for bottom preparation and observation. With design and construction in accordance with the recommendations of this report, settlements of drilled shaft foundations should be less than 1 inch.

Lateral drilled shaft capacities will depend on lateral loads, shaft diameters and length, allowable head deflection and soil parameters surrounding the drilled shafts. We can provide recommendations for lateral capacity analyses as the necessary design information becomes available.

Site challenges associated with drilled shaft foundations are the variable bedrock surface and discontinuous bedrock. Additionally, ground water may be encountered in some shaft excavations either perched on the bedrock surface or in seams and discontinuities within the bedrock. Where cavities, very soft zones or soil seams are encountered within the dolomite and limestone bedrock, shaft bearing elevation and sockets depths will need to be extended below the cavities, very soft zones and soil seams.

6.3.1.2 Drilled Shaft Construction

Based on the exploration, hard rock coring/drilling tools will generally be required to remove the strong dolomite and limestone bedrock. It is preferred to construct in the shaft using the dry method by using casing, if possible, to facilitate observation of the shaft sockets and bottoms. The exploration generally indicates the drilled shafts should be able to be constructed using the dry method. We anticipate ground water may be encountered in some of the shaft excavations and pumping of ground water will be needed to construct shafts using the dry method.

As the drilled shaft hole is advanced, temporary steel casing or casings (as required) should be installed in the hole. A properly designed casing is needed to allow worker access to clean and observe the bearing materials, prevent collapse of the excavation sidewalls and to reduce the inflow of mud and water into the excavation.

Temporary casings may be removed as concrete is placed. Temporary casing should not be left in rock sockets as it would interfere with the concrete/rock side friction. Temporary casings should not be moved until the concrete is above the ground-water level. A minimum 5-foot head of concrete should be maintained above the bottom of the casing during withdrawal and the contractor should prevent concrete from hanging up inside the casing, which can allow soils and water intrusion below the casing.

Ground-water conditions may require the use of special procedures to achieve a satisfactory foundation installation if substantial localized ground-water flows are encountered in a drilled shaft hole. If water is flowing into the drilled shaft hole at 12 inches per hour (1 inch per 5 minutes) or less, pumps should be used to maintain less than 3 inches of water in the hole to facilitate observation of the shaft bottom/socket. After verification that rock in the bottom and socket appears adequate and sufficiently clean, the pumps should be removed and concreting commenced immediately.

If more than 12 inches per hour (1 inch per 5 minutes) of water is flowing into the drilled shaft excavation, the water should be allowed to stabilize in the shaft excavation before attempting to place the concrete. After the water has stabilized, the concrete should be placed into the drilled shaft through a tremie pipe. The end of the pipe should be installed to the bottom of the drilled shaft so the water will be displaced out the top of the shaft during concrete placement.

If the water is pumped out of the driller shaft, we recommend the concrete placement be directed through a centering chute or other commonly used methods at the surface so the concrete falls vertically down the center of the shaft. Concrete should not be allowed to hit the reinforcing steel or sides to reduce the potential for segregation of the concrete.

Concrete slumps of about 9 inches are recommended for drilled shaft construction. Concrete with a slump of about 9 inches will flow through the rebar cage and fill any voids or irregularities in the shaft sidewalls.

Placement of concrete in the drilled shaft should be performed on the same day the bedrock in the shaft is excavated. In the event a shaft excavation is left open overnight, the sidewalls and bottom will need to be re-cleaned as needed to remove and mud and soft or loose material.

The differential weathering of the bedrock causes required drilled shaft lengths to vary. These variable conditions require field observation of actual conditions at each drilled shaft location. Therefore, a representative of the geotechnical engineer should observe the drilled shaft construction to document the shaft diameter, depth, cleanliness, plumbness, suitability of the rock in the shaft sockets and bottom, and concrete placement. Significant deviations from the specified or anticipated conditions will be reported to the owner's representative and to the foundation designer. Each drilled shaft excavation should be observed after the bottom of the shaft is leveled, cleaned of any mud or extraneous material and dewatered.

The contractor must be required to drill at least one probe hole in the bottom of each drilled shaft excavation. The probe hole should be at least 2 inches in diameter and is typically drilled with a pneumatic percussion drill. The probe holes should be drilled to a depth of two the shaft diameters or to a maximum depth of 10 feet. Each hole should be observed by qualified personnel with a steel feeler rod to assess the rock continuity. If this observation indicates discontinuous rock and compressible seams that could contribute to excessive settlement, the drilled shaft should be excavated through these discontinuities. Additional probe holes may be required to check drilled shafts supported on suspect materials.

6.3.2 Mini-Piles

Mini-piles are relatively small diameter (typically less than 12 inches) drilled and grouted piles. They are constructed with equipment like that used for anchoring and grouting. Rotary drilling/percussion is used to advance a casing into the bond zone/supporting strata. The piles typically develop their geotechnical capacity through grout to rock side friction/bond in the bond zone. The drilling techniques used to install mini-piles can readily penetrate bedrock and the mini-piles can be readily extended to accommodate the variable bedrock depths encountered at the subject site. In addition, mini-piles can be readily extended through any cavities, soils seams or soft zones within the bedrock mass.

