Report of Geotechnical Exploration

South 5th Street and Summer Place
Nashville, Davidson County, Tennessee

Prepared for
Gresham Smith
Nashville, Tennessee

Prepared by:
TTL, Inc.
Nashville, Tennessee

Project No. 000220804344.00
January 12, 2023

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January 12, 2023

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RE: Report of Geotechnical Exploration
South 5th Street at Summer Place
Nashville, Davidson County, Tennessee
TTL Project No. 000220804344.00

Dear Mr. Bell:

We have completed the authorized geotechnical exploration for the project in Nashville, Tennessee. Our services were provided in general accordance with the scope outlined in our proposal dated November 17, 2022. Our services were authorized under the terms of AIA Document C402 dated December 8, 2022.

The purposes of the exploration were to obtain subsurface data at the property and develop geotechnical recommendations for the planned building and associated infrastructure. This report summarizes our understanding of the planned construction, the site and subsurface conditions encountered, and our geotechnical recommendations. The scope of this geotechnical exploration did not include environmental assessment of the site.

We appreciate the opportunity to be of service to you during this stage of the project. Please contact us if you have questions regarding the information or recommendations contained in the report.

Respectfully submitted,

TTL, Inc.

Douglas O. Bell

DOUGLAS O. BELL
No. 00117767

12 JAN 2023

Douglas O. Bell, PE
Principal Engineer

Leanna S. Whitwell, PE
Regional Manager/Principal Engineer

Attachments

CONTENTS

EXECUTIVE SUMMARY

The following is a summary of the geotechnical recommendations included in this report.

- Existing fill or possible fill was encountered to depths of about 3 feet below existing grades at two borings. We recommend undercutting existing or possible fill and replacing it with compacted fill where it is present below final grades.
- Weak near-surface soils were encountered by most borings at the site. These soils will likely be unstable and require undercutting or stabilization in-place at the time of construction. Undercut depths on the order of 3 feet should be expected.
- The heavy foundation loads for the building should be supported by groups of micropile foundations bearing in competent bedrock below the site. The top of competent bedrock appears to be at or below approximately elevation 384 feet, which is between about 44 feet and 68 feet below existing grades. We recommend micropiles have a minimum rock socket length of 5 feet into competent bedrock.
- Shallow foundations for lightly loaded incidental structures or equipment, such as light poles, HVAC pads, or site retaining walls less than 8 feet tall which are separate from the structure, can be designed to bear on undisturbed alluvial soils with an allowable net bearing pressure of 1,500 pounds per square foot (psf). Individual shallow footings should not be larger than 6 feet in plan dimension and continuous wall footings should not be wider than 3 feet.
- Floor slabs may be constructed as concrete slabs on grade.
- We recommend seismic site class C for seismic design.
- The alluvial soils encountered by the borings can be used as compacted fill.
- The contractor should be prepared to provide temporary control of groundwater before making site excavations. Permanent groundwater control measures below the building are not considered necessary based on the expected groundwater level being at or lower than about elevation 416 feet.

This summary is provided for convenience only. The report must be read in its entirety to understand fully the information and recommendations provided for the project.

1.0 PROJECT INFORMATION

1.1 Project Description

Information about the project was provided by Mr. Bell (GS) in email communications and telephone conversations since November 8, 2022. Our understanding of the project is summarized below:

Item	Description
Project Location	The project is located at South 5th Street and Summer Place in Nashville, Tennessee. The site is approximately 1.8 acres at the southeast corner of the intersection. A Site Location Map is provided in Appendix A.
Proposed Construction	A concrete- and wood-framed residential building with a footprint of about 130 feet by 290 feet and a total of five and six floors. The western half of the lowest level of the building will have a basement wall about 15 feet tall near the north-south centerline of the structure. The lower level will have finished floor elevations (FFE) of 429 feet and 436 feet. The 2nd level of the building will have a FFE of 450.67 feet and will consist of a parking garage. The two lower levels of the building will be concrete-framed and will support four levels of wood-framed residential apartments above.
Maximum Loads	Loading information has not been provided. We assume maximum column loads on the order of 500 kips and maximum wall loads on the order of 8 kips per linear foot.
Grading	We expect excavation as deep as 15 feet below ground in the middle part of the site where the basement wall separating Level 1 from Level 2 will be located. The grading plan provided shows existing grades between about elevation 453 feet and 429 feet in the building footprint. Based on FFEs discussed above, we expect excavations as deep as 15 feet and fills as tall as 18 feet relative to existing site grades.
Pavements	We understand the 2nd level of the structure will provide parking for passenger vehicles. We understand an asphalt surface parking lot for passenger vehicles will be located north of the building.

Please contact us if the above information is not correct so we can make the necessary modifications to this document and our evaluation and recommendations, if needed.

1.2 Scope of Services and Authorization

The purposes of our services were to explore the subsurface conditions and develop geotechnical recommendations for the project. We drilled six soil and rock core borings, measured shear wave velocity profile below one array, performed laboratory testing of recovered soil and rock samples, and prepared this geotechnical report. Assessment of environmental conditions was beyond the scope of this exploration. Our services were provided in general accordance with the scope outlined in our Proposal No. 000220804344.00, dated November 17, 2022. Our services were authorized under the terms of AIA Document C402 dated December 8, 2022.

2.0 EXPLORATION FINDINGS

2.1 Site Conditions

Item	Description
Existing Conditions	The site is currently undeveloped except for a retaining wall along part of South 5th Street and asphalt parking lots with incidental sidewalks on the north and south ends of the site. The undeveloped area of the site is covered by grass and contains a few isolated trees.
Existing Topography	Existing grades generally slope down from the north and east toward the south and west, with about 20 feet of total relief.

Photographs showing general conditions of the site at the time of our field activities are shown below.

View looking south along the eastern property margin from near boring B-02.

View looking north along the eastern property margin from near boring B-02.

View looking southwest from near boring B-01 in the northeast part of the site.

2.2 Site Geology

The Geologic Map of the Nashville West Quadrangle, Tennessee (Tennessee Division of Geology and dated 1966), shows the site is underlain by the Hermitage Limestone Formation, but the site is less than a few hundred feet west of Quaternary-aged Alluvium, which are relatively thick layers of clay and sand deposited by the Cumberland River.

The Hermitage Formation typically consists of a medium to dark, bluish-gray, thinly bedded limestone with interbedded layers of shale. The formation weathers to produce residual soil layers that are typically clays, silts and sands. The residual soils often grade with increasing depth from silts and clays to sands. The lower sandy soils often contain thin zones of high plasticity wet clays which are often termed “phosphates” by local contractors.

Limestone is susceptible to solution weathering and development of karst features, such as sinkholes. Although karst features are possible, the risk of karst features in the Hermatige is less than other limestone formations because the interbedded shale layers tend to inhibit the movement of water through the limestone. We did not observe indications of karst features or sinkholes at the site during our field activities, although existing improvements and heavy vegetation could have masked such indicators. The Geologic Map of the Nashville West Quadrangle did not indicate mapped depressions on the site but contained several large mapped depressions within 2,000 feet north, west, and south of the site. The scale of the map often precludes the mapping of smaller features.

Despite the geologic map showing the site within the boundaries of the Hermitage limestone formation, the borings encountered thick deposits of alluvial soil overlying bedrock. These soils can be relatively deep and are typically highly variable in thickness and lateral continuity because of repeated episodes of deposition and scour. Alluvial soils are often soft and wet because they have not been subjected to overburden pressures greater than the existing overburden pressures, and they can sometimes include ancient organic materials, such as leaves, sticks, or even logs and tree trunks.

Alluvial deposits can also contain “floating” boulders or layers of gravel and cobbles within the alluvial matrix well above the underlying bedrock level.

Existing fill or possible fill was encountered above the underlying deposits by two borings. Fill material is typically soil, but may include rock particles, placed by the actions of man. Fill can be problematic for site development when it has not been compacted in thin lifts. Uncompacted or poorly compacted fill can be a source of unpredictable and excessive settlements or other measures of poor structural performance. Fill that has been placed without engineering observation or documentation can sometimes contain objectionable inclusions or constituents, such as fibrous organic pieces (tree trunks or brush piles), junk and debris, trash, excessively wet or high plasticity soils, or large rock boulders. When such undesirable inclusions are present, the consistency or density of the fill cannot necessarily be correlated with conventional indicators, such as drive-sample blow counts or estimates of unconfined compressive strength of cohesive soils. For this reason, consistency descriptions of fill layers are typically not included on boring logs.

2.3 Subsurface Stratigraphy

2.3.1 Soil and Rock Conditions

The exploration included drilling six exploratory borings (labeled B-01 through B-06) at the approximate locations shown on the Exploration Location Plan in Appendix A. Three of the borings were extended below auger refusal depths by rock coring to evaluate the composition and quality of the refusal materials. The field exploration and laboratory testing methods are described in Appendix B. Soil descriptions follow the Unified Soil Classification System (USCS), which is described in ASTM D2487 and D2488. A Generalized Subsurface Profile showing soil and rock symbols and some sampling data from the borings, as well as other information, is included in Appendix A. The profile uses approximate elevations interpolated from topographic contours on the provided grading plan drawing.

Additional information about the subsurface stratigraphy encountered at the boring locations is provided on boring logs in Appendix A. The boring logs represent our interpretation of the subsurface conditions at the test boring locations based on tests and observations performed during the drilling operations, visual classification of the soil and rock samples by a geoprofessional, and laboratory tests conducted on the recovered soil samples. The lines designating the interfaces between various strata on the boring logs represent the approximate strata boundaries; however, the transition between strata may be more gradual than shown, especially where indicated by a broken line. Conditions may vary at locations away from or between the boring locations. Information about rock core samples is also provided on Rock Core Photographs sheets in Appendix A. The rock core photographs include tables summarizing the coring data, percent Recovery, RQD, and a qualitative assessment of rock quality based on RQD.

A summary of the conditions encountered by the borings is presented in the table below.

Stratum	Approximate Thickness	Material Description	Stratum Properties
Surface Material	2 inches to 3 inches	Topsoil	Not Applicable
Fill and Possible Fill (Borings B-01 and B-02 only)	3 feet	Lean Clay (USCS - CL), black, with trace limestone gravel (fine) and fine roots, dry to moist *(Excessive organics, trash, debris, and junk were not encountered in the fill at the borings.)*	N-values: 6 bpf and 8 bpf
Upper Clayey Alluvium	13 feet to 30 feet	Mostly Lean Clay (USCS - CL) with some layers of Sandy Lean Clay (USCS – CL), mostly stiff to very stiff, brown, brown and gray, tan, and tan and gray, with zones of black mineral staining, moist	N-values: 5 bpf to 27 bpf, but most between 9 bpf and 19 bpf MC: 11% to 23%, most between 15% and 22% LL: 27 to 37 PI: 9 to 17 -200: 60% and 76%
Sandy and Gravelly Alluvium	5 feet to 15 feet	Mostly Sandy Lean Clay (USCS – CL), soft to stiff, brown or tan, moist, with layers of Clayey Sand (USCS – SC) and Clayey Gravel (USCS – GC), loose, brown or tan, moist to wet	N-values: Most from 3 bpf to 9 bpf MC: 19% and 20% LL: 27 PI: 10 -200: 53% and 64%
Lower Clayey Alluvium	14 feet to 25 feet	Mostly Lean Clay (USCS – CL), soft to stiff, brown or tan, moist to wet, with layers of Sandy Lean Clay (USCS – CL) and Clayey Sand (USCS – SC) Limestone cobbles and boulders were encountered in layers less than 2 feet thick between depths of about 45 feet and 51 feet at borings B-05 and B-06	N-values: 2 bpf to 10 bpf MC: 29% to 45% -200: 64%
Bedrock (based on borings B-04, B-05, and B-06)	Depths to top of competent bedrock ranged from 42 feet to 58 feet. Approximate elevation of top of competent bedrock ranged from about 386 feet to about 384 feet.	Limestone, hard, gray and dark gray with black bands, fine- to medium-grained, thin-bedded, intensely to moderately fractured, slightly weathered to fresh	REC: 86% to 104% RQD: 77% to 98%

[1] Includes N-values of applicable samples, not including amplified N-values. MC = Moisture Content, LL = Liquid Limit, PI = Plasticity Index, and -200 = Percent Fines of select soil samples only. REC = Percent Recovered and RQD = Rock Quality Designation of rock cores entirely penetrating competent bedrock.

2.3.2 Shear Wave Velocity Profile

The shear wave velocity profile developed from our field measurements indicates velocities between 293 feet per second (fps) and 3,006 fps. The estimated shear wave velocities from our field measurements are tabulated below for convenience.

SHEAR WAVE VELOCITY PROFILE VALUES

Layer Top Depth (feet)[1]	Layer Bottom Depth (feet)[1]	Shear Wave Velocity (fps)[1]
0	4	293
4	12	1,444
12	37	965
37	48	1,546
48	70	2,899
70	100	3,006

[1] Rounded to the nearest foot and whole number value (fps).

The IBC and ASCE 7 use a velocity of 2,500 fps as the boundary between soil and bedrock, with lower velocities representing soil and soft, fractured, or weathered rock layers and higher velocities representing bedrock layers. Based on that criterion, the shear wave velocity profile measurements suggest the boundary between soil/soft rock and bedrock below the site occurs near depths of about 48 feet below existing grades along the array shown in the Exploration Location Plan in Appendix A. The velocity model is a one-dimensional model based on average or aggregated behavior along the entire length of the geophone array used to obtain the field measurements. Consequently, the depth to bedrock could vary below the array and may be shallower or deeper than implied by the velocity profile.

2.4 Groundwater Conditions

Groundwater was encountered in the borings between depths of 18 feet and 46 feet below ground surface while drilling. Groundwater levels measured at the completion of drilling but before rock coring ranged between 18 feet and 46 feet below ground surface (corresponding to elevations between about 422 feet and 405 feet). Water levels measured after completion of coring ranged between 12.5 feet and 25 feet below ground surface (corresponding to elevations between about 415 feet and 416 feet). We believe some of the measured water levels, but not all of them, were affected by water introduced into the boreholes while coring. We consider the groundwater level indicated by the boring data was likely at or below approximately elevation 416 feet. This interpretation is supported by soil moisture content data showing wet conditions at or below about elevation 410 feet and dry to moist conditions above that elevation. Observing boreholes for groundwater the day after drilling or at later times was not in our scope of services.

Groundwater generally means a continuous water surface present below the ground surface year-round resulting from long-term accumulation of water above or between relatively impervious subsurface strata, such as clays or bedrock. The primary water source is usually from infiltration of surface water into the subsurface, but it can also come from lateral flows of subsurface water from adjacent aquifers. The groundwater surface, sometimes called the “water table,” can fluctuate up or down throughout the year due to seasonal changes in climate, precipitation, vegetation, surface runoff, water levels in nearby water bodies, and other factors. The groundwater level below the site may fluctuate up or down in response to such changes and may be at different levels than indicated on the exploration logs at times after the exploration.

Sometimes shallow temporary subsurface water conditions can develop as a result of above-normal rates of precipitation or surface runoff that exceed the rate at which the infiltration can pass through the subsurface strata. These temporary water levels are called “perched” or “trapped” water levels, and they can change rapidly over short horizontal distances and short durations of time. It is often not possible to distinguish between a temporary perched water level or the groundwater table based on one-time observations of water levels in open boreholes.

3.0 GEOTECHNICAL CONSIDERATIONS

The following geotechnical considerations are based on the data collected or developed during this project, our understanding of the proposed construction, and our knowledge of sites with similar surface and subsurface conditions.

3.1 Existing Fill

Existing fill or possible fill was encountered in the top 3 feet at borings B-01 and B-02. The fill encountered by the borings was of variable consistency and contained variable concentrations of limestone and chert gravel and fine roots. Direct observations made during fill construction in conjunction with performing in-place density tests (soil fills) is the industry standard for evaluating whether or not the fill material and compaction procedures meet the project specifications and are appropriate for the type of construction planned. Documentation of the fill placement or fill thickness is not available. We did not observe widespread indications of abundant organic or deleterious debris, trash, or junk in the fill at the borings. However, trace amounts of roots were noted, and these conditions could exist at other locations within the fill.

Based on our understanding of existing and proposed grades, it appears the existing fill encountered by the borings will likely be removed by excavating down to the planned building subgrade in that part of the site. However, fill can sometimes be variable in thickness or location and therefore may exist below the planned building subgrade in some parts of the building footprint. We generally expect the existing fill is probably not very much thicker than a few feet and can readily be undercut and replaced during site preparation without significant additional cost. Therefore, we recommend completely removing existing fill or possible fill and replacing it with compacted fill during site preparation. This approach avoids risks of future poor performance of foundations, slabs, or pavements constructed over existing fill. We should be contacted for additional recommendations if a significant volume of existing fill to be removed and replaced is discovered during construction. We suggest the project budget and schedule include a contingency for removing and replacing existing fill below the floor slabs and pavement areas.

3.2 Weak Subgrade Conditions

Near-surface fill and alluvial soils encountered by most borings have N-values lower than about 9 bpf and could be unstable when exposed to construction traffic. We recommend the project budget and schedule plan for undercutting as deep as 3 feet to 5 feet below final cut subgrades or existing ground surface below fill areas of the site. Borings B-05 and B-06 do not indicate weak near-surface conditions, although such conditions could exist between or nearby these areas.

3.3 Foundation Considerations

We recommend the additions be supported by deep foundations bearing on or in the competent limestone bedrock below the site. The generalized subsurface profile in Appendix A indicates the apparent top of competent bedrock suitable for support of deep foundation elements is at approximately elevation 384 feet.

Although many types of deep foundations are available, we recommend using groups of micropiles to support the building. Micropiles are commonly drilled using air-rotary percussion drilling equipment and can easily and cost-effectively penetrate limestone boulders and cobbles contained within the alluvium overlying the bedrock, while drilled shaft foundations will be difficult to install where boulders are encountered in the alluvium above the competent bedrock level. The borings encountered limestone cobbles and boulders at elevations above the competent bedrock. Borings B-01 through B-03 encountered refusal conditions above the apparent bedrock level, so we expect the potential to encounter such boulders and cobbles above the bedrock level across the site. Additionally, micropile foundation sizes and rock socket lengths can be adjusted to avoid excessive capacity with respect to anticipated foundation loads. Our recommendations for design and construction of micropile foundations for the additions are discussed later, in Section 6.0.

