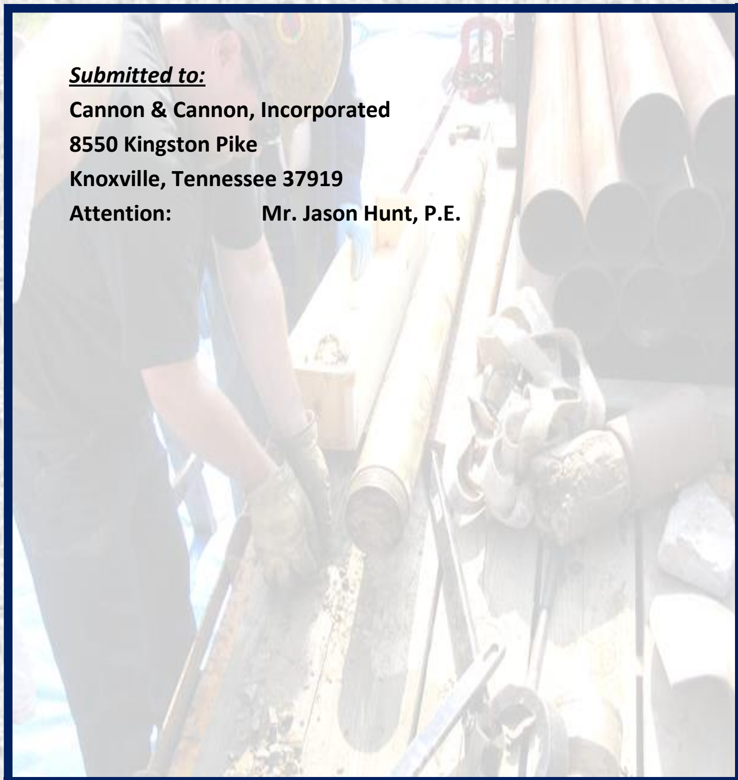

Submitted to:
Cannon & Cannon, Incorporated
8550 Kingston Pike
Knoxville, Tennessee 37919
Attention: Mr. Jason Hunt, P.E.

REPORT OF PRELIMINARY GEOTECHNICAL EXPLORATION

KCDC Development – Vine Avenue

Knoxville, Tennessee

Submitted by:
GEOServices, LLC
2561 Willow Point Way
Knoxville, Tennessee 37931

Phone (865) 539-8242
Fax (865) 539-8252

GEOServices, LLC-Geotechnical and Materials Engineers

GEOSERVICES, LLC

PROJECT NO. 21-19354

May 20, 2019

Cannon & Cannon, Incorporated
8550 Kingston Pike
Knoxville, Tennessee 37919

Attention: Mr. Jason Hunt, P.E.
jhunt@cannnon-cannon.com

Subject: **REPORT OF PRELIMINARY GEOTECHNICAL EXPLORATION**
KCDC Development – Vine Avenue
Knoxville, Tennessee
GEOServices Project No. 21-19354

Dear Mr. Hunt:

We are submitting this report of preliminary geotechnical exploration performed for the subject project. Our services were performed in accordance with our Proposal Number 11-19071, dated February 14, 2019, and as authorized by you. The following report presents our findings and preliminary recommendations for the subject property. Should you have any questions regarding this report, or if we can be of any further assistance, please contact us at your convenience.

Sincerely,
GEOServices, LLC

THOMAS BRIAN WILLIAMSON
REGISTERED ENGINEER
AGRICULTURE
COMMERCE
No. 118861
STATE OF TENNESSEE
5/20/19

Matthew B. Haston, P.E.
Senior Geotechnical Engineer

T. Brian Williamson, P.E.
Geotechnical Department Manager

TBW/MBH/tbw

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1.0 INTRODUCTION

1.1 PURPOSE

The purpose of our services was to provide preliminary geotechnical information for potential development of the site. The geotechnical exploration provides general subsurface conditions across the site and geotechnical recommendations for general site development and depth of refusal materials.

This exploration is preliminary in nature and should be used for general site planning and feasibility evaluation only. Due to the relatively limited information available at this preliminary stage of the project, preparation of a complete report of geotechnical study with specific recommendations for foundation design and site will require supplemental exploration and analysis. Project details and performance criteria should; however, be initially further developed. As project details are developed, additional exploration, field and laboratory testing, and engineering analysis will be required.

1.2 PROJECT INFORMATION AND SITE DESCRIPTION

Project information was provided by Mr. Jason Hunt via email on February 5, 2019, which included an aerial image of the project site as well as some preliminary information regarding the anticipated project. The project site is located at the southwest quadrant at the intersection of South Gay Street and West Vine Avenue in Knoxville, Tennessee. Based on a review of available topographic information (i.e. KGIS), it appears the site is generally sloping downhill from the west to the east with existing grades ranging from about 940 to 918 feet MSL. Although detailed construction information has not been provided, we understand that KCDC is evaluating the site for possibly a mid-to high-rise structure (i.e. 3, or more stories). We were requested to perform a geotechnical exploration in efforts to provide preliminary geotechnical recommendations for the project.

The project site currently exists as a surface parking lot. Ground cover exists as asphalt pavement and grass areas. The project site is bordered by West Vine Avenue to the north, South Gay Street to the east, West Summit Hill Drive to the south and by an existing hotel to the west. As mentioned, the site currently exists as a parking lot. However, a review of available historical aerial imagery of the site revealed the site was previously occupied by multiple structures.

1.3 SCOPE OF STUDY

The study involved a site reconnaissance, field drilling, laboratory testing, and engineering analysis. The following sections of this report present discussions of the field exploration, site conditions, and conclusions and preliminary recommendations. Following the text of this report, Appendix A presents figures and test boring record. Appendix B presents the results of the laboratory testing.

The scope of our geotechnical engineering services did not include an environmental assessment for determining the presence or absence of wetlands, or hazardous or toxic materials in the soil, bedrock, surface water, groundwater, or air, on, or 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.