Bedrock supported drilled and grouted mini-piles with diameters ranging from 6 to 11 inches routinely achieve ultimate capacities ranging from 75 to 300 tons. An ultimate grout/bedrock side resistance/bond value of 200 psi should be available for sizing mini-pile diameters and load transfer/bond zone socket lengths in the dolomite and limestone bedrock. Design side resistance/bond values should be verified through load testing. We estimate that 6-inch diameter piles can develop ultimate capacities of about 220 kips and allowable capacities of about 70 kips with a safety factor of about 3 by penetrating 5 feet into the underlying fresh dolomite and limestone bedrock.

Mini piles are typically provided on projects through geotechnical contractor design-build methods. This is because methods of design, specification, and installation of mini pile underpinning systems in the United States have been developed by geotechnical specialty contractors. Mini pile capacities depend on the installation techniques used to install them. Geotechnical specialty contractors are typically the most adept at selecting the most efficient installation technique to meet the project requirements.

The specialty contractor's responsibilities concerning the underpinning performance should be clearly established in the contract documents. Mini pile design and performance criteria (i.e., design loads, deflection/movement limits and any additional performance requirements) should be provided to the specialty contractor through the project drawings and specifications.

The top of bedrock varies across the site, which will cause mini-pile lengths to vary. If very soft zones or soil seams are encountered in the bedrock, mini-pile sockets will need to be extended below the very soft zones and soil seams. We recommend the geotechnical engineer have an on-site representative to observe pile installation, confirm the subcontractor's determination of penetration into fresh bedrock and document any problems that may arise.

7.0 Limitations

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

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

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

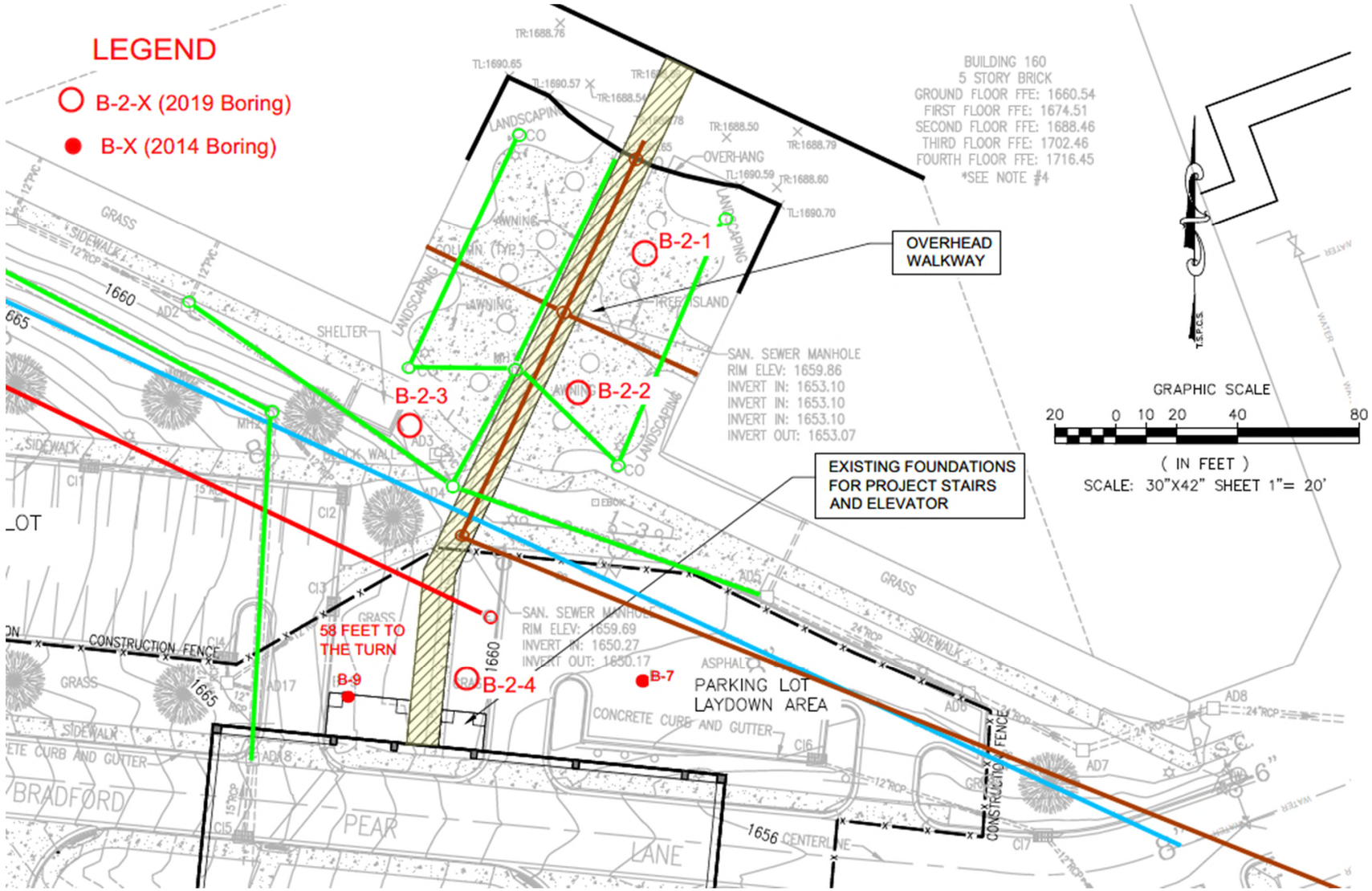
Unless specifically noted otherwise, our field exploration program did not include an assessment of regulatory compliance, environmental conditions or pollutants or presence of any biological materials (mold, fungi, bacteria).