4.0 EARTHWORK RECOMMENDATIONS

4.1 Subgrade Preparation and Stabilization

4.1.1 Stripping and Demolition

Subgrade preparation should begin with demolition of existing pavements and sidewalks affected by the proposed construction, followed by clearing and grubbing of trees and stripping to remove organic-laden topsoil from planned construction areas.

- Stripping should extend 10 feet beyond construction limits.
- Organic-laden strippings should be removed from the site or disposed of at designated on-site areas located outside limits of current or future development.
- Strippings may be stockpiled for re-use as topsoil during landscaping if they are suitable for that purpose.
- Strippings should not be used to build permanent slopes.
- Existing pavement asphalt should be removed from the construction area and disposed of off-site.
- Existing pavement basestone may be left in place as a working surface, provided it is stable, but it should not be reused as basestone for new pavements or grade slabs.

Site preparation should also include removal of existing underground utilities that will not be retained and incorporated into the final project.

- Existing utilities should be completely removed, including bedding and backfill.
- Excavations resulting from demolition and removal of structures and utilities should be backfilled with compacted fill (see Section 4.3, below).

Existing fill should be completely removed during site preparation. The existing fill may be reused as compacted fill if it meets criteria given in Section 4.3 below.

4.1.2 Proofrolling

After stripping, the stability of exposed subgrades in areas to receive fill and the stability of subgrades exposed by cutting to final grades should be evaluated by proofrolling.

- Perform proofrolling with a rubber-tired vehicle having a gross vehicle weight of at least 20 tons (such as a loaded, tandem-axle dump truck).
- Proofrolling equipment should make multiple closely-spaced overlapping passes in perpendicular directions over the subgrade at a walking pace.
- The subgrade should be relatively smooth and free of wheel ruts, sheepsfoot roller dimples, loose clods of soil, or loose gravel, and the subgrade should not be desiccated, cracked, wet, or frozen at the time of proofrolling.
- A representative of the geotechnical engineer should observe the proofrolling to identify, document, and mark areas of unstable subgrade response, such as pumping, rutting, or shoving, if any.

4.1.3 Subgrade Stabilization

Depending on final grading and time of year construction takes place, we expect weak or unstable subgrades could be encountered across most of the site, although surface conditions at borings B-05 and B-06 appear to have greater stiffness than at other borings. The following methods are options for producing stable subgrade conditions depending on the nature of the unstable condition, the location and size of the unstable area, and the time available to address the unstable condition. Other subgrade repair considerations may be possible based on actual conditions encountered at the time of construction.

- Undercutting
    - This means simply excavating to remove the unstable soil conditions.
    - It is usually the most expedient and cost-effective means of dealing with unstable conditions when less than about 3 feet to 5 feet of undercutting is needed, although deeper undercutting may be justified when volumes are small.
    - Requires disposing of the excavated unstable soils and replacing the undercut excavation with new compacted fill.
    - It may be possible to improve the condition of the unstable materials that were undercut (usually by drying) so they can be reused as compacted fill in another part of the site.

- Bridging with Clean Shot-Rock Fill (Below Pavements Only)
    - This means placing a single lift of clean shot-rock fill thick enough that the surface can be made relatively stable by repeated passes of tracked construction equipment.

    - The thickness of the bridge lift needed to create a stable condition depends on the depth of unstable material. Generally, bridge lifts using clean shot-rock fill range from 2 feet to 3 feet thick.
    - We recommend covering the shot-rock bridge lift with a 4-inch- to 6-inch-thick layer of crushed mineral aggregate base to close-off openings in the surface of the shot-rock that could allow raveling of soil fill with future infiltration of water.

- Bridging with Geogrid/Geotextile and Crushed Stone (Below Pavements Only)
    - Place a biaxial geogrid (Tensar BX1100, or equal), a triaxial geogrid (Tensar TriAx TX-5, or equal), or a woven geotextile fabric (Mirafi HP270, or equal) over the unstable subgrade and backfill with a single lift of crushed stone (TDOT No. 57 or mineral aggregate base (MAB)). The type of geogrid or geotextile fabric and the thickness of crushed stone will vary with the nature of the unstable subgrade.
    - Generally, 1 foot or more of crushed stone is needed over the geogrid or geotextile fabric, and often it is necessary to use as much as 2 feet of crushed stone to stabilize especially weak subgrades. The crushed stone should be densified by repeated passes of a smooth-drum roller operating without vibration.
    - This approach should not be used below foundations or elements where future utility excavations will be deeper than the geogrid or geotextile fabric to avoid tearing the geogrid or fabric during utility installation.
- Scarifying and Recompacting
    - This means scarifying the subgrade to a depth of 8 inches to dry the soil, and then recompacting the scarified layer to recommendations given for compacted fill in Section 4.3 below.
    - This method is usually only used when the proofrolled subgrade ruts without significant pumping.

Weak subgrades identified in the portion of the building area where a slab-on-grade floor will be constructed or where pavements will be constructed should be undercut to stable materials. The boring data suggest undercut depths could be as deep as 3 feet to 5 feet below final subgrades.

Bridging is not an exact science but more of a trial-and-error approach in which a bridge lift thickness is tried over a small test section and then adjustments are made until the appropriate thickness needed to create a stable condition is achieved. Test excavations into the unstable subgrade may be needed to help establish an appropriate thickness for the bridge lift. We recommend a test section be constructed to confirm the proposed stabilization approach will produce the desired result before implementing the method across large areas. Test sections should typically be 40 feet to 50 feet long and at least 20 feet wide, but they can be larger or smaller if needed. Where bridging is performed, the bridge lift should be spread from the perimeter of the unstable area using low-pressure tracked equipment which should make multiple passes over the surface of the bridge lift until it is stable enough to support rubber-tired equipment.

4.2 Excavation Conditions

The existing fill and alluvial overburden soils encountered by the test borings can be excavated by conventional earthmoving equipment. Limestone boulders were encountered in two borings at depths much deeper than expected excavations, but boulders could still be encountered at locations between borings. Removing such boulders may require larger tracked equipment and possibly hoe-ramming to break down the boulders into smaller pieces. Based on our understanding of the planned construction, we do not anticipate excavations deep enough to encounter competent bedrock.

4.2.1 Temporary Slopes/OSHA Soil Types/Sheeting/Shoring

Temporary construction excavations less than 20 feet deep should be sloped or shored by the contractor in accordance with OSHA requirements. The on-site fill and alluvial soils within expected excavation depths appear to be OSHA Type B and C soils. OSHA requires temporary excavation slopes no steeper than 1-horizontal to 1-vertical (1H:1V) through Type B soils and no steeper than 1.5H:1V through Type C soils. The contractor's "competent person" should evaluate temporary excavations daily and determine the specific soil types and temporary slope or shoring measures necessary according to OSHA requirements. Temporary excavations taller/deeper than 20 feet must be designed specifically by a registered engineer and cannot be made based on OSHA soil types.

The contractor is responsibile for temporary shoring designs, the details of construction, and the maintenance and stability of adjacent surfaces, structures, and utilities. TTL assumes no responsibility for excavations, shoring, or job site safety, which are the sole responsibility of the general contractor.

4.2.2 Temporary Groundwater Control

The boring data generally indicate groundwater was below about elevation 416 feet (depths between about 12 feet and 25 feet below existing site grades) at the time of the exploration. The contractor should be prepared to lower groundwater levels prior to making excavations that extend below the groundwater level. This will require the contractor to first explore the excavation to determine the depth of groundwater at the time of construction so they can be prepared to lower groundwater levels, if needed. Localized zones of "trapped" or "perched" water can sometimes develop in the alluvial soil overburden or the existing fill, especially after extended wet weather. Groundwater inflow can normally be removed from construction excavations by pumping from a sump near the point of seepage. However, the alluvial soils at the site have high sand content and could allow for faster groundwater inflows than are otherwise typical for the Nashville area. Our geotechnical engineer should be contacted for guidance if heavy seepage occurs or there is evidence of soil particle migration. Detailed design of the temporary groundwater control measures for construction is the responsibility of the contractor.

4.3 Compacted Fill

Compacted fill is new fill material (typically soil, but can also include crushed stone and shot-rock) placed as backfill in undercut excavations and utility excavations, or placed to raise final site grades above existing site grades below slopes, pavements, and structures. Fill that is placed outside of current or proposed development areas is sometimes called "common fill" or "general fill." Materials that do not meet compacted fill requirements may sometimes be used in these “common” or “general” fill areas. In addition, materials that meet requirements for compacted fill may also be used in these areas. Junk, garbage, organic strippings, and other deleterious materials (which can decay, rot, or corrode over time) should not be used as fill in any site areas.

Criteria for fill characteristics, compaction procedures, and compaction control are listed in the table below. Fill placement and compaction should be observed by our representative on a full-time basis. We recommend low plasticity (PI<25) lean clays (USCS CL) be used as compacted fill. The existing fill appears to mostly consist of lean clay, but it also contains some clayey limestone gravel. We believe much of the clayey gravel existing fill can also be used as compacted fill if placed more than 4 feet below final subgrade elevations, but it should not be used shallower than 4 feet below final grades.

We recommend cutting benches (vertical excavations less than 2 feet high) into the sides of existing slopes steeper than about 4H:1V before placing fill against the slope. The benches should generally parallel the face of the existing slope so new fill can be placed and compacted in horizontal lifts of consistent thickness. We caution against cutting benches taller than 2 feet high or longer than about 100 feet because removing material from the toe of the existing slope could destabilize the slope surface and cause the slope to creep or slide, especially for steep slopes. The length and height of benches may also need to be adjusted if the existing slope was constructed by filling because placing and compacting fill at the face of a slope often results in soft or loose soil conditions.

If grading occurs during wet, cool weather, when drying soils is more difficult and time-consuming, the grading contractor may have difficulty achieving suitable moisture conditions for proper compaction of soil fill. As an alternative, mineral aggregate base (MAB) stone may be considered for use as fill. The MAB will not be as moisture sensitive as soil, but some weather delays may still be experienced if MAB is utilized as fill. Refer to the soil fill compaction procedures and compaction control listed in the table below for placing MAB.

Shot-rock fill materials can be used to backfill undercut excavations or as new fill below the building footprint, at least up to micropile cap and grade beam levels. Clean shot-rock fill materials can be placed without moisture conditioning and can typically be placed and compacted during wet, cool weather with little delay. These materials are also typically able to accommodate construction traffic with only limited subgrade degradation. Therefore, if grading occurs during wet, cool weather, consideration should be given to importing these materials for compacted fill.

Shallow excavations made within shot-rock fill may be significantly larger than similar excavations in soil due to the particle size of the rocks within the shot-rock fill. We recommend limiting the maximum rock particle size to 12 inches or less in the upper zone of shot-rock fill to reduce the size of utility and micropile cap and grade beam excavations through shot-rock fill. We recommend the contractor consult with the micropile contractor to determine if large shot-rock particles will adversely affect the installation of micropiles. If so, then we recommend limiting the maximum shot-rock particle to a size small enough to not interfere with micropile installation.

Material Type	Characteristics	Compaction Procedures	Compaction Control [1,2]
Lean Clay and Sandy Lean Clay	Maximum particle size – 3 inches Maximum gravel and oversize particle content – 15 percent retained on a ¾-inch sieve. Maximum allowable organic content– 3 percent by weight, but large roots should not be allowed. Liquid Limit: Not more than 50. Plasticity Index: Not more than 25.	Maximum loose lift thickness: 8 inches for ride-on equipment; 6 inches for hand-held or walk-behind/ remote controlled equipment. Compaction Requirement: Compaction should be to at least 95 percent of the standard Proctor maximum dry density (ASTM D 698). Moisture content at time of compaction: Within plus to minus 2 percent of the material's optimum moisture content	General Fill Areas – One moisture-density test every 5,000 square feet, with at least two tests per lift. Small Fill Areas – One moisture-density test for every 50 cubic yards, with at least two tests per lift. Field density tests of compacted soil fill shall be done using either the nuclear method (ASTM D6938), the sand cone method (ASTM D1556), or the drive-cylinder method (ASTM D2937), as appropriate for the fill materials. Proofrolling lifts of fill shall not be permitted as a means of evaluating compaction of soil fill for compliance with these recommendations.
Clayey Gravel or Clayey Sand	Maximum particle size – 3 inches Gravel and oversize particle content – not more than 30 percent retained on a ¾-inch sieve. Maximum allowable organic content– 3 percent by weight, but large roots should not be allowed. Liquid Limit: Not more than 50. Plasticity Index: Not more than 25.		
Mineral Aggregate Base (MAB)	Type A, Grading C or D in accordance with Section 903.05 of the Tennessee Department of Transportation (TDOT) specifications.		
Limestone Shot-Rock	Maximum shot-rock size: Not more than 18 inches. Percentage of soil: Not more than 10 percent by volume, and high plasticity clay is not allowed. Gradation: Adequate fines and smaller rock pieces to effectively "choke" the larger rock pieces by filling voids or open spaces between rock pieces.	Spreading: The larger rock pieces should lie flat and not overlap each other. Maximum lift thickness: Not more than 24 inches. Compaction Requirements: The fill should be compacted by making multiple passes with a CAT D8 bulldozer, or equal. The number of passes should be sufficient to demonstrate the material is densified and stable	A technician working under the direction of our geotechnical engineer should observe shot-rock fill placement and compaction techniques. The technician should document fill constituents, lift thickness and compaction techniques.

[1] For preliminary planning only, our technician/ engineer should determine the actual test frequency.

[2] In addition, the fill must be stable under the influence of compaction equipment. After the fill is placed and compacted, it will be advisable to limit the amount of heavy construction traffic on the soil subgrade.

If lifts of shot-rock thinner than the maximum lift thickness are required based on site grades, the maximum particle size of the shot-rock should be reduced to be at least 6 inches less than the lift thickness. We recommend placing a layer of MAB stone over shot-rock fill to close voids at the surface to reduce pathways for soil migration into the shot-rock over time.

The surface of any filled area can experience settlement due to compression of the underlying soils, and sometimes additional settlement results from consolidation of thick soil fills due to their own self-weight. We expect settlements of fills indicated by the provided grading plan should occur mostly during the course of construction. We should be notified to evaluate settlement potential for areas where deeper fills may be needed.

4.4 Drainage Considerations

4.4.1 Surface Water

The clay soils at the site are sensitive to elevated moisture contents. When dry, they may exhibit good strength characteristics and be relatively stable under moving rubber-tired equipment. However, when they are moist as a result of local precipitation and climatic conditions or exposure with depth of cut, they can become unstable, particularly under repeated loading from heavy construction equipment. This can occur even for soil that has been moisture conditioned and properly compacted. Protection of the prepared subgrades will be important with regard to the construction schedule and grading costs for this project. Drying the soil may not be practical during wet seasons of the year.

If possible, site development should be performed during seasonably dry weather (typically May through October), and excavation/site preparation should not be performed during or immediately following periods of heavy precipitation or freezing temperatures. Positive surface drainage should be maintained during grading operations and construction to prevent water from ponding on the surface. Surface water from off-site areas should be diverted around the site using berms or ditches. The surface can be rolled smooth to enhance drainage if precipitation is expected, but should then be scarified and moisture conditioned, as needed, prior to resuming fill placement. When work activities are interrupted by heavy rainfall, fill operations should not be resumed until the moisture content and density of the previously placed fill materials are as recommended in this report. Subgrades damaged by construction equipment should be promptly repaired to reduce potential for water ponding or further degradation in adjacent areas. Degradation of the exposed soils should be expected if they are subjected to freeze/thaw, excessive precipitation, or ponded water. These considerations generally do not apply to aggregate base or clean limestone shot rock fill.

4.4.2 Permanent Groundwater Control

Groundwater was encountered in the soil test borings, but the apparent elevation of the water table is deep enough below the proposed lower floors that we believe a permanent groundwater control system below the floor slabs is not necessary. This recommendation does not apply to retaining walls, which should include drainage zones as discussed in Section 5.2, below. We should be contacted to provide additional guidance if shallow groundwater conditions are present during construction.

4.5 Karst Considerations

The site is in a region that is historically susceptible to the development of sinkholes and other karst features. We did not observe existing surface depressions or other indications of possible sinkholes or other karst features at the site during our exploration, although existing improvements and ground cover could have masked such conditions. It is possible that sinkholes could develop at the site in the future. It has been our experience that new sinkholes are more likely to occur during site grading than afterward because incipient sinkholes not visible at the surface may appear in response to the natural drainage mechanisms being disrupted by removal of vegetation or altering of grades. Because sinkholes typically result from movement of water through the subsurface regime, it is important to reduce the quantity of surface water allowed to infiltrate the building addition areas. The recommendations below are provided to reduce the potential for sinkhole development as a result of construction activities:

- Control storm water drainage by properly grading the site to promote complete and rapid runoff of surface water away from construction areas and avoid the ponding of water in open excavations.
- Locate detention/retention ponds as far as practical from buildings, roads, or utilities.
- Construct underground plumbing systems in a leak-proof manner.
- Provide ditches or pipes for discharge of storm water, to the extent practical.
- Evaluate areas of suspected sinkhole development, such as abnormally thick topsoil deposits, depressions, and locations of soil collapse or voids within the overburden.
- Where sinkholes or incipient sinkholes are detected, perform remedial treatment as recommended by our geotechnical engineer, based on the actual conditions encountered.