2.0 EXPLORATION AND TESTING PROGRAMS

2.1 FIELD EXPLORATION

The existing subsurface conditions was explored with a total of five (5) soil test borings drilled at the approximate locations shown on Figure 2 in Appendix A attached to this report. The boring locations were selected and located in the field by GEOServices LLC. (GEOS) personnel Drilling was performed on April 18, 2019. The borings were advanced using 2.25-inch inside diameter hollow stem augers (HSA) with a Geoprobe® track-mounted drill rig. Detailed logs for soil test borings can be found in Appendix A of this report.

Within each boring, SPT and split-spoon sampling were performed at approximately 2.5-foot intervals in the upper 10 feet, and 5 feet intervals thereafter. Rock coring to explore auger refusal material was performed in accordance with ASTM D 2113. Standard Penetration Tests and split-spoon sampling were performed in accordance with ASTM D 1586.

In split–spoon sampling, a standard 2-inch O.D. split-spoon sampler is driven into the bottom of the boring with a 140-pound hammer falling a distance of 30 inches. The number of blows required to advance the sampler the last 12 inches of the standard 18 inches of total penetration is recorded as the Standard Penetration Resistance (N-value). These N-values are indicated on the boring logs at the testing depth, and provide an indication of strength of cohesive materials.

2.2 LABORATORY TEST PROGRAM

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 Unified Soil Classification System (USCS – ASTM D 2487) by a GEOServices geotechnical professional. Select samples were then tested for moisture content (ASTM D 2216) and Atterberg limits (ASTM D 4318). The laboratory test results are further discussed in the following sections of this report and a summary is provided in Appendix B.

3.0 SUBSURFACE CONDITIONS

3.1 GEOLOGIC CONDITIONS

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.

Published geologic information indicates that this site is underlain by bedrock of the Ottosee Shale Formation of the Chickamauga Group. This formation is primarily composed of calcareous shale with minor amounts of coarsely crystalline, fossiliferous limestone (i.e. marble). The Ottosee Shale formation typically weathers to produce a tan or yellowish-brown clay residual soil with weathered shale fragments.

The boundary between soil and rock is not sharply defined in this geologic setting and there often is a transitional zone, termed "weathered rock" overlying competent bedrock. Weathering is facilitated by fractures, joints, and the presence of less resistant rock types. Consequently, the profile of the weathered rock and hard rock is quite irregular and erratic, even over short horizontal distances. Also, it is not unusual to find lenses and boulders of hard rock and/or zones of weathered rock within the soil mantle well above the general bedrock level.

Since the bedrock formations underlying the site consist of dolomite, they are susceptible to the typical carbonate 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 the hazards. Of these hazards, the occurrence of sinkholes is potentially the most damaging. In East Tennessee, sinkholes occur primarily due to differential weathering of the bedrock and "flushing" or "raveling" of overburden soils into the cavities in the bedrock. The loss of solids creates a cavity or "dome" in the overburden. Growth of the dome over time or excavation over the dome can create a condition in which rapid, local 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 at any site located within carbonate geologic settings. A rigorous effort to assess the potential for sinkhole development at this site was beyond our scope-of-services for this project. However, typical characteristics of sinkhole conditions (SPT N-values significantly decreasing with depth with moisture content significantly increasing with depth) were not encountered during our subsurface exploration. Furthermore, we did not observe any closed contour depressions, which are indicative of past sinkhole activity, on the USGS (Knoxville, TN Quadrangle) topographic map within the vicinity of the site.

It is our opinion that the risk of sinkhole development at this site is no greater than at other sites located within similar geologic settings which have been developed successfully. However, the owner must be willing to accept a slight risk of future sinkhole development at this site. The risk of sinkhole development can be reduced by following the recommendations provided in the *Sinkhole Risk Reduction and Corrective Actions* (Section 5.5) section of this report.

3.2 SOIL STRATIGRAPHY

The following subsurface description is of a generalized nature to highlight the subsurface stratification features and material characteristics at the boring locations. The boring logs included in Appendix A of this report should be reviewed for specific information at each boring location. Information on actual subsurface conditions exists only at the specific boring locations and is relevant only to the time that this exploration was performed. Variations may occur and should be expected at the site

Surficial Layer

A surficial layer of asphalt pavement consisting of 4 of asphalt underlain by 8 inches of gravel was encountered in each boring.

Fill Soils

Underlying the surficial layers encountered in each boring, existing fill soils were encountered to depths ranging from 8 to 22 feet beneath the existing ground surface. Fill is generally classified as soils that have been transported and placed by man. The fill generally consisted of orangish brown and dark brown fat clay (CH) with varying quantities of gravel, chert, brick and asphalt fragments. The SPT N-value of the existing fill ranged from about 4 blows per foot (bpf) to 50 blows with 0 inches of penetration, indicating a relative consistency of soft to very hard. We note that the N-values indicating very hard fill soils were often encountered within zones of brick and or gravel layers. The fill material was generally soft to stiff in consistency. Moreover, auger refusal was encountered within the fill material in three of the borings (B-1, B-2 and B-4) and the depth of fill may be deeper in these areas. Moisture contents of selected fill samples ranged between 18.6 to 31.6 percent. Atterberg limit testing on a selected fill sample results in a liquid limit (LL) of 58 percent and plasticity index (PI) of 35 percent, indicating the tested fill classified as fat (higher plasticity) clay (CH).

Residual Soil

Residual soils were encountered underlying the existing fill soils in two of the borings (B-3 and B-5) to depths ranging from 20 to 26 feet beneath the existing ground surface. The residual soil generally consisted of yellowish brown and brown lean (lower plasticity) clays (CL) with varying amounts of chert fragments and sand. The SPT N-values used to evaluate the consistency of the residual soil encountered ranged 2 to 6 bpf, indicating a relative consistency of very soft to firm. Moisture content testing on select residual soil samples ranged from 26 to 46.6 percent.

Weathered Rock

Weathered rock was encountered in boring B-3 below the residual soils to a depth of 26 feet below existing ground surface. The weathered rock encountered consisted of very hard brown to black siltstone with sand. The moisture content of the tested weathered rock samples was 19.9 percent.