If there is a concern about these items, other studies should be performed. S&ME can provide a proposal and perform these services if requested.

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

Appendices

Appendix I – Boring Location Plan



REFERENCE:
25% SCHEMATIC DESIGN DRAWING (UNDATED) PROVIDED BY AKEA DESIGN INC

BORING LOCATION PLAN

MOUNTAIN HOME VA HOSPITAL ELEVATED WALKWAY
BRADFORD PEAR LANE TO BUILDING 160
JOHNSON CITY, TENNESSEE

SCALE:
AS SHOWN

DATE:
10-17-2019

PROJECT NUMBER
1243-19-046

FIGURE NO.
1

Appendix II – Test Boring Logs

Project:	Mountain Home VA Center Elevated Walkway Johnson City, Tennessee 1243-19-046	BORING LOG: B-2-1 *Sheet 1 of 2*

DATE DRILLED: 09/29/2019	**ELEVATION:**	**NOTES:** Sample descriptions based on visual observation of obtained samples.
DRILL RIG: Diedrich D-50 (ATC)	**DATUM:** NAVD88	
DRILLER: T. Bunch	**BORING DEPTH:** 45.7 ft	
HAMMER TYPE: Auto Hammer (140 lb)	**CLOSURE:** Backfilled with cuttings	
DRILLING METHOD: 3 1/4" H.S.A.	**LOGGED BY:** J. Feuge	**Latitude:** **Longitude:**
SAMPLING METHOD: SS	**PROJECT COORDINATE SYSTEM -** NAD 1983 StatePlane Tennessee FIPS 4100 Feet	

DEPTH (feet)	NOTES	GRAPHIC	SAMPLE NO. (RECOVERY)	MATERIAL DESCRIPTION	BLOW COUNT DATA (SPT N-value)	STANDARD PENETRATION TEST DATA (○ NMC, ⊢⊣ PL---LL, △ % Fines; 20 40 60 80)	ELEVATION
0	Residuum - Fat Clay			TOPSOIL, 2 inches			0
				FAT CLAY (CH), few fine sand, trace roots, stiff, brown, slightly moist	10-5-5 N= 10		
				FAT CLAY (CH), little fine sand, stiff, tan, slightly moist	4-6-5 N= 11		
5							-5
				FAT CLAY (CH), trace fine sand, stiff, brown and orange brown, moist	4-5-5 N= 10		
				FAT CLAY (CH), few fine sand, soft to very soft, light brown and orange brown, wet	2-2-1 N= 3		
10							-10
					0-0-0 N= 0		
15							-15
					0-0-0 N= 0		
20							-20
					0-0-0 N= 0		
25							-25
					0-0-0 N= 0		
30							-30

GROUNDWATER		DATE/TIME	DEPTH (FT)	REMARKS
DURING ADVANCE	▽			
END OF DRILLING	▼			
AFTER DRILLING	▼			
AFTER DRILLING	▼			

GROUNDWATER DEPTHS ARE NOT EXACT AND MAY VARY SUBSTANTIALLY FROM THOSE INDICATED.
LL=Liquid Limit, PL = Plastic Limit, NM = Natural Moisture Content, PPV = Pocket Penetrometer (tsf), PTV = Pocket Torvane (tsf)

Project:	Mountain Home VA Center Elevated Walkway Johnson City, Tennessee 1243-19-046	BORING LOG: B-2-1 *Sheet 2 of 2*

DATE DRILLED: 09/29/2019	**ELEVATION:**	**NOTES:** Sample descriptions based on visual observation of obtained samples.
DRILL RIG: Diedrich D-50 (ATC)	**DATUM:** NAVD88	
DRILLER: T. Bunch	**BORING DEPTH:** 45.7 ft	
HAMMER TYPE: Auto Hammer (140 lb)	**CLOSURE:** Backfilled with cuttings	
DRILLING METHOD: 3 1/4" H.S.A.	**LOGGED BY:** J. Feuge	**Latitude:** **Longitude:**
SAMPLING METHOD: SS	**PROJECT COORDINATE SYSTEM -** NAD 1983 StatePlane Tennessee FIPS 4100 Feet	