5.0 INFRASTRUCTURE RECOMMENDATIONS

5.1 Buried Utilities

We recommend trenches for buried utilities with lateral dimension of 6 inches or more be at least 24 inches wider than the width or diameter of the utility to provide room on either side of the utility to place and compact backfill beside the utility. We recommend a minimum trench width of 18 inches for utilities smaller than 6 inches diameter to provide sufficient width for compaction of backfill. The depth of the trench will depend on the type and size/diameter of the utility being installed: pipes and conduits larger than 6 inches typically require a bedding layer of sand or gravel below them to make it easier to maintain the design grade and slope of the utility and to provide bearing capacity to support the utility as backfill is compacted above it, while wire or cable utilities can often be placed directly on the bottom of the excavated trench. Where bedding is required below and beside the utility, the type and thickness of the bedding is typically determined based on the size, shape, and material of the utility. We recommend bedding materials consist of either clean crushed stone (TDOT No. 57 or No. 67 stone), dense graded aggregate (TDOT MAB – see Section 4.3 of this report), or sand (USCS SP, SW, SP-SM, or SW-SM), or as specified by Metro Nashville requirements. The thickness of the bedding layer below the bottom of the pipe/conduit should be at least ½ of the width/diameter of the pipe/conduit, with a minimum thickness of 6 inches. We recommend the bedding layer extend up to at least 6 inches above the top of the pipe/conduit to provide a level and compacted subgrade to support compaction of the remaining backfill. Inadequate compaction of trench backfill can lead to excessive settlement of the backfill and premature distress of overlying pavements, slabs, or structures. Therefore, we recommend the following:

- Whenever possible, trench and install utilities prior to placing slab-on-grade foundations, mats, or other surface treatments.
- Place, moisture-condition, and compact the backfill in accordance with the compaction recommendations outlined in Section 4.3 of this report.

If free-draining backfill, such as clean sand or clean crushed stone, is used as bedding and/or trench backfill, we recommend providing a gravity drainage outlet for the bedding and backfill to prevent the build-up of water that may infiltrate into the backfill from the surface or that may enter the backfill through leaky pipes. Preventing the build-up of water in the backfill is important to reduce the potential for softening of the clay soils at the bottoms and sides of the utility trenches. We also recommend providing a nonwoven, needle-punched filter fabric (Mirafi 140N, or equal) between the free-draining backfill and the surrounding soils to reduce potential for loss of soil into the backfill over time. We recommend overlapping the ends and edges of the filter fabric per the manufacturer's recommendatioins with a minimum lap of 1 foot.

We recommend considering flowable fill as an alternative backfill material for bedding and backfill layers. When designed with a maximum unconfined compressive strength less than about 200 pounds per square inch (psi), the flowable fill can be excavated at a later date, if required. Flowable fill will harden and will not settle which should prevent damage to overlying pavements or floor slabs. It is faster to place than soil or stone, which require compaction to reduce potential settlements. And since it is not necessary to compact the flowable fill, it can be placed in narrower trenches than are necessary when placing soil backfill. We recommend a minimum trench width of 8 inches wider than the pipe/conduit being placed when backfilling with flowable fill. The minimum trench width for wire or cable utilities that do not require a bedding layer can be 2 inches wider than the wire or cable, but we still recommend a minimum width of 6 inches. A filter fabric between the bedding and the surrounding soil is not necessary when using flowable fill as bedding.

5.2 Retaining Walls

We expect cast-in-place concrete retaining walls will be used along basement walls for the building, and we presume mechanically stabilized earth (MSE) walls will be used as site walls.

5.2.1 Concrete or Structural Masonry Walls

Cast-in-place concrete or reinforced masonry retaining walls should be designed using the earth pressure recommendations below.

Backfill Material	Total Unit Weight, pcf	Ultimate Passive Earth Pressure Coefficient, k_p*	Ultimate At-Rest Earth Pressure Coefficient, k_o	Ultimate Active Earth Pressure Coefficient, k_a
Compacted Lean Clay (CL)	120	2.22	0.60	0.45
Clayey Gravel (GC)	130	3.22	0.47	0.31
TDOT No. 57 Stone	100	3.70	0.43	0.27
Shot-rock**	145	4.55	0.36	0.22

*Passive pressure should be neglected in the upper 2 feet below grade unless the grade is covered by a floor slab or pavement that will reduce the potential for the soil to shrink away from the wall or footing. We recommend dividing the computed passive force by a factor of 2 to help reduce potential lateral deflections needed to develop the passive resistance.

**Shot-rock used as retaining wall backfill should have a maximum particle size of 8 inches.

The parameters above are subject to the following requirements:

- Use the at-rest earth pressure condition if the top of the wall is restrained against rotation or if rotation of the wall is not desired.
- Use the active earth pressure condition if the wall is free to rotate outward at least 1 percent of the height of the wall.
- The zone of backfill behind the wall extends upward from the back of the retaining wall foundation at a slope of 1H:1V, or flatter.
- The grade behind the top of the wall will be horizontal. Different geometry behind the wall will produce different earth pressures, and sloping backfill will generally increase the earth pressures applied to retaining walls.
- The earth pressure coefficients can also be used to estimate the increased earth pressure from uniform surcharge loads on the backfill behind the walls.
- Hydrostatic pressures are not included in the earth pressure coefficient or unit weights.
- Seismic forces are not included in the earth pressure coefficients or unit weights.
- Lateral and overturning stability of the retaining wall should include a factor of safety at least 1.5 or as required by the building code or local codes.

We recommend providing a drainage zone behind the wall to collect and drain groundwater or surface water infiltration from behind the wall. The drainage zone should meet the following requirements:

- It should consist of TDOT No. 57 clean crushed stone at least 1 foot wide behind the wall, extending from about 1 foot below the top of the wall down to the top of the wall footing.
- It should be separated from the retaining wall backfill material by a non-woven needle-punched geotextile filter fabric (Mirafi 140N, or equal). Ends and edges of the geotextile sheets should overlap at least 1 foot to help prevent gapping open at joints. If clean crushed stone is used as backfill behind the wall, the filter fabric should be placed between the backfill and the sloping soil subgrade instead of 1 foot behind the wall stem within the crushed stone.
- A perforated plastic collector pipe (at least 4 inches diameter) should be provided at the base of the drainage zone to collect water from the zone and drain it from behind the wall via gravity to a suitable daylight outlet away from buildings or pavements. It may be feasible to connect the wall drains to storm drains nearby, or, where possible, through weep holes through the face of the wall. We recommend all daylight outlets of drains include rodent guards to prevent animals from nesting in the pipes and clogging them.
- Basement walls should be waterproofed.

Shallow foundations for site retaining walls (not basement walls or walls that are part of the building) should be designed and constructed using the recommendations given in Section 6.2 below.

5.2.2 MSE Walls

Where these walls can be used, they usually are less expensive than concrete walls. However, MSE walls must deflect laterally to develop the active earth pressure condition and to engage the geogrid reinforcing layers which provide the stability for the wall. Locally available soil compacted fill (clay soils from on site or off site) cannot be used within the reinforced zone behind the wall because it can take several months or years for the clay soils to fully develop the active earth pressure condition, and the movements occurring over that time frame can be detrimental to retained grades, structures, or pavements. Therefore, MSE walls should be designed and constructed with free-draining backfill materials, typically TDOT No. 57 crushed stone, in the Reinforced Backfill zone, which is the zone of backfill containing the geogrid reinforcing layers. Soil fill, or TDOT No. 57 stone, or clean shot-rock can be used in the Retained Backfill zone, which is the fill zone behind the ends of the geogrid reinforcing layers, or directly behind the wall facing blocks when geogrids are not needed for stability. We recommend the following design parameters for these types of walls:

Backfill Material	Total Unit Weight, pcf	Effective Friction Angle (phi), degrees	Effective Cohesion, psf
Compacted Soil Fill (for Retained Backfill Zone Only)	120	22	0
Shot-Rock Fill (for Retained Backfill Zone Only	140	40	0
TDOT No. 57 Stone (for Reinforced and Retained Backfill Zones)	100	35	0

Additionally:

- The design parameters above do not account for hydrostatic pressures from water behind the walls because the TDOT No. 57 stone used in the Reinforced Zone behind the wall face will serve as the drainage material.
- The stone should be separated from the Retained Backfill material by a non-woven needle-punched geotextile filter fabric (Mirafi 140N, or equal). Ends and edges of the geotextile sheets should overlap per manufacturer’s recommendation, but at least 1 foot to help prevent gapping open at joints.
- A perforated plastic collector pipe (at least 4 inches diameter) should be provided at the base of the Reinforced Zone behind the face of the wall to collect and drain water from behind the wall via gravity to a suitable daylight outlet away from buildings or pavements. It may be feasible to connect the wall drains to storm drains nearby, or where possible, through weep holes through the face of the wall. We recommend outlets of drains include rodent guards to prevent animals from nesting in the pipes and clogging them.
- The MSE wall designer should confirm their wall design meets global stability, including sloping surfaces present above and in front of the wall.

Civil design using MSE walls should account for the specific intricacies of the MSE systems available. In particular, most MSE wall facing materials are specially-shaped dry-stacked concrete masonry units or concrete blocks that are designed to provide a sloping (battered) face. The face batter angle varies between manufacturers, but a slope of 1H:8V is common (most MSE masonry units are 8 inches high and use a 1-inch setback per course). For short walls, less than about 6 feet tall, the batter may not present a problem for design or construction, but for taller walls the batter can cause the top of the wall to be more than 1 foot behind the bottom face of the wall. If not accounted for in site layout and grading, the batter can sometimes encroach into the space behind the top of the wall which can interfere with other elements of the site like guardrails, fencing, or pavements. Also, the batter can make "outside" corners or tight "outside" radii difficult to achieve without saw-cutting the units, which adds to the cost by slowing down the construction. Outside corners or curves are those where the angle between straight parts of the wall on either side of the corner or curve measured on the outside of the wall, meaning the wall face, is greater than 180 degrees. We encourage civil design of MSE walls use a radius of 10 feet, or more when possible, for outside curves.

The MSE walls depend on the geogrid reinforcing layers within the reinforced backfill for stability, so these layers cannot be removed or cut by utility installations, light pole foundations, or other underground structures or utilities after the walls have been constructed. Civil design should strive to avoid utilities below or closer behind the top of wall than two times the height of the wall. Where utilities, especially storm drainage pipes, must cross through the reinforced zone behind the wall, the utility should be oriented perpendicular to the face of the wall as much as possible to reduce the interference with geogrid layers. Storm drainage structures, such as curb inlets, yard inlets, or manholes, should not be located within the Reinforced Backfill because the backfill and geogrid layers will move which can shift the drainage structures and cause leaks that can lead to saturation of fill and foundation materials or that can overwhelm the wall drainage system and lead to unanticipated hydrostatic pressures within the wall.

Civil design of MSE walls along cut sections of the site near property boundaries must consider that geogrid reinforcing layers extend well behind the wall face. Cut walls must be located far enough from the property or construction limits to allow room for the geogrid layers and for the temporary excavation slope or shoring needed behind the ends of the geogrid to permit construction. We suggest locating the face of MSE walls in cut situations at least two times the exposed wall height away from the property boundary. Where permanent slopes will be constructed above MSE walls, the surcharge weight from the slope often requires longer-than-usual geogrid layers, so the wall face should be at least two times the total grade change produced by the wall-slope combination away from a property line behind the wall.

There are other design considerations regarding how MSE walls impact and interact with civil design requirements for a project. The MSE wall design is usually left to the contractor as a design-build process during construction. We encourage the designer and owner to consider having MSE walls designed as part of the Construction Document process so retaining wall design drawings and construction specifications can be bid by the contractor. This should provide more competitive bidding for the wall construction, and it will allow for better coordination of the MSE wall design with the civil design for the project.

5.3 Pavements

Specific pavement loading and design information has not been provided. We have assumed light-duty pavements will be used by passenger cars, SUV's, delivery trucks, and occasional garbage trucks, with infrequent heavy truck traffic. We expect asphalt pavements will be used in surface parking areas. We expect concrete pavement will be used for dumpster pad areas and for ground level parking under the building. We recommend dumpster bins be placed on a concrete pad long enough to support the bins and truck. Otherwise, a punching shear failure could develop in front of the bins due to the high wheel stresses generated by the trucks during waste transfer. The recommendations provided below are based on minimum pavement sections typically used in Middle Tennessee and our experience with pavements in similar conditions. All pavement layers should be sloped to promote drainage away from pavement and building areas and reduce the potential for standing water on pavement subgrades.

5.3.1 Flexible (Asphalt) Pavements

We recommend using the flexible pavement section thicknesses in the table below. If these layer thicknesses are less than minimum sections required by local codes or ordinances, the local minimum sections should be used.

Pavement Use	Layer Thickness		
	Asphalt Surface Course	Asphalt Binder Course	Mineral Aggregate Base[1]
Light-Duty (No Trucks)	2.5 inches	-	8 inches
Medium Duty (No more than 3 trucks per week)	2.0 inches	2.5 inches	8 inches

[1]Mineral aggregate base should be compacted to 100 percent of the maximum dry density as determined by the standard Proctor (ASTM D698).

5.3.2 Rigid (Concrete) Pavements

We recommend using the rigid pavement section thicknesses in the table below. If these layer thicknesses are less than minimum sections required by local codes or ordinances, the local minimum sections should be used.

Pavement Use	Layer Thickness	
	Portland Cement Concrete	Mineral Aggregate Base[1]
Medium Duty	5 inches	4 inches
Heavy-Duty	6 inches	4 inches
Dumpster Pad	8 inches	4 inches

[1]Mineral aggregate base should be compacted to 100 percent of the maximum dry density as determined by the standard Proctor.

The concrete for rigid pavements should have a 28-day compressive strength of at least 4,000 psi. Exterior concrete exposed to rain and snow should also contain entrained air to improve durability. The air content should be compatible with the maximum aggregate size and the project location. The concrete pavement should be designed and constructed in accordance with applicable ACI guidelines, including joint spacing. Additional considerations for pavement design and construction are provided below:

- Construction joints should be sawed as soon as the concrete will allow. The joints should be subsequently sealed to reduce surface water infiltration into the prepared subbase.
- Construction joints (excluding saw joints) should be underlain by a non-woven geotextile (about 2 feet wide) to reduce the potential for the upward movement of soil fines through the joints.
- Loading (traffic) should not be allowed until the concrete has achieved at least 85 percent of its design strength.

5.3.3 General Pavement Recommendations

Pavement recommendations in this report do not account for construction traffic. We recommend construction traffic not be allowed on asphalt pavement layers. If desired, construction traffic can use mineral aggregate base (MAB) layers as long as the MAB is re-evaluated and repaired prior to placement of asphalt. Repair may require removal of some, or all, of the MAB if it has become contaminated with soil. The soil subgrade may also require repair consisting of undercutting and replacing with compacted fill or additional MAB. Subgrade repairs needed as a result of construction traffic on the MAB should not result in extra charges to the Owner as the use of pavement subgrades for construction traffic falls under the contractor's means and methods of construction.

Site grading is generally accomplished early in the construction. The subgrade should be evaluated at the time of pavement construction by proofrolling with a loaded, tandem-axle dump truck. Particular attention should be given to high traffic areas that were rutted and disturbed earlier and to areas where backfilled trenches are located. Areas where unsuitable conditions are detected should be repaired by removing unstable materials and replacing with compacted fill. Maintenance is essential to good, long-term performance of pavements. Distressed areas should be promptly repaired. Cracks and joints should be sealed annually with a heavy-duty sealer to accomplish site grading.

6.0 STRUCTURAL RECOMMENDATIONS

6.1 Seismic Site Classification

Based on the 2021 edition of the International Building Code (IBC), the results of shear wave velocity profile testing, and our interpretations of the site conditions, we recommend Seismic Site Class C for seismic design of the building additions.

6.2 Foundations

The existing fill and alluvial soils encountered at the site should not be used to directly support heavily-loaded foundations for the building because of potential for excessive foundation settlements. We recommend the building be supported by deep foundations consisting of groups of micropile foundations bearing in competent bedrock. Lightly-loaded incidental equipment, such as isolated light poles or equipment pads, such as for transformers or HVAC equipment, and for site retaining walls separate from the building and not more than 8 feet tall can be supported on shallow foundations bearing on alluvial soils using a low bearing pressure. Recommendations for both foundation types are given in the following sections.

6.2.1 Shallow Foundations

The table below summarizes design considerations for shallow foundations.

Design Considerations	Value
Suitable bearing soil	Stiff lean clay alluvium or new compacted fill.
Allowable net bearing pressure for sustained and transient loads (no increase for wind or seismic loads)	1,500 psf
Minimum bearing depth below exterior grade	1.5 feet
Minimum footing sizes	2 feet for isolated spread footings 1.5 feet for continuous wall footings
Maximum footing sizes	6 feet for isolated spread footings 3 feet for continuous wall footings
Expected total foundation settlement	Less than 1 inch
Maximum expected differential settlement between new shallow foundations	Less than ½ inch*
*(Differential settlement between shallow foundations and micropile-supported building will approach the total settlement of the shallow foundations.)	
Ultimate coefficient of friction between concrete and bearing soil for lateral load resistance	0.35
Minimum factor of safety for lateral resistance from friction	1.5
Ultimate passive pressure from soil against vertical face of footing for lateral load resistance (Do Not Use if footing is formed)	250 psf per vertical foot Neglect resistance in top 1 foot unless ground surface is protected by floor slab or pavement
Minimum factor of safety for lateral resistance from passive soil pressure	2.0
Total unit weight for backfill over footings for computing ultimate uplift resistance	100 pcf
Minimum factor of safety for uplift resistance from soil backfill weight	2.0

Below are construction considerations for shallow foundations:

- Foundation bearing surfaces should be level and free of loose or soft soil, unsuitable material (such as debris, trash, etc.), ponded water, or desiccation cracks.
- Foundation excavations should be backfilled with concrete the same day the footing excavations are open.
- If footing excavations are left open for more than a day, the contractor should protect the bearing materials against degradation from exposure. One method commonly used is placing a “mud-mat” of lean concrete, but other similar methods may also be used. Protective layers should not be placed until after the bearing surfaces have been evaluated to confirm they are suitable for the design bearing pressure.

- Surface water should not be allowed to flow into footing excavations. Water that enters the excavation, either from surface flow, precipitation, or other sources, should be promptly removed, even if the bearing level is covered by a protective layer, like a mud-mat.
- Footings should be poured directly against the sides of the excavation. If the footings are formed, the excavated space around the finished footing should be cleaned of soft or loose soil and then backfilled as soon as practical using soil compacted to the requirements for compacted fill given in Section 4.3 above.

Our geoprofessional should observe the materials exposed at the footing support level to check whether or not the exposed soils can support the foundation. Undercut excavations to remove weak or unsuitable soils should be backfilled with lean concrete or flowable fill.

