Future MTSU Student Housing Project

Geotechnical Engineering Report

November 15, 2024 | Terracon Project No. 18245169

Prepared for:

Rodney L. Wilson Consulting
205 Rolling Mill Ct.
Old Hickory, TN 37138

Terracon

1922 Old Murfreesboro Pike, Bldg 900, Ste 905
Nashville, TN 37217
P (615) 333-6444
Terracon.com

November 15, 2024

Rodney L. Wilson Consulting
205 Rolling Mill Ct.
Old Hickory, TN 37138

Attn: Mr. Rodney Wilson
P: 615-476-2055
E: rwilson@rlwconsult.com

Re: Geotechnical Engineering Report
Future MTSU Student Housing Project
1835 Alumni Drive
Murfreesboro, Tennessee
Terracon Project No. 18245169

Dear Rodney:

We have completed the scope of Geotechnical Engineering services for the above referenced project in general accordance with Terracon Proposal No. P18245169 dated June 12, 2024, and Supplemental Change Order dated August 17, 2024. This report presents the findings of the subsurface exploration and provides geotechnical recommendations concerning earthwork and the design and construction of foundations, slabs and pavements for the proposed project.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report or if we may be of further service, please contact us.

Sincerely,

Terracon

ASHFAQ AHMED MEMON
REGISTERED ENGINEER
AGRICULTURE
COMMERCE
No. 106145
STATE OF TENNESSEE

Juan Vazquez, E.I.
Staff Engineer

Ashfaq Memon, P.E.
Senior Engineer

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Note: This report was originally delivered in a web-based format. **Blue Bold** text in the report indicates a referenced section heading. The PDF version also includes hyperlinks which direct the reader to that section and clicking on the Terracon logo will bring you back to this page. For more interactive features, please view your project online at **client.terracon.com**.

Refer to each individual Attachment for a listing of contents.

Introduction

This report presents the results of our subsurface exploration and Geotechnical Engineering services performed for the proposed five-story housing structures and a five-story parking garage to be located at 1835 Alumni Drive in Murfreesboro, Tennessee. The purpose of these services was to provide information and geotechnical engineering recommendations relative to:

- Subsurface soil and rock conditions
- Groundwater conditions
- Seismic site classification per IBC
- Site preparation and earthwork
- Demolition considerations
- Foundation design and construction
- Floor slab design and construction
- Lateral earth pressures
- Pavement design and construction

The geotechnical engineering Scope of Services for this project included the advancement of test borings, laboratory testing, engineering analysis, and preparation of this report.

Drawings showing the site and boring locations are shown on the **Site Location** and **Exploration Plan**, respectively. The results of the laboratory testing performed on soil samples obtained from the site during our field exploration are included on the boring logs in the **Exploration Results** section.

Project Description

Our initial understanding of the project was provided in our proposal and was discussed during project planning. A period of collaboration has transpired since the project was initiated, and our final understanding of the project conditions is as follows:

Item	Description
Information Provided	Information provided by email communications between Mr. Rodney Wilson of RLW Consulting, PLLC to Mr. Will McCloy of Terracon. Provided information included a site aerial image with preliminary boring locations prepared by RLW Consulting. Subsequent information provided by Mr. Rodney Wilson included a site aerial image with additional boring locations prepared by RLW Consulting.
Project Description	The project will include MTSU student housing facility containing three at-grade 5-story housing buildings and a 5-story parking garage.
Proposed Structure	Three wood framed five-story student housing structures and a five-level concrete parking garage. A precise final building layout was unavailable at the time of this report. However, an approximate preliminary building layout was provided to us.
Building Construction	Steel frame or concrete frame Load-bearing masonry walls Cast in place or precast concrete for parking garage Slab-on-grade (non-basement)
Finished Floor Elevation	Not provided; exploration depths have assumed that finished floor is within 3 feet of existing grades
Structural Loads	Approximate preliminary structural loads for housing units were provided by RLW consulting. Parking garage loads were assumed based on our past experience on similar projects. ▪ Columns: 100-200 kips (housing structures) ▪ Columns: 700-800 kips (parking garage) ▪ Walls: 2-4 kips per linear foot (housing structures) ▪ Walls: 5-6 kips per linear foot (parking garage) ▪ Slabs: 100 pounds per square foot (psf)
Grading/Slopes	A grading plan with building locations was not available. Based on existing grades and planned construction, we expect up to 3 feet of fill and less than 2 feet of cut may be required to develop final grades.
Below-Grade Structures	None anticipated
Free-Standing Retaining Walls	Retaining walls are not assumed to be constructed as part of site development to achieve final grades.
Building Code	2018 IBC

Item	Description
Pavements	Assumed traffic is as follows: ▪ Autos/light pickup trucks: 500 vehicles per day ▪ Light delivery vehicles: 5 vehicles per day ▪ Trash collection trucks: 2 vehicle per week ▪ Heavy-duty (semi) delivery trucks: 1 vehicle per week The pavement design period is 20 years.

Terracon should be notified if any of the above information is inconsistent with the planned construction, especially the grading limits, as modifications to our recommendations may be necessary.

Site Conditions

The following description of site conditions is derived from our site visit in association with the field exploration and our review of publicly available geologic and topographic maps.

Item	Description
Parcel Information	The project is located on MTSU Campus at 1835 Alumni Drive Murfreesboro, Tennessee. Site is approximately 14 acres Lat/Long: 35.8450° N/86.3597° W See **Site Location**
Existing Improvements	Existing MTSU apartment buildings, sidewalks and landscaping Existing pavements consisting of asphalt and/or concrete
Current Ground Cover	Grass, asphalt, concrete and a few scattered trees along site perimeter
Existing Topography (From *Murfreesboro GIS Data dated 2023*)	Approximate maximum grades vary from about 624 feet to 630 feet, MSL.

We also collected photographs at the time of our field exploration program(s) and select samples. Representative photos are provided in our **Photography Log**.

Geotechnical Characterization

We have developed a general characterization of the subsurface conditions based upon our review of the subsurface exploration, laboratory data, geologic setting and our understanding of the project. This characterization, termed GeoModel, forms the basis of our geotechnical calculations and evaluation of the site. Conditions observed at each exploration point are indicated on the individual logs. The individual logs can be found in the **Exploration Results** and the GeoModel can be found in the **Figures** attachment of this report.

As part of our analyses, we identified the following model layers within the subsurface profile. For a more detailed view of the model layer depths at each boring location, refer to the GeoModel.

Model Layer	Layer Name	General Description
1	Surficial Cover	Approximately 3 to 7 inches of topsoil
2	Fill/Possible Fill	Lean clay with some limestone rock fragments, trace of roots and mineral nodules
3	Lean Clay	Low plasticity clay, medium stiff to very stiff
4	Fat Clay	Moderately high plasticity clay, medium stiff to very stiff
5	Limestone Bedrock	Moderately to slightly weathered, highly to slightly fractured, thin to medium bedded (RQD = 68 to 100% and REC = 90 to 100%)

Borings B-1 through B-8 were performed during our preliminary study. Borings B-9 through B-18 were performed during our final study. Both studies were performed based on preliminary building layout. Borings B-1 through B-4, and B-9 through B-18 were drilled within/near the proposed housing building pads and borings PG-1 through PG-4A and B-5 through B-8 were drilled within/near the proposed parking garage area.

The borings typically encountered about 3 to 7 inches of topsoil cover at the surface. Underlying the topsoil cover, borings B-5 through B-7, PG-1, and PG-3 through PG-4A encountered about 2½ to 3¼ feet of existing fill consisting of lean and fat clay with some limestone rock fragments. Boring B-5 encountered auger refusal at about 3 feet below existing grade likely on large size rocks within the existing fill. The fill exhibited highly erratic Standard Penetration Test (SPT) N-values ranging from 4 to 50 blows per foot (bpf). The higher N-values are probably exaggerated due to the presence of limestone fragments within the existing fill and do not represent the true strength of the existing fill.

Natural fat clay was encountered beneath the existing fill and beneath the surface cover where fill is absent and extended to auger refusal/termination depths ranging from about 5½ to 22 feet below existing grade. The natural clay is typically stiff to very stiff but occasionally medium stiff based on SPT N-values varying from 6 to 31 bpf. Relatively lower strength soils (N-values of 6 to 7 bpf) were encountered in borings B-13 and B-15 within portions of the proposed middle and east housing buildings. Some higher N-vales greater than 50 bpf occurred near auger refusal are probably exaggerated due to the presence of weathered limestone rock.

The depth to auger refusal/termination at our boring locations varied from about 3 to 24¾ feet below the existing ground surface. The following table summarizes auger refusal depths at each location.

Boring No.	Approx. Auger Refusal Depth (feet)	Boring No.	Approx. Auger Refusal Depth (feet)
B-1	5 ½	B-13	9 ¾
B-2	7 ¾	B-14	12 ¼
B-3	11 ¼	B-15	15 ¼
B-4	5 ¾	B-16	6 ¼
B-5	3	B-17[1]	See Note 1
B-6	18 ¾	B-18	11
B-7	9 ½	PG-1[2]	3 ¼
B-8	18 ½	PG-2	22
B-9	10 ¾	PG-3	17 ½
B-10[1]	See Note 1	PG-4	17 ¼
B-11	9 ¼	PG-4A[2]	14 ¾
B-12	9 ½	--	--

1. Boring terminated at target depth without encountering auger refusal
2. Rock core location

Rock coring procedures are generally required to determine the character and continuity of the auger refusal material and these factors must be considered when evaluating the depth to auger refusal in those test borings that are not cored. Rock core operations were performed at borings PG-1, PG-2, and PG-4A to better explore the auger refusal materials at these locations. Boring PG-2 encountered false auger refusal at about 9½ feet below existing grade and encountered a 5-inch thick rock lens suspended within the clay overburden. This boring location encountered auger refusal again at about 22 feet below existing grade likely on bedrock. At boring locations, PG- 1 and PG-4A, auger refusal occurred on limestone bedrock. The bedrock materials sampled from the borings consist of gray, moderately to slightly weathered, thin to medium bedded limestone. Bedrock cores obtained from borings PG-1 and PG-4A were relatively intact and rock quality was fair to excellent based on RQD values ranging from about 68 to 100 percent.

Unconfined compressive strength tests were performed on two selected rock core samples taken from borings PG-1 and PG-4A at depths of about 3 ¼ to 14 ¾ feet below grade, respectively. The unconfined compressive strength values were about 6,730 psi and 6,630 psi.

Groundwater Conditions

Groundwater was not observed in the borings while drilling, or for the short duration the borings could remain open. This does not necessarily mean the borings terminated above groundwater, or the water levels summarized above are stable groundwater levels. Due to the low permeability of the soils encountered in the borings, a relatively long period may be necessary for a groundwater level to develop and stabilize in a borehole. Long term observations in piezometers or observation wells sealed from the influence of surface water are often required to define groundwater levels in materials of this type.

Groundwater level fluctuations occur due to seasonal variations in the amount of rainfall, runoff and other factors not evident at the time the borings were performed. Therefore, groundwater levels during construction or at other times in the life of the structure may be higher or lower than the levels indicated on the boring logs. Perched water can also develop on top of bedrock or within the porous fill material. The possibility of groundwater level fluctuations and perched water should be considered when developing the design and construction plans for the project.

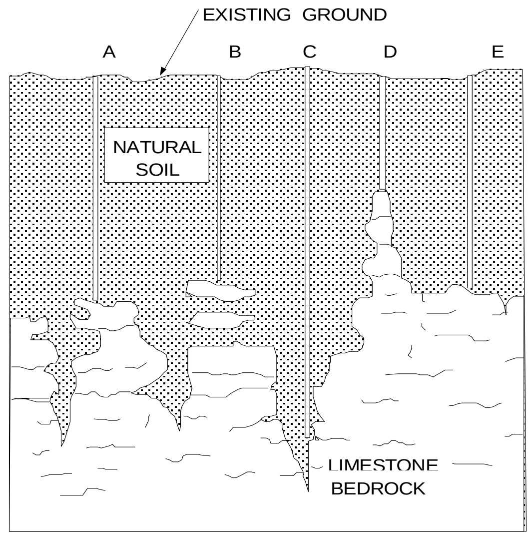
Geologic Hazards

Formation [1]	Description
Ridley Limestone Formation	**Ridley Limestone** - brownish-gray to yellowish brown cryptocrystalline to very fine-grained limestone with thin lenses of chert. Thickness is about 100 feet

1. Geologic Map of the Dillton Quadrangle, Tennessee published by the State of Tennessee Department of Conservation, Division of Geology (1964).

In an area of existing fill, auger refusal can occur on man-made material, such as boulders, "shot rock" or construction debris. In an area of limestone bedrock, auger refusal can result on slabs of unweathered limestone suspended in the residual soil matrix ("floaters"), on rock "pinnacles" rising above the surrounding bedrock surface, in widened joints that may extend well below the surrounding bedrock surface, or on the upper surface of continuous bedrock. Several of these possible auger refusal conditions are illustrated in the adjacent figure.

AUGER REFUSAL ILLUSTRATION



THIS FIGURE IS FOR ILLUSTRATIVE PURPOSES ONLY AND DOES NOT NECESSARILY DEPICT THE SPECIFIC BEDROCK CONDITIONS AT THIS SITE

The Ridley Limestone bedrock formation is known for producing several obstructions that can cause the augers to refuse above sound bedrock. These obstructions can range from floaters to rock pinnacles as illustrated in examples A, B, C, and D in the above figure. Depth to competent bedrock in areas of karst geology can vary greatly over short distances. The possibility of varying depths to bedrock should be considered when developing the design and construction plans for this project.

The site is underlain by carbonate limestone that is highly susceptible to dissolution along joints and bedding planes in the rock mass. This results in voids and solution channels within the rock strata and a highly irregular bedrock surface. The weathering of the bedrock and subsequent collapse or erosion of the overburden into these openings results in what is referred to as karst topography. Any construction in karst topography is accompanied by some degree of risk for future internal soil erosion and ground subsidence that could affect the stability of the soil supported structures. Our review of the available topographic and geologic mapping did not note any sinkholes on the site. Furthermore, the borings drilled at the site did not disclose any obvious signs of impending overburden collapse or soil softening at depth or deep soil slots (cutters) in bedrock due to karst activity within the depths explored.

Seismic Site Class

The seismic design requirements for buildings and other structures are based on Seismic Design Category. Site Classification is required to determine the Seismic Design Category for a structure. The Site Classification is based on the upper 100 feet of the site profile defined by a weighted average value of either shear wave velocity, standard penetration resistance, or undrained shear strength in accordance with Section 20.4 of ASCE 7 and the International Building Code (IBC). Based on the soil/bedrock properties observed at the site and as described on the exploration logs and results, our professional opinion is that a **Seismic Site Classification of C** be considered for the project. Subsurface explorations at this site were extended to a maximum depth of 24 ¾ feet. The site properties below the boring depth and extending to 100 feet below the lowest planned building elevation were estimated based on our experience and knowledge of geologic conditions of the general area. Additional deeper borings or geophysical testing may be performed to confirm the conditions below the current boring or test depth.

Geotechnical Overview

Based on our borings, in our opinion the site is generally suitable for the proposed housing and parking garage development provided our recommendations outlined herein are followed.

In general, the subsurface profile at the site typically consists of medium stiff to very stiff, moderate to highly plastic clays over limestone bedrock. Surficial undocumented cohesive fill was also in a few borings. Bedrock depth varies significantly across the site as evidenced by auger refusal in our borings ranging from 3 to over 22 feet below existing grade. The site is underlain by a limestone formation that is known for irregular weathering, rock pinnacles, soil filled joints (cutters) and solution weathering due to karst activity.

Based on the information obtained from the subsurface exploration, the following geotechnical considerations were identified.

- **Existing Fill -** About 2 ½ to 3 ¼ feet of undocumented fill was encountered in borings B-5 through B-7, PG-1, and PG-3 through PG-4A. The fill typically consisted of lean and fat clay with some samples containing limestone fragments. Documentation regarding fill compaction and quality control was not available. Based on the potential presence of large size rocks, trace organics and erratic and some low SPT N-values indicate existing fill is suspect and appears to have been placed without proper quality control and under the observation of a technical person. These factors pose a potential risk for excessive building settlement if directly supported on the existing fill without remediation. Therefore, we recommend, existing fill, where present within the building footprints and a contiguous 10-foot (minimum width) envelopes, be undercut in its entirety to suitable natural subgrade. The undercut areas should be backfilled with approved engineered fill per our recommendations outlined herein. Based on our borings, fill is expected to be on the order of about 2 to 3 feet thick and is anticipated in the southern half at/near boring locations B-5, B-6 and B-7. Some fill and/or disturbed soils should also be expected within/near the existing structures and roadways that may require some remediation.

The existing fill in pavement areas should also be undercut as necessary to achieve at least 1½-foot thick "buffer" layer of new engineered fill below finished subgrade provided the underlying fill subgrade passes a proofroll and/or recompacted to non-yielding state. Where grading plan requires more than 1½ feet of new engineered fill to reach desired finished subgrade, existing fill may remain provided the fill subgrade passes a proofroll and some risk of higher pavement maintenance is acceptable.

- **High Plasticity "Fat" Clay (CH) -** High plasticity clay was encountered in a few borings near the surface in our borings drilled within the proposed construction footprint. Fat clay has some potential to shrink and swell with changes in moisture content. This volume change potential presents a risk of some objectionable slab or pavement movement and/or cracking in response to changes in the soil's moisture content. Where these soils are exposed at/near finished floor slab subgrade, the upper 1 foot of subgrade should be undercut and replaced with low volume change engineered fill (LL≤45). Delineation of fat clay should be performed in the field by a qualified geotechnical engineer.

- **Moisture Sensitive Soils -** The near surface cohesive soils are moisture sensitive and could become unstable during wet weather and under repetitive construction traffic. Therefore, effective drainage should be implemented early in the construction sequence and maintained after construction to avoid potential subgrade instability issues. We recommend the grading be performed during the warmer and drier times of the year. If grading is performed during the wet season, widespread subgrade instability issues may arise that may require undercutting and replacement of unstable subgrade.

- **Potential Rock Excavation –** Relatively shallow auger refusal depths on the order of 3 to 5 ½ feet below existing grade was encountered in borings B-1, B-4, B-5 and PG-1 drilled within the proposed development area. Depending upon the proposed grading cuts, depth to bedrock and considering relatively shallow auger refusal depths on the order of 3 to 6 ½ feet encountered in boings B-1, B-5, B-16 and PG-1, it is possible some building and parking garage foundations and deep utility excavations in some isolated areas could engage the bedrock surface and may require rock excavation techniques to achieve desired excavation depths. Depending on the quality and depth of excavation and excavation depths, we expect the use of rock excavation equipment such as rock trenchers, hoe ram equipment, line drilling, hydraulic splitting, and possibly blasting will be required to remove bedrock and achieve desired excavation depths. If blasting is performed, we recommend a pre-blast survey should be performed.