DEPTH (feet)	NOTES	GRAPHIC	SAMPLE NO. (RECOVERY)	MATERIAL DESCRIPTION	BLOW COUNT DATA (SPT N-value)	STANDARD PENETRATION TEST DATA (○ NMC, ⊢⊣ PL---LL, △ % Fines; 20, 40, 60, 80)	ELEVATION
				FAT CLAY (CH), few fine sand, soft to very soft, light brown and orange brown, wet			
35					0-0-0 N= 0		-35
40					0-0-0 N= 0		-40
					0-0-50/2" N= 100		
45	Residuum - Weathered Rock Auger refusal at 45.7 feet Borehole terminated at 45.7 feet			WEATHERED ROCK very hard, gray, slightly moist			-45
50							-50
55							-55
60							-60

GROUNDWATER		DATE/TIME	DEPTH (FT)	REMARKS
DURING ADVANCE	▽			
END OF DRILLING	▼			
AFTER DRILLING	▼			
AFTER DRILLING	▼			

GROUNDWATER DEPTHS ARE NOT EXACT AND MAY VARY SUBSTANTIALLY FROM THOSE INDICATED.
LL=Liquid Limit, PL = Plastic Limit, NM = Natural Moisture Content, PPV = Pocket Penetrometer (tsf), PTV = Pocket Torvane (tsf)

Project:	Mountain Home VA Center Elevated Walkway Johnson City, Tennessee 1243-19-046	BORING LOG: B-2-2 *Sheet 1 of 1*

DATE DRILLED: 09/29/2019	**ELEVATION:**	**NOTES:** Sample descriptions based on visual observation of obtained samples.
DRILL RIG: Diedrich D-50 (ATC)	**DATUM:** NAVD88	
DRILLER: T. Bunch	**BORING DEPTH:** 23.0 ft	
HAMMER TYPE: Auto Hammer (140 lb)	**CLOSURE:** Backfilled with cuttings	
DRILLING METHOD: 3 1/4" H.S.A., NQ Core	**LOGGED BY:** J. Feuge	**Latitude:** **Longitude:**
SAMPLING METHOD: SS, RC	**PROJECT COORDINATE SYSTEM -** NAD 1983 StatePlane Tennessee FIPS 4100 Feet	

DEPTH (feet)	NOTES	GRAPHIC	SAMPLE NO. (RECOVERY)	MATERIAL DESCRIPTION	BLOW COUNT DATA (SPT N-value)	STANDARD PENETRATION TEST DATA (○ NMC, ⊢⊣ PL---LL, △ % Fines; 20, 40, 60, 80)	ELEVATION
0	Fill - Fat Clay			CONCRETE, 7 inches			0
				FAT CLAY (CH), some rock fragments, trace chert, firm, red brown, moist to very moist	3-3-3 N= 6		
	Residuum - Fat Clay			FAT CLAY (CH), trace sand, firm, gray and brown, slightly moist	4-2-4 N= 6		
5	Residuum - Elastic Silt			ELASTIC SILT (MH), firm, orange brown, moist	4-50/2" N= 100		-5
	Residuum - Weathered Rock			WEATHERED ROCK very hard, tan, dry			
	Auger refusal at 7.3 feet Limestone			Limestone, gray, thinly bedded, competent high quality finely crystalline, slightly weathered			
10			REC-100% RQD-96%				-10
			REC-100% RQD-94%				
15							-15
			REC-99% RQD-96%				
20							-20
			REC-96% RQD-96%				
	Borehole terminated at 23.0 feet						
25							-25
30							-30

GROUNDWATER		DATE/TIME	DEPTH (FT)	REMARKS
DURING ADVANCE	▽			
END OF DRILLING	▼			
AFTER DRILLING	▼			
AFTER DRILLING	▼			

GROUNDWATER DEPTHS ARE NOT EXACT AND MAY VARY SUBSTANTIALLY FROM THOSE INDICATED.
LL=Liquid Limit, PL = Plastic Limit, NM = Natural Moisture Content, PPV = Pocket Penetrometer (tsf), PTV = Pocket Torvane (tsf)

Project:	Mountain Home VA Center Elevated Walkway Johnson City, Tennessee 1243-19-046	BORING LOG: B-2-3 *Sheet 1 of 1*

DATE DRILLED: 09/28/2019	**ELEVATION:**	**NOTES:** Sample descriptions based on visual observation of obtained samples.
DRILL RIG: Diedrich D-50 (ATC)	**DATUM:** NAVD88	
DRILLER: T. Bunch	**BORING DEPTH:** 16.9 ft	
HAMMER TYPE: Auto Hammer (140 lb)	**CLOSURE:** Backfilled with cuttings	
DRILLING METHOD: 3 1/4" H.S.A.	**LOGGED BY:** J. Feuge	**Latitude:** **Longitude:**
SAMPLING METHOD: SS, UD	**PROJECT COORDINATE SYSTEM -** NAD 1983 StatePlane Tennessee FIPS 4100 Feet	