Auger Refusal

Auger refusal was encountered in each of the five borings conducted on site at depths ranging from 13 to 26 feet beneath the existing ground surface. Auger refusal is a designation applied to any material that cannot be penetrated by the power auger. As mentioned previously, refusal was encountered in three of the borings prior to penetrating the fill. Coring of refusal materials was beyond the scope of our services. Given the depths of refusal and the fact that portions of the site were previously occupied by structures, borings B-1, B-2 and B-4 likely refused on buried concrete foundations or slabs. However, based on our experience in this geologic setting and the samples retrieved, the refusal materials in borings B-3 and B-5 likely correspond to top of pinnacles, ledges or competent bedrock.

Subsurface Water

Subsurface water was encountered in borings B-3 and B-5 at a depth of 18 feet at the completion of drilling. It is noted that subsurface water levels may fluctuate due to seasonal changes in precipitation amounts. Additionally, areas of perched water may exist in the overburden and/or near the contact with refusal materials.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 SITE ASSESSMENT

The results of the subsurface exploration indicate that the site is generally underlain by existing fill material that was generally soft to stiff in consistency that contained varying deleterious materials (e.g. brick, gravel, asphalt, etc.). Moreover, three of the borings did not penetrate the fill due to the presents of what is believed to be buried foundations or concrete slabs. The residual soils, where encountered, were generally soft to firm in consistency. As the project is in the early conceptual phase, detailed information regarding proposed foundation loading and building support is currently unavailable. However, based on the subsurface conditions encountered, the site does present certain challenges that should be address and understood prior to any development. These challenges include the presence of undocumented existing fill, previous structures / buried foundations, karst geology and groundwater conditions. Each of these challenges are discussed herein.

Existing Fill

Existing fill material that consisted of higher plasticity clay was encountered in each boring to depths ranging from 8 to 22 feet below. Moreover, three of the borings did not penetrate the existing fill soils. Given the SPT N-values within the zone of existing fill, it is likely that the fill was not “engineered” fill. As such, observations of the fill placement nor compaction testing was likely not performed. There are certain risks associated with construction on these types of fill. The risk primarily consists of excessive and/or non-uniform settlement caused by extensive zones or pockets of soft, loose, or uncompacted material (such as those that were encountered across the site in the borings performed).

Previous Structures

As review of available historical aerial photography of the site revealed that site was once occupied by multiple structures. We understand the buildings were demolished around 1980 and the site was converted into a surface parking lot. The size of the previous buildings is currently unknown to us. As such, it is unknown if the buildings did have below grade portions or underground utilities. However, given the subsurface conditions encountered (i.e. refusal within fill material and buried brick), it is likely that at least a portion of the buildings did have below grade elevations (i.e. basements). As with any site that was previously occupied, there is inherent risks of encountering buried building remnants (e.g. foundations, walls, slabs, etc.) or buried construction debris (e.g. brick, concrete, etc.). Given the data retrieved from the drilling and sampling performed, it is likely that these materials are present at this site. Further exploration could and should be performed across the site once design progresses to better understand the impacts of these materials for the development.

Karst Geology

Any site underlain by carbonic bedrock is susceptible to sinkhole formation. Generally, sites underlain by a proximately shale bedrock formation (such as this one) have lower susceptibility to sinkhole formation. Typical subsurface conditions that would indicate higher karst (i.e. sinkhole) potential consist of significantly decreasing SPT N-values and increasing moisture contents with depth. Based on the subsurface conditions encountered, it is our professional opinion that the risk of sinkhole development is no higher at this site as the sites surrounding that have been successfully developed previously. We understand that the parking lot has previously experienced subsidence. Furthermore, we understand that area was repaired by removing the distressed asphalt, removal of soft unsupportive soils and the placement of a new concrete patch. Based on the subsurface conditions encountered in Boring B-2, which was performed in the general vicinity of the patch, it is our opinion the subsidence was likely due to settlement of existing fill and not the results of karst solutioning (i.e. sinkhole activity).

Groundwater Conditions

Where the fill was penetrated, subsurface water was encountered at a depth of approximately 18 feet below existing ground surface. The elevations of the ground water ranged between 912 and 912 feet. As the finished floor elevation for the proposed building is currently unavailable, we are providing this discussion. If finished floor is proposed to be within about 4 feet of the elevations where ground water was encountered, the owner should anticipate a temporary dewatering system will be required during construction and a permanent dewatering system will be required as part of the building design.

4.2 PRELIMINARY FOUNDATION RECOMMENDATIONS

As the project is in the conceptual design phase, detailed information regarding the proposed building is currently unavailable. However, we understand the owner is evaluating the construction of a midrise (3 to 5) story building. Furthermore, the proposed building may require below grade portions. Based on our experience with similar construction, we expect structural loads may range between 300 and 700 kips. Given the presence of undocumented existing fill, the feasibility of utilizing conventional shallow foundations bearing on the existing fill likely not permissible. Furthermore, given the depths of existing fill, a complete undercut of the existing fill would likely not be an economic option. As such, we expect either an intermediate or deep foundation will be required. Each option is described below.

Aggregate Pier – Foundation Loads less than 500 Kips (maximum per column)

An intermediate foundation system would consist of an aggregate pier remediated subgrade and may be used as an alternative to deep foundations. The aggregate pier support elements are constructed by drilling a hole, removing a volume of soil, then building a bottom bulb of clean, open-graded stone while vertically pre-stressing and pre-straining subsoils underlying the bottom bulb. The aggregate pier shaft is built on top of the bottom bulb, using well-graded high base course stone placed in thin lifts (12-inches compacted thickness). Aggregate pier foundation systems would reinforce the existing subsoils on this site to allow shallow foundation support of structural loads less than 500 kips for the proposed construction. After the installation of the aggregate pier, the proposed structure may be supported using a conventional shallow foundation system. Aggregate pier foundations have recently been used to support structures with fairly high structural loads (such as parking garages and mid-rise buildings).