6.2.2 Micropile Foundations

Individual micropile foundation elements may consist of a single central steel bar (typically a threaded bar, like a Dywidag bar) encased in neat cement grout (comprised only of cement and water without fine or coarse aggregates) installed within a small-diameter hole (typically not more than 10 inches) that is usually drilled with rotary percussive drilling equipment. Other designs utilize a high-strength (80 ksi yield stress is typical) steel pipe encased in grout instead of or in conjunction with a central threaded bar. Micropile foundations derive their capacity from adhesion developed between the grout backfill and the adjacent soil or rock. Micropiles should be installed by a specialty geotechnical contractor experienced with this method of construction. The detailed design of the micropile foundation elements is typically performed as a design-build contract with the micropile contractor because the contractor's specific means and methods (equipment and procedures) often result in the most competitive pricing compared to requiring various contractors to bid a standard design.

We recommend the following for design of micropiles for the project.

Item	Value
Ultimate Bond Stress between Grout and Soil or Weathered Limestone	Neglect
Ultimate Bond Stress between Grout and Competent Limestone	200 psi
Ultimate Bond Stress between Grout and Central Steel Casing	300 psi
Minimum Grout Thickness Around Micropile Central Reinforcement	1 inch
Allowable End Bearing Pressure for Axial Compression	Neglect
Highest Apparent Elevation of Competent Rock	384 feet
Minimum Length of Competent Rock Bond Zone	5 feet
Minimum Diameter of Micropile Drilled Hole	4 inches
Minimum Center-to-Center Spacing of Micropiles in Groups	Greater of 3 diameters or 16 inches
Reduction Factor for Axial Compressive Capacity of Micropiles in Groups	1.0
Reduction Factor for Axial Uplift Capacity of Micropiles in Groups	0.75

We typically expect micropile lengths may vary from column to column or along wall foundations because the top of competent bedrock elevation can vary. Data from the borings at the site suggest the elevation of the top of competent rock is relatively consistent at about elevation 384 feet. We recommend using this elevation for bidding and design and to help evaluate as-built conditions. We noted there is potential for shallow limestone boulders and cobbles to be present at shallow depths above the general bedrock level, so we recommend extending micropiles to about elevation 384 feet before starting to measure the rock socket embedment into competent rock. The specialty contractor performing the design-build of the micropiles should take the varying subsurface conditions into account in their design and should be prepared to lengthen micropiles or add piles, if needed, in response to specific conditions at each foundation location. TTL should be authorized to provide additional services to review the contractor's design submittal for correct interpretation and implementation of our recommendations.

Groups of micropiles without lateral bracing from grade beams should contain at least three micropiles, while groups with lateral bracing from grade beams can contain as few as one micropile or two micropiles. Considering the maximum foundation load of 500 kips assumed in Section 1.1 of the report, and assuming a group of three micropiles so lateral bracing from grade beams is not required, the apparent average compressive load per micropile would be about 167 kips. To support this load, a single micropile with a rock socket diameter of 6 inches would need to be embedded at least 7.5 feet into competent bedrock (using the design parameters above). Similarly, a rock socket diameter of 8 inches would need to be embedded at least 5.5 feet into competent rock to support the average load.

The lateral capacities of groups of micropile foundations, or the response of the micropile groups in terms of expected lateral deflections and moments, will depend on variables such as the number and arrangement of micropiles in the group, the design (diameter, length, bar size, casing, grout strength, etc.) of the individual micropiles, and the specific subsurface conditions at those locations required to resist lateral loads. Consequently, analysis of micropiles to resist lateral loads was not performed as part of this exploration. The design-build specialty contractor should provide lateral load analyses to confirm adequate lateral capacity and acceptable lateral deflections as part of the foundation design. If needed, micropiles can be battered (installed at an angle) and/or can include larger-diameter steel casing to provide additional lateral capacity, although this can add to the cost and time required to install the foundations. Once preliminary design of micropile foundations is complete, we should be authorized for additional services to review those critical and typical foundation groups to confirm the lateral load capacities.

Because the limestone may contain open voids, uncased micropiles through the voids could require much larger volumes of grout to fill the micropile hole than calculated based on the theoretical volume of the drilled hole. We recommend the micropiles be designed and installed with steel casing to penetrate the voids and allow grout to reach the deeper bearing strata without excessive grout loss. The use of steel casing for installation can also be accounted for in design of the micropiles. The casing can increase axial compressive capacity and lateral capacity of the micropiles, and it can provide additional resistance to prevent column buckling failure of micropiles that penetrate significant open voids, if any. The foundation installer should consider this possibility for bidding and design purposes. We recommend the design-build contractor advance several air percussion boreholes at the site to verify their depth assumptions once the preliminary design has been completed. The additional data can also be used to better estimate casing requirements and the likelihood of excessive grout takes during backfilling.

Pile load tests should be performed to verify the axial compressive and uplift capacities of the micropiles. Load tests should be performed on "sacrificial" test piles within the building footprint, but outside of actual foundation areas. The load test for compressive capacity should be performed on a different test pile than the one tested for uplift capacity. The test piles should be installed using the same materials and procedures that will be used for production piles. The installation of test piles and the load testing of the piles should be observed by our geotechnical engineer.

The procedures for conducting axial compressive and uplift pile load tests are outlined in ASTM D 1143. We recommend using the Quick Test procedure, with no more than a 1-hour hold at the maximum test load since the bedrock materials are not typically susceptible to creep displacements. Each test pile should be loaded in the same direction as the test is supposed to represent. Specialty contractors will possibly ask to perform the "compressive" load test as a tension test to avoid the cost of installing anchor piles and providing a compressive load test frame. We recommend this approach should not be allowed, as the purpose of the load test is to model the response of the pile to loads expected on the production piles supporting the structure.

The test piles should be loaded to at least 2.5 times the design load for the pile. At the contractor's discretion, the test piles can be loaded to "failure," which is generally considered as the inability to add load to the pile without excessive plunging or pullout of the pile. The purpose of "failing" the test pile would be to assess the apparent ultimate bond stress available from the site-specific conditions with the objective of modifying the design to reduce pile diameter, or pile embedment, or the number of piles to save cost and/or schedule. We should review the results of the load tests and any proposed design modifications for compatibility with the original design intent of the foundations.

Because site conditions are variable, we recommend the construction specifications include a requirement for the contractor to be prepared to conduct "proof" tests on production piles in case review of the installation records suggests the capacity of a pile may be in question. We suggest a budget for proof testing 1 percent of the production piles installed or a minimum of 5 piles, whichever is more. Conditions such as excessive grout volume, or erratic drill rates within the foundation zone, or delays in grouting that could create a cold joint within the grout are examples of installation irregularities that could be cause for proof testing. Proof testing should be performed as an uplift load test with a center-pull hydraulic ram to avoid the need for a large reaction frame and reaction piles. Proof tests should be conducted using the Quick Test method and should load the piles to 1.5 times the design load of the piles.

Considerable judgment and experience will be required to evaluate pile lengths relative to the materials needed to provide the design adhesion. The contractor selected for micropile construction should have at least 5 years of experience installing micropiles in geologic settings and subsurface conditions similar to those at this site. The installation of all micropile foundations should be observed and documented on a full-time basis by a TTL representative.

6.3 Floor Slabs

A concrete slab-on-grade floor is expected to perform satisfactorily provided the subgrade is prepared in accordance with the recommendations presented in this report. We recommend the slab-on-grade:

- Be placed on a 4-inch-thick layer of dense-graded crushed limestone aggregate compacted according to requirements of Section 4.3 of this report; and
- Be structurally separated from walls and columns and contain an appropriate number of control joints to accommodate differential movement that may occur.

Foundation and utility line installation, weather, and other construction activities can disturb a soil floor slab subgrade between completion of grading and slab construction. A TTL representative should observe a proofroll (see Section 4.1.2, above) of the soil subgrade immediately prior to placing the base material and slab concrete. Other methods, such as Dynamic Cone Penetrometer testing, should be used if proofrolling is not feasible. Unstable or soft areas should be undercut and replaced as recommended by our geotechnical engineer.

7.0 LIMITATIONS

This geotechnical engineering report has been prepared for the exclusive use of our Client for specific application to this project. The report has been prepared in accordance with generally accepted geotechnical engineering practices using that level of care and skill ordinarily exercised by licensed members of the engineering profession currently practicing under similar conditions in the same locale. No warranties, expressed or implied, are intended or made.

TTL understands this geotechnical engineering report will be used by the Client and various designers and contractors involved with the design and construction of the project. Individuals and companies receiving a copy of this report shall recognize it is for information only and cannot legally be relied on without first entering into a Secondary Client Agreement with TTL. We should be invited and authorized to attend project meetings (in person or teleconferencing) or to address applicable issues relating to the geotechnical engineering aspects of the project. TTL should also be retained to review the final construction plans and specifications to evaluate if the information and recommendations in this geotechnical engineering report has been properly interpreted and implemented in the design and specifications. This report has not been prepared as, and should not be used as, a design or specification document to be directly implemented by the contractor. The contractor and applicable subcontractors should familiarize themselves with this report prior to the start of their construction activities, contact TTL for any interpretation or clarification of the report, retain the services of their own consultants to interpret this report, or perform additional geotechnical testing prior to bidding and construction.

This geotechnical engineering report is based upon the information provided to us by the Client and various other individuals and entities associated with the project, exploratory borings drilled within the project limits, laboratory testing of selected soil samples recovered from the borings, and our engineering analyses and evaluation. The Client and readers of this geotechnical engineering report should realize that subsurface variations and anomalies can and may exist across the site and between the exploratory boring locations. Site conditions can change due to the modifying effects of seasonal and climatic conditions, such that conditions at times after the exploration may be different than reported herein. The nature and extent of such site or subsurface variations may not become evident until construction commences or is in progress. If site and subsurface anomalies or variations are encountered, TTL should be contacted immediately and authorized to evaluate such conditions and, if necessary, provide applicable recommendations.

Unless stated otherwise in this report or in the contract documents between TTL and Client, our scope of services for this project did not include, either specifically or by implication, any environmental or biological assessment of the site, or any identification or prevention of pollutants, hazardous materials or conditions at the site. If the Client is concerned about the potential for such contamination or pollution, TTL should be contacted to provide a scope of additional services to address the environmental concerns. Also, permitting, site safety, excavation support, and dewatering requirements are the responsibility of others.

Should the nature, design, or location of the project, be modified, the geotechnical engineering recommendations and guidelines provided in this document will not be considered valid unless TTL is authorized to review the changes and either verifies or modifies the applicable recommendations in writing.

Additional information about the use and limitations of a geotechnical report is provided within the Geoprofessional Business Association document included at the end of this report.

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will <u>not</u> likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will <u>not</u> be adequate to develop geotechnical design recommendations for the project.

Do <u>not</u> rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do <u>not</u> rely on an executive summary. Do <u>not</u> read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept* *responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are not final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

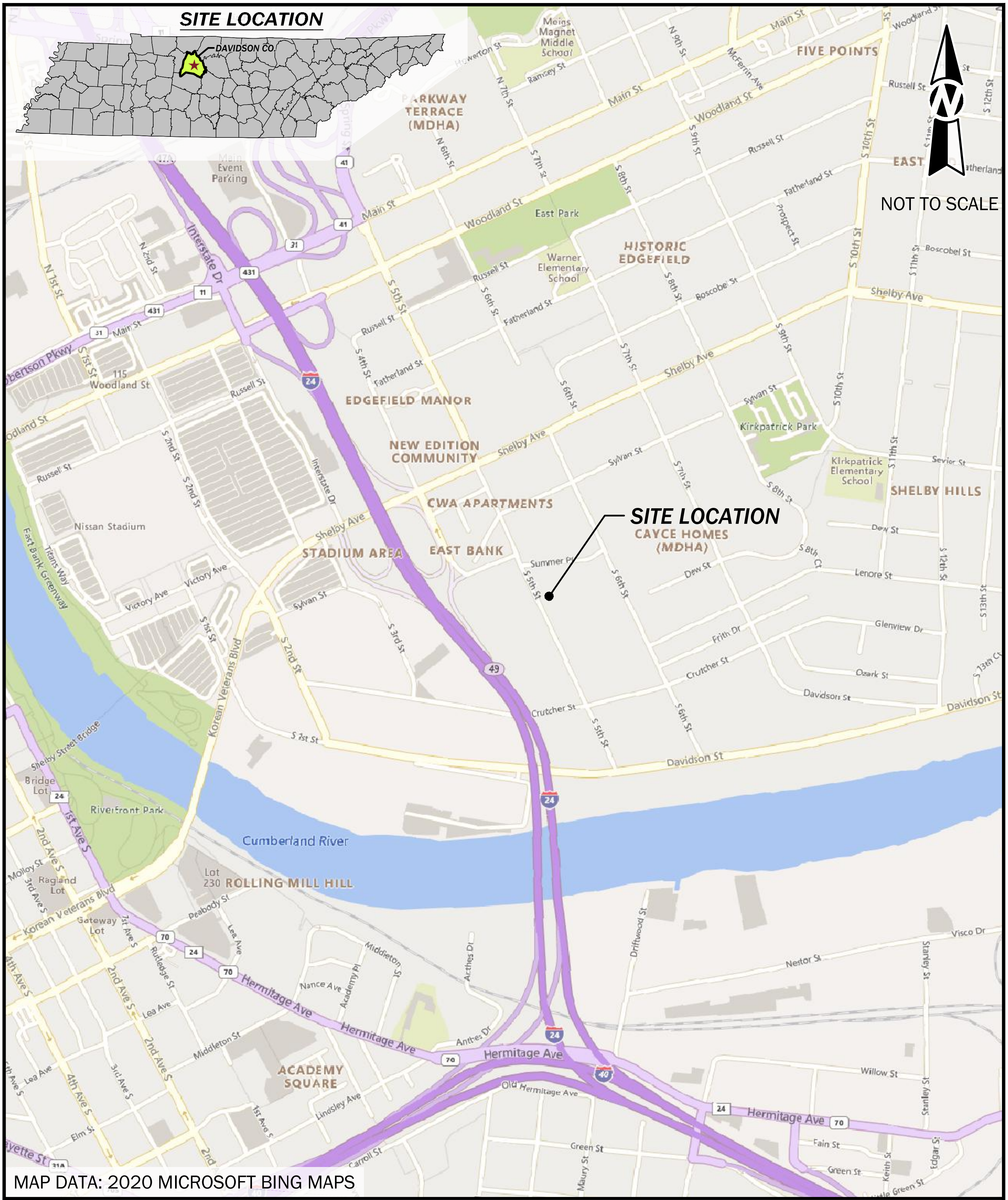
Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GBA GEOPROFESSIONAL BUSINESS ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

APPENDIX A
ILLUSTRATIONS



MAP DATA: 2020 MICROSOFT BING MAPS

624 Grassmere Park, Ste 14| Nashville, TN 37211
615.331.7770 | www.ttlusa.com

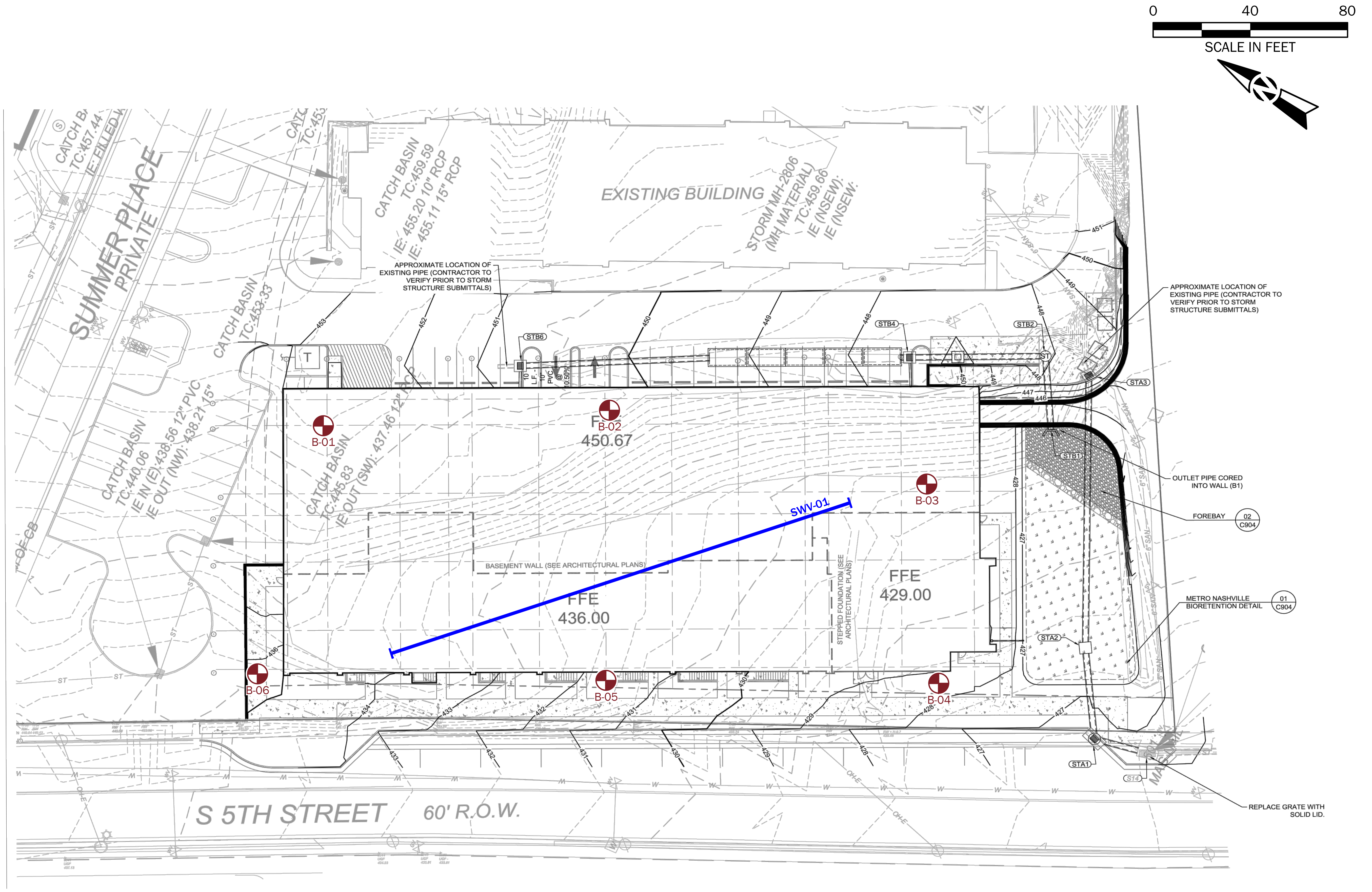
SOUTH 5TH STREET AND SUMMER PLACE

GRESHAM SMITH

SOUTH 5TH STREET AND SUMMER PLACE
NASHVILLE, DAVIDSON COUNTY, TENNESSEE

Drawn By: M H M
Checked By: D O B
Date: 01/09/2023
Proj. No.: 000220804344.00
File Name: 22.80.4344.SLM.dwg

Sheet Title
SITE LOCATION MAP



NOTES:

1. BORING LOCATIONS SHOWN ON THIS DRAWING ARE APPROXIMATE.
2. BORING LOCATION SYMBOLS ONLY REPRESENT THE LOCATION OF THE BORING AND DO NOT REPRESENT THE SIZE OR SHAPE OF THE ACTUAL BORING METHOD.