DEPTH (feet)	NOTES	GRAPHIC	SAMPLE NO. (RECOVERY)	MATERIAL DESCRIPTION	BLOW COUNT DATA (SPT N-value)	STANDARD PENETRATION TEST DATA (○ NMC, ⊢⊣ PL---LL, △ % Fines; 20, 40, 60, 80)	ELEVATION
0	Fill - Fat Clay			TOPSOIL, 2 inches			0
				FAT CLAY (CH), few rock fragments, very stiff, red brown, slightly moist	5-19-10 N= 29		
	Residuum - Fat Clay			FAT CLAY (CH), trace fine sand, stiff, brown, slightly moist	3-4-6 N= 10		
5							-5
				FAT CLAY (CH), little fine sand, trace chert, stiff, brown and gray, slightly moist	6-6-8 N= 14		
			REC-100%				
10					4-5-6 N= 11		-10
				FAT CLAY (CH), few chert, very soft, brown, wet			
					1-1-0 N= 1		
15							-15
	Auger refusal at 16.9 feet Borehole terminated at 16.9 feet						
20							-20
25							-25
30							-30

GROUNDWATER		DATE/TIME	DEPTH (FT)	REMARKS
DURING ADVANCE	▽			
END OF DRILLING	▼			
AFTER DRILLING	▼			
AFTER DRILLING	▼			

GROUNDWATER DEPTHS ARE NOT EXACT AND MAY VARY SUBSTANTIALLY FROM THOSE INDICATED.
LL=Liquid Limit, PL = Plastic Limit, NM = Natural Moisture Content, PPV = Pocket Penetrometer (tsf), PTV = Pocket Torvane (tsf)

Project:	Mountain Home VA Center Elevated Walkway Johnson City, Tennessee 1243-19-046	BORING LOG: B-2-4 *Sheet 1 of 1*

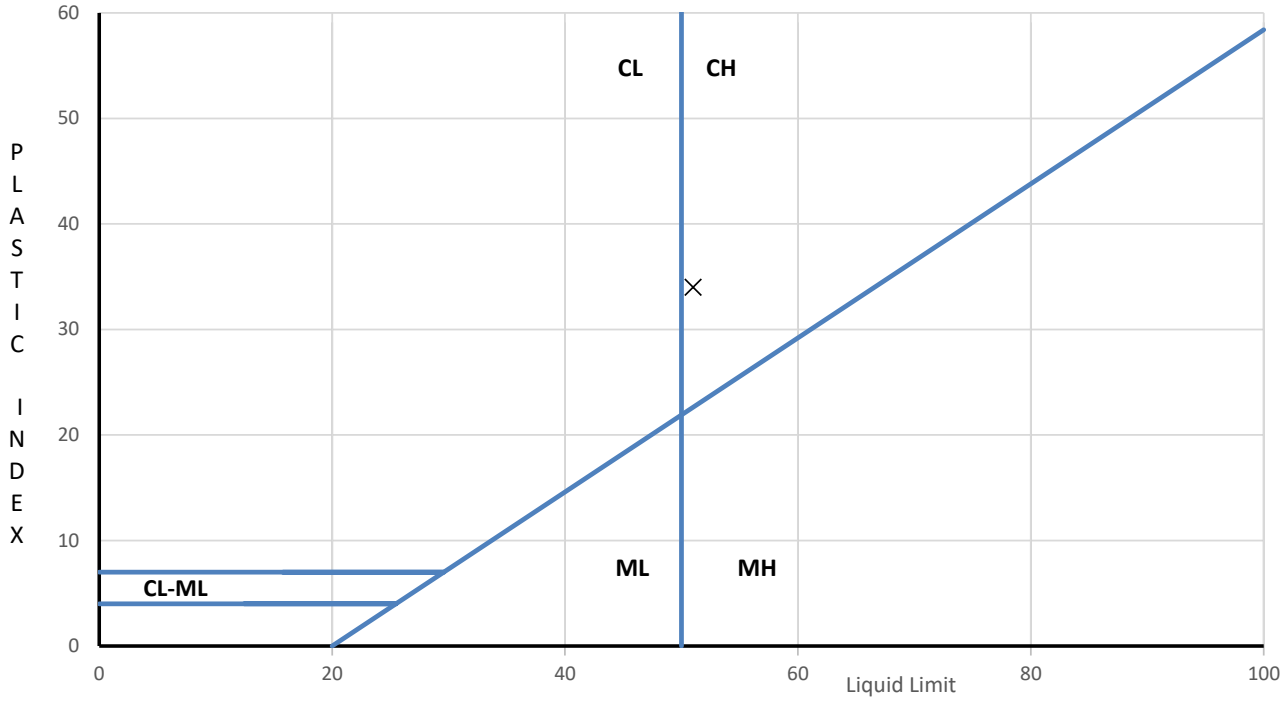
DATE DRILLED: 09/28/2019	**ELEVATION:**	**NOTES:** Sample descriptions based on visual observation of obtained samples.
DRILL RIG: Diedrich D-50 (ATC)	**DATUM:** NAVD88	
DRILLER: T. Bunch	**BORING DEPTH:** 12.5 ft	
HAMMER TYPE: Auto Hammer (140 lb)	**CLOSURE:** Backfilled with cuttings	
DRILLING METHOD: 3 1/4" H.S.A.	**LOGGED BY:** J. Feuge	**Latitude:** **Longitude:**
SAMPLING METHOD: SS, UD	**PROJECT COORDINATE SYSTEM -** NAD 1983 StatePlane Tennessee FIPS 4100 Feet	