Housing Building Foundations - The proposed housing buildings can be supported on a shallow foundation system and ground supported floor slabs after proper subgrade remediation and improvement as discussed herein. The existing suspect fill such as noted in borings B-5 through B-7, and any low to moderate strength soils (N $\leq$6 bpf) such as encountered in boring B-15, where present, within the building footprints and a contiguous 10-foot (minimum width) envelopes, should be undercut to suitable natural subgrade. The undercut areas should be backfilled with approved engineered fill per our recommendations outlined herein. Considerations should be given to perform additional post demo exploration and proofroll to further evaluate subgrade soils and delineate existing fill and low strengths soils requiring remediation. The extent of subgrade remediation should be finalized based on the results of additional evaluation and proofrolling and upon reviewing the final building locations and grading plan.

After building demolition, site clearing, undercutting of existing fill and any low strength soils, the completion of planned grading, the proposed buildings may be supported on shallow foundations over stiff natural soils and/or new engineered fill extending to suitable soils. Foundations supported on onsite stiff soils or new engineered fill may be designed for a maximum allowable bearing pressure of 2,250 psf. This assumes that column loads will not exceed 200 kips and low strength soils and existing fill will be undercut and replaced with new engineered fill to control settlement to tolerable limits. A higher bearing pressure on the order of 4,500 psf may be available if foundations are supported on aggregate pier modified ground

Parking Garage Foundations – Based on the expected moderately high foundation loads, the presence of moderate strength soils, and to control settlement to tolerable limits, it will be necessary to support the at-grade 5-story parking garage foundations on reinforced (modified) ground. Ground reinforcement can be performed via aggregate piers and a shallow foundation system can be used to support the proposed structure on improved ground. Undercutting of existing fill and low strength soils will not be necessary below foundation bearing if the ground is improved with aggregate piers. An allowable bearing pressure of 4,500 psf can be used for design when foundations are supported on aggregate pier modified ground

As an alternate to ground improvement, the proposed structure can be supported on bedrock bearing deep foundation system. A rock bearing deep foundation system consists of either drilled shafts or micropiles. Based on our borings, depth to bedrock within the proposed parking garage is expected to vary from about 3¼ to 22 feet below existing grade, depending upon the location. Relatively shallower depth to bedrock, possibly rock pinnacles, are expected to be encountered in the northwest portion of the garage footprint. Bedrock in the remaining portion is expected to be relatively deeper.

More details regarding ground improvement and deep foundation options are discussed later in this report.

This study was performed based on preliminary buildings layout and in the absence of a grading plan. Therefore, recommendations outlined herein, should be confirmed upon reviewing final building layout and grading plan. Depending upon final building locations and grading configurations and structural loads, post demo additional exploration may be necessary to confirm and/or update recommendations outlined herein.

Additional site preparation recommendations, including subgrade improvement and fill placement, are provided in the **Earthwork** section.

The recommendations contained in this report are based upon the results of field and laboratory testing (presented in the **Exploration Results**), engineering analyses, and our current understanding of the proposed project. The **General Comments** section provides an understanding of the report limitations.

Earthwork

Earthwork is anticipated to include demolition, clearing and grubbing, excavations, and engineered fill placement. The following sections provide recommendations for use in the preparation of specifications for the work. Recommendations include critical quality criteria, as necessary, to render the site in the state considered in our geotechnical engineering evaluation for foundations and floor slabs.

Demolition

The proposed housing structures and parking garage will be constructed within the footprint of the existing Womack Lane Apartments which will need to be demolished, as well as exterior sidewalks, pavements, and utilities. We recommend all existing foundations, slabs, pavements, any walls, and utilities be removed from within the proposed building footprints and at least 10 feet beyond the outer edge of foundations. Below grade excavation required to remove buried structures should be backfilled with approved engineered fill pe our recommendations outlined herein. Any buried utilities outside the construction footprints that are left in the ground should be properly sealed and decommissioned.

Site Preparation

Prior to placing new fill but after site clearing and necessary grading cuts, existing vegetation, topsoil, trees including stumps and root mats should be removed from the entire construction footprint.

Where fill is placed on existing slopes steeper than 5H:1V, benches should be cut into the existing slopes prior to fill placement. The benches should have a minimum vertical face height of 1 foot and a maximum vertical face height of 3 feet and should be cut wide enough to accommodate the compaction equipment. This benching will help provide a positive bond between the fill and natural soils and reduce the possibility of failure along the fill/natural soil interface.

Although no evidence of underground structures (such as septic tanks, cesspools, basements, and utilities) was observed during the exploration and site reconnaissance, such features could be encountered during construction. If underground facilities are encountered, such features should be removed, and the excavation thoroughly cleaned prior to backfill placement and/or construction.

Subgrade Preparation

As discussed earlier, after site clearing, the existing fill and any low strength or disturbed soils such as encountered in some of our borings, should be undercut to suitable natural subgrade and replaced with approved engineered fill as discussed herein. We recommend additional subgrade evaluation and exploration via test pits, proofrolling and DCP testing be performed in the presence of a Terracon representative to delineate existing fill and low strength soils that may require remediation. The extent of subgrade remediation should be finalized based on the results of additional subgrade evaluation and upon reviewing the grading plan.

The subgrade should be proofrolled with a fully-loaded tandem-axle dump truck. The proofrolling should be performed under the observation of the Geotechnical Engineer or representative. Areas excessively deflecting under proofroll should be delineated and subsequently addressed by the Geotechnical Engineer. Such areas should either be removed/replaced or recompacted and/or modified via chemical stabilization. Excessively wet or dry material should either be removed or moisture conditioned and recompacted.

All exposed areas which will receive new fill, once properly cleared and benched where necessary, should be scarified to a minimum depth of 10 inches, moisture conditioned as necessary, and compacted per the compaction requirements in this report. Compacted engineered fill soils should then be placed to the proposed design grade and the moisture content and compaction of subgrade soils should be maintained until foundation or pavement construction.

Based upon the subsurface conditions determined from the geotechnical exploration, subgrade soils exposed during construction are anticipated to be relatively workable; however, the workability of the subgrade may be affected by precipitation, repetitive construction traffic or other factors. If unworkable conditions develop, workability may be improved by scarifying and drying.

As noted in **Geotechnical Characterization,** high plasticity "fat" clay (CH) was encountered near the surface in a few borings within the proposed construction footprint. Where these soils are exposed at/near finished floor slab subgrade, the upper 2-foot of subgrade should be undercut/replaced with low volume change engineered fill (LL$\leq$45).

The on-site clayey soils are susceptible to disturbance and loss of strength from construction activity, particularly if the soil has a high natural moisture content and is wetted by surface water or seepage. Therefore, care should be taken during the site grading operation to provide adequate site drainage and minimize disturbance of the bearing soils. Heavy equipment traffic directly on bearing surfaces should be avoided in wet clay soils.

Existing Fill

As noted in **Geotechnical Characterization**, borings B-5 through B-7, PG-1, and PG-3 through PG-4A encountered previously placed fill to depths ranging from about 2½ to 3¼ feet. We have no records to indicate the degree of control, and consequently, the fill is considered unreliable for support of foundation, floor slabs and pavements. After site clearing, the existing fill, where present, should be undercut in its entirety within the proposed building pads including 10 feet beyond the lateral limits of the building footprints. Following this overexcavation, the entire area should be proofrolled with heavy, rubber tire construction equipment, to aid in delineating areas of soft or otherwise unsuitable soil. Once unsuitable materials have been remediated, and the subgrade has passed the proofroll test, backfill to finished subgrade elevation can begin. The existing undocumented fill that was removed can be evaluated for reuse as engineered fill.

The existing fill in pavement areas should also be undercut as necessary to achieve at least 1½-foot thick "buffer" layer of new engineered fill below finished subgrade provided the underlying fill subgrade passes a proofroll and/or recompacted to non-yielding state. Where grading plan requires more than 1½ feet of new engineered fill to reach desired finished subgrade, existing fill may remain provided the fill subgrade passes a proofroll and/or recompacted to non-yielding state and some risk of higher pavement maintenance is acceptable.

Excavation

We anticipate that most of the excavations for the proposed construction can be accomplished with conventional earth moving equipment. However, depending upon the proposed grading cuts and depth to bedrock, it is possible that some foundation and utility excavations within portions of the building pads, parking garage and deep utilities will engage the bedrock surface and will require rock excavation techniques to achieve the desired excavations depths. We expect use of rock trenchers, hoe ram equipment, line drilling, hydraulic splitting, etc. and blasting will be required to remove bedrock and achieve desired excavation depths.

Soil Stabilization

Methods of subgrade improvement, as described below, could include scarification, moisture conditioning and recompaction, removal of unstable materials and replacement with granular fill (with or without geosynthetics), and chemical stabilization. The appropriate method of improvement, if required, would be dependent on factors such as schedule, weather, the size of area to be stabilized, and the nature of the instability. More detailed recommendations can be provided during construction as the need for subgrade stabilization occurs. Performing site grading operations during warm seasons and dry periods would help reduce the amount of subgrade stabilization required.

If the exposed subgrade is unstable during proofrolling operations, it could be stabilized using one of the methods outlined below.

- **Scarification and Recompaction -** It may be feasible to scarify, dry, and recompact the exposed soils. The success of this procedure would depend primarily upon favorable weather and sufficient time to dry the soils. Stable subgrades likely would not be achievable if the thickness of the unstable soil is greater than about 1 foot, if the unstable soil is at or near groundwater levels, or if construction is performed during a period of wet or cool weather when drying is difficult.
- **Crushed Stone** - The use of crushed stone or crushed gravel is a common procedure to improve subgrade stability. Typical undercut depths would be expected to range from about 18 to 24 inches below finished subgrade elevation. The use of high modulus geotextiles (i.e., engineering fabric or geogrid) could also be considered after underground work such as utility construction is completed. Prior to placing the fabric or geogrid, we recommend that all below grade construction, such as utility line installation, be completed to avoid damaging the fabric or geogrid. Equipment should not be operated above the fabric or geogrid until one full lift of crushed stone fill is placed above it. The maximum particle size of granular material placed over geotextile fabric or geogrid should not exceed 1-1/2 inches.
- **Chemical Modification** - Improvement of subgrades with portland cement or class C fly ash could be considered for improving unstable soils. Chemical modification should be performed by a pre-qualified contractor having experience with successfully stabilizing subgrades in the project area on similar sized projects with similar soil conditions. Results of chemical analysis of the additive materials should be provided to the geotechnical engineer prior to use. The hazards of chemicals blowing across the site or onto adjacent property should

also be considered. Additional testing would be needed to develop specific recommendations to improve subgrade stability by blending chemicals with the site soils. Additional testing could include, but not be limited to, determining the most suitable stabilizing agent, the optimum amounts required, the presence of sulfates in the soil, and freeze-thaw durability of the subgrade.

- **"Shot Rock"** – Clean, well graded blasted limestone (commonly called "shot rock") can also be used to stabilize unstable subgrade. The thickness of "shot rock" required to achieve bridging of the unstable subgrade will depend upon the extent of subgrade stability. A test strip should be initially prepared at the site to determine the minimum required shot rock fill thickness to achieve subgrade stability. "Shot rock" particle size should not exceed 12 inches and should be compacted using a heavy-duty vibratory roller or D-6 size bulldozer.

Further evaluation of the need and recommendations for subgrade stabilization can be provided during construction as the geotechnical conditions are exposed.

Fill Material Types

Fill required to achieve design grade should be classified as engineered fill and general fill. Engineered fill is material used below, or within 10 feet of structures, concrete slabs or constructed slopes. General fill is material used to achieve grade outside of these areas.

Fill materials should meet the following material property requirements. Regardless of its source, compacted fill should consist of approved materials that are free of organic matter and debris. Frozen material should not be used, and fill should not be placed on a frozen subgrade.

Excavated on-site soil may be selectively reused as fill below, or within 10 feet of structures, pavement, concrete slabs, and any compacted slopes. Material property requirements for on-site soil and offsite borrow material for use as engineered fill and general fill are noted in the table below:

Soil Type [1]	USCS Classification	Acceptable Parameters (for Engineered Fill)	Acceptable Parameters (for General Fill)
Low Plasticity Cohesive	CL	Liquid Limit less than 50 Plasticity index less than 30 Can be used in all areas except where confined footing undercut prevents compactive efforts	Can be used in all areas

Soil Type [1]	USCS Classification	Acceptable Parameters (for Engineered Fill)	Acceptable Parameters (for General Fill)
High Plasticity Cohesive	CH[2]	Liquid limit greater than 50 but less than 60 Plasticity index less than 35 (Not recommended in building pads and within upper 2 feet of pavement subgrade). Liquid limit greater than 60 is not recommended for reuse .	Can be used in all areas
Granular	GW[3]	Can be used in all areas	Can be used in all areas
Existing Fill	--	Most of the existing fill is expected to be lean and fat clay with rock fragments and is not recommended for reuse as engineered fill.	Can be used in landscaping areas.

1. Engineered and general fill should consist of approved materials free of organic matter and debris. Frozen material should not be used, and fill should not be placed on a frozen subgrade. A sample of each material type should be submitted to the Geotechnical Engineer for evaluation prior to use on this site. Additional geotechnical consultation should be provided prior to use of uniformly graded gravel on the site.
2. CH soils should not be used in building pads
3. Similar to TDOT Section 903.05 Type A, Grading D crushed limestone aggregate, limestone screenings, or granular material such as well graded gravel or crushed stone.

Fill Placement and Compaction Requirements

Engineered and general fill should meet the following compaction requirements.

Item	Engineered Fill	General Fill
Maximum Lift Thickness	8 inches or less in loose thickness when heavy, self-propelled compaction equipment is used 4 to 6 inches in loose thickness when hand-guided equipment (i.e. jumping jack or plate compactor) is used 12 to 18 inches for surge and "shot rock"[4]	Same as engineered fill
Minimum Compaction Requirements [1,2,3]	98% of max. below foundations, floor slabs, and pavements Surge and "shot rock" to be compacted with heavy-duty vibratory smooth drum roller or D-6 class dozer making ten passes (five in one direction and 5 at right angle to initial passes) or until the material is not yielding under the load.	92% of max.
Water Content Range [1]	Low plasticity cohesive: -1% to +3% of optimum High plasticity cohesive: 0 to +3% of optimum Granular: -2% to +2% of optimum	As required to achieve min. compaction requirements

1. Maximum density and optimum water content as determined by the standard Proctor test (ASTM D 698).
2. High plasticity cohesive fill should not be compacted to more than 100% of standard Proctor maximum dry density.
3. If the granular material is a coarse sand or gravel, or of a uniform size, or has a low fines content, compaction comparison to relative density may be more appropriate. In this case, granular materials should be compacted to at least 70% relative density (ASTM D 4253 and D 4254). Materials not amenable to density testing should be placed and compacted to a stable condition observed by the Geotechnical Engineer or representative.

Utility Trench Backfill

Any soft or unsuitable materials encountered at the bottom of utility trench excavations should be removed and replaced with engineered fill or bedding material in accordance with public works specifications for the utility be supported. This recommendation is particularly applicable to utility work requiring grade control and/or in areas where subsequent grade raising could cause settlement in the subgrade supporting the utility. Trench excavation should not be conducted below a downward 1:1 projection from existing foundations without engineering review of shoring requirements and geotechnical observation during construction.

Trench backfill should be mechanically placed and compacted as discussed earlier in this report. Compaction of initial lifts should be accomplished with hand-operated tampers or other lightweight compactors. Where trenches are placed beneath slabs or footings, the backfill should satisfy the gradation and expansion index requirements of engineered fill discussed in this report. Flooding or jetting for placement and compaction of backfill is not recommended.

Grading and Drainage

All grades must provide effective drainage away from the buildings and structures during and after construction and should be maintained throughout the life of the structures. Water retained next to the building can result in soil movements greater than those discussed in this report. Greater movements can result in unacceptable differential floor slab and/or foundation movements, cracked slabs and walls, and roof leaks. The roof should have gutters/drains with downspouts that discharge onto splash blocks at a distance of at least 10 feet from the buildings.

Exposed ground should be sloped and maintained at a minimum 5% away from the building for at least 10 feet beyond the perimeter of the building. Locally, flatter grades may be necessary to transition ADA access requirements for flatwork. After building construction and landscaping have been completed, final grades should be verified to document effective drainage has been achieved. Grades around the structures should also be periodically inspected and adjusted, as necessary, as part of the structure's maintenance program. Where paving or flatwork abuts the structures, a maintenance program should be established to effectively seal and maintain joints and prevent surface water infiltration.

Earthwork Construction Considerations

Upon completion of filling and grading, care should be taken to maintain the subgrade water content prior to construction of soil-supported improvements such as floor slabs and pavements. Construction traffic over the completed subgrades should be avoided. The site should also be graded to prevent ponding of surface water on the prepared subgrades or in excavations. Water collecting over or adjacent to construction areas should be removed. If the subgrade freezes, desiccates, saturates, or is disturbed, the affected material should be removed, or the materials should be scarified, moisture conditioned, and recompacted prior to floor slab construction.

Most shallow excavations for the proposed structures are anticipated to be accomplished with conventional construction equipment. Considering shallow auger refusal depths on the order of 3 to 5 ½ feet in some of our borings and depending upon the planned grading configuration, some deep utility cuts and foundation excavations are anticipated to engage limestone bedrock. Rippability of the bedrock will vary across the site depending on rock quality and depth of excavation. Highly weathered limestone is expected to be rippable with heavy-duty machinery equipped with rock rippers. Relatively intact bedrock, or rock with high RQD values, will require use of rock excavation equipment such as hoe-rams, jack hammers, and rock trenchers or blasting for removal to achieve desired finished grades and/or excavation depths. The client should review with their selected contractor regarding the various means and methods for removal on this site and for specific structures.

As a minimum, excavations should be performed in accordance with OSHA 29 CFR, Part 1926, Subpart P, "Excavations" and its appendices, and in accordance with any applicable local and/or state regulations.

Construction site safety is the sole responsibility of the contractor who controls the means, methods, and sequencing of construction operations. Under no circumstances shall the information provided herein be interpreted to mean Terracon is assuming responsibility for construction site safety or the contractor's activities; such responsibility shall neither be implied nor inferred.

Construction Observation and Testing

The earthwork efforts should be observed by the Geotechnical Engineer (or others under their direction). Observation should include documentation of adequate removal of surficial materials (vegetation, topsoil, and pavements), evaluation and remediation of existing fill materials, as well as proofrolling and mitigation of unsuitable areas delineated by the proofroll.

Each lift of compacted fill should be tested, evaluated, and reworked, as necessary, as recommended by the Geotechnical Engineer prior to placement of additional lifts. Each lift of fill should be tested for density and water content at a frequency of at least one test for every 2,500 square feet of compacted fill in the building areas and 5,000 square feet in pavement areas. Where not specified by local ordinance, one density and water content test should be performed for every 100 linear feet of compacted utility trench backfill and a minimum of one test performed for every 10 vertical inches of compacted backfill.

In areas of foundation excavations, the bearing subgrade should be evaluated by the Geotechnical Engineer. For bedrock supported deep foundations, an airtrack probe hole should be performed at the planned footing locations for the Geotechnical Engineer's evaluation and to confirm continuous and fair quality bedrock is encountered at the bearing depths. The hole should be a minimum of 2-inches in diameter and extend into bedrock a depth equal to at least two times the drilled shaft diameter but not less than 8 feet. If unanticipated conditions are observed, the Geotechnical Engineer should prescribe mitigation options.