Micropiles - Foundation Loads Greater than 500 Kips

Micropiles are installed by drilling a steel pipe (i.e., casing) to the underlying bedrock. The hole is then extended, without casing, through competent bedrock creating a socket (the pile bond length). Once the appropriate socket is penetrated (a function of rock quality and design bond strength), a steel reinforcing bar is centered in the casing which extends from the bottom of the socket to the pile cut-off length. Finally, the entire cased length is pumped full of grout using the tremie method. Construction techniques and methods associated with micropiles are very flexible and may vary from this general description in some ways. However, based on the subsurface conditions encountered, it is our professional opinion that un-cased micropiles will **not** be appropriate for this project. If un-cased, the micropile excavation will likely collapse (or neck) upon removal of the drilling equipment and not allow for proper grouting or retain the required diameter for the entire depth. Therefore, we recommend that cased micropiles be utilized on this project.

5.0 CONSTRUCTION CONSIDERATIONS

5.1 EXCAVATIONS

Auger refusal materials were encountered in each of the borings conducted on site. Auger refusal conditions generally correspond to materials which require hoe-ramming and/or blasting for removal. Typically, soils penetrated by augers can be removed with conventional earthmoving equipment. However, excavation equipment varies, and field refusal conditions may vary. Generally, the weathering process is erratic and variations in the rock profile can occur in small lateral distances. Therefore, it is possible that some partially weathered rock and/or rock pinnacles or ledges requiring difficult excavation techniques may be encountered in site areas between our boring locations. The owner should be aware that some partially weathered rock, bedrock or boulders may be encountered which will require blasting and/or mechanical breakers (hoe-ram) for removal. Furthermore, buried concrete slabs or foundations, if encountered, will require hoe-ramming for removal.

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

5.2 HIGH PLASTICITY SOIL CONSIDERATIONS

Based on our experience in the East Tennessee area, soils with plasticity indices (PI) less than 30 percent have a slight potential for volume changes with changes in moisture content, and soils with a PI greater than 50 percent are highly susceptible to volume changes. Between these values, we consider the soils to be slightly to moderately susceptible to volume changes.

Highly plastic soils have the potential to shrink or swell with significant changes in moisture content. Unlike other areas of the country where high plasticity soils cause considerable foundation problems, East Tennessee does not typically endure long periods of severe drought or wet weather. However, in recent years, drought conditions have been sufficient to cause soil shrinkage and related structural distress of buildings, floor slabs and pavements at sites underlain by high plasticity soils.

At sites that have high plasticity soils, certain precautions should be considered to minimize or eliminate the potential for volume changes. The most effective way to eliminate the potential for volume changes is to remove highly plastic soils and replace them with compacted fill of non-expansive material. Testing and recommendations for the required depth of removal can be provided, if needed. If removal of the highly plastic soils is not desirable, then measures should be taken to protect the soils from excessive amounts of wetting or drying. In addition, modification of the soils by lime or cement treatment can be utilized to reduce the soil plasticity. Several construction considerations may reduce the potential for volume changes in the subgrade soils. Foundations should be excavated, checked, and concreted in the same day to prevent excessive wetting or drying of the foundation soils. The floor subgrade should be protected from excessive drying and wetting by covering the subgrade prior to slab construction. The site should be graded in order to drain surface water away from the building both during and after construction. Installing moisture barriers around the perimeter of the slab will help limit the moisture variation of the soil and reduce the potential for shrinking or swelling. In addition, roof drains should discharge water away from the building area and foundations. Heat sources should be isolated from foundation soils to minimize drying of the foundation soils. Trees and large shrubs can draw large amounts of moisture from the soil during dry weather and should be kept well away from the building to prevent excessive drying of the foundation soils. Watering of lawns or landscaped areas should be performed to maintain moisture levels during dry weather.

Structural details to make the building flexible should be considered to accommodate potential volume changes in the subgrade. Floor slabs should be liberally jointed to control cracking, and the floor slab should not be structurally connected to the walls. Walls should incorporate sufficient expansion/contraction joints to allow for differential movement.

5.3 MOISTURE SENSITIVE SOILS

The moderately plastic fine-grained soils encountered at this site will be sensitive to disturbances caused by construction traffic and changes in moisture content. During wet weather periods, increases in the moisture content of the soil can cause significant reduction in the soil strength and support capabilities. Construction traffic patterns should be varied to prevent the degradation of previously stable subgrade. In addition, the soils at this site which become wet may be slow to dry and thus significantly retard the progress of grading and compaction activities. We caution if site grading is performed during the wet weather season increases in the undercut volume required due to the marginal fills should be expected. Further for site fills, methods such as discing and allowing the material to dry will be required to meet the required compaction recommendations. It will, therefore, be advantageous to perform earthwork and foundation construction activities during dry weather. However, November through March is typically the difficult grading period due to the limited drying conditions that exist.

5.4 DRAINAGE AND SURFACE WATER CONCERNS

To reduce the potential for undercutting, water should not be allowed to collect in the foundation excavations, on floor slab areas, or on prepared subgrades of the construction area either during or after construction. Undercut or excavated areas should be sloped toward one corner to facilitate removal of any collected rainwater, subsurface water, or surface runoff. Positive site surface drainage should be provided to reduce infiltration of surface water around the perimeter of the building(s) and beneath the floor slab(s). The grades should be sloped away from the building(s) and surface drainage should be collected and discharged such that water is not permitted to infiltrate the backfill and floor slab areas of the building(s).

5.5 SINKHOLE RISK REDUCTION AND CORRECTIVE ACTIONS

Based on our experience, corrective actions can also be performed to reduce the potential for sinkhole development at this site. These corrective actions would decrease but not eliminate the potential for sinkhole development. Much can be accomplished to decrease the potential of future sinkhole activity by proper grade selection and positive site drainage.

In general, the portions of a site that are excavated to achieve the desired grades will have a higher risk of sinkhole development than the areas that are filled, because of the exposure of relic fractures in the soil to rainfall and runoff. On the other hand, those portions of a site that receive a modest amount of fill (or that have been filled in the past) will have a decreased risk of sinkhole development caused by rainfall or runoff because the placement of a cohesive soil fill over these areas effectively caps the area with a relatively impervious “blanket” of remolded soil.

Therefore, the recommendations that follow incorporate a modest remedial treatment program designed to make the surface of the soil in excavated areas less permeable.