DRAWING SOURCE: "GRADING PLAN, 5TH AND SUMMER PLACE," SHEET C301, PREPARED BY GRESHAM SMITH, DATED JULY 26, 2022.

LEGEND

⊕ B-01	SOIL BORING LOCATION AND IDENTIFIER
SWV-01	MASW ARRAY APPROXIMATE LOCATION

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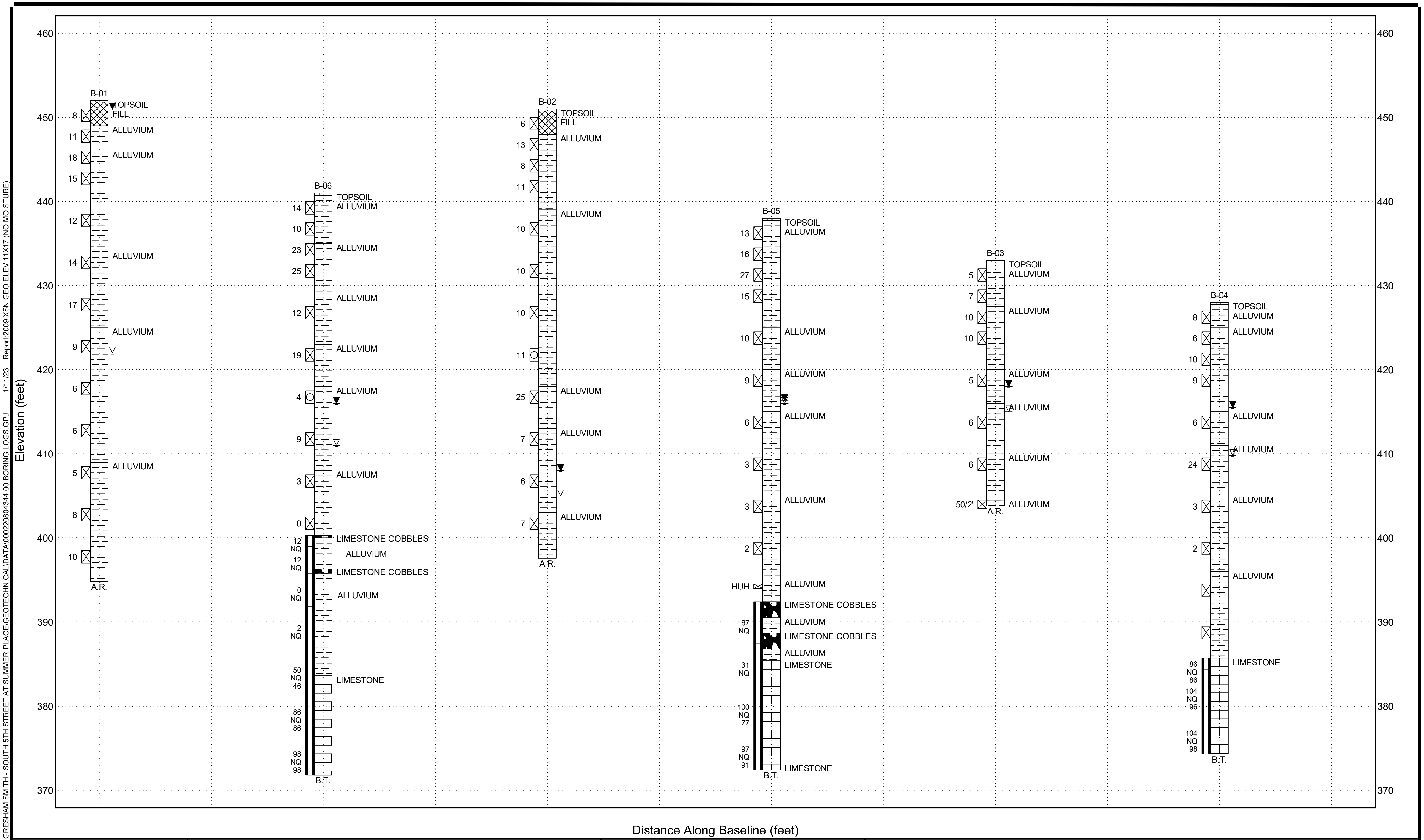
SOUTH 5TH STREET AND SUMMER PLACE

GRESHAM SMITH

SOUTH 5TH STREET AND SUMMER PLACE
NASHVILLE, DAVIDSON COUNTY, TENNESSEE

Drawn By: M H M
Checked By: D O B
Date: 01/12/2022
Proj. No.: 000220804344.00
File Name: 22.80.4344.ELP.dwg

Sheet Title
EXPLORATION LOCATION PLAN



GENERALIZED SUBSURFACE PROFILE

Client: Gresham Smith
Project: South 5th Street and Summer Place
Location: Nashville, Davidson County, Tennessee
Project Number: 000220804344.00

LEGEND: See Legend Key Sheet for explanation of symbols

N (bpf) ☒ Material Symbol % Moisture Classification
Recovery% Core Size RQD%
▽ Water level at time of boring
▼ Delayed water level
Cave-in depth
Perched water level.
B.T. = Boring Terminated A.R. = Auger Refusal

Notes:
For illustration purposes only. Elevations were interpolated from topographic contours on the provided grading plan and should be considered approximate.

Scale: None
* N-value was affected by unbalanced water pressure and is not reported.

X:\2022\08\22-08-04344.00 - GRESHAM SMITH - SOUTH 5TH STREET AT SUMMER PLACE\GEOTECHNICAL\DATA\000220804344.00 BORING LOGS.GPJ 1/11/23 Report:2009 XSN GEO ELEV 11X17 (NO MOISTURE)

SOIL LEGEND

FINE- AND COARSE-GRAINED SOIL INFORMATION

FINE-GRAINED SOILS (SILTS AND CLAYS)

SPT N-Value	Consistency	Estimated Q_u (TSF)
0 - 1	Very Soft	0 - 0.25
2 - 4	Soft	0.25 - 0.5
5 - 8	Firm	0.5 - 1.0
9 - 15	Stiff	1.0 - 2.0
16 - 30	Very Stiff	2.0 - 4.0
31+	Hard	4.0+

Q_u = Unconfined Compression Strength

COARSE-GRAINED SOILS (SANDS AND GRAVELS)

SPT N-Value	Relative Density
0 - 4	Very Loose
5 - 10	Loose
11 - 30	Medium Dense
31 - 50	Dense
51+	Very Dense

PARTICLE SIZE

Name	Size (US Std. Sieve)
Boulders	>300 mm (>12 in.)
Cobbles	75 mm to 300 mm (3 - 12 in.)
Coarse Gravel	19 mm to 75 mm (3/4 - 3 in.)
Fine Gravel	4.75 mm to 19 mm (#4 - 3/4 in.)
Coarse Sand	2 mm to 4.75 mm (#10 - #4)
Medium Sand	0.425 mm to 2 mm (#40 - #10)
Fine Sand	0.075 mm to 0.425 mm (#200 - #40)
Silts and Clays	< 0.075 mm (< #200)

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Terms	Percent of Dry Weight
"Trace"	< 15
"With"	15 - 30
Modifier	> 30

RELATIVE PROPORTIONS OF CLAYS AND SILTS

Descriptive Terms	Percent of Dry Weight
"Trace"	< 5
"With"	5 - 12
Modifier	> 12

CRITERIA FOR DESCRIBING MOISTURE CONDITION

Description	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp, but no visible water
Wet	Visible free water, usually soil is below water table

CRITERIA FOR DESCRIBING CEMENTATION

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure
Moderate	Crumbles or breaks with considerable finger pressure
Strong	Will not crumble or break with finger pressure

CRITERIA FOR DESCRIBING STRUCTURE

Description	Criteria
Stratified	Alternating layers of varying material or color with layers at least 6 mm thick; note the thickness
Laminated	Alternating layers of varying material or color with the layers less than 6 mm thick; note thickness
Fissured	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided	Fracture planes appear polished or glossy, sometimes striated
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed	Inclusion of small pockets of different soils such as small lenses of sand scattered through a mass of clay; note thickness
Homogeneous	Same color and appearance throughout

ABBREVIATIONS AND ACRONYMS

WOH	Weight of Hammer	N-Value	Sum of the blows for last two 6-in increments of SPT
WOR	Weight of Rod		
Ref.	Refusal	NA	Not Applicable or Not Available
ATD	At Time of Drilling	OD	Outside Diameter
DCP	Dynamic Cone Penetrometer	PPV	Pocket Penetrometer Value
Elev.	Elevation	SFA	Solid Flight Auger
ft.	feet	SH	Shelby Tube Sampler
HSA	Hollow Stem Auger	SS	Split-Spoon Sampler
ID	Inside Diameter	SPT	Standard Penetration Test
in.	inches	USCS	Unified Soil Classification System
lbs	pounds		

SAMPLERS AND DRILLING METHODS

▌	AUGER CUTTINGS
◇	BAG/BULK SAMPLE
✋	GRAB SAMPLE
▤	CONTINUOUS SAMPLES
■	SHELBY TUBE SAMPLE
⧗	PITCHER SAMPLE
☒	STANDARD PENETRATION SPLIT-SPOON SAMPLE
○	SPLIT-SPOON SAMPLE WITH NO RECOVERY
▼	DYNAMIC CONE PENETROMETER
▮	ROCK CORE

WATER LEVEL SYMBOLS

▽	WATER LEVEL AT TIME OF DRILLING
≡	PERCHED WATER OBSERVED AT DRILLING
▼	DELAYED WATER LEVEL OBSERVATION
▨	CAVE-IN DEPTH
≈	OBSERVED SEEPAGE

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

COARSE GRAINED SOILS (>50% of the material is larger than the #200 sieve)	GRAVELS (>50% of coarse fraction is larger than the #4 sieve)	CLEAN GRAVEL WITH <5% FINES	Cu > 4 Cc = 1-3	▨	GW	Well-graded gravels, gravel-sand mixtures with trace or no fines
			Cu ≤ 4 and/or Cc < 1 Cc > 3	▨	GP	Poorly-graded gravels, gravel-sand mixtures with trace or no fines
		GRAVEL WITH 5% TO 12% FINES	Cu > 4 Cc = 1-3	▨	GW-GM	Well-graded gravels, gravel-sand mixtures with silt fines
				▨	GW-GC	Well-graded gravels, gravel-sand mixtures with clay fines
			Cu ≤ 4 and/or Cc < 1 Cc > 3	▨	GP-GM	Poorly-graded gravels, gravel-sand mixtures with silt fines
				▨	GP-GC	Poorly-graded gravels, gravel-sand mixtures with clay fines
		GRAVEL WITH MORE THAN 12% FINES		▨	GM	Silty gravels, gravel-silt-sand mixtures
				▨	GC	Clayey gravels, gravel-sand-clay mixtures
				▨	GC-GM	Clayey gravels, gravel-sand-clay-silt mixtures
	SANDS (>50% of coarse fraction is smaller than the #4 sieve)	CLEAN SAND WITH <5% FINES	Cu > 6 Cc = 1-3	▨	SW	Well-graded sands, sand-gravel mixtures with trace or no fines
			Cu ≤ 6 and/or Cc < 1 Cc > 3	▨	SP	Poorly-graded sands, sand-gravel mixtures with trace or no fines
		SAND WITH 5% TO 12% FINES	Cu > 6 Cc = 1-3	▨	SW-SM	Well-graded sands, sand-gravel mixtures with silt fines
				▨	SW-SC	Well-graded sands, sand-gravel mixtures with clay fines
			Cu ≤ 6 and/or Cc < 1 Cc > 3	▨	SP-SM	Poorly-graded sands, sand-gravel mixtures with silt fines
				▨	SP-SC	Poorly-graded sands, sand-gravel mixtures with clay fines
		SAND WITH MORE THAN 12% FINES		▨	SM	Silty sands, sand-gravel-silt mixtures
				▨	SC	Clayey sands, sand-gravel-clay mixtures
				▨	SC-SM	Clayey sands, sand-gravel-clay-silt mixtures
FINE GRAINED SOILS (>50% of material is smaller than the #200 sieve)		SILTS & CLAYS (Liquid Limit less than 50)		▨	ML	Inorganic silts with low plasticity
				▨	CL	Inorganic clays of low plasticity, gravelly or sandy clays, silty clays, lean clays
				▨	CL-ML	Inorganic clay-silts of low plasticity, gravelly clays, sandy clays, silty clays, lean clays
				▨	OL	Organic silts and organic silty clays of low plasticity
		SILTS & CLAYS (Liquid Limit more than 50)		▨	MH	Inorganic silts of high plasticity, elastic silts
				▨	CH	Inorganic clays of high plasticity, fat clays
				▨	OH	Organic clays and organic silts of high plasticity

USCS - HIGHLY ORGANIC SOILS

Primarily organic matter, dark in color, organic odor

▨	PT	Peat, humus, swamp soils with high organic contents

OTHER MATERIALS

■	BITUMINOUS CONCRETE (ASPHALT)
▨	CONCRETE
▨	CRUSHED STONE/AGGREGATE BASE
▨	TOPSOIL
▨	FILL
▨	UNDIFFERENTIATED ALLUVIUM
▨	UNDIFFERENTIATED OVERBURDEN
▨	BOULDERS AND COBBLES

UNIFORMITY COEFFICIENT

$$C_u = D_{60}/D_{10}$$

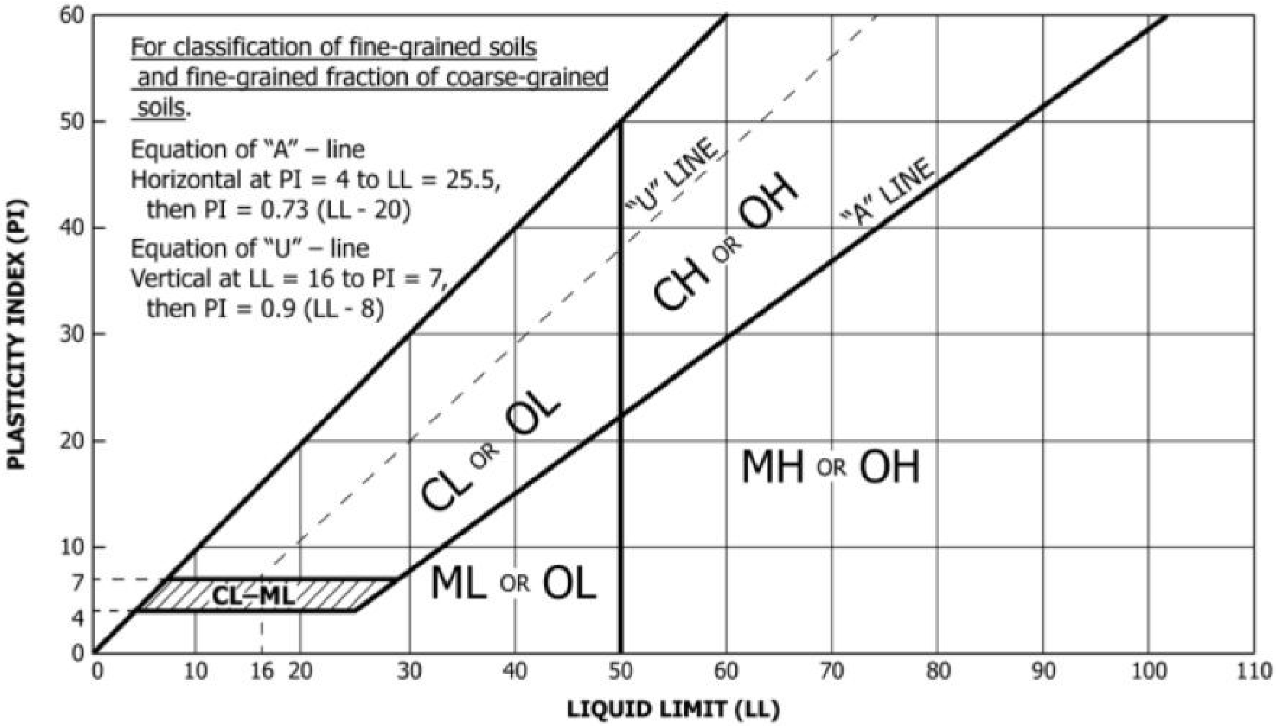
COEFFICIENT OF CURVATURE

$$C_C = (D_{30})^2/(D_{60}xD_{10})$$

Where:

D_{60} = grain diameter at 60% passing
D_{30} = grain diameter at 30% passing
D_{10} = grain diameter at 10% passing

PLASTICITY CHART FOR USCS CLASSIFICATION OF FINE-GRAINED SOILS



IMPORTANT NOTES ON TEST BORING RECORDS

1) The report and graphics key are an integral part of these logs. All data and interpretations in this log are subject to the explanations and limitations stated in the report.

2) Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual or differ from those shown. Solid lines are used to indicate a change in the material type, particularly a change in the USCS classification. Dashed lines are used to separate two materials that have the same material type, but that differ with respect to two or more other characteristics (e.g. color, consistency).

3) No warranty is provided as to the continuity of soil or rock conditions between individual sample locations.

4) Logs represent general soil and rock conditions observed at the point of exploration on the date indicated.