In addition to the documentation of the essential parameters necessary for construction, the continuation of the Geotechnical Engineer into the construction phase of the project provides the continuity to maintain the Geotechnical Engineer's evaluation of subsurface conditions, including assessing variations and associated design changes.

Shallow Foundations

If the site has been prepared in accordance with the requirements noted in **Earthwork**, the following design parameters are applicable for shallow foundations.

Design Parameters – Compressive Loads

Item	Description
Maximum Net Allowable Bearing Pressure [1, 2, 8]	Housing Buildings: 2,250 psf - foundations bearing upon stiff to very stiff soils (N-Value ≥8) and/or engineered fill Parking Garage: 4,500 psf (for improved ground via aggregate pier or undercut/replace soils to bedrock and backfill with flowable fill)
Minimum Foundation Dimensions	Per IBC 1809.7
Ultimate Passive Resistance[4] **(equivalent fluid pressures)**	295 pcf (cohesive backfill) 390 pcf (granular backfill)
Sliding Resistance [5]	0.35 for clayey soils 0.45 for granular soils
Minimum Embedment below Finished Grade [6]	Footings in unheated areas: 18 inches
Estimated Total Settlement from Structural Loads [2]	Less than about 1 inch
Estimated Differential Settlement [2, 7]	About ½ of total settlement

1. The maximum net allowable bearing pressure is the pressure in excess of the minimum surrounding overburden pressure at the footing base elevation. Values assume that exterior grades are no steeper than 20% within 10 feet of structure.

Item	Description

2. Values provided are for maximum loads noted in **Project Description**. These settlement values assume column loads not to exceed 200 kips and subgrade remediation is performed as recommended herein. Additional geotechnical consultation will be necessary if higher loads are anticipated and building locations are changed.
3. Existing fill or soft to medium stiff soils should be overexcavated and replaced per the recommendations presented in **Earthwork**.
4. Use of passive earth pressures require the sides of the excavation for the spread footing foundation to be nearly vertical and the concrete placed neat against these vertical faces or that the footing forms be removed and compacted engineered fill be placed against the vertical footing face. Assumes no hydrostatic pressure.
5. Can be used to compute sliding resistance where foundations are placed on suitable soil/materials. Frictional resistance for granular materials is dependent on the bearing pressure which may vary due to load combinations. For fine-grained materials, lateral resistance using cohesion should not exceed ½ the dead load. The settlement tolerance discussed herein should be confirmed by the specialty ground improvement contractor.
6. Embedment necessary to minimize the effects of frost and/or seasonal water content variations. For sloping ground, maintain depth below the lowest adjacent exterior grade within 5 horizontal feet of the structure.
7. Differential settlements are noted for equivalent-loaded foundations and bearing elevation as measured over a span of 50 feet.
8. **The allowable bearing pressure for housing structures assumes that columns loads will not exceed 200 kips and subgrade remediation is performed as discussed herein**. A higher bearing pressure on the order of 4,500 psf may be used if footings are supported on aggregate pier modified ground.

Design Parameters – Overturning and Uplift Loads

Shallow foundations subjected to overturning loads should be proportioned such that the resultant eccentricity is maintained in the center-third of the foundation (e.g., $e < b/6$, where b is the foundation width). This requirement is intended to keep the entire foundation area in compression during the extreme lateral/overturning load event. Foundation oversizing may be required to satisfy this condition.

Uplift resistance of spread footings can be developed from the effective weight of the footing and the overlying soils with consideration to the IBC basic load combinations.

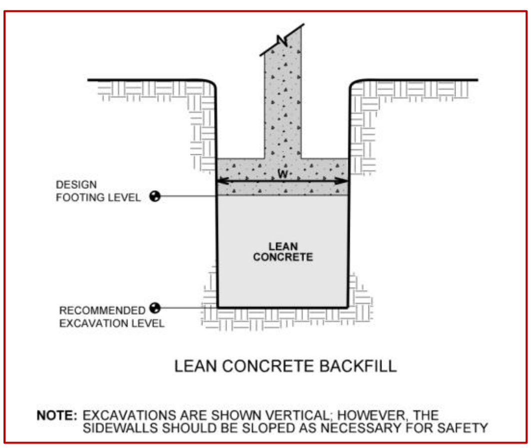
Item	Description
Soil Moist Unit Weight	100 pcf
Soil Effective Unit Weight[1]	40 pcf
Soil weight included in uplift resistance	Soil included within the prism extending up from the top perimeter of the footing at an angle of 20 degrees from vertical to ground surface

1. Effective (or buoyant) unit weight should be used for soil above the foundation level and below a water level. The high groundwater level should be used in uplift design as applicable.

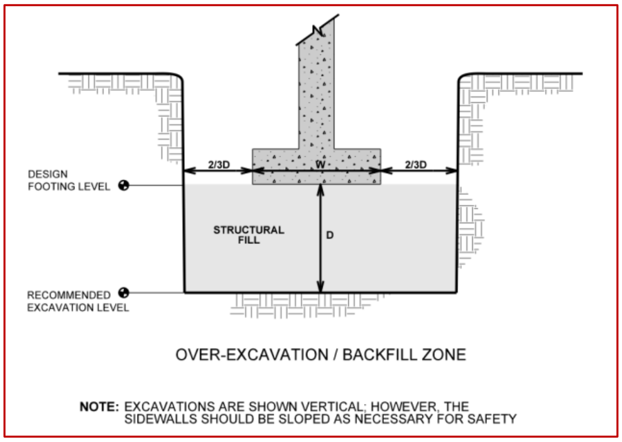
Foundation Construction Considerations

As noted in **Earthwork**, the footing excavations should be evaluated under the observation of the Geotechnical Engineer. The base of all foundation excavations should be free of water and loose soil/rock, prior to placing concrete. Concrete should be placed soon after excavating to reduce bearing soil disturbance. Care should be taken to prevent wetting or drying of the bearing materials during construction. Excessively wet or dry material or any loose/disturbed material in the bottom of the footing excavations should be removed/reconditioned before foundation concrete is placed.

If existing fill and/or low strength bearing soils are observed at the base of the planned footing excavation, the excavation should be extended deeper to suitable stiff natural soils, and the footings could bear directly on these soils at the lower level or on lean concrete backfill placed in the excavations. The lean concrete replacement zone is illustrated on the sketch below.



Overexcavation for engineered fill placement below footings should be conducted as shown below. The overexcavation should be backfilled up to the footing base elevation, with approved engineered fill placed, as recommended in the **Earthwork** section.



Ground Improvement

The parking structure foundations could be supported on onsite soils if ground improvement methods are utilized. Ground improvement methods are proprietary systems designed by licensed contractors who could provide further information regarding support options. A ground improvement alternative that may offer a more economical foundation to deep foundation support includes the installation of aggregate piers.

An aggregate pier consists of a stone-filled column constructed by excavating a cylindrical hole and backfilling it with crushed stone placed in lifts and applying a high degree of compaction effort resulting in stone filled piers. The aggregate pier construction process not only results in a rigid stone-filled column that lends support to the structure, but it also helps to densify the soils surrounding the pier. Aggregate pier improvements are a proprietary product and, should be designed and installed by a specialty contractor. Due to the specialty of this soil improvement procedure, we recommend that a performance specification be used for this system.

Footings supported on reinforced ground via aggregate piers can be designed for a maximum net allowable bearing pressure of 4,500 psf.

If ground improvement via aggregate piers is performed, a well graded crushed rock backfill material should be used to reinforce the ground. Considerations may be given to use cemented treated crushed rock fill plug to minimize penetration of surface water into the ground due to karst terrain.

We understand if aggregate pier improvements or other methods are utilized, the aggregate pier or other method design firm will be the geotechnical engineer of record for these foundations. As such, the design firm would provide the necessary design parameters for the planned foundation system including, but not limited to, allowable bearing capacity, settlement estimates and foundation-specific earthwork recommendations.

Deep Foundations-Parking Garage

As an alternate to ground improvement via aggregate piers, if desired, the proposed garage can be supported on deep foundation system (drilled shafts or micropiles) supported on relatively intact good quality bedrock below existing voids and weathered rock and soil seams. The following sections provide design parameters for rock bearing deep foundation system

Drilled Shaft Design Parameters

Soil design parameters are provided below in the **Drilled Shaft Design Summary** table for the design of drilled shaft foundations. The values presented for allowable side friction and end bearing include a factor of safety.

Approximate Depth (feet) [1]	Allowable Skin Friction (psf)	Allowable End Bearing Pressure (psf)	Allowable Passive Pressure (psf)	Cohesion (psf)	Internal Angle of Friction (Degrees)	Strain ε_{50}	Lateral Subgrade Modulus (pci)
0 – 3	Ignore	Ignore	Ignore	Ignore	Ignore	Ignore	Ignore
Fill	250	Ignore	500	500	--	0.02	40
Native Clay	400	Ignore	1,250	1,250	--	0.008	100
Intact Limestone Bedrock	2,500 [2]	50,000	5,000 [2]	50,000 [2]	--	0.00001	3,000

1. Based on existing grades, does not take into consideration proposed cut and fill. Terracon should observe the shaft installation to assist with adjustment of the shaft length if variable soil and rock conditions are encountered. A total unit weight of 110 pcf, 120 pcf and 150 pcf can be assumed for the fill, natural clay and limestone bedrock, respectively.
2. The parameters have been reduced to take into account the possibility of shallow overburden. The shafts may require embedment by the designer into limestone bedrock to mobilize these rock strength parameters. Furthermore, it is assumed the rock socket will be extended using coring techniques rather than blasting/shooting.
3. These values assume that drilled shafts or piles are extended into intact limestone bedrock (Min. REC/RQD = 90%/50% respectively) below any voids and clayey seams and rock condition should be field verified during construction.

The above indicated cohesion, lateral subgrade modulus and strain values have no factors of safety, and the allowable skin friction and the passive resistances have factors of safety of 2. The cohesion, lateral subgrade modulus, and strain values given in the above table are based on the results of borings, published values and our past experience with similar soil types. These values should, therefore, be considered approximate. The allowable end bearing pressure provided in the table has an approximate factor of safety of at least 3.

The upper 3 feet of overburden should be ignored due to the potential effects of frost action and construction disturbance. To avoid a reduction in lateral and uplift resistance caused by variable subsurface conditions, we recommend that drawings instruct the contractor to notify the engineer if subsurface conditions significantly different than encountered in our borings are disclosed during drilled shaft installations. Under these circumstances, it may be necessary to adjust the length of the shafts. To facilitate shaft length adjustments that may be necessary because of variable soil and rock conditions, we recommend that a Terracon representative observe the drilled shaft excavations.

A drilled shaft foundation should be designed with a minimum shaft diameter of 30 inches to facilitate clean out and possible dewatering of the shaft excavation. Temporary casing may be required during the shaft excavation in order to control possible groundwater seepage and support the sides of the excavation in weak soil zones. Care should be taken so that the sides and bottom of the excavations are not disturbed during construction. The bottom of the shaft should be free of loose soil or debris prior to reinforcing steel and concrete placement.

A concrete slump of at least 6 inches is recommended to facilitate temporary casing removal. Temporary casing will be required in areas of existing fill, soil overburden, and where poor quality weathered rock is encountered. It should be possible to remove the casing from a shaft excavation during concrete placement provided that the concrete inside the casing is maintained at a sufficient level to resist any earth and hydrostatic pressures outside the casing during the entire casing removal procedure. Tensile reinforcement should extend to the bottom of shafts subjected to uplift loading.

Drilled shafts should have a minimum (center-to-center) spacing of three diameters. Closer spacing may require a reduction in axial load capacity. Axial capacity reduction can be determined by comparing the allowable axial capacity determined from the sum of individual shafts in a group versus the capacity calculated using the perimeter and base of the shaft group acting as a unit. The lesser of the two capacities should be used in design.

The drilled shaft installation process should be performed under the direction of the Geotechnical Engineer. The Geotechnical Engineer should document the shaft installation process including soil/rock and any groundwater conditions encountered, consistency with expected conditions, and details of the installed shaft. If shaft locations are not pre-drilled to determine the target bearing elevation, the bottom of the shaft should have a probe hole drilled a minimum depth of twice the diameter of the shaft but not less than 8 feet. The geotechnical engineer will evaluate the hole for voids or weathered rock and clayey seams that could negatively impact the shaft's performance. The drilled shaft contractor should provide safe entry and air monitoring for the geotechnical engineer.

Concrete for "dry" drilled shaft construction should have a slump of about 5 to 7 inches. Concrete should be directed into the shaft utilizing a centering chute. Concrete for "wet" shaft construction would require higher slump concrete.

Drilled Shaft Construction Considerations

To prevent collapse of the sidewalls and/or to control possible groundwater seepage, the use of temporary steel casing and/or slurry drilling procedures may be required for construction of the drilled shaft foundations. Significant seepage could occur in case of excavations penetrating water-bearing sandy soil and/or highly broken bedrock layers. The drilled shaft contractor and foundation design engineer should be informed of these risks.

Use of a telescoping casing arrangement can be considered to avoid handling long casing lengths. The lower casing should be of sufficient length and stiffness and have an appropriate cutting edge to allow it to be firmly seated into the bedrock to seal out groundwater. If possible, excess water should be evacuated from the casing to place concrete in the "dry."

Care should be taken to not disturb the sides and bottom of the excavation during construction. The bottom of the shaft excavation should be free of loose material before concrete placement. Concrete should be placed as soon as possible after the foundation excavation is completed, to reduce potential disturbance of the bearing surface.

While withdrawing casing, care should be exercised to maintain concrete inside the casing at a sufficient level to resist earth and hydrostatic pressures acting on the casing exterior. Arching of the concrete, loss of seal and other problems can occur during casing removal and result in contamination of the drilled shaft. These conditions should be considered during the design and construction phases. Placement of loose soil backfill should not be permitted around the casing prior to removal.

The drilled shaft installation process should be performed under the observation of the Geotechnical Engineer. The Geotechnical Engineer should document the shaft installation process including soil/rock and groundwater conditions observed, consistency with expected conditions, and details of the installed shaft. For each drilled shaft foundation, a probe hole for scratch testing of the bedrock should be installed by the contractor at the bottom of the shaft for the Geotechnical Engineer's use. The hole should be a minimum of 2-inches in diameter and extend a depth equal to at least two times the foundation width and not less than 6-feet. The contractor should provide safe entry for the inspection, including a competent spotter and monitoring or control of air within the shaft.

Specialty Foundations

As an alternative to drilled shafts, it is our opinion that micropile foundations would provide a viable alternative for foundation support for the parking garage due to the magnitude of the column loads anticipated and site conditions including variable depth to rock. Micropile foundations generally consist of permanent steel casing that is advanced into the underlying bedrock and grouted in place. Axial capacity is developed both in end bearing and in skin friction along a grout bond zone within a rock socket beneath the tip of the steel casing.

For micropile foundations terminating in continuous limestone, a typical grout-to-rock bond strength of 150 pounds per square inch (psi) may be assumed. An allowable tip rock bearing pressure of 50,000 psf may be used for micropile design when supported within continuous intact bedrock (Min. REC/RGD = 90/50 percent) below any voids, clayey seams and highly weathered rock layers.

The micropile systems can be designed using either of the following approaches:

- Prescriptive Specifications – The owner provides the design and specific procedures that must be followed. In this case, the owner, through the design team, is responsible for the proper performance of the system. The contractor is responsible for satisfying the details of the specifications.

- Performance Specification – The contractor is permitted control over certain design and/or construction procedures but must demonstrate to the owner through testing and/or certification that the final product meets the specified performance criteria. This allows for innovative design based on contractor experience. The responsibility for the work is shared between the owner and the contractor. Micropile design-build contractors can often design and install Micropiles having a significantly higher capacity based on their experience, research, testing, and unique installation methods.

Additional information concerning Micropiles can be obtained from the FHWA Micropile design guide.

Floor Slabs

Design parameters for floor slabs assume the requirements for **Earthwork** have been followed. Specific attention should be given to positive drainage away from the structure and positive drainage of the aggregate base beneath the floor slab.

Low strength soils (N-value ≤ 6) may be encountered at the floor slab subgrade level within portions of the building pads. These soils should be replaced with engineered fill or recommended as discussed herein so the floor slab is supported on compacted suitable engineered fill or stable natural soils.

Existing fill materials and materials described as possible fill were observed at the site to depths of 3 to 5 ½ feet below existing grade. As previously described, any existing fill present beneath floor slabs should be completely removed and further evaluated by the Geotechnical Engineer to finalize extent of remediation.

Some of the subgrade soils are comprised of high plasticity clays exhibiting the potential to swell with increased water content. Construction of the floor slab, combined with the removal of trees, and revising site drainage creates the potential for gradual increased water contents within the clays. Increases in water content will cause the clays to swell and damage the floor slab. To reduce the swell potential to less than about 1 inch, at least the upper 12 inches of subgrade soils below the floor slab finished subgrade elevation (excluding the floor slab support course) should be an approved Low Volume Change (LVC) material consisting of granular fill or lean clay.

Floor Slab Design Parameters

Item	Description
Floor Slab Support[1]	Use minimum 4 inches base course meeting material specifications of ACI 302and compacted to at least 98% of ASTM D698 Subgrade compacted to recommendations in **Earthwork**
Estimated Modulus of Subgrade Reaction [2]	100 pounds per square inch per inch (psi/in) for point loads

1. Floor slabs should be structurally independent of building footings or walls to reduce the possibility of floor slab cracking caused by differential movements between the slab and foundation.
2. Modulus of subgrade reaction is an estimated value based upon our experience with the subgrade condition, the requirements noted in **Earthwork**, and the

Item	Description
	floor slab support as noted in this table. It is provided for point loads. For large area loads the modulus of subgrade reaction would be lower.

The use of a vapor retarder should be considered beneath concrete slabs on grade covered with wood, tile, carpet, or other moisture sensitive or impervious coverings, when the project includes humidity-controlled areas, or when the slab will support equipment sensitive to moisture. When conditions warrant the use of a vapor retarder, the slab designer should refer to ACI 302 and/or ACI 360 for procedures and cautions regarding the use and placement of a vapor retarder.

Saw-cut contraction joints should be placed in the slab to help control the location and extent of cracking. For additional recommendations, refer to the ACI Design Manual. Joints or cracks should be sealed with a waterproof, non-extruding compressible compound specifically recommended for heavy duty concrete pavement and wet environments.

Where floor slabs are tied to perimeter walls or turn-down slabs to meet structural or other construction objectives, our experience indicates differential movement between the walls and slabs will likely be observed in adjacent slab expansion joints or floor slab cracks beyond the length of the structural dowels. The Structural Engineer should account for potential differential settlement through use of sufficient control joints, appropriate reinforcing or other means.

Settlement of floor slabs supported on existing fill materials cannot be accurately predicted but could be larger than normal and result in some cracking. Mitigation measures, as noted in **Earthwork**, are critical to the performance of floor slabs. In addition to the mitigation measures, the floor slab can be stiffened by adding steel reinforcement, grade beams, and/or post-tensioned elements.