DEPTH (feet)	NOTES	GRAPHIC	SAMPLE NO. (RECOVERY)	MATERIAL DESCRIPTION	BLOW COUNT DATA (SPT N-value)	STANDARD PENETRATION TEST DATA (○ NMC, ⊢⊣ PL---LL, △ % Fines; 20, 40, 60, 80)	ELEVATION
0	Fill - Fat Clay			TOPSOIL, 2 inches FAT CLAY (CH), few chert, trace roots, stiff, red brown, slightly moist	7-6-6 N= 12		0
					15-7-6 N= 13		
5	Residuum - Fat Clay			FAT CLAY (CH), stiff, light brown and gray, moist	3-4-5 N= 9		-5
			REC-100%				
10					4-4-6 N= 10		-10
	Auger refusal at 12.5 feet Borehole terminated at 12.5 feet				50/0" N= 100		
15							-15
20							-20
25							-25
30							-30

GROUNDWATER		DATE/TIME	DEPTH (FT)	REMARKS
DURING ADVANCE	▽			
END OF DRILLING	▼			
AFTER DRILLING	▼			
AFTER DRILLING	▼			

GROUNDWATER DEPTHS ARE NOT EXACT AND MAY VARY SUBSTANTIALLY FROM THOSE INDICATED.
LL=Liquid Limit, PL = Plastic Limit, NM = Natural Moisture Content, PPV = Pocket Penetrometer (tsf), PTV = Pocket Torvane (tsf)

Appendix III – Laboratory Test Reports



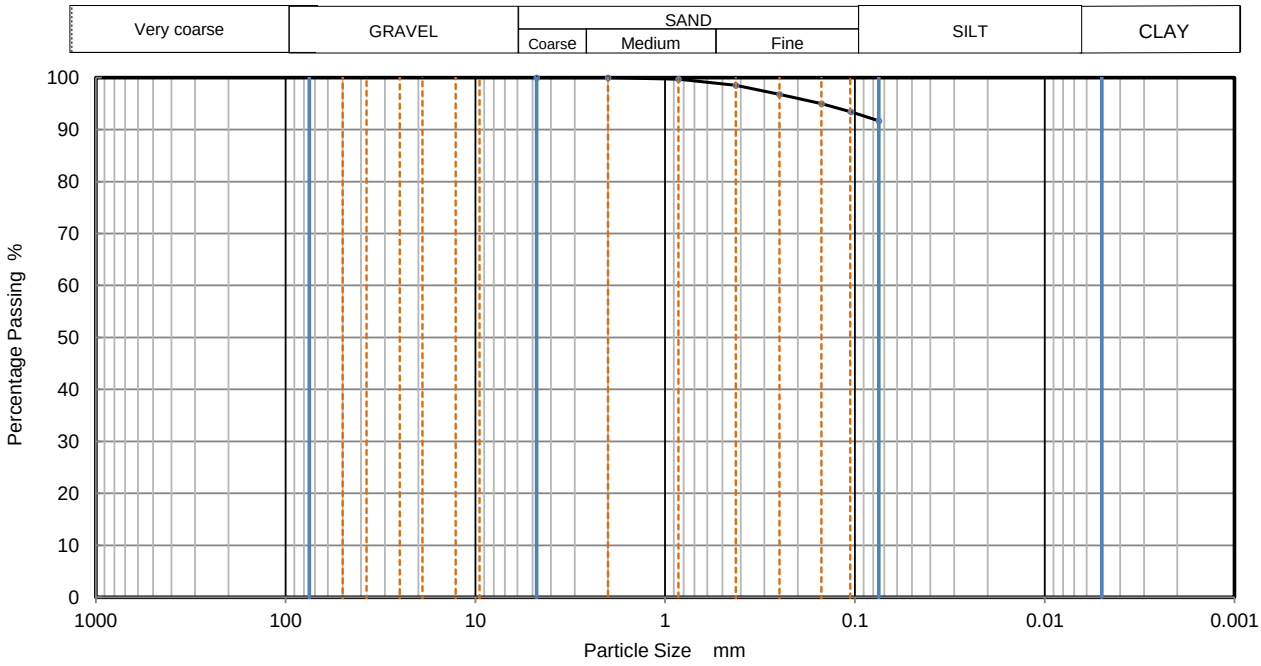
Specimen Identification				MC	LL	PL	PI	Fines	Classification (symbol is based on minus 40 material only when no grain size information is present.)	
	ID	No.	Top Depth						Symbol	Name
	B-2-1	**SS-2**	**3.5**	**23.7**						
	B-2-1	**SS-6**	**18.5**	**27.0**						
	B-2-1	**SS-8**	**28.5**	**42.8**						
	B-2-2	**SS-1**	**1**	**22.4**						
	B-2-2	**SS-2**	**3.5**	**22.3**						
	B-2-3	**SS-1**	**1**	**7.1**						
	B-2-3	**SS-2**	**3.5**	**35.0**						
	B-2-4	**SS-1**	**1**	**20.3**						
	B-2-4	**SS-2**	**3.5**	**17.5**						
X	**B-2-4**	**UD-1**	**8**		**51**	**17**	**34**	**91.7**	**CH**	**Fat clay**