5) In general, Unified Soil Classification System (USCS) designations presented on the logs were based on visual classification in the field and were modified where appropriate based on gradation and index property testing.

6) Fine-grained soils that plot within the hatched area on the Plasticity Chart, and coarse-grained soils with between 5% and 12% passing the #200 sieve require dual USCS symbols as presented on the previous page.

7) If the sampler is not able to be driven at least 6 inches, then 50/X" indicates that the sampler advanced X inches when struck 50 times with a 140-pound hammer falling 30 inches.

8) If the sampler is driven at least 6 inches, but cannot be driven either of the subsequent two 6-inch increments, then either 50/X" or the sum of the second 6-inch increment plus 50/X" for the third 6-inch increment will be indicated.
Example 1: Recorded SPT blow counts are 16 - 50/4", the SPT N-value will be shown as N = 50/4"
Example 2: Recorded SPT blow counts are 18 - 25 - 50/2", the SPT N-value will be shown as N = 75/8"

TEST BORING RECORD LEGEND FOR ROCK

ROCK CORE INFORMATION

ROCK QUALITY DESIGNATION (RQD)

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

ROCK HARDNESS CRITERIA

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

$$\text{Recovery (\%)} = \frac{\text{Length of Core Sample Recovered}}{\text{Length of the Core Run}} \text{ x } 100$$

$$\text{RQD (\%)} = \frac{\text{Sum of Lengths of Intact Rock Pieces of 4 in. and Longer}}{\text{Length of the Core Run}} \text{ x } 100$$

WEATHERING OR ALTERATION

Term	Description
Fresh	No evidence of alteration
Slightly Weathered	Slight discoloration on surface
Moderately Weathered	Discoloring evident; alteration penetrating well below rock surface
Highly Weathered	Entire rock mass discolored
Decomposed	Rock reduced to a soil with relict rock texture

JOINT ROUGHNESS COEFFICIENT (JRC)

Coefficient	Description
14 - 20	Very Rough: Near vertical edges evident
10 - 14	Rough: Smooth ridges, surface abrasion
6 - 10	Slightly Rough: Asperities on surface can be felt
2 - 6	Smooth: Appears and feels smooth
0 - 2	Slickensided: Visible polishing, striated surface

FRACTURE/JOINT DENSITY

Description	Observed Fracture Density
Intact	No fractures or joints less than 6 ft. apart
Slightly Fractured/Jointed	Lengths from 3 ft. to 6 ft.
Moderately Fractured/Jointed	Lengths from 1 ft. to 3 ft.
Highly Fractured/Jointed	Lengths from 4in. to 1 ft.
Intensely Fractured/Jointed	Lengths less than 4 inches

DISCONTINUITY TERMS

Fracture: Collective term for any natural break excluding shears, shear zones, and faults

Joint (JT): Planar break with little or no displacement

Foliation Joint (FJ) or Bedding Joint (BJ): Joint along foliation or bedding

Incipient Joint (IJ) or Incipient Fracture (IF): Joint or fracture not evident until wetted and dried; breaks along existing surface

Random Fracture (RF): Natural, very irregular fracture that does not belong to a set

Bedding Plane Separation or Parting: A separation along bedding after extraction from stress relief or slaking

Fracture Zone (FZ): Planar zone of broken rock without gouge

Mechanical Break (MB): Breaks due to drilling or handling; drilling break is denoted as (DB) and hammer break is denoted as (HB)

Shear (SH): Surface of differential movement evident by presence of slickensides, striations, or polishing

Shear Zone (SZ): Zone of gouge and rock fragments bounded by planar shear surfaces

Fault (FT): Shear zone of significant extent; differentiation from shear zone may be site-specific

BEDDING THICKNESS

Massive	> 3 ft.
Thick	1 ft. to 3 ft.
Medium	4 in. to 1 ft.
Thin	1-1/4 in. to 4 in.
Banded	1/4 in. to 1-1/4 in.
Parting	< 1/4 in.

APERTURE WIDTH

Term	Spacing
Very Tight	< 0.1 mm
Tight	0.1 to 0.25 mm
Partly Open	0.25 to 0.5 mm
Open	0.5 to 2.5 mm
Moderately Wide	2.5 to 10 mm
Wide	10 mm to 1 cm
Very Wide	1 to 10 cm
Extremely Wide	10 cm to 1 m
Cavernous	> 1 m

TTL

ROCK CLASSIFICATION

	▨	PARTIALLY WEATHERED ROCK (UNDIFFERENTIATED)
	▨	WEATHERED ROCK (UNDIFFERENTIATED)
CLASTIC SEDIMENTARY ROCKS	△	BRECCIA
	●	CONGLOMERATE
	●	SANDSTONE
	●	WEATHERED SANDSTONE
	×	SILTSTONE
	▨	CLAYSTONE
	☰	SHALE
	☰	WEATHERED SHALE
	■	COAL
CARBONATE SEDIMENTARY ROCKS	▦	LIMESTONE
	▦	WEATHERED LIMESTONE
	▨	DOLOMITE
	☼	CORAL
EVAPORITE ROCKS	▦	CHALK
	▦	WEATHERED CHALK
	◇	GYPSUM
	▦	HALITE
	○	CALCITE
INTRUSIVE IGNEOUS ROCKS	◁	GRANITE
	⁄	GRANO-DIORITE
	●	DIORITE
	^	GABBRO

EXTRUSIVE IGNEOUS ROCKS	○	TUFF
	▨	RYOLITE
	○	DACITE
	+	ANDESITE
	⬡	BASALT
METAMORPHIC ROCKS	∿	MARBLE
	∿	QUARTZITE
	☰	SLATE
	∿	PHYLLITE
	∿	SCHIST
	⌐	GNEISS
	∿	AMPHIBOLITE
	▤	METAGRAYWACKE

FEATURES WITHIN ROCK

■	FILLED CAVITY
◩	PARTIALLY FILLED CAVITY
▥	OPEN CAVITY

Gresham Smith
South 5th Street and Summer Place

Nashville, Davidson County, Tennessee

Log of B-01

Page 1 of 2

Drilling Co.:	*Tri-State Drilling, LLC*	**TTL** Project No.:	*000220804344.00*
Driller:	*B. Richardson*	Date Drilled:	*12/6/2022*
Logged by:	*J. Felts*	Boring Depth:	*57.2 feet*
Equipment:	*Diedrich D-50*	Boring Elevation:	*452 feet*
Hammer Type:	*Automatic*	Coordinates:	*Not Available*
Drilling Method:	*Hollow Stem Auger w/SPT Sampling and Wash Rotary*	▽ Water Level at Time of Drilling:	*30 ft BGS*
		Cave-In at Time of Drilling:	*N/A*

Remarks:
Borehole backfilled with auger cuttings.

Began wash rotary drilling at 30 feet.

Boring elevation was interpolated from 1-foot contours on provided drawing and is approximate.

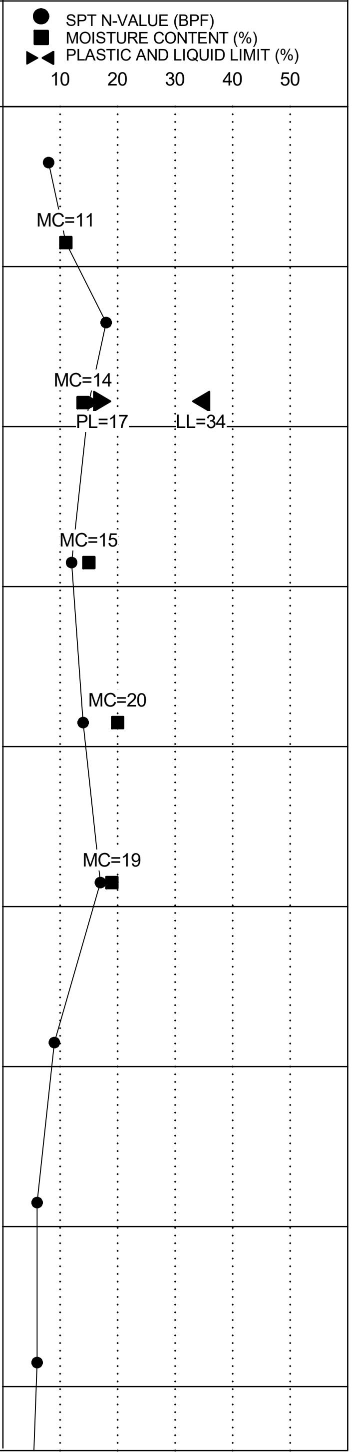
▼ Delayed Water Level: *1 ft BGS*

Delayed Water Observation Date: *12/6/2022*

ELEVATION (ft)	DEPTH (ft)	MATERIALS DESCRIPTION
		TOPSOIL (2 inches)
450		**FILL:** LEAN CLAY, black, with fine roots and trace limestone gravel (fine), dry (CL)
	5	**ALLUVIUM:** LEAN CLAY, stiff, brown, dry (CL)
445	10	**ALLUVIUM:** LEAN CLAY, very stiff to stiff, brown and gray to brown, moist (CL) - black mineral staining from 8.0 feet to 13.0 feet
440, 435	15	
430	20	**ALLUVIUM:** LEAN CLAY, stiff to very stiff, brown, with black mineral staining, moist (CL)
425	25	
420, 415	30, 35, 40	**ALLUVIUM:** SANDY LEAN CLAY, stiff to firm, tan, fine sand, moist (CL)

SAMPLE DATA

% PASSING #200 SIEVE	PPV (tsf)	TYPE	SPT/CORE DATA 1st 6" - 2nd 6" - 3rd 6" N-VALUE	RQD / % REC
		☒	4 - 4 - 4 N = 8	
		☒	4 - 5 - 6 N = 11	
		☒	6 - 7 - 11 N = 18	
		☒	5 - 7 - 8 N = 15	
		☒	4 - 6 - 6 N = 12	
		☒	5 - 6 - 8 N = 14	
		☒	5 - 6 - 11 N = 17	
		☒	3 - 4 - 5 N = 9	
		☒	3 - 3 - 3 N = 6	
		☒	6 - 4 - 2 N = 6	



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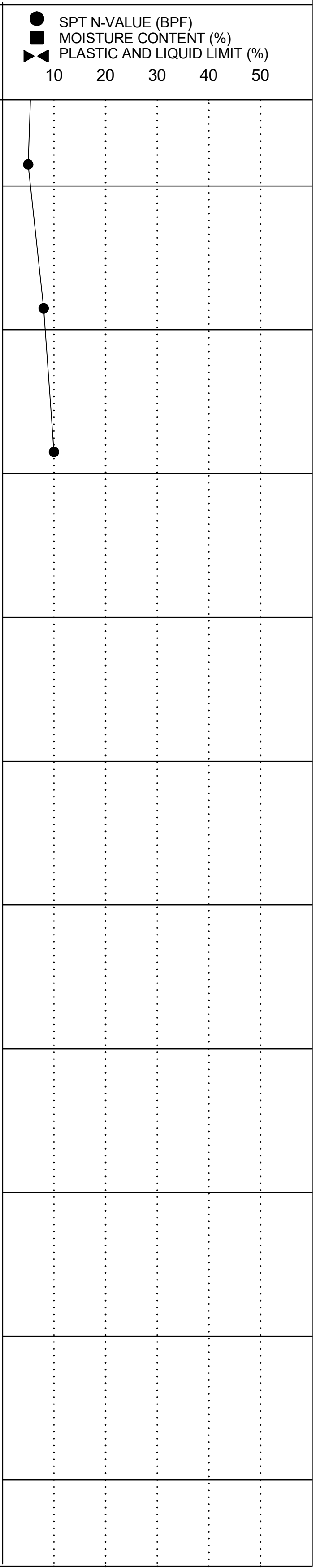
South 5th Street and Summer Place

Nashville, Davidson County, Tennessee

Log of B-01

SAMPLE DATA

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIALS DESCRIPTION	% PASSING #200 SIEVE	PPV (tsf)	TYPE	SPT/CORE DATA 1st 6" 2nd 6" 3rd 6" N-VALUE	RQD % REC
410			*Continued from previous page-* **ALLUVIUM:** SANDY LEAN CLAY, stiff to firm, tan, fine sand, moist (CL)			☒	4 - 3 - 2 N = 5	
	45		**ALLUVIUM:** LEAN CLAY, firm to stiff, brown, wet (CL)					
405						☒	4 - 4 - 4 N = 8	
	50							
400						☒	3 - 4 - 6 N = 10	
	55							
395			Auger refusal at 57.2 feet.					
	60							
390								
	65							
385								
	70							
380								
	75							
375								
	80							
370								
	85							
365								
	90							
360								



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Nashville, Davidson County, Tennessee

Log of
B-02

Page 1 of 2

Drilling Co.:	*Tri-State Drilling, LLC*	**TTL** Project No.:	*000220804344.00*
Driller:	*B. Richardson*	Date Drilled:	*12/6/2022*
Logged by:	*J. Felts*	Boring Depth:	*53.4 feet*
Equipment:	*Diedrich D-50*	Boring Elevation:	*451 feet*
Hammer Type:	*Automatic*	Coordinates:	*Not Available*
Drilling Method:	*Hollow Stem Auger w/SPT Sampling and Wash Rotary*	▽ Water Level at Time of Drilling:	*46 ft BGS*
		Cave-In at Time of Drilling:	*N/A*

Remarks:
Borehole backfilled with auger cuttings.

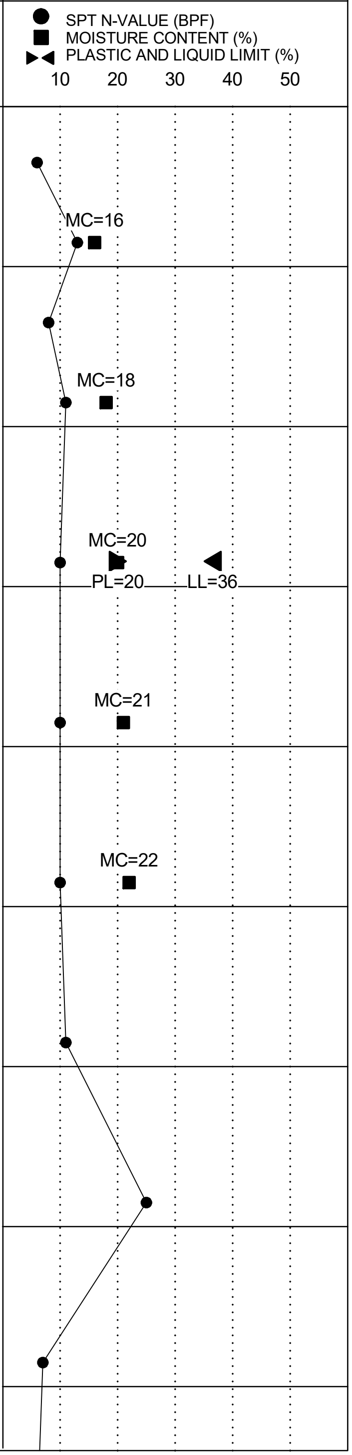
Boring elevation was interpolated from 1-foot contours on provided drawing and is approximate.

▼ Delayed Water Level: *43 ft BGS*

Delayed Water Observation Date: *12/6/2022*

SAMPLE DATA

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIALS DESCRIPTION	% PASSING #200 SIEVE	PPV (tsf)	TYPE	SPT/CORE DATA 1st 6" 2nd 6" 3rd 6" N-VALUE	RQD % REC
			TOPSOIL (3 inches)					
450			**POSSIBLE FILL:** SANDY LEAN CLAY, black, with trace limestone gravel (fine) and fine roots, moist (CL)			☒	3 - 3 - 3 N = 6	
			ALLUVIUM: SANDY LEAN CLAY, stiff and firm, brown to light brown, with some black mineral staining, moist (CL)			☒	3 - 2 - 11 N = 13	
	5							
445						☒	3 - 4 - 4 N = 8	
				76.1		☒	3 - 5 - 6 N = 11	
	10							
440								
			ALLUVIUM: LEAN CLAY, stiff, tan and gray, with some black mineral staining, moist (CL)					
						☒	3 - 5 - 5 N = 10	
	15							
435								
						☒	4 - 4 - 6 N = 10	
	20							
430								
						☒	5 - 6 - 4 N = 10	
	25							
425								
			- no recovery of sample from 28.5 feet			○	5 - 5 - 6 N = 11	
	30							
420								
			ALLUVIUM: CLAYEY GRAVEL with SAND, medium-dense, brown, fine to coarse gravel and fine sand, moist (GC)			☒	10 - 11 - 14 N = 25	
	35							
415								
			ALLUVIUM: LEAN CLAY, firm, tan, with fine sand partings, wet (CL)			☒	3 - 3 - 4 N = 7	
	40							
410								



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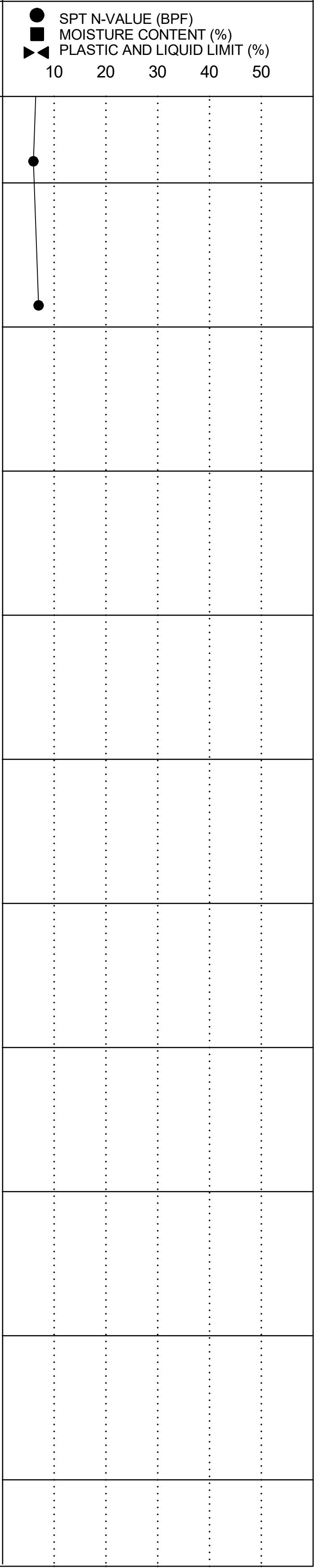
SAMPLE DATA

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIALS DESCRIPTION	% PASSING #200 SIEVE	PPV (tsf)	TYPE	SPT/CORE DATA 1st 6" 2nd 6" 3rd 6" N-VALUE	RQD % REC
			Continued from previous page -	▼				
	45		**ALLUVIUM:** LEAN CLAY, firm, tan, with fine sand partings, wet (CL)			☒	2 - 3 - 3 N = 6	
405				∇				
			ALLUVIUM: CLAYEY SAND, fine, loose, brown, wet (SC)			☒	2 - 2 - 5 N = 7	
400	50							
			Auger refusal at 53.4 feet.					
395	55							
390	60							
385	65							
380	70							
375	75							
370	80							
365	85							
360	90							



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South 5th Street and Summer Place

Nashville, Davidson County, Tennessee

Log of B-03

Page 1 of 1

Drilling Co.:	*Tri-State Drilling, LLC*	**TTL** Project No.:	*000220804344.00*
Driller:	*B. Richardson*	Date Drilled:	*12/7/2022*
Logged by:	*J. Felts*	Boring Depth:	*29.2 feet*
Equipment:	*Diedrich D-50*	Boring Elevation:	*433 feet*
Hammer Type:	*Automatic*	Coordinates:	*Not Available*
Drilling Method:	*Hollow Stem Auger w/SPT Sampling and Wash Rotary*	▽ Water Level at Time of Drilling:	*18 ft BGS*
		Cave-In at Time of Drilling:	*N/A*

Remarks:
Borehole backfilled with auger cuttings.