Floor Slab Construction Considerations

Finished subgrade, within and for at least 10 feet beyond the floor slab, should be protected from traffic, rutting, or other disturbance and maintained in a relatively moist condition until floor slabs are constructed. If the subgrade should become damaged or desiccated prior to construction of floor slabs, the affected material should be removed, and engineered fill should be added to replace the resulting excavation. Final conditioning of the finished subgrade should be performed immediately prior to placement of the floor slab support course.

Prior to construction of grade supported slabs, varying levels of remediation may be required to reestablish stable subgrades within slab areas due to construction traffic, rainfall, disturbance, desiccation, etc. As a minimum, the following measures are recommended.

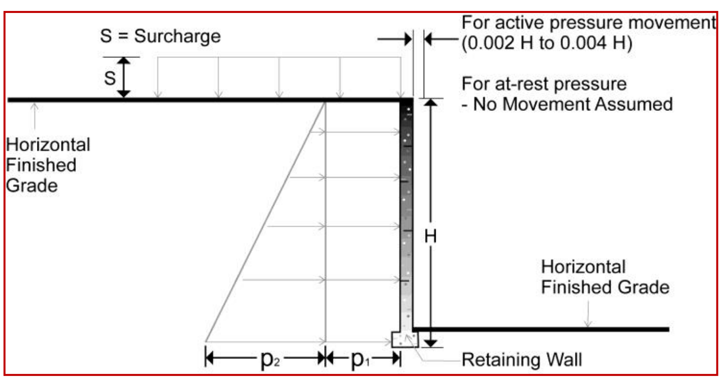
- Confirm that interior trench backfill placed beneath slabs is compacted in accordance with recommendations outlined in this report.
- All floor slab subgrade areas should be moisture-conditioned and properly compacted to the recommendations in this report immediately prior to placement of the stone base and concrete.

The Geotechnical Engineer should observe the condition of the floor slab subgrades immediately prior to placement of the floor slab support course, reinforcing steel, and concrete. Attention should be paid to high traffic areas that were rutted and disturbed earlier, and to areas where backfilled trenches are located.

Lateral Earth Pressures

Design Parameters

Structures with unbalanced backfill levels on opposite sides should be designed for earth pressures at least equal to values indicated in the following table. Earth pressures will be influenced by structural design of the walls, conditions of wall restraint, methods of construction, and/or compaction and the strength of the materials being restrained. Two wall restraint conditions are shown in the diagram below. Active earth pressure is commonly used for design of free-standing cantilever retaining walls and assumes wall movement. The "at-rest" condition assumes no wall movement and is commonly used for basement walls, loading dock walls, or other walls restrained at the top. The recommended design lateral earth pressures do not include a factor of safety and do not provide for possible hydrostatic pressure on the walls (unless stated).



Lateral Earth Pressure Design Parameters

Earth Pressure Condition [1]	Coefficient for Backfill Type [2]	Surcharge Pressure [3] p_1 (psf)	Equivalent Fluid Pressures (psf) [2,4]	
			Unsaturated [5]	Submerged [5]
Active (Ka)	Granular - 0.31 Fine Grained - 0.41	(0.31)S (0.41)S	(33)H (48)H	(80)H (85)H
At-Rest (Ko)	Granular - 0.47 Fine Grained - 0.58	(0.47)S (0.58)S	(50)H (70)H	(82)H (95)H
Passive (Kp)	Granular – 3.25 Granular – 2.46	--- ---	(390)H (295)H	(200)H (205)H

1. For active earth pressure, wall must rotate about base, with top lateral movements 0.002 H to 0.004 H, where H is wall height. For passive earth pressure, wall must move horizontally to mobilize resistance. Fat clay or other expansive soils should not be used as backfill behind the wall.
2. Uniform, horizontal backfill, with a maximum unit weight of 120 pcf for soils.
3. Uniform surcharge, where S is surcharge pressure.
4. Loading from heavy compaction equipment is not included.
5. To achieve "Unsaturated" conditions, follow guidelines in **Subsurface Drainage for Below-Grade Walls** below. "Submerged" conditions are recommended when drainage behind walls is not incorporated into the design.

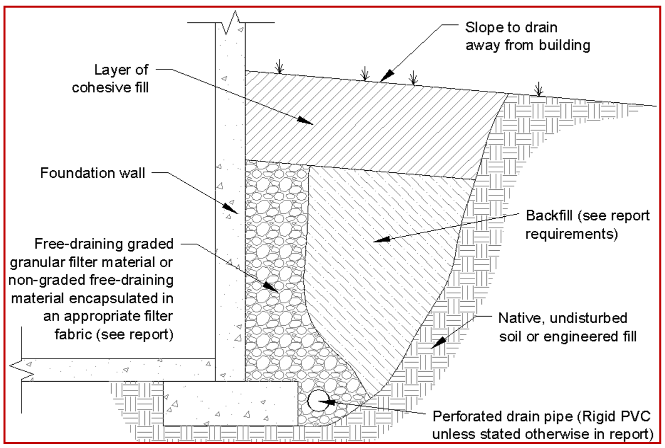
Backfill placed against structures should consist of granular soils or low plasticity cohesive soils. For the granular values to be valid, the granular backfill must extend out and up from the base of the wall at an angle of at least 45 degrees from vertical for the active case.

Footings, floor slabs or other loads bearing on backfill behind walls may have a significant influence on the lateral earth pressure. Placing footings within wall backfill and in the zone of active soil influence on the wall should be avoided unless structural analyses indicate the wall can safely withstand the increased pressure.

The lateral earth pressure recommendations given in this section are applicable to the design of rigid retaining walls subject to slight rotation, such as cantilever, or gravity type concrete walls. These recommendations are not applicable to the design of modular block - geogrid reinforced backfill walls (also termed MSE walls). Recommendations covering these types of wall systems are beyond the scope of services for this assignment. However, we would be pleased to develop a proposal for evaluation and design of such wall systems upon request.

Subsurface Drainage for Below-Grade Walls

A perforated rigid plastic drain line installed behind the base of walls and extends below adjacent grade is recommended to prevent hydrostatic loading on the walls. The invert of a drain line around a below-grade building area or exterior retaining wall should be placed near foundation bearing level. The drain line should be sloped to provide positive gravity drainage to daylight or to a sump pit and pump. The drain line should be surrounded by clean, free-draining granular material having less than 5% passing the No. 200 sieve, such as No. 57 aggregate. The free-draining aggregate should be encapsulated in a filter fabric. The granular fill should extend to within 2 feet of final grade, where it should be capped with compacted cohesive fill to reduce infiltration of surface water into the drain system.



As an alternative to free-draining granular fill, a prefabricated drainage structure may be used. A prefabricated drainage structure is a plastic drainage core or mesh which is covered with filter fabric to prevent soil intrusion and is fastened to the wall prior to placing backfill.

Pavements

General Pavement Comments

Pavement designs are provided for the traffic conditions and pavement life conditions as noted in **Project Description** and in the following sections of this report. A critical aspect of pavement performance is site preparation. Pavement designs noted in this section must be applied to the site which has been prepared as recommended in the **Earthwork** section. The section thicknesses and traffic conditions in this report do not account for construction traffic, incomplete placement of the full pavement section, or loads beyond what was assumed or provided. If the contractor or owners are aware or require additional sections or traffic count considerations, Terracon should be provided that information for our review.

We recommend the moisture content and density of the top 12 inches of the subgrade be evaluated and the pavement subgrades be proof rolled within two days prior to commencement of actual paving operations. Areas not in compliance with the required ranges of moisture or density should be moisture conditioned and recompacted. Particular attention should be paid to high traffic areas that were rutted and disturbed earlier and to areas where backfilled trenches are located. Areas where unsuitable conditions are located should be repaired by removing and replacing the materials with properly compacted fills. If a significant precipitation event occurs after the evaluation or if the surface becomes disturbed, the subgrade should be reviewed by qualified personnel immediately prior to paving. The subgrade should be in its finished form at the time of the final review.

As noted in **Geotechnical Characterization,** surficial undocumented fill was encountered in a few borings near the surface. The existing fill in pavement areas should be undercut, as needed, to construct a minimum 1½-foot "buffer" layer of new engineered fill beneath finished subgrade. Any remaining fill beneath this buffer should be thoroughly evaluated and recompacted to a non-yielding state or properly bridged as recommended by the Geotechnical Engineer.

Support characteristics of subgrade for pavement design do not account for shrink/swell movements of high plasticity clay subgrade, such as soils observed on this project. Thus, the pavement may be adequate from a structural standpoint, yet still experience cracking and deformation due to shrink/swell related movement of the subgrade.

Pavement Design Parameters

A California Bearing Ratio (CBR) of 4 was used for the subgrade for the asphaltic concrete (AC) pavement designs. A modulus of subgrade reaction of 120 pci was used for the Portland cement concrete (PCC) pavement designs. The value was empirically derived based upon our experience with the lean clay subgrade soils and our expectation of the quality of the subgrade as prescribed by the **Site Preparation** conditions as outlined in **Earthwork**. A modulus of rupture of 580 psi was used in design for the concrete (based on correlations with a minimum 28-day compressive strength of 4,000 psi).

Pavement Section Thicknesses

The following table provides our estimated minimum thickness of PCC pavements.

Layer	Asphaltic Concrete Design Thickness (inches)	
	Light Duty [1]	Heavy Duty [1]
AC Surface [2]	1 ½	1 ½
AC Binder [2]	2	2 ½
Aggregate Base [2]	6	8

1. See **Project Description** and design parameters discussed in the previous section for more specifics regarding Light Duty and Heavy-Duty traffic.
2. All materials should meet the current State of TN department of Transportation (TDOT) Standard Specifications for Highway and Bridge Construction.

- Asphaltic Surface – TDOT Section 903.11 for Surface Course, Grading E
- Asphaltic Base – TDOT Section 903.06 for Hot Mix Asphalt Leveling Course, Grading B-M
- Section 903.05 for Aggregate Base Course material, Class A, Grading D
- A minimum 1.5-inch surface course should be used on ACC pavements

The following table provides our estimated minimum thickness of PCC pavements.

Layer	Portland Cement Concrete Design Thickness (inches)		
	Light Duty [1]	Heavy Duty [1]	Dumpster Approach/Apron [3]
PCC [2,4]	5	6	7
Aggregate Base [2]	4	4	4

1. See **Project Description** and design parameters discussed in the previous section for more specifics regarding traffic classifications.
2. All materials should meet the current Tennessee Department of Transportation (TDOT) Standard Specifications for Highway and Bridge Construction.
3. In areas of anticipated heavy traffic, fire trucks, delivery trucks, or concentrated loads (e.g., dumpster approach/apron), and areas with repeated turning or maneuvering of heavy vehicles. Additional steel reinforcement within aprons is not common but the use of dowels at the connection of aprons to dumpster pads may help alleviate potential cracking from concentrated wheel loads.

Layer	Portland Cement Concrete Design Thickness (inches)		
	Light Duty [1]	Heavy Duty [1]	Dumpster Approach/Apron [3]

4. Portland cement concrete should be 4,000 psi compressive strength at 28 days. PCC pavements are recommended for trash container pads and in any other areas subjected to heavy wheel loads and/or turning traffic such as entrance aprons.

Areas for parking of heavy vehicles, concentrated turn areas, and start/stop maneuvers could require thicker pavement sections. Edge restraints (i.e. concrete curbs or aggregate shoulders) should be planned along curves and areas of maneuvering vehicles.

A minimum 4-inch thick base course layer is recommended to help reduce potential for slab curl, shrinkage cracking, and subgrade pumping through joints. Proper joint spacing will also be required to prevent excessive slab curling and shrinkage cracking. Joints should be sealed to prevent entry of foreign material and doweled where necessary for load transfer. PCC pavement details for joint spacing, joint reinforcement, and joint sealing should be prepared in accordance with ACI 330 and ACI 325.

Where practical, we recommend early-entry cutting of crack-control joints in PCC pavements. Cutting of the concrete in its “green” state typically reduces the potential for micro-cracking of the pavements prior to the crack control joints being formed, compared to cutting the joints after the concrete has fully set. Micro-cracking of pavements may lead to crack formation in locations other than the sawed joints, and/or reduction of fatigue life of the pavement.

Openings in pavements, such as decorative landscaped areas, are sources for water infiltration into surrounding pavement systems. Water can collect in the islands and migrate into the surrounding subgrade soils thereby degrading support of the pavement. Islands with raised concrete curbs, irrigated foliage, and low permeability near-surface soils are particular areas of concern. The civil design for the pavements with these conditions should include features to restrict or collect and discharge excess water from the islands. Examples of features are edge drains connected to the stormwater collection system, longitudinal subdrains, or other suitable outlets and impermeable barriers preventing lateral migration of water such as a cutoff wall installed to a depth below the pavement structure.

Pavement Drainage

Pavements should be sloped to provide rapid drainage of surface water. Water allowed to pond on or adjacent to the pavements could saturate the subgrade and contribute to premature pavement deterioration. In addition, the pavement subgrade should be graded to provide positive drainage within the granular base section. Appropriate sub-drainage or connection to a suitable daylight outlet should be provided to remove water from the granular subbase.

Pavement Maintenance

The pavement sections represent minimum recommended thicknesses and, as such, periodic upkeep should be anticipated. Preventive maintenance should be planned and provided for through an on-going pavement management program. Maintenance activities are intended to slow the rate of pavement deterioration and to preserve the pavement investment. Pavement care consists of both localized (e.g., crack and joint sealing and patching) and global maintenance (e.g., surface sealing). Additional engineering consultation is recommended to determine the type and extent of a cost-effective program. Even with periodic maintenance, some movements and related cracking may still occur, and repairs may be required.

Pavement performance is affected by its surroundings. In addition to providing preventive maintenance, the civil engineer should consider the following recommendations in the design and layout of pavements:

- Final grade adjacent to paved areas should slope down from the edges at a minimum 2%.
- Subgrade and pavement surfaces should have a minimum 2% slope to promote proper surface drainage.
- Install pavement drainage systems surrounding areas anticipated for frequent wetting.
- Install joint sealant and seal cracks immediately.
- Seal all landscaped areas in or adjacent to pavements to reduce moisture migration to subgrade soils.
- Place compacted, low permeability backfill against the exterior side of curb and gutter.
- Place curb, gutter and/or sidewalk directly on clay subgrade soils rather than on unbound granular base course materials.

General Comments

Our analysis and opinions are based upon our understanding of the project, the geotechnical conditions in the area, and the data obtained from our site exploration. Variations will occur between exploration point locations or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. Terracon should be retained as the Geotechnical Engineer, where noted in this report, to provide observation and testing services during pertinent construction phases. If variations appear, we can provide further evaluation and supplemental recommendations. If variations are noted in the absence of our observation and testing services on-site, we should be immediately notified so that we can provide evaluation and supplemental recommendations.

Our Scope of Services does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

Our services and any correspondence are intended for the sole benefit and exclusive use of our client for specific application to the project discussed and are accomplished in accordance with generally accepted geotechnical engineering practices with no third-party beneficiaries intended. Any third-party access to services or correspondence is solely for information purposes to support the services provided by Terracon to our client. Reliance upon the services and any work product is limited to our client and is not intended for third parties. Any use or reliance of the provided information by third parties is done solely at their own risk. No warranties, either express or implied, are intended or made.

Site characteristics as provided are for design purposes and not to estimate excavation cost. Any use of our report in that regard is done at the sole risk of the excavating cost estimator as there may be variations on the site that are not apparent in the data that could significantly affect excavation cost. Any parties charged with estimating excavation costs should seek their own site characterization for specific purposes to obtain the specific level of detail necessary for costing. Site safety and cost estimating including excavation support and dewatering requirements/design are the responsibility of others. Construction and site development have the potential to affect adjacent properties. Such impacts can include damages due to vibration, modification of groundwater/surface water flow during construction, foundation movement due to undermining or subsidence from excavation, as well as noise or air quality concerns. Evaluation of these items on nearby properties are commonly associated with contractor means and methods and are not addressed in this report. The owner and contractor should consider a preconstruction/precondition survey of surrounding development. If changes in the nature, design, or location of the project are planned, our conclusions and recommendations shall not be considered valid unless we review the changes and either verify or modify our conclusions in writing.

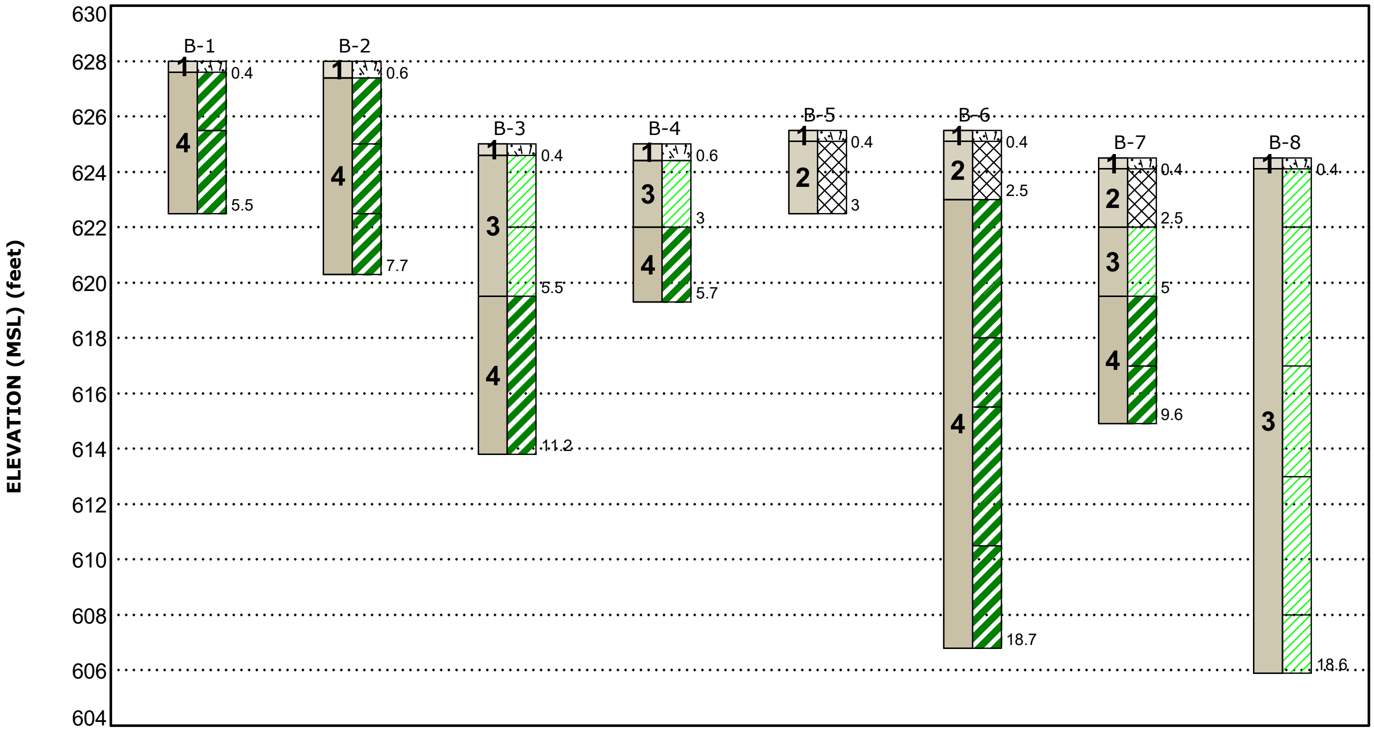
Figures

Contents:

GeoModel (3 pages)

1922 Old Murfreesboro Pike
Ste 905
Nashville, TN

GeoModel



This is not a cross section. This is intended to display the Geotechnical Model only. See individual logs for more detailed conditions.