Although it is our opinion that the risk of ground subsidence associated with sinkhole formation cannot be eliminated, we have found that several measures are useful in site design and development to reduce this potential risk. These measures include:

- Maintaining positive site drainage to route surface waters well away from structural areas both during construction and for the life of the structure.
- The scarification and re-compaction of the upper 6 to 10 inches of soil in earthwork cut areas.
- Verifying that subsurface piping beneath structures is carefully constructed and pressure tested prior to its placement in service.
- The use of pavement or lined ditches, particularly in cut areas, to collect and transport surface water to areas away from structures.

Considerations when building within a sinkhole prone area are to provide positive surface drainage away from any proposed building or parking area both during and after construction. Backfill in utility trenches of other excavations should consist of compacted, well-graded material such as dense graded aggregate or compacted on site soils. The use of an open graded stone such as No. 57 stone is not recommended unless the stone backfill is provided an exit path and not allowed to pond. If sinkhole conditions are observed, the type of corrective action is most appropriately determined by GEOServices on a case-by-case basis.

6.0 LIMITATIONS

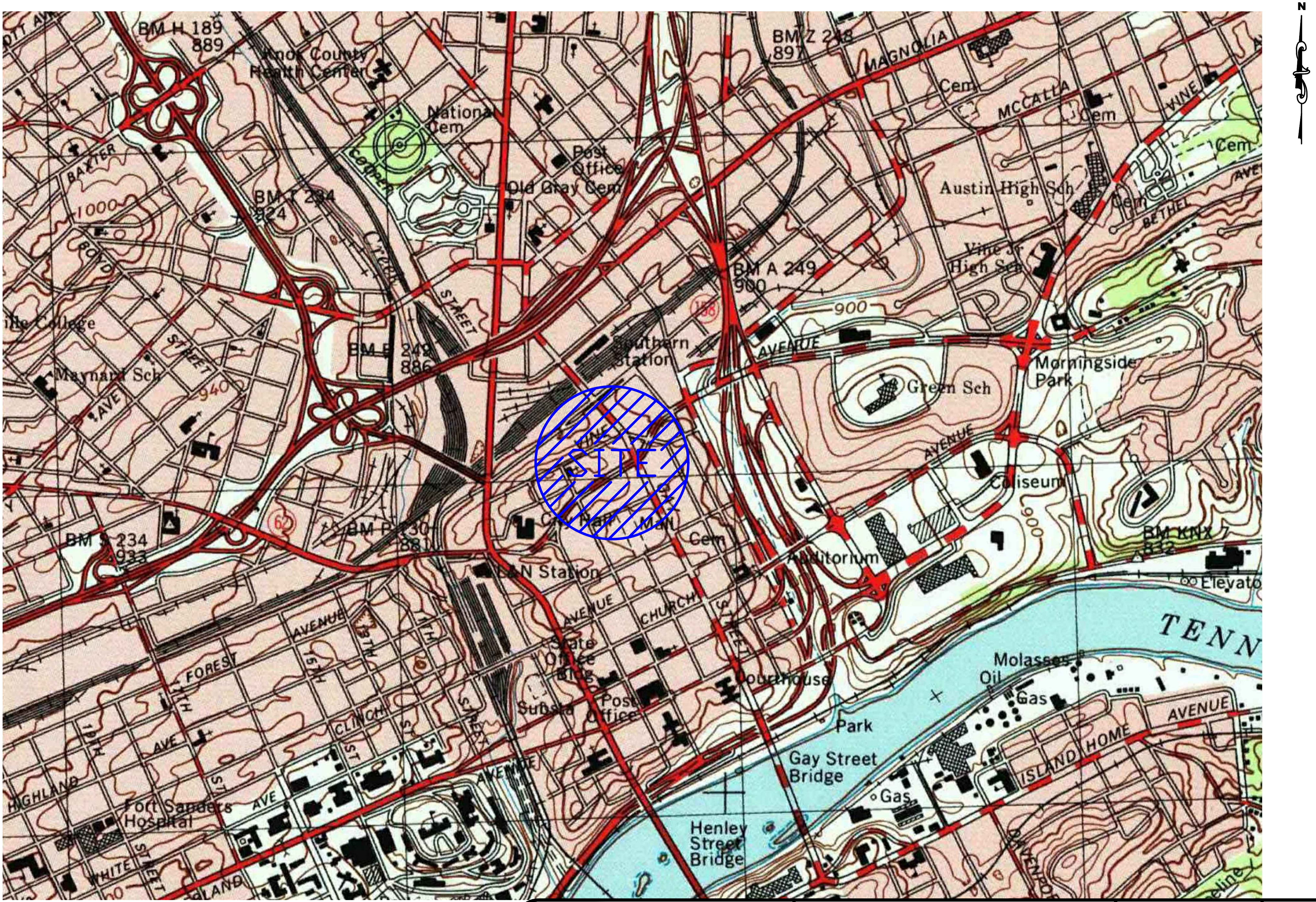
This report has been prepared in accordance with generally accepted geotechnical engineering practice for specific application to this project. This report is for our geotechnical work only, and no environmental assessment efforts have been performed. The conclusions and recommendations contained in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

The analyses and recommendations submitted herein are based, in part, upon the data obtained from the exploration. The nature and extent of variations between the borings will not become evident until construction. We recommend that GEOServices be retained to observe the project construction in the field. GEOServices cannot accept responsibility for conditions which deviate from those described in this report if not retained to perform construction observation and testing. If variations appear evident, then we will re-evaluate the recommendations of this report. In the event that any changes in the nature, design, or location of the structures are planned, the conclusions and recommendations contained in this report will not be considered valid unless the changes are reviewed and conclusions modified or verified in writing. Also, if the scope of the project should change significantly from that described herein, these recommendations may need to be re-evaluated.