INDEX TEST RESULTS

Project Name	Mountain Home VA Center Walkway		
Project Number	1243-19-046		
Tested by	Date	Approved by	Date
Derek Baker	10/14/2019	N. Randy Raimondi	10/14/2019

Report No. KNOX_19000129
Report Date 10/14/2019

	ASTM D6913: Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis	Project Number	**1243-19-046**
		Borehole/Pit No.	B-2-4
Project Name	Mountain Home VA Center Walkway	Sample No.	UD-1
USCS	Fat clay CH	Sample Top Depth, ft	8.00
Specimen No.	Specimen Depth ft	Sample Type	UD
Lab visual/manual description		Log No.	KNOX2019100710



Sieving		Hydrometer Sedimentation	
Particle Size mm	% Passing	Particle Size mm	% Passing
4.75	100		
2	100		
0.85	100		
0.425	99		
0.25	97		
0.15	95		
0.105	93		
0.075	92		

Dry Mass of sample, g	87.6

Sample Proportions	% dry mass
Very coarse, >75mm	
Gravel, 75mm - 4.75mm	0.0
Coarse Sand, 4.75 to 2.00mm	0.0
Medium Sand, 2.00 to 0.425mm	1.5
Fine Sand, 0.425 to 0.075mm	6.8
Fines <0.075mm	91.7