Model Layer	Layer Name	General Description
1	Surficial Cover	Approximately 3 to 7 inches of topsoil
2	Fill/Possible Fill	Lean clay with some limestone rock fragments, trace of roots and mineral nodules
3	Lean Clay	Low plasticity clay, medium stiff to very stiff
4	Fat Clay	Moderately high plasticity clay, medium stiff to very stiff
5	Limestone Bedrock	Moderately to slightly weathered, highly to slightly fractured, thin to medium bedded (RQD = 68 to 100% and REC = 90 to 100%)

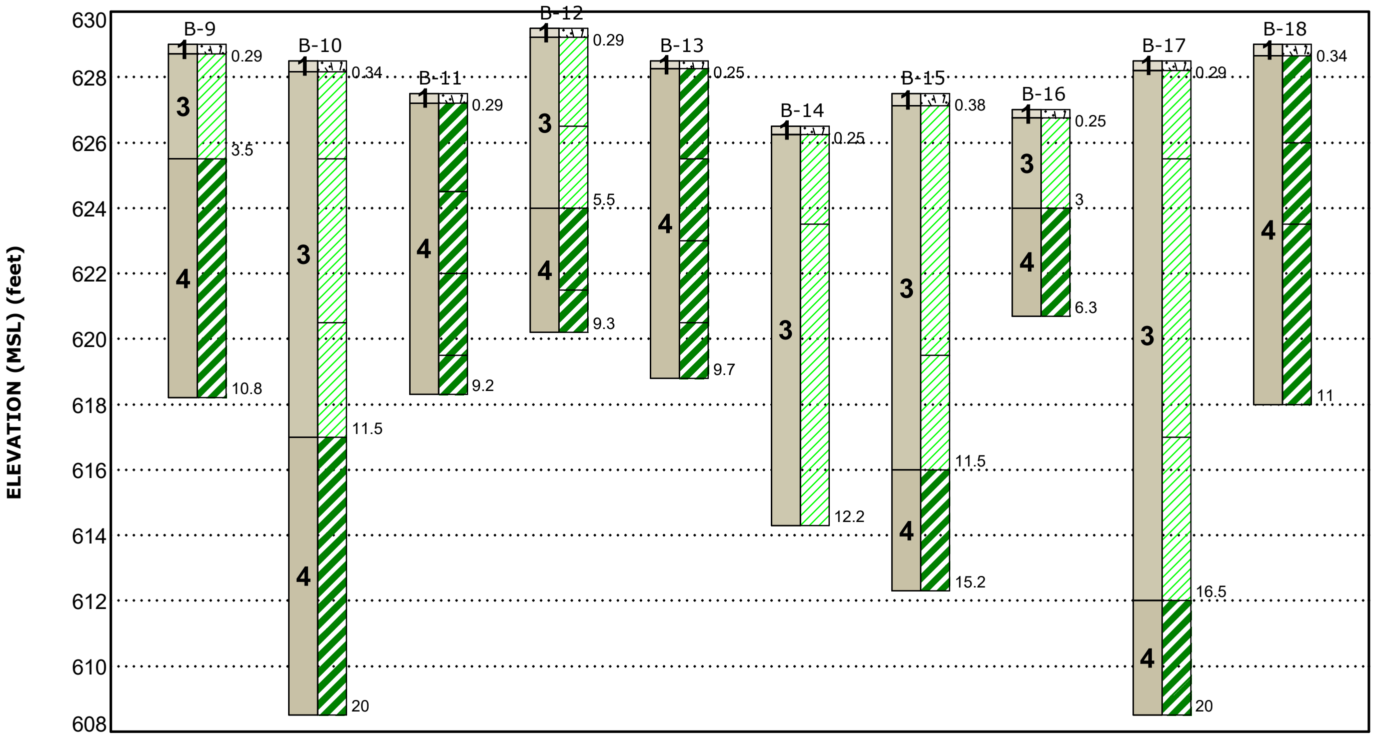
Legend

Topsoil
Fat Clay
Lean Clay
Fill

NOTES:
Layering shown on this figure has been developed by the geotechnical engineer for purposes of modeling the subsurface conditions as required for the subsequent geotechnical engineering for this project.
Numbers adjacent to soil column indicate depth below ground surface.

Terracon
1922 Old Murfreesboro Pike
Ste 905
Nashville, TN

GeoModel



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Legend

Topsoil
Lean Clay
Fat Clay

NOTES:
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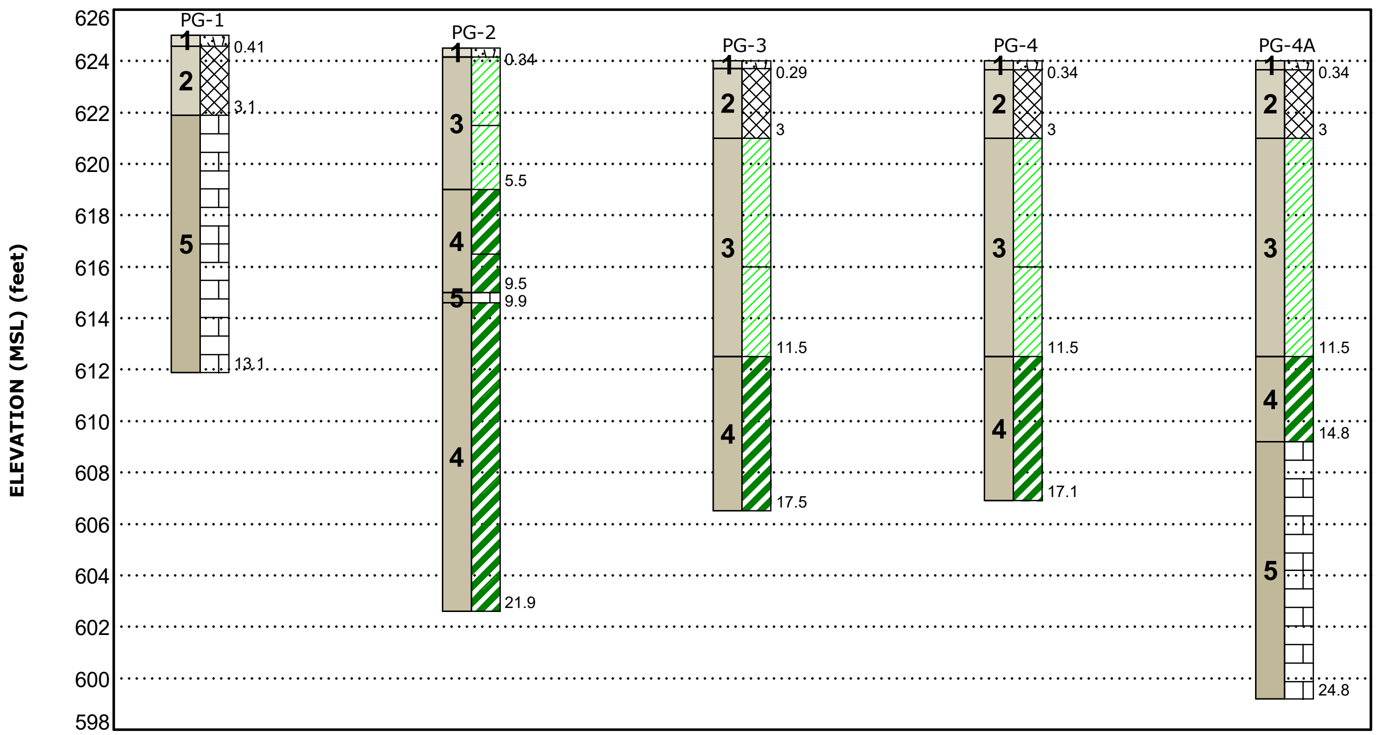
1922 Old Murfreesboro Pike
Ste 905
Nashville, TN

GeoModel



This is not a cross section. This is intended to display the Geotechnical Model only. See individual logs for more detailed conditions.

Model Layer	Layer Name	General Description
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Legend

Topsoil
Fill
Limestone
Lean Clay
Fat Clay

NOTES:
Layering shown on this figure has been developed by the geotechnical engineer for purposes of modeling the subsurface conditions as required for the subsequent geotechnical engineering for this project.
Numbers adjacent to soil column indicate depth below ground surface.

Attachments

Exploration and Testing Procedures

Field Exploration

Number of Exploration Points	Approximate Exploration Depth (feet)	Location
16	3 to 20	Proposed Housing Structures
7	13 ¼ to 22	Proposed Parking Garage

Boring Layout and Elevations: Terracon personnel provided the boring layout using handheld GPS equipment (estimated horizontal accuracy of about ±10 feet) and referencing existing site features. Approximate ground surface elevations were obtained by interpolation from a topographic survey titled *Middle Tennessee State University-Womack Lane* by *Civil Infrastructure Associates* dated *September 27,2024*. If elevations and a more precise boring layout are desired, we recommend borings be surveyed.

Subsurface Exploration Procedures: We advanced the borings with a track-mounted rotary drill rig using continuous flight augers (solid stem and/or hollow stem, as necessary, depending on soil conditions). Four samples were obtained in the upper 10 feet of each boring and at intervals of 5 feet thereafter. In the split-barrel sampling procedure, a standard 2-inch outer diameter split-barrel sampling spoon was driven into the ground by a 140-pound automatic hammer falling a distance of 30 inches. The number of blows required to advance the sampling spoon the last 12 inches of a normal 18-inch penetration is recorded as the Standard Penetration Test (SPT) resistance value. The SPT resistance values, also referred to as N-values, are indicated on the boring logs at the test depths. For safety purposes, all borings were backfilled with auger cuttings after their completion.

We also observed the boreholes while drilling and at the completion of drilling for the presence of groundwater. Groundwater was not observed at these times in the boreholes.

The sampling depths, penetration distances, and other sampling information was recorded on the field boring logs. The samples were placed in appropriate containers and taken to our soil laboratory for testing and classification by a Geotechnical Engineer. Our exploration team prepared field boring logs as part of the drilling operations. These field logs included visual classifications of the materials observed during drilling and our interpretation of the subsurface conditions between samples. Final boring logs were prepared from the field logs. The final boring logs represent the Geotechnical Engineer's interpretation of the field logs and include modifications based on observations and tests of the samples in our laboratory.

The “percent recovery” is the ratio of the sample length retrieved to the drilled length, expressed as a percent. An indication of the actual in-situ rock quality is provided by calculating the sample’s Rock Quality Designation (RQD). The RQD is the ratio of the cumulative length of 4 inch or longer core sections (discounting mechanical breaks) to the length of the core run. The percent recovery and RQD are related to rock soundness and quality as illustrated below:

Relation of RQD and In-situ Rock Quality	
Percentage	**Rock Quality**
90 - 100	Excellent
75 - 90	Good
50 - 75	Fair
25 - 50	Poor
0 -25	Very Poor

Laboratory Testing

The project engineer reviewed the field data and assigned laboratory tests. The laboratory testing program included the following types of tests:

- Moisture Content
- Atterberg Limits
- Unconfined compressive strength of rock

The laboratory testing program often included examination of soil samples by an engineer. Based on the results of our field and laboratory programs, we described and classified the soil samples in accordance with the Unified Soil Classification System.

Rock classification was conducted using locally accepted practices for engineering purposes; petrographic analysis may reveal other rock types. Rock core samples typically provide an improved specimen for this classification. Boring log rock classification was determined using the Rock Classification Notes.

Photography Log

Photo 1 B-3 Facing North

Photo 2 B-4 Facing West

Photo 3 B-8 Facing Northeast

Photo 4 B-7 Facing Southwest

Photo 5 B-6 Facing North

Photo 6 B-5 Facing North

Photo 7 B-1 Facing North

Photo 8 B-2 Facing Northwest

Photo 9 Site View Facing Northeast near B-5

Photo 10 Site View Facing Northeast near B-3

Photo 1 B-11 facing North

Photo 2 B-11 facing South

Photo 3 B-12 facing North

Photo 4 B-12 facing South

Photo 5 B-13 facing North

Photo 6 B-13 facing South


Photo 7 B-14 facing North


Photo 8 B-14 facing South


Photo 9 B-15 facing North


Photo 10 B-15 facing South


Photo 11 B-16 facing North


Photo 12 B-16 facing South

Photo 13 B-17 facing North

Photo 14 B-17 facing South

Photo 15 B-18 facing North

Photo 16 B-18 facing South

Photo 17 B-19 facing North

Photo 18 B-19 facing South


Photo 19 B-20 facing North


Photo 20 B-20 facing South


Photo 21 PG-1 facing North


Photo 22 PG-1 facing South


Photo 23 PG-2 facing North


Photo 24 PG-2 facing South

Photo 25 PG-3 facing North

Photo 26 PG-3 facing South

Photo 27 PG-4 facing North

Photo 28 PG-4 facing South

Photo 1 PG-1 - Run 1 (3.1'-8.1') & Run 2 (8.1'-13.1')

Photo 2 PG-4 - Run 1 (14.8'-19.8') & Run 2 (19.8'-24.8')

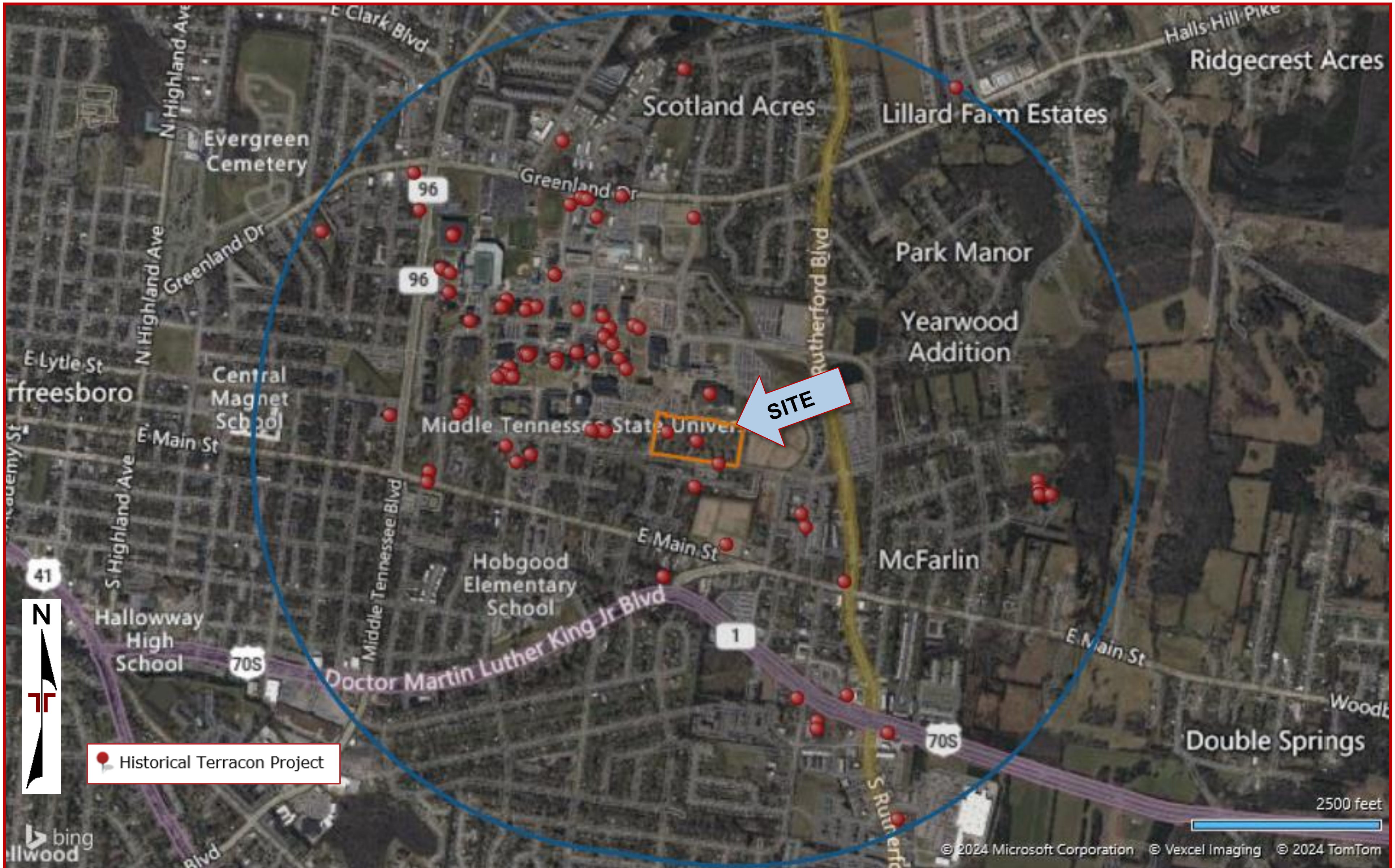
Site Location and Exploration Plans

Contents:

Site Location Plan
Exploration Plan (2 pages)

Note: All attachments are one page unless noted above.