Began wash rotary drilling at 18.5 feet.

Boring elevation was interpolated from 1-foot contours on provided drawing and is approximate.

▼ Delayed Water Level: *15 ft BGS*

Delayed Water Observation Date: *12/7/2022*

MATERIALS DESCRIPTION

ELEVATION (ft)	DEPTH (ft)
430	
	5
425	
	10
420	
	15
415	
	20
410	
	25
405	
	30
400	
	35
395	
	40

TOPSOIL (2 inches)

ALLUVIUM: LEAN CLAY, firm, tan and brown, with abundant black mineral staining and trace chert gravel (coarse), moist (CL)

ALLUVIUM: LEAN CLAY, stiff, brown with abundant gray mottling, moist (CL)

ALLUVIUM: SANDY LEAN CLAY, firm, brown, fine sand, wet (CL)

ALLUVIUM: LEAN CLAY, firm, dark brown and gray, with some chert gravel (fine to coarse), moist (CL)

ALLUVIUM: CLAYEY SAND, fine, loose, brown, wet (SC)

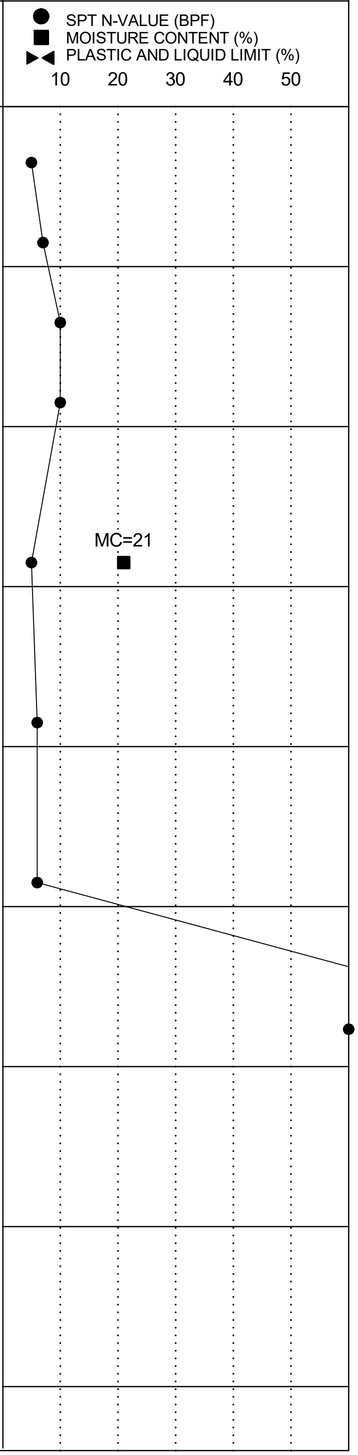
ALLUVIUM: LEAN CLAY, soft, brown, with chert gravel (fine), wet (CL)

- N-value at 28.5 feet was amplified by auger refusal material

Auger refusal at 29.2 feet.

SAMPLE DATA

% PASSING #200 SIEVE	PPV (tsf)	TYPE	SPT/CORE DATA 1st 6" 2nd 6" 3rd 6" N-VALUE	RQD % REC
		☒	3 - 2 - 3 N = 5	
		☒	3 - 3 - 4 N = 7	
		☒	3 - 4 - 6 N = 10	
		☒	4 - 4 - 6 N = 10	
60.2 ▼		☒	2 - 2 - 3 N = 5	
▽				
		☒	5 - 4 - 2 N = 6	
		☒	3 - 3 - 3 N = 6	
		☒	3 - 50/2 N = 50/2"	



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South 5th Street and Summer Place

Nashville, Davidson County, Tennessee

Log of
B-04

Page 1 of 2

Drilling Co.: *Tri-State Drilling, LLC*

Driller: *B. Richardson*

Logged by: *J. Felts*

Equipment: *Diedrich D-50*

Hammer Type: *Automatic*

Drilling Method: *Hollow Stem Auger w/SPT Sampling and Wash Rotary*

TTL Project No.: *000220804344.00*

Date Drilled: *12/2/2022*

Boring Depth: *53.7 feet*

Boring Elevation: *428 feet*

Coordinates: *Not Available*

▽ Water Level at Time of Drilling: *18.2 ft BGS*

Cave-In at Time of Drilling: *N/A*

Remarks:
Borehole backfilled with auger cuttings.

Driller reported 100% return of water while coring.

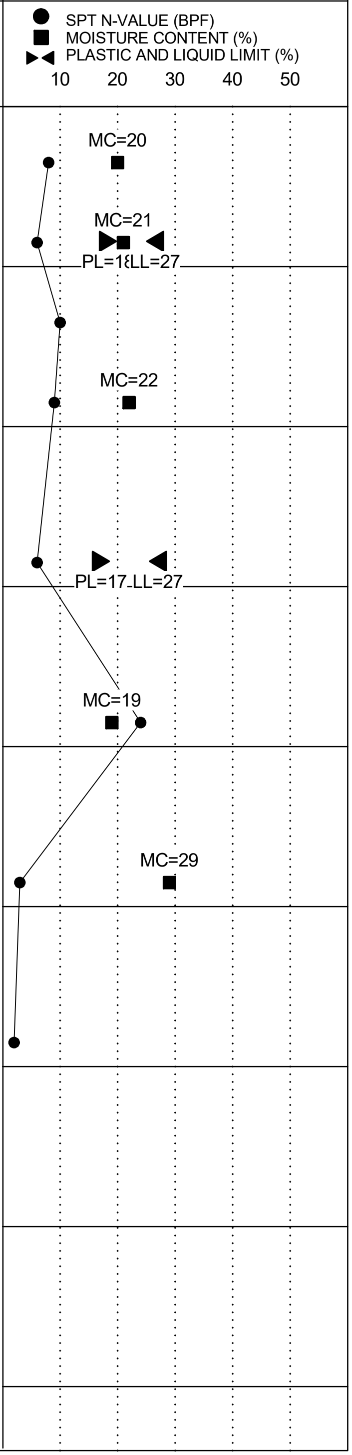
Boring elevation was interpolated from 1-foot contours on provided drawing and is approximate.

▼ Delayed Water Level: *12.5 ft BGS*

Delayed Water Observation Date: *12/2/2022*

SAMPLE DATA

ELEVATION (ft)	DEPTH (ft)	MATERIALS DESCRIPTION	% PASSING #200 SIEVE	SPT/CORE DATA 1st 6" 2nd 6" 3rd 6" N-VALUE
		TOPSOIL (3 inches)		
		ALLUVIUM: LEAN CLAY, firm, brown with trace dark brown mottling and some gray mottling, with trace black mineral staining, moist (CL)		3 - 4 - 4 N = 8
425		**ALLUVIUM:** LEAN CLAY, firm to stiff, brown to tan and gray, with black mineral stains, moist (CL)		3 - 3 - 3 N = 6
	5			4 - 5 - 5 N = 10
420				3 - 5 - 4 N = 9
	10		▼	
415		**ALLUVIUM:** SANDY LEAN CLAY, firm, tan, fine sand, wet (CL)	63.7	3 - 3 - 3 N = 6
	15			
410		**ALLUVIUM:** SANDY LEAN CLAY, very stiff, tan with trace gray mottling, with trace to abundant chert gravel (fine to coarse), with gray fat clay seams, moist (CL) - N-value at 18.5 feet was amplified by gravel	▽	8 - 10 - 14 N = 24
	20			
405		**ALLUVIUM:** LEAN CLAY, soft, tan, moist (CL)		2 - 1 - 2 N = 3
	25			
400				2 - 1 - 1 N = 2
	30			
395		**ALLUVIUM:** SANDY LEAN CLAY, brown, fine sand, wet (CL) - N-value at 33.5 feet affected by unbalanced water and is not reported	63.8	
	35			
390		- N-value at 38.5 feet affected by unbalanced water and is not reported		
	40			



ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIALS DESCRIPTION	% PASSING #200 SIEVE	PPV (tsf)	TYPE	SPT/CORE DATA: 1st 6" / 2nd 6" / 3rd 6" N-VALUE; RQD / % REC	● SPT N-VALUE (BPF) ■ MOISTURE CONTENT (%) ▶◀ PLASTIC AND LIQUID LIMIT (%) 10 20 30 40 50
385			*Continued from previous page -* **ALLUVIUM:** SANDY LEAN CLAY, brown, fine sand, wet (CL)				RQD=86 REC=86	Auger refusal at 42.3 feet. Begin NQ coring.
	45		LIMESTONE, very hard, gray, fine-grained, thin-bedded, highly fractured, slightly weathered				RQD=96 REC=104	
380	50						RQD=98 REC=104	
375	55		Boring terminated at 53.7 feet.					
370	60							
365	65							
360	70							
355	75							
350	80							
345	85							
340	90							

Gresham Smith
South 5th Street and Summer Place

Nashville, Davidson County, Tennessee

Log of
B-05

Page 1 of 2

Drilling Co.:	*Tri-State Drilling, LLC*	**TTL** Project No.:	*000220804344.00*
Driller:	*B. Richardson*	Date Drilled:	*12/2/2022*
Logged by:	*J. Felts*	Boring Depth:	*65.6 feet*
Equipment:	*Diedrich D-50*	Boring Elevation:	*438 feet*
Hammer Type:	*Automatic*	Coordinates:	*Not Available*
Drilling Method:	*Hollow Stem Auger w/SPT Sampling and Wash Rotary*	▽ Water Level at Time of Drilling:	*22 ft BGS*
		Cave-In at Time of Drilling:	*N/A*

Remarks:
Borehole backfilled with auger cuttings.

Driller reported 100% return of water while coring.

Boring elevation was interpolated from 1-foot contours on provided drawing and is approximate.

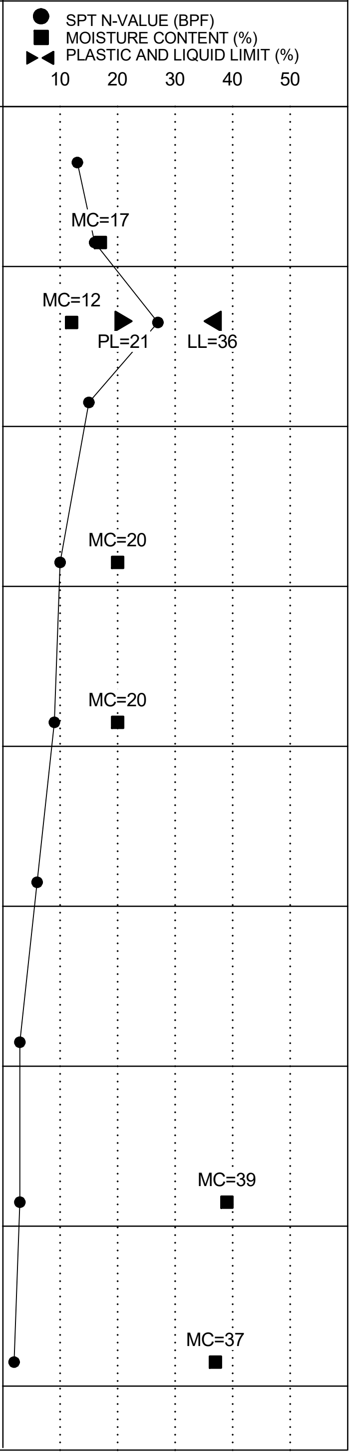
▼ Delayed Water Level: *21.7 ft BGS*

Delayed Water Observation Date: *12/2/2022*

ELEVATION (ft)	DEPTH (ft)	MATERIALS DESCRIPTION
		TOPSOIL (3 inches)
435	5	**ALLUVIUM:** LEAN CLAY, stiff and very stiff, brown to light brown with some tan mottling, with trace black mineral staining, dry (CL)
430	10	
425		**ALLUVIUM:** LEAN CLAY, stiff, tan, dry (CL)
	15	
420		**ALLUVIUM:** SANDY LEAN CLAY, stiff, tan, fine sand, wet (CL)
	20	
415		**ALLUVIUM:** SANDY LEAN CLAY, firm to soft, tan, fine sand, wet (CL)
	25	
410	30	
405		**ALLUVIUM:** LEAN CLAY, soft, brown and tan, wet (CL)
	35	
400		- with fine chert gravel below 38.0 feet
	40	

SAMPLE DATA

% PASSING #200 SIEVE	PPV (tsf)	TYPE	SPT/CORE DATA 1st 6" 2nd 6" 3rd 6" N-VALUE	RQD % REC
		☒	6 - 6 - 7 N = 13	
		☒	6 - 8 - 8 N = 16	
		☒	8 - 11 - 16 N = 27	
		☒	8 - 7 - 8 N = 15	
		☒	4 - 4 - 6 N = 10	
		☒	5 - 4 - 5 N = 9	
		☒	2 - 2 - 4 N = 6	
52.8		☒	2 - 1 - 2 N = 3	
		☒	2 - 2 - 1 N = 3	
		☒	2 - 1 - 1 N = 2	



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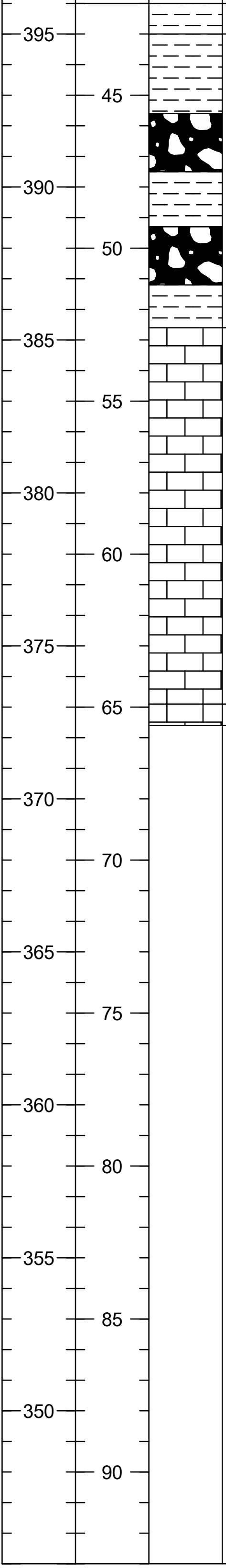
Gresham Smith
South 5th Street and Summer Place

Nashville, Davidson County, Tennessee

Log of B-05



MATERIALS DESCRIPTION

Continued from previous page -

ALLUVIUM: LEAN CLAY, soft, brown ant tan, wet (CL)

ALLUVIUM: LEAN CLAY, very soft, brown, with trace chert gravel (fine), moist (CL)
- 45.6 feet to 47.5 feet - LIMESTONE BOULDERS
- 49.3 feet to 51.2 feet - LIMESTONE BOULDERS

LIMESTONE, very hard, gray with black bands, fine-grained, thin-bedded, intensely fractured, slightly weathered
- with shale parting at 54.7 feet

LIMESTONE, hard, light gray, medium-grained, thin-bedded, highly fractured, slightly weathered

Boring terminated at 65.6 feet.

SAMPLE DATA

% PASSING #200 SIEVE	PPV (tsf)	TYPE	SPT/CORE DATA (1st 6" 2nd 6" 3rd 6" N-VALUE / RQD % REC)	● SPT N-VALUE (BPF) ■ MOISTURE CONTENT (%) ▶◀ PLASTIC AND LIQUID LIMIT (%) 10 20 30 40 50
		☒	WOH N = OOPS	
			REC=67	Auger refusal at 45.6 feet. Begin NQ coring.
			REC=31	
			RQD=77 REC=100	
			RQD=91 REC=97	

This boring log shall not be separated from the corresponding Instrument of Service; no third party may rely upon this boring log or the corresponding Instrument of Service absent a written TTL Secondary Client Agreement.