APPENDIX A

Figures and Test Boring Records

GEOS
GEOServices, LLC-Geotechnical and Materials Engineers



NOTES:

1.) BASE MAP: USGS QUADRANGLE (KNOXVILLE, TENNESSEE)

GEOS

GEOServices, LLC-Geotechnical and Materials Engineers

2561 Willow Point Way
Knoxville, Tennessee 37931

Office: 865-539-8242
Fax: 865-539-8252

SITE VICINITY MAP

PROPOSED KCDC DEVELOPMENT

WEST VINE AVENUE

KNOXVILLE, TENNESSEE

DRAWN BY:	KSR
APPROVED BY:	MRB
SCALE:	N.T.S.
JOB NO.:	21-19354
DATE:	4/30/2019

FIGURE

1

NOTES:
1.) BORING LOCATIONS ARE SHOWN IN GENERAL ARRANGEMENT ONLY.
2.) DO NOT USE BORING LOCATIONS FOR DETERMINATIONS OF DISTANCES OR QUANTITIES.
3.) BASE MAP PROVIDED BY: Cannon & Cannon, Inc.

⌖ LOCATION OF SOIL TEST BORINGS

SOIL TEST BORING LOCATION PLAN

PROPOSED KCDC DEVELOPMENT

WEST VINE AVENUE

KNOXVILLE, TENNESSEE

DRAWN BY:	KSR
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Figure 2

GENERAL NOTES

FINE AND COARSE GRAINED SOIL PROPERTIES

PARTICLE SIZE

BOULDERS:	GREATER THAN 300 mm
COBBLES:	75 mm to 300 mm
GRAVEL:	4.74 mm to 75 mm
COARSE SAND:	2 mm to 4.74 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

COARSE GRAINED SOILS (SANDS & GRAVELS)

N-VALUE	RELATIVE DENSITY
0 - 4	VERY LOOSE
5 - 10	LOOSE
11 - 30	MEDIUM DENSE
31 - 50	DENSE
OVER 50	VERY DENSE

FINE GRAINED SOILS (SILTS & CLAYS)

N-VALUE	CONSISTENCY	Qu, PSF
0 - 2	VERY SOFT	0 - 500
3 - 4	SOFT	500 - 1000
5 - 8	FIRM	1000 - 2000
9 - 15	STIFF	2000 - 4000
16 - 30	VERY STIFF	4000 - 8000
OVER 31	HARD	8000 +

STANDARD PENETRATION TEST (ASTM D1586)

THE STANDARD PENETRATION TEST AS DEFINED BY ASTM D1586 IS A METHOD TO OBTAIN A DISTURBED SOIL SAMPLE FOR EXAMINATION AND TESTING AND TO OBTAIN RELATIVE DENSITY AND CONSISTENCY INFORMATON. THE 1.4 INCH I.D./2.0 INCH O.D. SAMPLER IS DRIVEN 3-SIX INCH INCREMENTS WITH A 140 LB. HAMMER FALLING 30 INCHES. THE BLOW COUNTS REQUIRED TO DRIVE THE SAMPLER THE FINAL 2 INCREMENTS ARE ADDED TOGETHER AND DESIGNATED THE N-VALUE. AT TIMES, THE SAMPLER CAN NOT BE DRIVEN THE FULL 18 INCHES. THE FOLLOWING REPRESENTS OUR INTERPRETATION OF THE STANDARD PENETRATION TEST WITH VARIATIONS.

BLOWS/FOOT (N-VALUE)	DESCRIPTION
25	25 BLOWS DROVE SAMPLER 12" AFTER INITIAL 6" SEATING
75/10"	75 BLOWS DROVE SAMPLER 10" AFTER INITIAL 6" SEATING
50/PR	PENETRATION REFUSAL OF SAMPLER AFTER INITIAL 6" SEATING

SAMPLING SYMBOLS

ST:	UNDISTURBED SAMPLE
SS:	SPLIT SPOON SAMPLE
CORE:	ROCK CORE SAMPLE
AU:	AUGER OR BAG SAMPLE

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, TSF
DUW:	DRY UNIT WEIGHT, PCF

ROCK PROPERTIES

ROCK QUALITY DESIGNATION (RQD)

PERCENT	QUALITY
90 TO 100	EXCELLENT
75 TO 90	GOOD
50 TO 75	FAIR
25 TO 50	POOR
0 TO 25	VERY POOR

ROCK HARDNESS

VERY SOFT:	ROCK DISINTEGRATES OR EASILY COMPRESSES TO TOUCH: CAN BE HARD TO VERY HARD SOIL.
SOFT:	ROCK IS COHERANT BUT BREAKS EASILY TO THUMB PRESSURE AT SHARP EDGES AND CRUMBLES WITH FIRM HAND PRESSURE.
MODERATELY HARD:	SMALL PIECES CAN BE BROKEN OFF ALONG SHARP EDGES BY CONSIDERABLE HARD THUMB PRESSURE: CAN BE BROKEN BY LIGHT HAMMER BLOWS.
HARD:	ROCK CAN NOT BE BROKEN BY THUMB PRESSURE, BUT CAN BE BROKEN BY MODERATE HAMMER BLOWS.
VERY HARD:	ROCK CAN BE BROKEN BY HEAVY HAMMER BLOWS.

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LOG OF BORING B-1
SHEET 1 OF 1

DRILLER L. Knox
ON-SITE REP.

BORING NO. / LOCATION B-1
DRY ON COMPLETION ? YES

DATE April 18, 2019 SURFACE ELEV. 934.0 FT.
REFUSAL: Yes DEPTH 13.0 FT. ELEV. 921.0 FT.
SAMPLED 13.0 FT. 4.0 M
TOP OF ROCK DEPTH FT. ELEV. FT.
BEGAN CORING DEPTH FT. ELEV. FT.
FOOTAGE CORED (LF) FT.
BOTTOM OF HOLE DEPTH 13.0 FT. ELEV. 921.0 FT.

WATER LEVEL DATA (IF APPLICABLE)

COMPLETION:	DEPTH	DRY	FT.
	ELEV.		FT.
AFTER 1 HRS:	DEPTH	TNP	FT.
	ELEV.		FT.
AFTER 24 HRS.	DEPTH	TNP	FT.
	ELEV.		FT.

BORING ADVANCED BY: POWER AUGERING X
PROPOSED FFE: FT.