Site Location



IAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES
MAP PROVIDED BY MICROSOFT BING MAPS

Exploration Plan (Concept Layout)

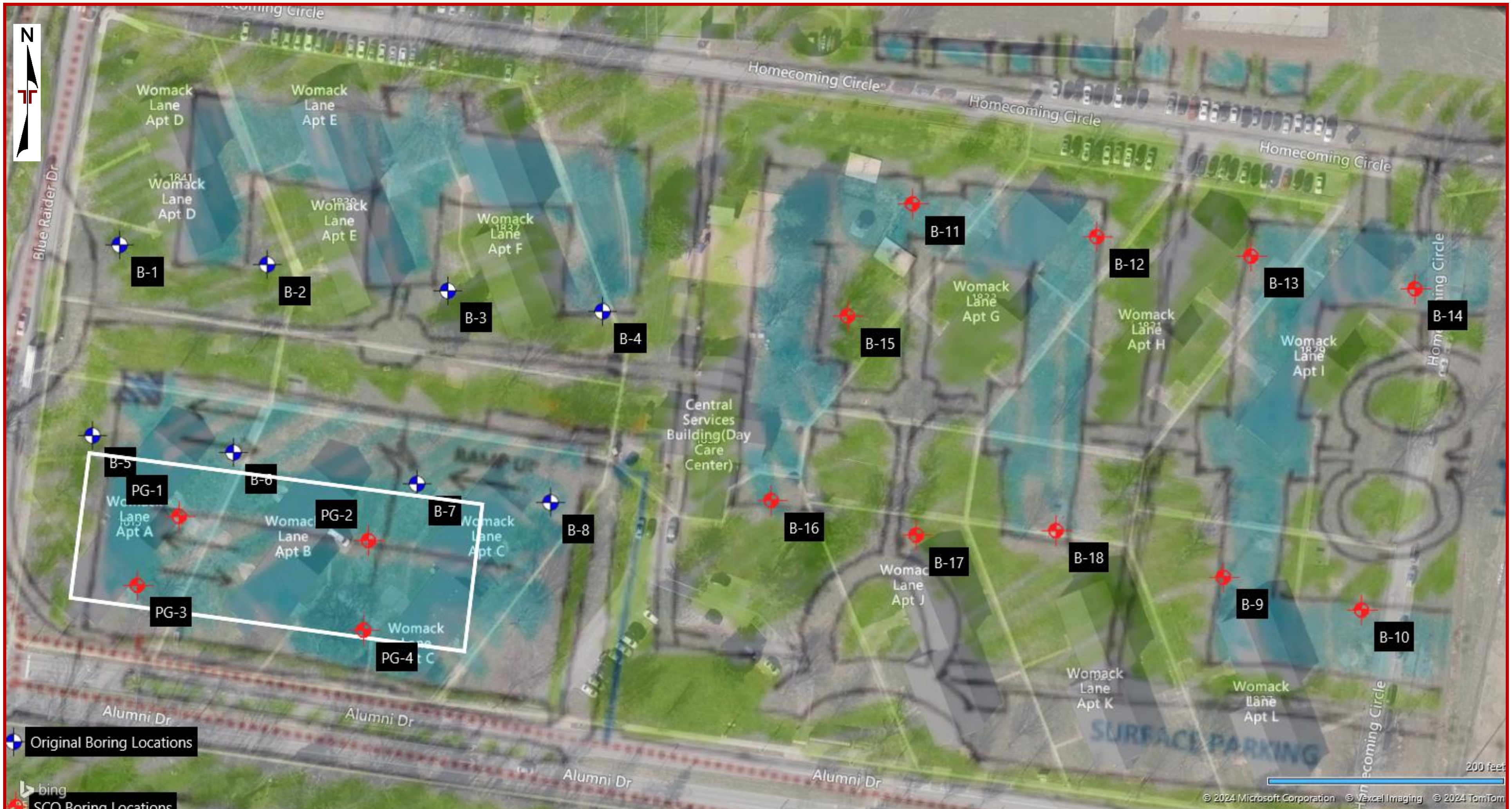


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

Exploration Plan (Parking Garage Overlay)

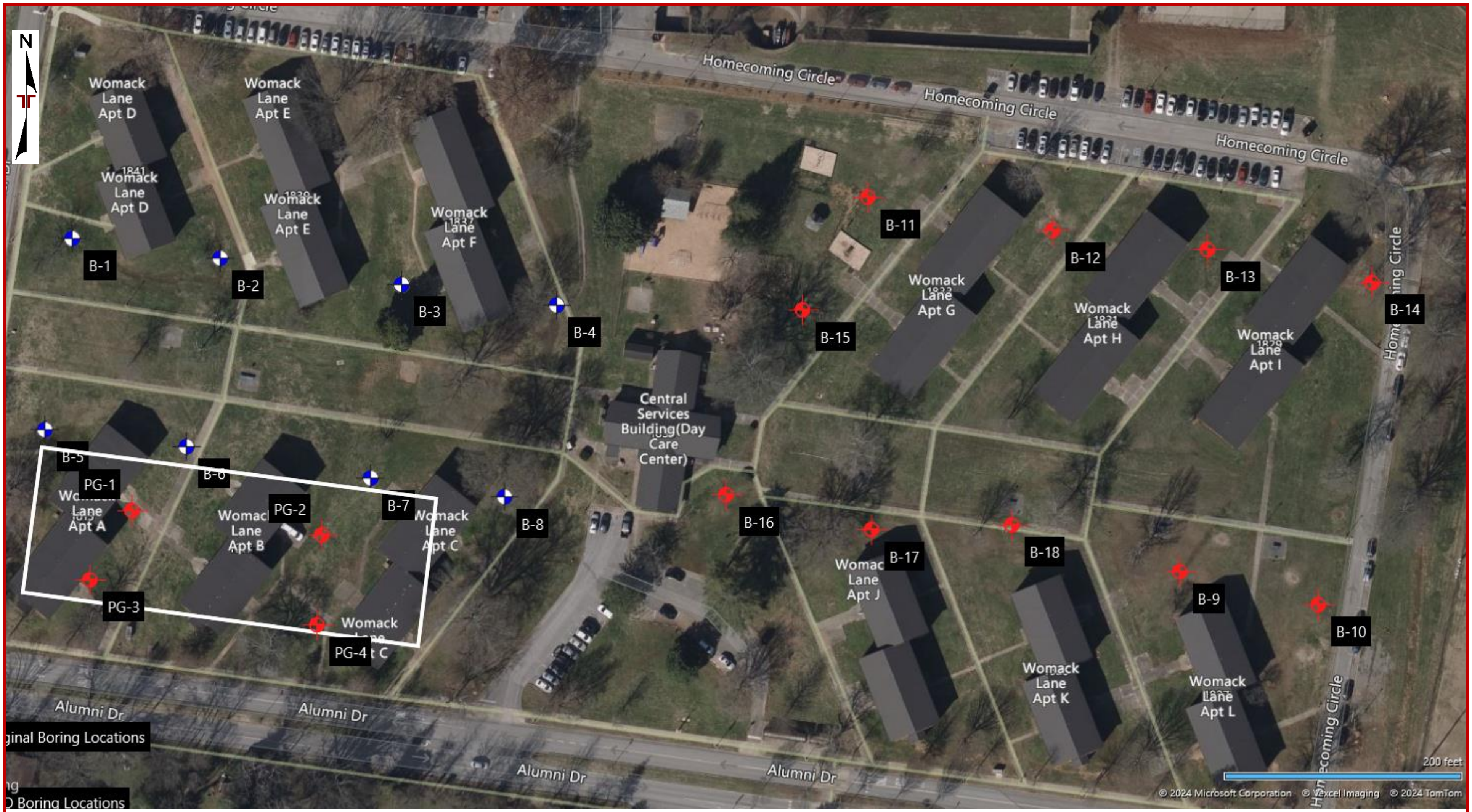


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

Exploration and Laboratory Results

Contents:

Boring Logs
(B-1 through B-18 and PG-1 through PG-4A)
Atterberg Limits

Note: All attachments are one page unless noted above.

Terracon
1922 Old Murfreesboro Pike Ste 905
Nashville, TN

Boring Log No. B-1

Location: See Exploration Plan

Latitude: 35.8454° Longitude: -86.3604°

Depth (Ft.) Elevation: 628 (Ft.) +/-

Model Layer	Graphic Log	Depth (Ft.)	Description	Elevation	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Recovery (%)	RQD (%)	Unconfined Compressive Strength (psi)	Water Content (%)	Atterberg Limits LL-PL-PI
1		0.4	**5" TOPSOIL**	627.6									
4		2.5	**FAT CLAY (CH)**, with chert, trace roots and mineral nodules, yellowish brown, hard	625.5			X	2-3-50/4"				31.2	
		5.5	**FAT CLAY (CH)**, trace mineral nodules, yellowish brown with gray mottling, stiff	622.5	5		X	3-5-7 N=12				29.1	

Auger Refusal at 5.5 Feet

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevation Reference: Elevations were interpolated from Murfreesboro GIS dated 2023 and therefore should be considered approximate and not for building construction use.

Water Level Observations
Groundwater not encountered

Drill Rig
Geoprobe

Hammer Type
Automatic

Driller
TSD

Notes

Advancement Method
Hollow Stem Augers

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Logged by
Matt H./Juan V.

Boring Started
08-05-2024

Boring Completed
08-05-2024

Terracon
1922 Old Murfreesboro Pike Ste 905
Nashville, TN

Boring Log No. B-2

Location: See Exploration Plan

Latitude: 35.8453° Longitude: -86.3599°

Depth (Ft.) — Elevation: 628 (Ft.) +/-

Model Layer	Depth (Ft.)	Material Description	Elevation	Sample Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Recovery (%)	RQD (%)	Unconfined Compressive Strength (psi)	Water Content (%)	Atterberg Limits LL-PL-PI
1	0.6	**7" TOPSOIL**	627.4									
4	3.0	**FAT CLAY (CH)**, with chert and trace mineral nodules, reddish brown, stiff	625			X	2-3-6 N=9				33.8	
4	5.5	**FAT CLAY (CH)**, few mineral nodules, yellowish brown, stiff	622.5	5		X	3-6-9 N=15				32.0	
4	7.7	**FAT CLAY (CH)**, trace mineral nodules, brown with gray mottling	620.3			X	7-9-50/2"				33.7	

Auger Refusal at 7.7 Feet

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevation Reference: Elevations were interpolated from Murfreesboro GIS dated 2023 and therefore should be considered approximate and not for building construction use.

Water Level Observations
Groundwater not encountered

Drill Rig
Geoprobe

Hammer Type
Automatic

Driller
TSD

Notes

Advancement Method
Hollow Stem Augers

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Logged by
Matt H./Juan V.

Boring Started
08-05-2024

Boring Completed
08-05-2024

Terracon
1922 Old Murfreesboro Pike Ste 905
Nashville, TN

Boring Log No. B-3

Location: See Exploration Plan

Latitude: 35.8453° Longitude: -86.3594°

Elevation: 625 (Ft.) +/-

Model Layer	Depth (Ft.)	Description	Elevation	Sample Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Recovery (%)	RQD (%)	Unconfined Compressive Strength (psi)	Water Content (%)	Atterberg Limits LL-PL-PI
1	0.4	**5" TOPSOIL**	624.6									
3	3.0	**LEAN CLAY (CL)**, trace mineral nodules and roots, brown, stiff	622				5-7-7 N=14				21.7	
3	5.5	**LEAN CLAY (CL)**, few mineral nodules, brown, medium stiff	619.5	5			3-3-4 N=7				24.0	
4	11.2	**FAT CLAY (CH)**, trace mineral nodules, dark yellowish brown with gray mottling, stiff	613.8	10			2-3-9 N=12				27.2	
							3-5-9 N=14				28.2	

Auger Refusal at 11.2 Feet

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevation Reference: Elevations were interpolated from Murfreesboro GIS dated 2023 and therefore should be considered approximate and not for building construction use.

Water Level Observations
Groundwater not encountered

Drill Rig
Geoprobe

Hammer Type
Automatic

Driller
TSD

Notes

Advancement Method
Hollow Stem Augers

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Logged by
Matt H./Juan V.

Boring Started
08-05-2024

Boring Completed
08-05-2024

Terracon
1922 Old Murfreesboro Pike Ste 905
Nashville, TN

Boring Log No. B-4

Location: See Exploration Plan

Latitude: 35.8452° Longitude: -86.3590°

Elevation: 625 (Ft.) +/-

Model Layer	Depth (Ft.)	Material Description	Elevation	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Recovery (%)	RQD (%)	Unconfined Compressive Strength (psi)	Water Content (%)	Atterberg Limits LL-PL-PI
1	0.6	**7" TOPSOIL**	624.4									
3	3.0	**LEAN CLAY (CL)**, trace roots and few mineral nodules, yellowish brown, stiff	622			X	2-4-6 N=10				18.7	
4	5.7	**FAT CLAY (CH)**, with chert and trace mineral nodules, yellowish brown, stiff	619.3	5		X	3-4-7 N=11				23.1	54-19-35

Auger Refusal at 5.7 Feet

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
See Supporting Information for explanation of symbols and abbreviations.
Elevation Reference: Elevations were interpolated from Murfreesboro GIS dated 2023 and therefore should be considered approximate and not for building construction use.

Water Level Observations
Groundwater not encountered

Drill Rig
Geoprobe

Hammer Type
Automatic

Driller
TSD

Notes

Advancement Method
Hollow Stem Augers

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Logged by
Matt H./Juan V.

Boring Started
08-05-2024

Boring Completed
08-05-2024

Terracon
1922 Old Murfreesboro Pike Ste 905
Nashville, TN

Boring Log No. B-5

Model Layer	Graphic Log	Location: See Exploration Plan Latitude: 35.8449° Longitude: -86.3604° Depth (Ft.) Elevation: 625.5 (Ft.) +/-	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Recovery (%)	RQD (%)	Unconfined Compressive Strength (psi)	Water Content (%)	Atterberg Limits LL-PL-PI
1		0.4 **5" TOPSOIL** 625.1									
2		**POSSIBLE FILL - LEAN CLAY (CL)**, No Recovery. Assume the same as nearby boring B-4. 3.0 622.5									
		Auger Refusal at 3 Feet									

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevation Reference: Elevations were interpolated from Murfreesboro GIS dated 2023 and therefore should be considered approximate and not for building construction use.

Water Level Observations
Groundwater not encountered

Drill Rig
Geoprobe

Hammer Type
Automatic

Driller
TSD

Notes

Advancement Method
Hollow Stem Augers

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Logged by
Matt H./Juan V.

Boring Started
08-06-2024

Boring Completed
08-06-2024

Terracon
1922 Old Murfreesboro Pike Ste 905
Nashville, TN

Boring Log No. B-6

Location: See Exploration Plan

Latitude: 35.8449° Longitude: -86.3600°

Elevation: 625.5 (Ft.) +/-

Model Layer	Depth (Ft.)	Material Description	Elevation	Sample Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Recovery (%)	RQD (%)	Unconfined Compressive Strength (psi)	Water Content (%)	Atterberg Limits LL-PL-PI
1	0.4	**5" TOPSOIL**	625.1									
2	2.5	**FILL - FAT CLAY (CH)**, with limestone rock fragments, trace roots and mineral nodules, pale brown, hard	623	1.0–1.8		SPT	2-50/4"				27.4	56-23-33
4	7.5	**FAT CLAY (CH)**, with chert, trace mineral nodules, pale brown with gray mottling, stiff to very stiff	618	3.5–5		SPT	2-5-7 N=12				28.2	
				6–7.5		SPT	6-9-15 N=24				32.1	
4	10.0	**FAT CLAY (CH)**, with chert, trace mineral nodules and silt, reddish brown, very stiff	615.5	8.5–10		SPT	4-5-10 N=15				30.1	
4	15.0	**FAT CLAY (CH)**, with chert, trace sand, mineral nodules and limestone rock fragments, dark reddish brown with gray mottling, very stiff	610.5	13.5–15		SPT	6-7-14 N=21				36.9	
4	18.7	**FAT CLAY (CH)**, with chert and trace mineral nodules, dark browm	606.8	18.5–18.7		SPT	50/2"				35.0	

Auger Refusal at 18.7 Feet

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevation Reference: Elevations were interpolated from Murfreesboro GIS dated 2023 and therefore should be considered approximate and not for building construction use.

Water Level Observations
Groundwater not encountered

Drill Rig
Geoprobe

Hammer Type
Automatic

Driller
TSD

Notes

Advancement Method
Hollow Stem Augers

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Logged by
Matt H./Juan V.

Boring Started
08-06-2024

Boring Completed
08-06-2024

Terracon
1922 Old Murfreesboro Pike Ste 905
Nashville, TN

Boring Log No. B-7

Location: See Exploration Plan

Latitude: 35.8448° Longitude: -86.3595°

Elevation: 624.5 (Ft.) +/-

Model Layer	Depth (Ft.)	Material Description	Elevation	Water Level Observations	Sample Type	Field Test Results	Recovery (%)	RQD (%)	Unconfined Compressive Strength (psi)	Water Content (%)	Atterberg Limits LL-PL-PI
1	0.4	**5" TOPSOIL**	624.1								
2	2.5	**FILL - LEAN CLAY (CL)**, with chert and limestone rock fragments, trace root, mineral nodules and brick fragments, dark brown	622		X	2-6-5 N=11				24.3	
3	5.0	**LEAN CLAY (CL)**, with chert and trace silt, sand and mineral nodules, brown, stiff	619.5		X	4-5-7 N=12				20.8	
4	7.5	**FAT CLAY (CH)**, with chert and trace mineral nodules, brown, very stiff	617		X	6-6-12 N=18				28.7	
4	9.6	**FAT CLAY (CH)**, with chert and limestone rock fragments, trace sand, brown	614.9		X	3-50/4"				39.8	

Auger Refusal at 9.6 Feet

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevation Reference: Elevations were interpolated from Murfreesboro GIS dated 2023 and therefore should be considered approximate and not for building construction use.

Water Level Observations
Groundwater not encountered

Drill Rig
Geoprobe

Hammer Type
Automatic

Driller
TSD

Notes

Advancement Method
Hollow Stem Augers

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Logged by
Matt H./Juan V.

Boring Started
08-06-2024

Boring Completed
08-06-2024

Terracon
1922 Old Murfreesboro Pike Ste 905
Nashville, TN

Boring Log No. B-8

Location: See Exploration Plan

Latitude: 35.8448° Longitude: -86.3591°

Elevation: 624.5 (Ft.) +/-

Model Layer	Depth (Ft.)	Material Description	Elevation (Ft.)	Sample Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Recovery (%)	RQD (%)	Unconfined Compressive Strength (psi)	Water Content (%)	Atterberg Limits LL-PL-PI
1	0.4	**4" TOPSOIL**	624.1									
3	2.5	**LEAN CLAY (CL)**, trace mineral and roots, dark reddish brown, stiff	622			X	3-5-7 N=12				22.3	
3	7.5	**LEAN CLAY (CL)**, with chert, trace mineral nodules, silt and sand, dark reddish brown, stiff	617	5		X	3-5-8 N=13				17.7	
						X	3-6-9 N=15				23.9	
3	11.5	**LEAN CLAY (CL)**, trace mineral nodules, brown, stiff	613	10		X	4-5-9 N=14				21.7	
3	16.5	**LEAN CLAY (CL)**, with chert and trace mineral nodules, yellowish brown, stiff	608	15		X	4-6-8 N=14				21.9	
3	18.6	**LEAN CLAY (CL)**, with limestone rock fragments and trace mineral nodules, yellowish brown, stiff	605.9			X	50/2"				21.3	
		Auger Refusal at 18.6 Feet										

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevation Reference: Elevations were interpolated from Murfreesboro GIS dated 2023 and therefore should be considered approximate and not for building construction use.

Notes

Water Level Observations
Groundwater not encountered

Advancement Method
Hollow Stem Augers

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Drill Rig
Geoprobe

Hammer Type
Automatic

Driller
TSD

Logged by
Matt H./Juan V.

Boring Started
08-05-2024

Boring Completed
08-05-2024

Terracon
1922 Old Murfreesboro Pike Ste 905
Nashville, TN

Boring Log No. B-9

Location: See Exploration Plan

Latitude: 35.8446° Longitude: -86.3572°

Model Layer	Graphic Log	Depth (Ft.) / Description / Elevation: 629 (Ft.) +/-	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Recovery (%)	RQD (%)	Unconfined Compressive Strength (psi)	Water Content (%)	Atterberg Limits LL-PL-PI
1		0.3 **3.5" TOPSOIL** 628.71									
3		**LEAN CLAY (CL)**, trace mineral nodules and chert, dark yellowish brown, stiff				4-5-6 N=11				19.1	
		3.5 625.5									
4		**FAT CLAY (CH)**, trace mineral nodules, yellowish red, stiff to very stiff				7-6-5 N=11				28.7	
			5			6-7-8 N=15				27.5	
						7-8-9 N=17				34.6	
		10.8 618.2	10								
		Auger Refusal at 10.8 Feet									

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevation Reference: Elevations were interpolated from a topographic survey titiled Middle Tennessee State University-Womack Lane by Civil Infrastructure Associates dated September 27, 2024.