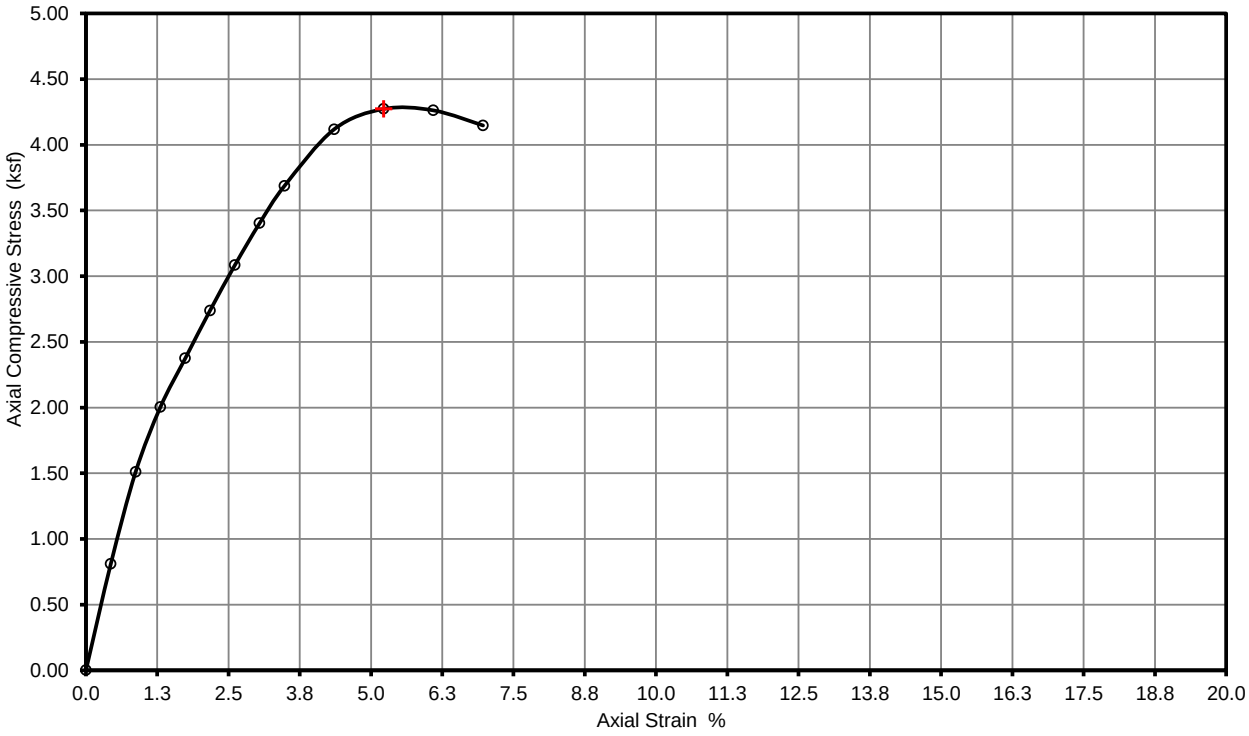
Remarks

Reviewed and approved by N.Randy Rainwater 10/14/19

	Unconfined Compressive Strength of Soils ASTM D2166	Project Number	1243-19-046
		Borehole/Pit No.	B-2-4
Project Name	Mountain Home VA Center Walkway	Sample No.	UD-1
Client Name		Sample Top Depth, ft	8.00
Lab visual/manual description	FAT CLAY (CH), brown and gray	Log No.	KNOX2019100710

Average Length		5.7428	in
Average Diameter		2.8175	in
Moisture Content		22.6	%
Dry Density		102.7	pcf
Rate of Strain		0.87	%/min
At failure	Axial Strain	5.2	%
	Unconfined Compressive Strength	4.273	ksf
	Mode of Failure	Shear	

Axial Compressive Stress v Axial Strain



Remarks

Axial compressive stress corrected for area change.

Report No.	KNOX_19000130
Report Date	10/14/2019

Tested by	Date	Approved by	Date
Derek Baker	10/14/2019	[illegible]	10/14/2019

Form No. TR-43-D7012C-02
Revision No. : 0
Revision Date: 08/22/18

UNCONFINED COMPRESSION
(ASTM D7012 Method C)

S&ME, Inc. - Knoxville 1413 Topside Road, Louisville, TN 37777

Project Name: Mountain Home VA Walkway
Project Number: 1243-19-046

Report Date: October 10, 2019
Reviewed By: John B. Peason

Boring No.	Sample No.	Depth (ft)	Dimensions, in.		Shape (See Key)	Area (in^2)	Unit Weight (lbs/ft^3)	Loading Rate (psi/sec)	Maximum Load (lbs)	Strength (psi)	Moisture (%)
			Length	Diameter							
B-2-2	Run 1	10.1 - 10.5	4.16	1.99	A	3.11	167.1	89	57,946	18,632	0.0

NOTES: Effective (as received) unit weight as determined by RTH 109-93.
Loading rates were selected to target reaching failure between 2 and 15 minutes.
Test results for specimens not meeting the requirements of ASTM D4543-08$^{\epsilon 1}$ may differ from a test specimen that meets the requirements of ASTM D4543.

SHAPE KEY

ASTM D4543-08$^{\epsilon 1}$ *Standard Practice for Preparing Rock Core as Cylindrical Test Specimens and Verifying Conformance to Dimensional and Shape Tolerance* Section 1.2 - "Rock is a complex engineering material that can vary greatly as a function of lithology, stress history, weathering, moisture content and chemistry, and other natural geologic processes. As such, it is not always possible to obtain or prepare rock core specimens that satisfy the desirable tolerances given in this practice. Most commonly, this situation presents itself with weaker, more porous, and poorly cemented rock types and rock types containing significant or weak (or both) structural features. For these and other rock types which are difficult to prepare, all reasonable efforts shall be made to prepare a specimen in accordance with this practice and for the intended test procedure. However, when it has been determined by trial that this is not possible, prepare the rock specimen to the closest tolerances practicable and consider this to be the best effort and report it as such and if allowable or necessary for the intended test, capping the ends of the specimen as discussed in this practice is permitted."

A Test specimen measurements met the desired shape tolerances of ASTM D4543-08$^{\epsilon 1}$ (side straightness, end flatness & parallelism, and end perpendicularity to axis)

B Test specimen measurements met the desired shape tolerances of ASTM D4543-08$^{\epsilon 1}$ for end flatness & parallelism, and end perpendicularity to axis. Specimen did not meet the desired tolerance for side straightness. Specimen prepared to closest tolerances practicable.

C Test specimen measurements met the desired shape tolerances of ASTM D4543-08$^{\epsilon 1}$ for end flatness & parallelism. Specimen did not meet the desired tolerances for side straightness and end perpendicularity to axis. Specimen prepared to closest tolerances practicable.

D Test specimen measurements met the desired shape tolerances of ASTM D4543-08$^{\epsilon 1}$ for end flatness. Specimen did not meet the desired tolerances for side straightness, parallelism and end perpendicularity to axis. Specimen prepared to closest tolerances practicable.

E Test specimen measurements met the desired shape tolerances of ASTM D4543-08$^{\epsilon 1}$ for end flatness and end perpendicularity to axis. Specimen did not meet the desired tolerance for side straightness and parallelism. Specimen prepared to closest tolerances practicable.

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Date: 10/9/2019

Photographer: Tori Igoe

1	**Location / Orientation**	B-2-2, Run 1 (10.1′ – 10.5′)
	Remarks	Unconfined Compressive Strength of Rock Core Specimen Before/After (ASTM D7012 Method C)