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	SAMPLE DEPTH TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qu	LABORATORY RESULTS LL	PI	%M	STRATUM DESCRIPTION
											Asphalt (4 Inches)
											Basestone (8 Inches)
		1..0	2.5	1	SS	2 - 3 - 3 N = 6					Fat CLAY (CH) - with chert fragments and trace gravel - orangish brown and grayish brown - moist (FILL)
2.5	931.5										
		3.5	5.0	2	SS	3 - 3 - 3 N =6					
5.0	929.0										
		6.0	7.5	3	SS	3 - 2 - 6 N = 8					
7.5	926.5										
		8.5	8.7	4	SS	50/2 " N = 50/2 "					
10.0	924.0										
12.5	921.5										
											Auger Refusal at 13.0 Feet
15.0	919.0										
17.5	916.5										
20.0	914.0										

REMARKS:

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LOG OF BORING B-2
SHEET 1 OF 1

DRILLER: L. Knox
ON-SITE REP.:

BORING NO. / LOCATION: B-2
DRY ON COMPLETION ? YES

DATE: April 18, 2019
SURFACE ELEV. 924.0 FT.
REFUSAL: Yes DEPTH 13.5 FT. ELEV. 910.5 FT.
SAMPLED 13.5 FT. 4.1 M
TOP OF ROCK DEPTH FT. ELEV. FT.
BEGAN CORING DEPTH FT. ELEV. FT.
FOOTAGE CORED (LF) FT.
BOTTOM OF HOLE DEPTH 13.5 FT. ELEV. 910.5 FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH DRY FT. ELEV. FT.
AFTER 1 HRS: DEPTH TNP FT. ELEV. FT.
AFTER 24 HRS. DEPTH TNP FT. ELEV. FT.

BORING ADVANCED BY: .
POWER AUGERING X
PROPOSED FFE: FT.

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qu	LABORATORY RESULTS LL	PI	%M	STRATUM DESCRIPTION
											Asphalt (4 Inches)
											Basestone (8 Inches)
		1..0	2.5	1	SS	2 - 1 - 4 N = 5					Fat CLAY (CH) - with trace gravel, chert fragments, brick fragments at depth and trace organics - orangish brown and brown - moist to dry (FILL)
2.5	921.5										
		3.5	5.0	2	SS	2 - 3 - 3 N = 6					
5.0	919.0										
		6.0	7.5	3	SS	2 - 2 - 2 N = 4					
7.5	916.5										
		8.5	10.0	4	SS	17 - 12 - 17 N = 19					
10.0	914.0										
12.5	911.5										
		13.5	13.5	5	SS	50/0 " N = 50/0 "					Auger Refusal at 13.5 Feet
15.0	909.0										
17.5	906.5										
20.0	904.0										

REMARKS:

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LOG OF BORING B-3
SHEET 1 OF 1
DRILLER: L. Knox
ON-SITE REP.:

BORING NO. / LOCATION: B-3
DRY ON COMPLETION ?: NO

DATE: April 18, 2019
SURFACE ELEV. 934.0 FT.
REFUSAL: Yes DEPTH 26.0 FT. ELEV. 916.0 FT.
SAMPLED: 26.0 FT. 7.9 M
TOP OF ROCK DEPTH FT. ELEV. FT.
BEGAN CORING DEPTH FT. ELEV. FT.
FOOTAGE CORED (LF) FT.
BOTTOM OF HOLE DEPTH 26.0 FT. ELEV. 908.0 FT.
BORING ADVANCED BY: POWER AUGERING X
PROPOSED FFE: FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH 18.0 FT. ELEV. 916.0 FT.
AFTER 1 HRS: DEPTH FT. ELEV. FT.
AFTER 24 HRS. DEPTH TNP FT. ELEV. FT.

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qu	LABORATORY RESULTS LL	PI	%M	STRATUM DESCRIPTION
											Asphalt (4 Inches)
											Basestone (8 Inches)
		1..0	2.5	1	SS	3 - 2 - 3 N =5				21.2	Fat CLAY (CH) - with trace gravel, trace topsoil and chert fragments - reddish brown and brown - moist (FILL)
2.5	931.5										
		3.5	5.0	2	SS	2 - 1 - 3 N =4				18.6	
5.0	929.0										
		6.0	7.5	3	SS	3 - 3 - 6 N = 9				28.1	
7.5	926.5										
		8.5	10.0	4	SS	2 - 3 - 3 N = 6				46.6	Lean CLAY (CL) - with chert fragments - yellowish brown and oragnish brown - moist to wet - firm to very soft (RESIDUUM)
10.0	924.0										
12.5	921.5										
		13.5	15.0	5	SS	1 - 1 - 1 N = 2				40.2	
15.0	919.0										
17.5	916.5										
		18.5	20.0	6	SS	2 - 1 - 1 N = 2				39.4	
20.0	914.0										

Continued

REMARKS:

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LOG OF BORING B-3
SHEET 1 OF 1

DRILLER L. Knox
ON-SITE REP.

BORING NO. / LOCATION B-3
DRY ON COMPLETION ? NO

DATE April 18, 2019 SURFACE ELEV. 934.0 FT.
REFUSAL: Yes DEPTH 26.0 FT. ELEV. 916.0 FT.
SAMPLED 26.0 FT. 7.9 M
TOP OF ROCK DEPTH FT. ELEV. FT.
BEGAN CORING DEPTH FT. ELEV. FT.
FOOTAGE CORED (LF) FT.
BOTTOM OF HOLE DEPTH 26.0 FT. ELEV. 908.0 FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH 18.0 FT. ELEV. 916.0 FT.
AFTER 1 HRS: DEPTH FT. ELEV. FT.
AFTER 24 HRS. DEPTH TNP FT. ELEV. FT.

BORING ADVANCED BY: . POWER AUGERING X
PROPOSED FFE: FT.