Water Level Observations
Groundwater not encountered

Drill Rig
Geoprobe

Hammer Type
Automatic

Driller
TSD

Notes

Advancement Method
Hollow Stem Augers

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Logged by
JV

Boring Started
10-16-2024

Boring Completed
10-16-2024

Terracon
1922 Old Murfreesboro Pike Ste 905
Nashville, TN

Boring Log No. B-10

Location: See Exploration Plan

Latitude: 35.8445° Longitude: -86.3568°

Elevation: 628.5 (Ft.) +/-

Model Layer	Depth (Ft.)	Material Description	Elevation (Ft.)	Sample Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Recovery (%)	RQD (%)	Unconfined Compressive Strength (psi)	Water Content (%)	Atterberg Limits LL-PL-PI
1	0.3	**4" TOPSOIL**	628.16									
3	3.0	**LEAN CLAY (CL)**, trace mineral nodules, yellow brown, stiff	625.5	1.0-2.5		SPT	4-5-6 N=11				9.4	
3	8.0	**LEAN CLAY (CL)**, trace mineral nodules, reddish brown, medium stiff to stiff	620.5	3.5-5.0		SPT	4-3-5 N=8				20.6	
				6.0-7.5		SPT	4-5-5 N=10				21.4	
3	11.5	**LEAN CLAY (CL)**, trace mineral nodules, reddish brown, stiff	617	8.5-10.0		SPT	2-4-5 N=9				21.9	
4	20.0	**FAT CLAY (CH)**, trace mineral nodules, yellowish brown, stiff	608.5	13.5-15.0		SPT	4-6-8 N=14				22.7	
				18.5-20.0		SPT	4-5-7 N=12				20.8	

Boring Terminated at 20 Feet

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevation Reference: Elevations were interpolated from a topographic survey titiled Middle Tennessee State University-Womack Lane by Civil Infrastructure Associates dated September 27, 2024.

Water Level Observations
Groundwater not encountered

Drill Rig
Geoprobe

Hammer Type
Automatic

Driller
TSD

Notes

Advancement Method
Hollow Stem Augers

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Logged by
JV

Boring Started
10-16-2024

Boring Completed
10-16-2024

Terracon
1922 Old Murfreesboro Pike Ste 905
Nashville, TN

Boring Log No. B-11

Location: See Exploration Plan

Latitude: 35.8455° Longitude: -86.3581°

Depth (Ft.) Elevation: 627.5 (Ft.) +/-

Model Layer	Depth (Ft.)	Material Description	Elevation	Sample Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Recovery (%)	RQD (%)	Unconfined Compressive Strength (psi)	Water Content (%)	Atterberg Limits LL-PL-PI
1	0.3	**3.5" TOPSOIL**	627.21									
4	3.0	**FAT CLAY (CH)**, trace mineral nodules, dark yellowish brown to reddish brown, stiff	624.5			X	5-6-8 N=14				25.0	63-23-40
4	5.5	**FAT CLAY (CH)**, trace mineral nodules and silt, yellowish brown, very stiff	622	5		X	6-8-10 N=18				20.6	
4	8.0	**FAT CLAY (CH)**, trace mineral nodules, yellowish brown with gray mottling, very stiff	619.5			X	6-8-10 N=18				32.2	
4	9.2	**FAT CLAY (CH)**, trace mineral nodules, yellowish brown with gray mottling	618.3			X	50/5"				26.0	

Auger Refusal at 9.2 Feet

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevation Reference: Elevations were interpolated from a topographic survey titiled Middle Tennessee State University-Womack Lane by Civil Infrastructure Associates dated September 27, 2024.

Water Level Observations
Groundwater not encountered

Drill Rig
Geoprobe

Hammer Type
Automatic

Driller
TSD

Notes

Advancement Method
Hollow Stem Augers

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Logged by
JV

Boring Started
10-16-2024

Boring Completed
10-16-2024

Terracon
1922 Old Murfreesboro Pike Ste 905
Nashville, TN

Boring Log No. B-12

Location: See Exploration Plan

Latitude: 35.8454° Longitude: -86.3576°

Elevation: 629.5 (Ft.) +/-

Model Layer	Depth (Ft.)	Description	Elevation (Ft.)	Field Test Results	Water Content (%)
1	0.3	**3.5" TOPSOIL**	629.21		
3	3.0	**LEAN CLAY (CL)**, with chert, trace roots and mineral nodules, reddish brown, stiff	626.5	6-8-6 N=14	15.7
3	5.5	**LEAN CLAY (CL)**, trace mineral nodules, yellowish brown, stiff	624	4-6-8 N=14	24.3
4	8.0	**FAT CLAY (CH)**, trace mineral nodules, yellowish brown with gray mottling, stiff	621.5	7-8-7 N=15	33.6
4	9.3	**FAT CLAY (CH)**, with limestone rock fragment, trace mineral nodules, yellowish red	620.2	3-50/1"	30.8

Auger Refusal at 9.3 Feet

Columns also shown (no entries): Water Level Observations, Sample Type, Recovery (%), RQD (%), Unconfined Compressive Strength (psi), Atterberg Limits LL-PL-PI

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
See Supporting Information for explanation of symbols and abbreviations.
Elevation Reference: Elevations were interpolated from a topographic survey titiled Middle Tennessee State University-Womack Lane by Civil Infrastructure Associates dated September 27, 2024.

Water Level Observations
Groundwater not encountered

Drill Rig
Geoprobe

Hammer Type
Automatic

Driller
TSD

Notes

Advancement Method
Hollow Stem Augers

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Logged by
JV

Boring Started
10-16-2024

Boring Completed
10-16-2024

Terracon
1922 Old Murfreesboro Pike Ste 905
Nashville, TN

Boring Log No. B-13

Location: See Exploration Plan

Latitude: 35.8454° Longitude: -86.3571°

Elevation: 628.5 (Ft.) +/-

Model Layer	Depth (Ft.)	Material Description	Elevation (Ft.)	Sample Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Recovery (%)	RQD (%)	Unconfined Compressive Strength (psi)	Water Content (%)	Atterberg Limits LL-PL-PI
1	0.3	**3" TOPSOIL**	628.25									
4	3.0	**FAT CLAY (CH)**, trace roots and mineral nodules, dark yellowish brown, medium stiff	625.5	1.0–2.5		SPT	6-3-4 N=7				28.0	
4	5.5	**NO RECOVERY**, assume the same soil as the sample above, medium stiff	623	3.5–5.0		SPT	2-3-4 N=7					
4	8.0	**FAT CLAY (CH)**, trace mineral nodules, dark reddish brown to yellowish brown, stiff	620.5	6.0–7.5		SPT	4-5-8 N=13				26.9	
4	9.7	**FAT CLAY (CH)**, trace mineral nodules, with weathered limestone rock fragments, dark yellowish brown	618.8	8.5–9.7		SPT	4-7-50/2"				26.7	

Auger Refusal at 9.7 Feet

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevation Reference: Elevations were interpolated from a topographic survey titiled Middle Tennessee State University-Womack Lane by Civil Infrastructure Associates dated September 27, 2024.

Water Level Observations
Groundwater not encountered

Drill Rig
Geoprobe

Hammer Type
Automatic

Driller
TSD

Notes

Advancement Method
Hollow Stem Augers

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Logged by
JV

Boring Started
10-16-2024

Boring Completed
10-16-2024

Terracon
1922 Old Murfreesboro Pike Ste 905
Nashville, TN

Boring Log No. B-14

Location: See Exploration Plan

Latitude: 35.8453° Longitude: -86.3567°

Elevation: 626.5 (Ft.) +/-

Model Layer	Depth (Ft.)	Material Description	Elevation (Ft.)	Water Level Observations	Sample Type	Field Test Results	Recovery (%)	RQD (%)	Unconfined Compressive Strength (psi)	Water Content (%)	Atterberg Limits LL-PL-PI
1	0.3	**3" TOPSOIL**	626.25								
3		**LEAN CLAY (CL)**, trace roots and mineral nodules, brown to reddish brown, very stiff			X	7-10-10 N=20				16.7	
	3.0		623.5								
3		**LEAN CLAY (CL)**, trace mineral nodules and silt, brown, very stiff			X	14-12-15 N=27				14.0	
					X	6-10-14 N=24				15.6	
		hard			X	10-17-14 N=31				11.0	
	12.2		614.3								

Auger Refusal at 12.2 Feet

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
See Supporting Information for explanation of symbols and abbreviations.
Elevation Reference: Elevations were interpolated from a topographic survey titiled Middle Tennessee State University-Womack Lane by Civil Infrastructure Associates dated September 27, 2024.

Water Level Observations
Groundwater not encountered

Drill Rig
Geoprobe

Hammer Type
Automatic

Driller
TSD

Notes

Advancement Method
Hollow Stem Augers

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Logged by
JV

Boring Started
10-16-2024

Boring Completed
10-16-2024

Terracon
1922 Old Murfreesboro Pike Ste 905
Nashville, TN

Boring Log No. B-15

Location: See Exploration Plan

Latitude: 35.8452° Longitude: -86.3583°

Depth (Ft.) Elevation: 627.5 (Ft.) +/-

Model Layer	Depth (Ft.)	Material Description	Elevation	Sample Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Recovery (%)	RQD (%)	Unconfined Compressive Strength (psi)	Water Content (%)	Atterberg Limits LL-PL-PI
1	0.4	**4.5" TOPSOIL**	627.12									
3		**LEAN CLAY (CL)**, trace mineral nodules, reddish brown, medium stiff				X	2-3-4 N=7				20.8	
						X	3-3-3 N=6				23.3	
		stiff		5		X	4-5-9 N=14				21.8	
	8.0		619.5									
		LEAN CLAY (CL), trace mineral nodules, yellowish brown with gray mottling, very stiff				X	6-8-13 N=21				19.5	
	11.5		616	10								
4		**FAT CLAY (CH)**, trace mineral nodules and silt, dark yellowish brown, very stiff				X	5-8-9 N=17				27.7	
	15.2		612.3	15								

Auger Refusal at 15.2 Feet

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevation Reference: Elevations were interpolated from a topographic survey titiled Middle Tennessee State University-Womack Lane by Civil Infrastructure Associates dated September 27, 2024.

Water Level Observations
Groundwater not encountered

Drill Rig
Geoprobe

Hammer Type
Automatic

Driller
TSD

Notes

Advancement Method
Hollow Stem Augers

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Logged by
JV

Boring Started
10-16-2024

Boring Completed
10-16-2024

Terracon
1922 Old Murfreesboro Pike Ste 905
Nashville, TN

Boring Log No. B-16

Location: See Exploration Plan

Latitude: 35.8448° Longitude: -86.3585°

Model Layer	Graphic Log	Depth (Ft.)	Material Description	Elevation: 627 (Ft.) +/-	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Recovery (%)	RQD (%)	Unconfined Compressive Strength (psi)	Water Content (%)	Atterberg Limits LL-PL-PI
1		0.3	**3" TOPSOIL**	626.75									
3			**LEAN CLAY (CL)**, trace mineral nodules, reddish brown, very stiff					5-6-11 N=17				22.4	
		3.0		624									
4			**FAT CLAY (CH)**, trace mineral nodules, reddish brown to yellowish brown, stiff		5			3-5-5 N=10				30.4	
		6.3		620.7									
			Auger Refusal at 6.3 Feet										

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevation Reference: Elevations were interpolated from a topographic survey titiled Middle Tennessee State University-Womack Lane by Civil Infrastructure Associates dated September 27, 2024.

Water Level Observations
Groundwater not encountered

Drill Rig
Geoprobe

Hammer Type
Automatic

Driller
TSD

Notes

Advancement Method
Hollow Stem Augers

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Logged by
JV

Boring Started
10-16-2024

Boring Completed
10-16-2024

Terracon
1922 Old Murfreesboro Pike Ste 905
Nashville, TN

Boring Log No. B-17

Location: See Exploration Plan

Latitude: 35.8447° Longitude: -86.3581°

Elevation: 628.5 (Ft.) +/-

Model Layer	Depth (Ft.)	Material Description	Elevation (Ft.)	Sample Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Recovery (%)	RQD (%)	Unconfined Compressive Strength (psi)	Water Content (%)	Atterberg Limits LL-PL-PI
1	0.3	**3.5" TOPSOIL**	628.21									
3	3.0	**LEAN CLAY (CL)**, trace mineral nodules, red, medium stiff	625.5				3-4-4 N=8				23.1	
3	11.5	**LEAN CLAY (CL)**, trace mineral nodules, red, stiff	617				5-6-7 N=13				20.4	
							4-5-6 N=11				25.5	
							5-6-7 N=13				24.4	
3	16.5	**LEAN CLAY (CL)**, trace mineral nodules and silt, red with yellowish brown mottling, stiff	612				4-5-6 N=11				24.3	
4	20.0	**FAT CLAY (CH)**, trace mineral nodules, yellowish brown, stiff	608.5				7-8-5 N=13				25.7	

Boring Terminated at 20 Feet

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
See Supporting Information for explanation of symbols and abbreviations.
Elevation Reference: Elevations were interpolated from a topographic survey titiled Middle Tennessee State University-Womack Lane by Civil Infrastructure Associates dated September 27, 2024.

Water Level Observations
Groundwater not encountered

Notes

Advancement Method
Hollow Stem Augers

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Drill Rig
Geoprobe

Hammer Type
Automatic

Driller
TSD

Logged by
JV

Boring Started
10-16-2024

Boring Completed
10-16-2024

Terracon
1922 Old Murfreesboro Pike Ste 905
Nashville, TN

Boring Log No. B-18

Location: See Exploration Plan

Latitude: 35.8447° Longitude: -86.3577°

Depth (Ft.) Elevation: 629 (Ft.) +/-

Model Layer	Depth (Ft.)	Material Description	Elevation	Sample Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Recovery (%)	RQD (%)	Unconfined Compressive Strength (psi)	Water Content (%)	Atterberg Limits LL-PL-PI
1	0.3	**4" TOPSOIL**	628.66									
4		**NO RECOVERY**, assume the same soil as the sample below, stiff				X	4-5-6 N=11					
	3.0		626									
		FAT CLAY (CH), trace mineral nodules, yellowish brown, stiff				X	5-7-7 N=14				30.6	79-30-49
	5.5		623.5	5								
		FAT CLAY (CH), trace mineral nodules, yellowish brown with gray mottling, very stiff to stiff				X	6-8-10 N=18				25.8	
						X	4-6-8 N=14				29.5	
	11.0		618	10								

Auger Refusal at 11 Feet

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
See Supporting Information for explanation of symbols and abbreviations.
Elevation Reference: Elevations were interpolated from a topographic survey titiled Middle Tennessee State University-Womack Lane by Civil Infrastructure Associates dated September 27, 2024.

Water Level Observations
Groundwater not encountered

Drill Rig
Geoprobe

Hammer Type
Automatic

Driller
TSD

Notes

Advancement Method
Hollow Stem Augers

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Logged by
JV

Boring Started
10-16-2024

Boring Completed
10-16-2024

Terracon
1922 Old Murfreesboro Pike Ste 905
Nashville, TN

Boring Log No. PG-1

Location: See Exploration Plan
Latitude: 35.8447° Longitude: -86.3602°
Elevation: 625 (Ft.) +/-

Model Layer	Depth (Ft.)	Description	Elevation	Field Test Results	Recovery (%)	RQD (%)	Unconfined Compressive Strength (psi)	Water Content (%)	Atterberg Limits LL-PL-PI
1	0.4	**5" TOPSOIL**	624.59						
2	3.1	**FILL - LEAN CLAY (CL)**, trace mineral nodules, with limestone rock fragments, brown	621.9	2-2-4 N=6				23.4	
		Auger Refusal at 3.1 Feet, Begin Rock Coring		50/2"					
5	8.1	**LIMESTONE**, slightly weathered, moderately fractured, medium bedded, gray	616.9		90	90	6730		
5	13.1	**LIMESTONE**, slightly weathered, slightly fractured, medium bedded, gray	611.9		100	100			

Boring Terminated at 13.1 Feet

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevation Reference: Elevations were interpolated from a topographic survey titiled Middle Tennessee State University-Womack Lane by Civil Infrastructure Associates dated September 27, 2024.

Water Level Observations
Groundwater not encountered

Drill Rig
Geoprobe

Hammer Type
Automatic

Driller
TSD

Notes

Advancement Method
Hollow Stem Augers

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Logged by
JV

Boring Started
10-08-2024

Boring Completed
10-08-2024

Terracon
1922 Old Murfreesboro Pike Ste 905
Nashville, TN

Boring Log No. PG-2

Location: See Exploration Plan

Latitude: 35.8447° Longitude: -86.3597°

Elevation: 624.5 (Ft.) +/-

Model Layer	Depth (Ft.)	Material Description	Elevation (Ft.)	Sample Depth (Ft.)	Field Test Results	Water Content (%)
1	0.3	**4" TOPSOIL**	624.16			
3	3.0	**LEAN CLAY (CL)**, trace mineral nodules and roots, brown, medium stiff	621.5	1.0–2.5	2-2-3 N=5	16.9
3	5.5	**LEAN CLAY (CL)**, trace mineral nodules, reddish brown, medium stiff	619	3.5–5.0	2-3-2 N=5	17.8
4	8.0	**FAT CLAY (CH)**, trace mineral nodules, yellowish brown, stiff	616.5	6.0–7.5	4-5-5 N=10	21.0
4	9.5	**FAT CLAY (CH)**, trace silt and mineral nodules, yellowish brown to reddish brown	615	8.5–9.4	3-50/5"	21.5
		Auger Refusal at 9.5 Feet, Begin Rock Coring				
5	9.9	**Rock lenses**, Drillers noted 5 inches of rock lenses	614.6			
4	21.9	**FAT CLAY (CH)**, Drillers noted that during rock coring, they broke through the encountered rock lenses and encountered refusal again at 21.9 Feet Assumed to be the same soil as boring PG-4	602.6			

Auger Refusal at 21.9 Feet

Column headings also shown (no data): Water Level Observations, Recovery (%), RQD (%), Unconfined Compressive Strength (psi), Atterberg Limits LL-PL-PI

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevation Reference: Elevations were interpolated from a topographic survey titiled Middle Tennessee State University-Womack Lane by Civil Infrastructure Associates dated September 27, 2024.

Water Level Observations
Groundwater not encountered

Drill Rig
Geoprobe

Hammer Type
Automatic

Driller
TSD

Notes

Advancement Method
Hollow Stem Augers

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Logged by
JV

Boring Started
10-07-2024

Boring Completed
10-07-2024

Terracon
1922 Old Murfreesboro Pike Ste 905
Nashville, TN

Boring Log No. PG-3

Location: See Exploration Plan

Latitude: 35.8446° Longitude: -86.3603°

Elevation: 624 (Ft.) +/-

Model Layer	Depth (Ft.)	Description	Elevation	Sample Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Recovery (%)	RQD (%)	Unconfined Compressive Strength (psi)	Water Content (%)	Atterberg Limits LL-PL-PI
1	0.3	**3.5" TOPSOIL**	623.71									
2	3.0	**FILL - LEAN CLAY (CL)**, trace mineral nodules, with chert, brown to yellowish brown	621			X	2-2-2 N=4				23.0	
3	8.0	**LEAN CLAY (CL)**, trace mineral nodules, reddish brown, stiff to medium stiff	616			X	3-4-6 N=10				14.5	
				5		X	4-3-5 N=8				17.1	
	11.5	**LEAN CLAY (CL)**, trace mineral nodules, reddish brown, stiff	612.5			X	3-5-6 N=11				15.4	
				10								
4	17.5	**NO RECOVERY (CH)**, assume same soil as other nearby borings below this depth, stiff	606.5			X	2-4-7 N=11					
				15								

Auger Refusal at 17.5 Feet

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevation Reference: Elevations were interpolated from a topographic survey titiled Middle Tennessee State University-Womack Lane by Civil Infrastructure Associates dated September 27, 2024.

Water Level Observations
Groundwater not encountered

Drill Rig
Geoprobe

Hammer Type
Automatic

Driller
TSD

Notes

Advancement Method
Hollow Stem Augers

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Logged by
JV

Boring Started
10-08-2024

Boring Completed
10-08-2024

Terracon
1922 Old Murfreesboro Pike Ste 905
Nashville, TN

Boring Log No. PG-4

Location: See Exploration Plan

Latitude: 35.8445° Longitude: -86.3597°

Depth (Ft.) Elevation: 624 (Ft.) +/-

Model Layer	Depth (Ft.)	Material Description	Elevation	Sample Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Recovery (%)	RQD (%)	Unconfined Compressive Strength (psi)	Water Content (%)	Atterberg Limits LL-PL-PI
1	0.3	**4" TOPSOIL**	623.66									
2	3.0	**FILL - LEAN CLAY (CL)**, with limestone rock fragments, trace roots and mineral nodules, brown	621			X	2-2-4 N=6				22.7	
3	8.0	**LEAN CLAY (CL)**, trace mineral nodules, reddish brown, medium stiff	616			X	6-3-3 N=6				23.0	43-19-24
				5		X	2-3-4 N=7				23.1	
3	11.5	**LEAN CLAY (CL)**, trace mineral nodules, yellowish brown with gray mottling, stiff	612.5			X	3-4-5 N=9				20.0	
				10								
4	17.1	**FAT CLAY (CH)**, trace mineral nodules, with chert and limestone rock fragments, yellowish brown to dark yellowish brown, stiff	606.9			X	2-3-7 N=10				28.1	
				15								

Auger Refusal at 17.1 Feet

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Elevation Reference: Elevations were interpolated from a topographic survey titiled Middle Tennessee State University-Womack Lane by Civil Infrastructure Associates dated September 27, 2024.

Water Level Observations
Groundwater not encountered

Drill Rig
Geoprobe

Hammer Type
Automatic

Driller
TSD

Notes

Advancement Method
Hollow Stem Augers

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Logged by
JV

Boring Started
10-07-2024

Boring Completed
10-07-2024

Terracon
1922 Old Murfreesboro Pike Ste 905
Nashville, TN

Boring Log No. PG-4A

Location: See Exploration Plan

Latitude: 35.8445° Longitude: -86.3597°

Elevation: 624 (Ft.) +/-

Model Layer	Depth (Ft.)	Material Description	Elevation (Ft.)	Sample Type	Recovery (%)	RQD (%)	Unconfined Compressive Strength (psi)
1	0.3	**4" TOPSOIL**	623.66				
2	3.0	**FILL - LEAN CLAY (CL)**, assumed to be the same soil as boring PG-4	621				
3	11.5	**LEAN CLAY (CL)**, assumed to be the same soil as boring PG-4	612.5				
4	14.8	**FAT CLAY (CH)**, assumed to be the same soil as boring PG-4 **Auger Refusal at 14.8 Feet, Begin Rock Coring**	609.2				
5	19.8	**LIMESTONE**, slightly weathered, slightly fractured, medium bedded, argillaceous, gray	604.2	Rock core	93	93	6630
5	24.8	**LIMESTONE**, moderately weathered, moderately fractured, medium to thin bedded, argillaceous, gray	599.2	Rock core	100	68	

Column headings also include: Graphic Log, Water Level Observations, Field Test Results, Water Content (%), Atterberg Limits LL-PL-PI.

Boring Terminated at 24.8 Feet

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
See Supporting Information for explanation of symbols and abbreviations.
Elevation Reference: Elevations were interpolated from a topographic survey titled Middle Tennessee State University-Womack Lane by Civil Infrastructure Associates dated September 27, 2024.

Water Level Observations
Groundwater not encountered

Drill Rig
Geoprobe

Hammer Type
Automatic

Driller
TSD

Notes

Advancement Method
Hollow Stem Augers

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Logged by
JV

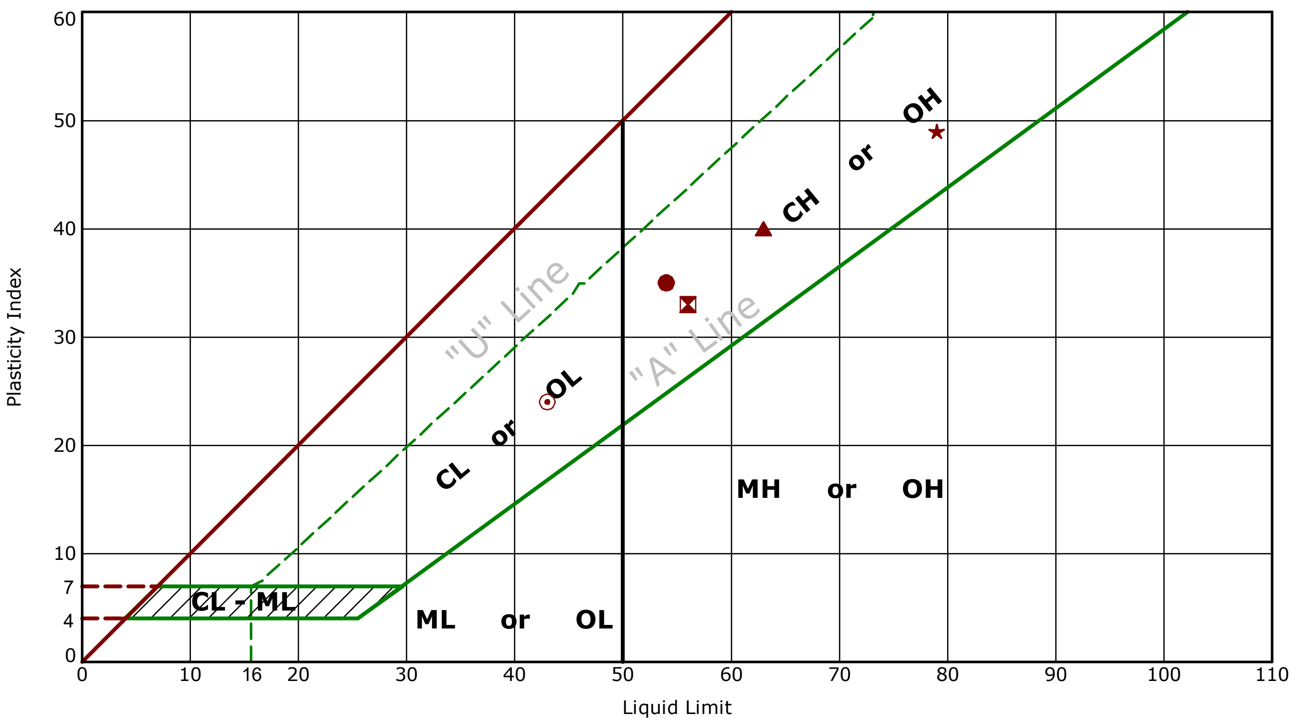
Boring Started
10-16-2024

Boring Completed
10-16-2024

Terracon
1922 Old Murfreesboro Pike Ste 905
Nashville, TN

Atterberg Limit Results

ASTM D4318



	Boring ID	Depth (Ft)	LL	PL	PI	Fines	USCS	Description
●	B-4	3.5 - 5	54	19	35		CL	CL- Lean CLAY
⊠	B-6	1 - 1.8	56	23	33		CL	CL- Lean CLAY
▲	B-11	1 - 2.5	63	23	40		CH	CH- Fat CLAY
★	B-18	3.5 - 5	79	30	49		CH	CH- Fat CLAY
⊙	PG-4	3.5 - 5	43	19	24		CL	CL- Lean CLAY

Supporting Information

Contents:

General Notes
Unified Soil Classification System
Rock Classification Notes

Note: All attachments are one page unless noted above.

1922 Old Murfreesboro Pike
Ste 905
Nashville, TN

General Notes

Sampling	Water Level		Field Tests	
Standard Penetration Test		Water Initially Encountered	N	Standard Penetration Test Resistance (Blows/Ft.)
		Water Level After a Specified Period of Time	(HP)	Hand Penetrometer
		Water Level After a Specified Period of Time	(T)	Torvane
		Cave In Encountered	(DCP)	Dynamic Cone Penetrometer
	Water levels indicated on the soil boring logs are the levels measured in the borehole at the times indicated. Groundwater level variations will occur over time. In low permeability soils, accurate determination of groundwater levels is not possible with short term water level observations.		UC	Unconfined Compressive Strength
			(PID)	Photo-Ionization Detector
			(OVA)	Organic Vapor Analyzer

Descriptive Soil Classification

Soil classification as noted on the soil boring logs is based Unified Soil Classification System. Where sufficient laboratory data exist to classify the soils consistent with ASTM D2487 "Classification of Soils for Engineering Purposes" this procedure is used. ASTM D2488 "Description and Identification of Soils (Visual-Manual Procedure)" is also used to classify the soils, particularly where insufficient laboratory data exist to classify the soils in accordance with ASTM D2487. In addition to USCS classification, coarse grained soils are classified on the basis of their in-place relative density, and fine-grained soils are classified on the basis of their consistency. See "Strength Terms" table below for details. The ASTM standards noted above are for reference to methodology in general. In some cases, variations to methods are applied as a result of local practice or professional judgment.

Location And Elevation Notes

Exploration point locations as shown on the Exploration Plan and as noted on the soil boring logs in the form of Latitude and Longitude are approximate. See Exploration and Testing Procedures in the report for the methods used to locate the exploration points for this project. Surface elevation data annotated with +/- indicates that no actual topographical survey was conducted to confirm the surface elevation. Instead, the surface elevation was approximately determined from topographic maps of the area.

Strength Terms

Relative Density of Coarse-Grained Soils (More than 50% retained on No. 200 sieve.) Density determined by Standard Penetration Resistance		**Consistency of Fine-Grained Soils** (50% or more passing the No. 200 sieve.) Consistency determined by laboratory shear strength testing, field visual-manual procedures or standard penetration resistance		
Relative Density	**Standard Penetration or N-Value (Blows/Ft.)**	**Consistency**	**Unconfined Compressive Strength Qu (tsf)**	**Standard Penetration or N-Value (Blows/Ft.)**
Very Loose	0 - 3	Very Soft	less than 0.25	0 - 1
Loose	4 - 9	Soft	0.25 to 0.50	2 - 4
Medium Dense	10 - 29	Medium Stiff	0.50 to 1.00	4 - 8
Dense	30 - 50	Stiff	1.00 to 2.00	8 - 15
Very Dense	> 50	Very Stiff	2.00 to 4.00	15 - 30
		Hard	> 4.00	> 30

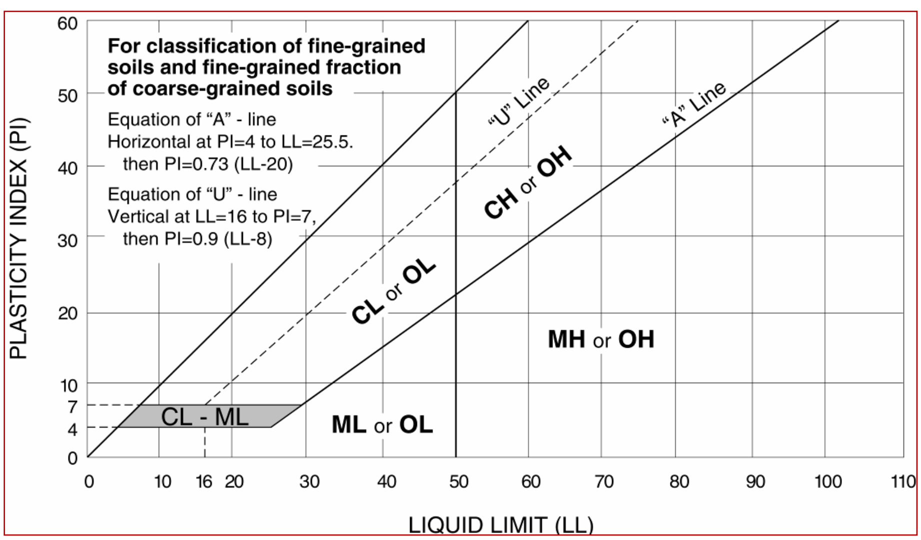
Relevance of Exploration and Laboratory Test Results

Exploration/field results and/or laboratory test data contained within this document are intended for application to the project as described in this document. Use of such exploration/field results and/or laboratory test data should not be used independently of this document.

Unified Soil Classification System

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [A]				Soil Classification	
				Group Symbol	**Group Name** [B]
Coarse-Grained Soils: More than 50% retained on No. 200 sieve	**Gravels:** More than 50% of coarse fraction retained on No. 4 sieve	**Clean Gravels:** Less than 5% fines [C]	Cu≥4 and 1≤Cc≤3 [E]	GW	Well-graded gravel [F]
			Cu<4 and/or [Cc<1 or Cc>3.0] [E]	GP	Poorly graded gravel [F]
		Gravels with Fines: More than 12% fines [C]	Fines classify as ML or MH	GM	Silty gravel [F, G, H]
			Fines classify as CL or CH	GC	Clayey gravel [F, G, H]
	Sands: 50% or more of coarse fraction passes No. 4 sieve	**Clean Sands:** Less than 5% fines [D]	Cu≥6 and 1≤Cc≤3 [E]	SW	Well-graded sand [I]
			Cu<6 and/or [Cc<1 or Cc>3.0] [E]	SP	Poorly graded sand [I]
		Sands with Fines: More than 12% fines [D]	Fines classify as ML or MH	SM	Silty sand [G, H, I]
			Fines classify as CL or CH	SC	Clayey sand [G, H, I]
Fine-Grained Soils: 50% or more passes the No. 200 sieve	**Silts and Clays:** Liquid limit less than 50	**Inorganic:**	PI > 7 and plots above "A" line [J]	CL	Lean clay [K, L, M]
			PI < 4 or plots below "A" line [J]	ML	Silt [K, L, M]
		Organic:	$\frac{LL\ oven\ dried}{LL\ not\ dried} < 0.75$	OL	Organic clay [K, L, M, N]
					Organic silt [K, L, M, O]
	Silts and Clays: Liquid limit 50 or more	**Inorganic:**	PI plots on or above "A" line	CH	Fat clay [K, L, M]
			PI plots below "A" line	MH	Elastic silt [K, L, M]
		Organic:	$\frac{LL\ oven\ dried}{LL\ not\ dried} < 0.75$	OH	Organic clay [K, L, M, P]
					Organic silt [K, L, M, Q]
Highly organic soils:	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A] Based on the material passing the 3-inch (75-mm) sieve.
[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.
[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay.
[E] $Cu = D_{60}/D_{10}$ $\quad Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$
[F] If soil contains ≥ 15% sand, add "with sand" to group name.
[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.
[H] If fines are organic, add "with organic fines" to group name.
[I] If soil contains ≥ 15% gravel, add "with gravel" to group name.
[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.
[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.
[L] If soil contains ≥ 30% plus No. 200 predominantly sand, add "sandy" to group name.
[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.
[N] PI ≥ 4 and plots on or above "A" line.
[O] PI < 4 or plots below "A" line.
[P] PI plots on or above "A" line.
[Q] PI plots below "A" line.



Rock Classification Notes

WEATHERING	
Term	**Description**
Fresh	Mineral crystals appear bright; show no discoloration. Features show little or no staining on surfaces. Discoloration does not extend into intact rock.
Slightly weathered	Rock generally fresh except along fractures. Some fractures stained and discoloration may extend <0.5 inches into rock.
Moderately weathered	Significant portions of rock are dull and discolored. Rock may be significantly weaker than in fresh state near fractures. Soil zones of limited extent may occur along some fractures.
Highly weathered	Rock dull and discolored throughout. Majority of rock mass is significantly weaker and has decomposed and/or disintegrated; isolated zones of stronger rock and/or soil may occur throughout.
Completely weathered	All rock material is decomposed and/or disintegrated to soil. The rock mass or fabric is still evident and largely intact. Isolated zones of stronger rock may occur locally.

STRENGTH OR HARDNESS		
Description	**Field Identification**	**Uniaxial Compressive Strength, psi**
Extremely strong	Can only be chipped with geological hammer. Rock rings on hammer blows. Cannot be scratched with a sharp pick. Hand specimens require several hard hammer blows to break.	>36,000
Very strong	Several blows of a geological hammer to fracture. Cannot be scratched with a 20d common steel nail. Can be scratched with a geologist's pick only with difficulty.	15,000-36,000
Strong	More than one blow of a geological hammer needed to fracture. Can be scratched with a 20d nail or geologist's pick. Gouges or grooves to ¼ inch deep can be excavated by a hard blow of a geologist's pick. Hand specimens can be detached by a moderate blow.	7,500-15,000
Medium strong	One blow of geological hammer needed to fracture. Can be distinctly scratched with 20d nail. Can be grooved or gouged 1/16 in. deep by firm pressure with a geologist's pick point. Can be fractured with single firm blow of geological hammer. Can be excavated in small chips (about 1-in. maximum size) by hard blows of the point of a geologist's pick;	3,500-7,500
Weak	Shallow indent by firm blow with geological hammer point. Can be gouged or grooved readily with geologist's pick point. Can be excavated in pieces several inches in size by moderate blows of a pick point. Small thin pieces can be broken by finger pressure.	700-3,500
Very weak	Crumbles under firm blow with geological hammer point. Can be excavated readily with the point of a geologist's pick. Pieces 1-in. or more in thickness can be broken with finger pressure. Can be scratched readily by fingernail.	150-700

DISCONTINUITY DESCRIPTION			
Fracture Spacing (Joints, Faults, Other Fractures)		**Bedding Spacing (May Include Foliation or Banding)**	
Description	**Spacing**	**Description**	**Spacing**
Intensely fractured	< 2.5 inches	**Laminated**	< ½-inch
Highly fractured	2.5 – 8 inches	**Very thin**	½ – 2 inches
Moderately fractured	8 inches to 2 feet	**Thin**	2 inches – 1 foot
Slightly fractured	2 to 6.5 feet	**Medium**	1 – 3 feet
Very slightly fractured	> 6.5 feet	**Thick**	3 – 10 feet
		Massive	> 10 feet

ROCK QUALITY DESIGNATION (RQD) [1]	
Description	**RQD Value (%)**
Very Poor	0 - 25
Poor	25 – 50
Fair	50 – 75
Good	75 – 90
Excellent	90 - 100

1. The combined length of all sound and intact core segments equal to or greater than 4 inches in length, expressed as a percentage of the total core run length.