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qu	LABORATORY RESULTS LL	PI	%M	STRATUM DESCRIPTION
											Continued
22.5	911.5										
		23.5	24.4	8	SS	9 - 50/5 " N = 50/5 "				19.9	Weathered ROCK (WR) - siltstone with sand - brown and black - moist - stiff
25.0	909.0										
27.5	906.5										Auger Refusal at 26.0 Feet
30.0	904.0										
32.5	901.5										
35.0	899.0										
37.5	896.5										
40.0	894.0										

REMARKS:

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LOG OF BORING B-4
SHEET 1 OF 1

DRILLER: L. Knox
ON-SITE REP.:

BORING NO. / LOCATION: B-4
DRY ON COMPLETION ?: YES

DATE	April 18, 2019		SURFACE ELEV.	930.0	FT.	
REFUSAL:	Yes	DEPTH 22.0 FT.	ELEV.	908.0	FT.	
SAMPLED	22.0 FT.	6.7 M				
TOP OF ROCK		DEPTH FT.	ELEV.		FT.	
BEGAN CORING		DEPTH FT.	ELEV.		FT.	
FOOTAGE CORED (LF)		FT.				
BOTTOM OF HOLE DEPTH		22.0 FT.	ELEV.	908.0	FT.	

WATER LEVEL DATA (IF APPLICABLE)

COMPLETION:	DEPTH	DRY	FT.
	ELEV.		FT.
AFTER 1 HRS:	DEPTH	TNP	FT.
	ELEV.		FT.
AFTER 24 HRS.	DEPTH	TNP	FT.
	ELEV.		FT.

BORING ADVANCED BY: POWER AUGERING X

PROPOSED FFE: FT.

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qu	LABORATORY RESULTS LL	PI	%M	STRATUM DESCRIPTION
											Asphalt (4 Inches)
											Basestone (8 Inches)
		1..0	2.5	1	SS	8 - 6 - 6 N = 12				21.2	
2.5	927.5										
		3.5	5.0	2	SS	3 - 3 - 4 N = 7		58	35	31.1	
5.0	925.0										
		6.0	7.5	3	SS	1 - 2 - 1 N = 3				32.3	
7.5	922.5										
		8.5	10.0	4	SS	8 - 4 - 2 N = 6				26.4	Fat CLAY (CH) - with chert fragments, trace gravel and asphalt in the upper 10 feet and gravel at depth - grayish brown and orangish brown - moist (FILL)
10.0	920.0										
12.5	917.5										
		13.5	15.0	5	SS	3 - 3 - 4 N = 7				20.6	
15.0	915.0										
17.5	912.5										
		18.5	20.0	6	SS	3 - 3 - 22 N = 25				31.6	
20.0	910.0										Auger Refusal at 22.0 Feet

REMARKS:

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Knoxville, Tennessee
GEOServices Project # 21-19354

LOG OF BORING B-5
SHEET 1 OF 1
DRILLER L. Knox
ON-SITE REP.

BORING NO. / LOCATION B-5
DRY ON COMPLETION ? NO

DATE April 18, 2019 | SURFACE ELEV. 930.0 FT.
REFUSAL: Yes DEPTH 23.0 FT. | ELEV. 907.0 FT.
SAMPLED 23.0 FT. 7.0 M
TOP OF ROCK DEPTH FT. | ELEV. FT.
BEGAN CORING DEPTH FT. | ELEV. FT.
FOOTAGE CORED (LF) FT.
BOTTOM OF HOLE DEPTH 23.0 FT. | ELEV. 907.0 FT.

WATER LEVEL DATA (IF APPLICABLE)
COMPLETION: DEPTH 18.0 FT. ELEV. 912.0 FT.
AFTER 1 HRS: DEPTH TNP FT. ELEV. FT.
AFTER 24 HRS. DEPTH TNP FT. ELEV. FT.

BORING ADVANCED BY: POWER AUGERING X
PROPOSED FFE: FT.

STRATUM DEPTH FT.	ELEV.	SAMPLE DEPTH FROM FT.	TO FT.	SAMPLE OR RUN NO.	SAMPLE TYPE	FIELD RESULTS N-Value	Qu	LABORATORY RESULTS LL	PI	%M	STRATUM DESCRIPTION
											Asphalt (4 Inches)
											Basestone (8 Inches)
		1..0	2.5	1	SS	1 - 3 - 4 N = 7				24.5	Fat CLAY (CH) - with trace gravel and gravel at depth - orangish brown and brown - moist (FILL)
2.5	927.5										
		3.5	5.0	2	SS	4 - 4 - 12 N = 16				23.7	
5.0	925.0										
		6.0	7.5	3	SS	8 - 10 - 15 N = 25				28.6	
7.5	922.5										
		8.5	10.0	4	SS	4 - 2 - 1 N = 3				36.8	Lean CLAY (CL) - with chert fragments, trace sand and trace gravel in the upper 3 feet - orangish brown and yellowish brown - moist to dry - very soft to firm (RESIDUUM)
10.0	920.0										
12.5	917.5										
		13.5	15.0	5	SS	1 - 1 - 1 N = 2					
15.0	915.0										
17.5	912.5										
		18.5	20.0	6	SS	7 - 1 - 5 N = 6				36	
20.0	910.0										

Auger Refusal at 23.0 Feet

REMARKS:

APPENDIX B

Laboratory Testing Results

GEOServices, LLC-Geotechnical and Materials Engineers

KCDC Vine Avenue

GEOServices Project No. 21-19354

May 6, 2019

SOIL DATA SUMMARY

Boring Number	Sample Number	Depth (feet)	Natural Moisture Content	Atterberg Limits			Soil Type	Percent Organic Content
				LL	PL	PI		
B-3	1	1.0-2.5'	21.2%					
	2	3.5-5.0'	18.6%					
	3	6.0-7.5'	28.1%					
	4	8.5-10.0'	46.6%					
	5	13.5-15.0'	40.2%					
	6	18.5-20.0'	39.4%					
	7	23.5-25.0'	19.9%					
B-4	1	1.0-2.5'	21.2%					
	2	3.5-5.0'	31.1%	58	23	35	CH	
	3	6.0-7.5'	32.3%					
	4	8.5-10.0'	26.4%					
	5	13.5-15.0'	20.6%					
	6	18.5-20.0'	31.6%					
B-5	1	1.0-2.5'	24.5%					
	2	3.5-5.0'	23.7%					
	3	6.0-7.5'	28.6%					
	4	8.5-10.0'	36.8%					
	6	18.5-20.0'	36.0%					