Gresham Smith
South 5th Street and Summer Place

Nashville, Davidson County, Tennessee

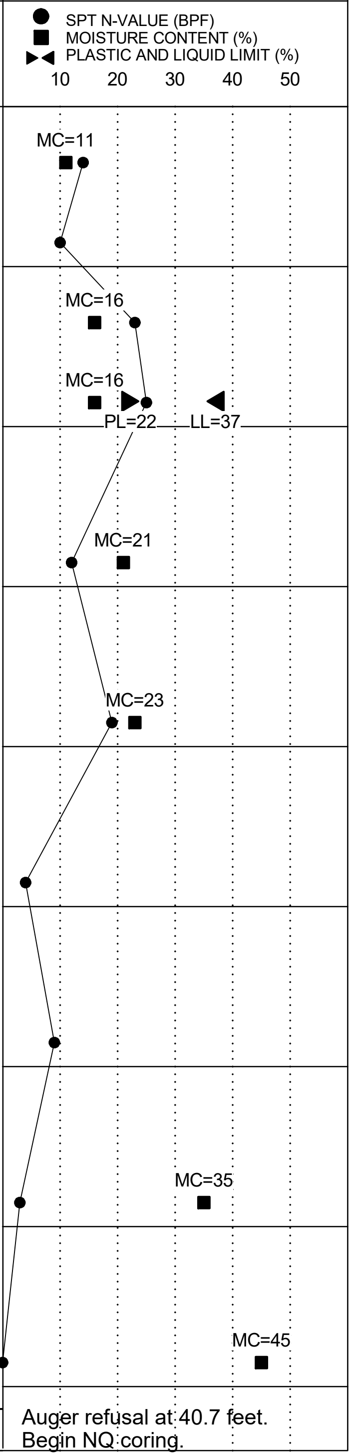
Log of B-06

Page 1 of 2

Drilling Co.:	*Tri-State Drilling, LLC*	**TTL** Project No.:	*000220804344.00*	Remarks:	Borehole backfilled with auger cuttings. Driller reported 100% loss of water at 54.2 feet while coring. Boring elevation was interpolated from 1-foot contours on provided drawing and is approximate.
Driller:	*B. Richardson*	Date Drilled:	*12/5/2022*		
Logged by:	*J. Felts*	Boring Depth:	*69.2 feet*		
Equipment:	*Diedrich D-50*	Boring Elevation:	*441 feet*		
Hammer Type:	*Automatic*	Coordinates:	*Not Available*		
Drilling Method:	*Hollow Stem Auger w/SPT Sampling and Wash Rotary*	▽ Water Level at Time of Drilling: *30 ft BGS* Cave-In at Time of Drilling: *N/A*		▼ Delayed Water Level: *25 ft BGS* Delayed Water Observation Date: *12/5/2022*	

SAMPLE DATA

ELEVATION (ft)	DEPTH (ft)	MATERIALS DESCRIPTION	% PASSING #200 SIEVE	PPV (tsf)	TYPE	SPT/CORE DATA 1st 6" - 2nd 6" - 3rd 6" / N-VALUE	RQD / % REC
		TOPSOIL (3 inches)					
440		**ALLUVIUM:** LEAN CLAY, stiff, brown with light brown mottling, with black mineral staining, dry (CL)			☒	3 - 5 - 9 N = 14	
					☒	3 - 5 - 5 N = 10	
	5						
435		**ALLUVIUM:** SANDY LEAN CLAY, very stiff, brown, with trace black mineral staining, moist (CL)			☒	8 - 11 - 12 N = 23	
					☒	7 - 11 - 14 N = 25	
	10						
430							
		ALLUVIUM: LEAN CLAY, stiff, brown and gray, with trace black mineral staining, moist (CL)			☒	7 - 6 - 6 N = 12	
	15						
425							
		ALLUVIUM: LEAN CLAY, very stiff, brown and gray, moist (CL)			☒	6 - 8 - 11 N = 19	
	20						
420							
		ALLUVIUM: CLAYEY SAND, fine, very loose to loose, brown, wet (SC)			○	2 - 2 - 2 N = 4	
	25	- no recovery of sample from 23.5 feet	▼				
415							
			▽		☒	3 - 4 - 5 N = 9	
	30						
410							
		ALLUVIUM: SANDY LEAN CLAY, fine sand, soft to very soft, brown and gray, wet (CL)			☒	2 - 2 - 1 N = 3	
	35						
405							
					☒	WOH - WOH - WOH N = 0	
	40						
400		- 40.7 feet to 41.0 feet - LIMESTONE COBBLES			▮		REC=12



Auger refusal at 40.7 feet.
Begin NQ coring.

This boring log shall not be separated from the corresponding Instrument of Service; no third party may rely upon this boring log or the corresponding Instrument of Service absent a written TTL Secondary Client Agreement.

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIALS DESCRIPTION	% PASSING #200 SIEVE	PPV (tsf)	TYPE	SPT/CORE DATA 1st 6" 2nd 6" 3rd 6" N-VALUE	RQD / % REC

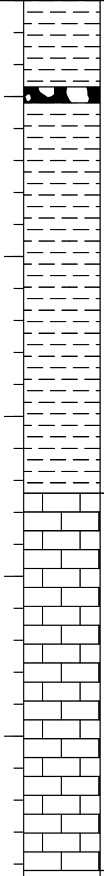
SAMPLE DATA

- ● SPT N-VALUE (BPF)
- ■ MOISTURE CONTENT (%)
- ▶◀ PLASTIC AND LIQUID LIMIT (%)

10 20 30 40 50

Elevation (ft): 395, 390, 385, 380, 375, 370, 365, 360, 355, 350

Depth (ft): 45, 50, 55, 60, 65, 70, 75, 80, 85, 90



Continued from previous page -

ALLUVIUM: SANDY LEAN CLAY, fine sand, soft to very soft, brown and gray, wet (CL)

-44.7 feet to 45.2 feet - LIMESTONE COBBLES

- with one piece of rounded limestone gravel (coarse) between 45.2 feet and 57.4 feet

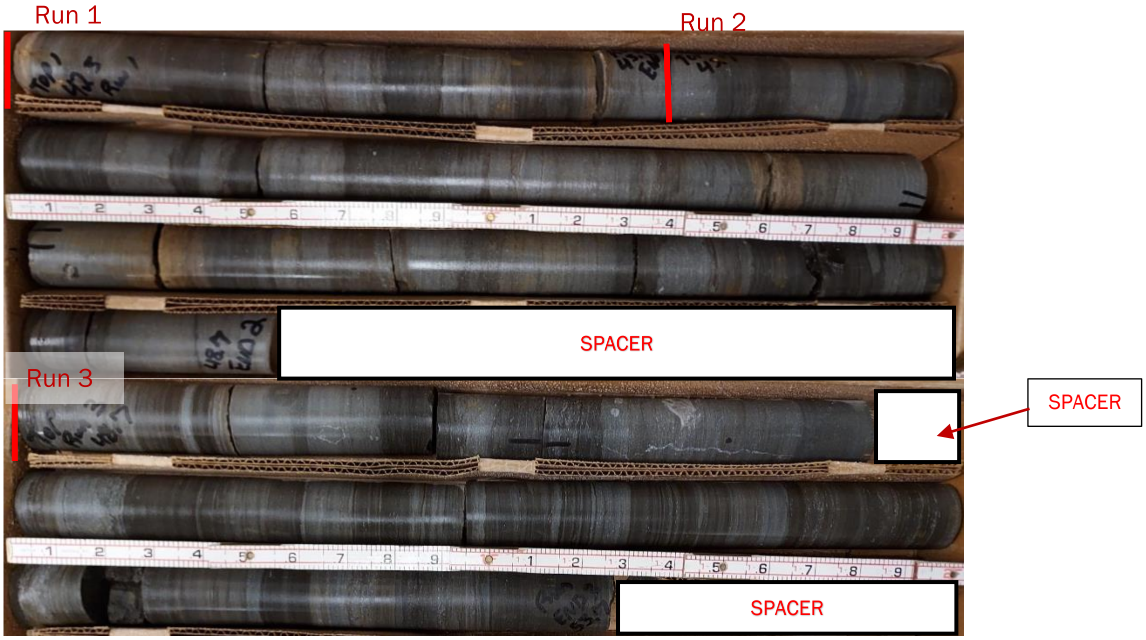
LIMESTONE, very hard, gray with black bands, fine-grained, thin-bedded, highly fractured to moderately fractured, fresh

Boring terminated at 69.2 feet.

REC=12	
REC=0	
REC=2	
RQD=46	REC=50
RQD=86	REC=86
RQD=98	REC=98

ROCK CORE PHOTOGRAPHS
SOUTH 5TH STREET AND SUMMER PLACE
NASHVILLE, DAVIDSON COUNTY, TENNESSEE
TTL PROJECT NO. 0002204344.00

Boring B-04
42.3 feet to 53.7 feet



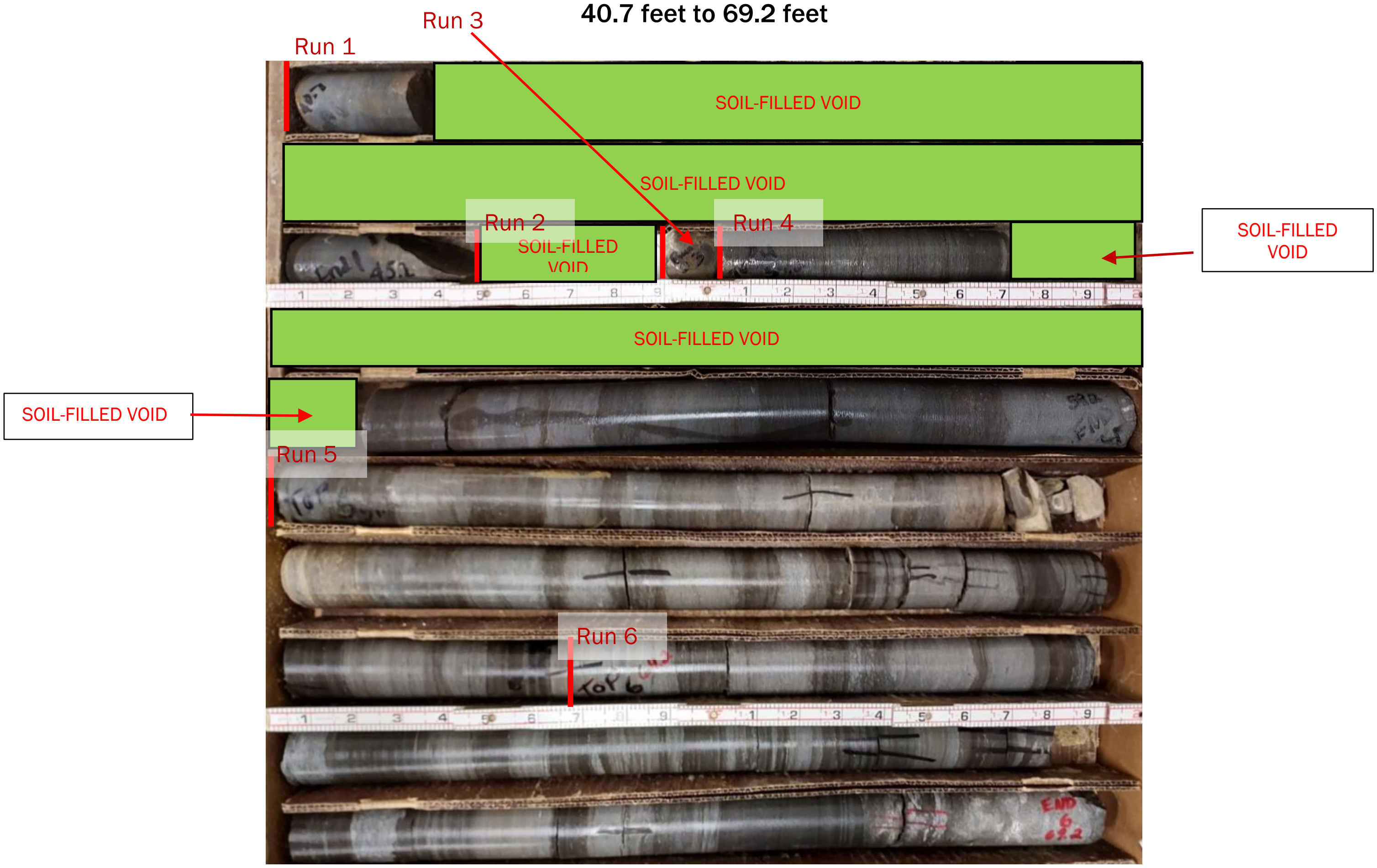
Run No.	Depth (feet)	Recovery (percent)	RQD (percent)	Rock Quality
1	42.3 to 43.7	86	86	Good
2	43.7 to 48.7	104	96	Excellent
3	48.7 to 53.7	104	98	Excellent

Boring B-05
45.6 feet to 65.6 feet

Run No.	Depth (feet)	Recovery (percent)	RQD (percent)	Rock Quality
1	45.6 to 50.6	67	N/A	N/A
2	50.6 to 55.6	31	N/A	N/A
3	55.6 to 60.6	100	77	Good
4	60.6 to 65.6	97	91	Excellent

ROCK CORE PHOTOGRAPHS
SOUTH 5TH STREET AND SUMMER PLACE
NASHVILLE, DAVIDSON COUNTY, TENNESSEE
TTL PROJECT NO. 0002204344.00

Boring B-06
40.7 feet to 69.2 feet



Run No.	Depth (feet)	Recovery (percent)	RQD (percent)	Rock Quality
1	40.7 to 45.2	12	N/A	N/A
2	45.2 to 49.2	0	N/A	N/A
3	49.2 to 54.2	2	N/A	N/A
4	54.2 to 59.2	50	46	Poor
5	59.2 to 64.2	86	86	Good
6	64.2 to 69.2	98	98	Excellent

Boring	Depth	Date Sampled	Classification	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	%<#200 Sieve
B-01	3.5	12/6/2022		11				
B-01	8.5	12/6/2022	CL	14	34	17	17	
B-01	13.5	12/6/2022		15				
B-01	18.5	12/6/2022		20				
B-01	23.5	12/6/2022		19				
B-02	3.5	12/6/2022		16				
B-02	8.5	12/6/2022		18				76
B-02	13.5	12/6/2022	CL	20	36	20	16	
B-02	18.5	12/6/2022		21				
B-02	23.5	12/6/2022		22				
B-03	13.5	12/7/2022		21				60
B-04	1	12/2/2022		20				
B-04	3.5	12/2/2022	CL	21	27	18	9	
B-04	8.5	12/2/2022		22				
B-04	13.5	12/2/2022	CL		27	17	10	64
B-04	18.5	12/2/2022		19				
B-04	23.5	12/2/2022		29				
B-04	33.5	12/2/2022						64
B-05	3.5	12/2/2022		17				
B-05	6	12/2/2022	CL	12	36	21	15	
B-05	13.5	12/2/2022		20				
B-05	18.5	12/2/2022		20				
B-05	28.5	12/2/2022						53
B-05	33.5	12/2/2022		39				
B-05	38.5	12/2/2022		37				
B-06	1	12/5/2022		11				
B-06	6	12/5/2022		16				
B-06	8.5	12/5/2022	CL	16	37	22	15	
B-06	13.5	12/5/2022		21				
B-06	18.5	12/5/2022		23				
B-06	33.5	12/5/2022		35				
B-06	38.5	12/5/2022		45				

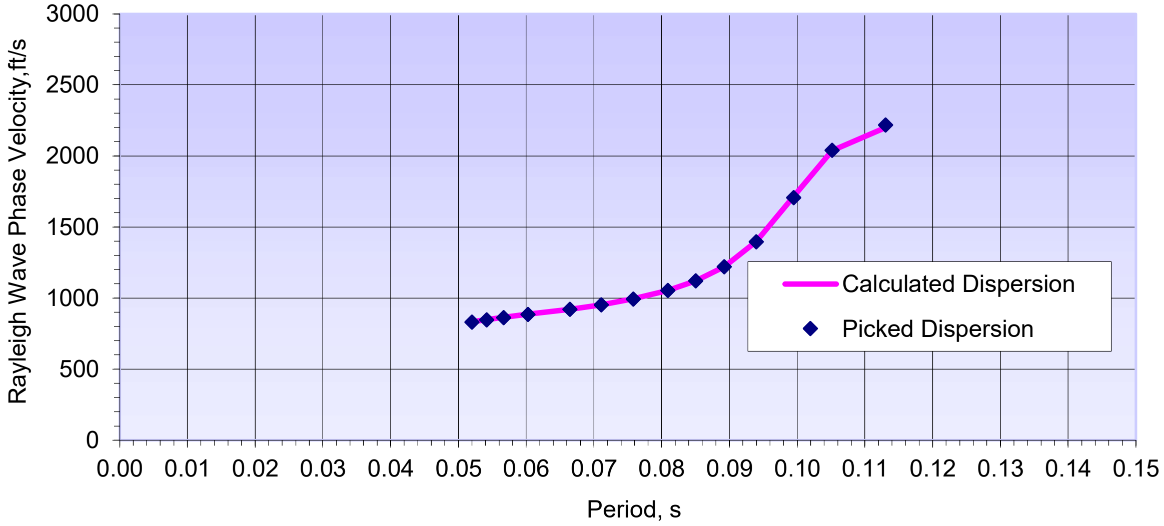
Note:

Summary of Laboratory Results

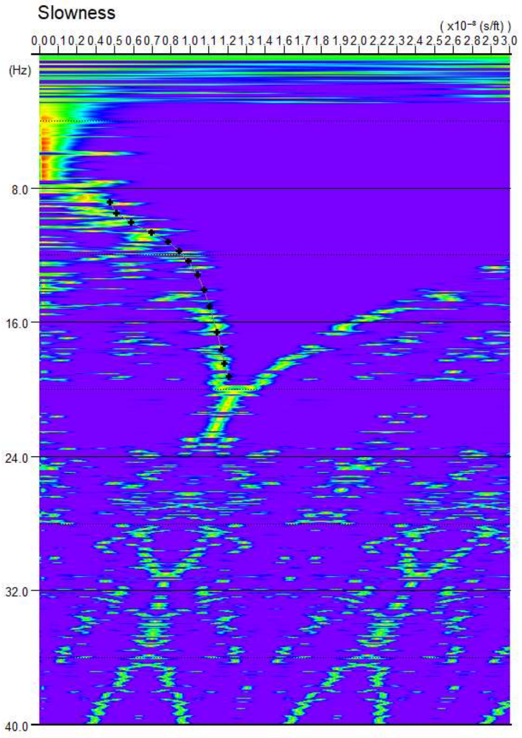
Client: Gresham Smith
Project: South 5th Street and Summer Place
Location: Nashville, Davidson County, Tennessee
Project Number: 000220804344.00

DISPERSION CURVE AND SLOWNESS SPECTRUM
SOUTH 5TH STREET AND SUMMER PLACE
NASHVILLE, DAVIDSON COUNTY, TENNESSEE
TTL PROJECT NUMBER 000220804344.00

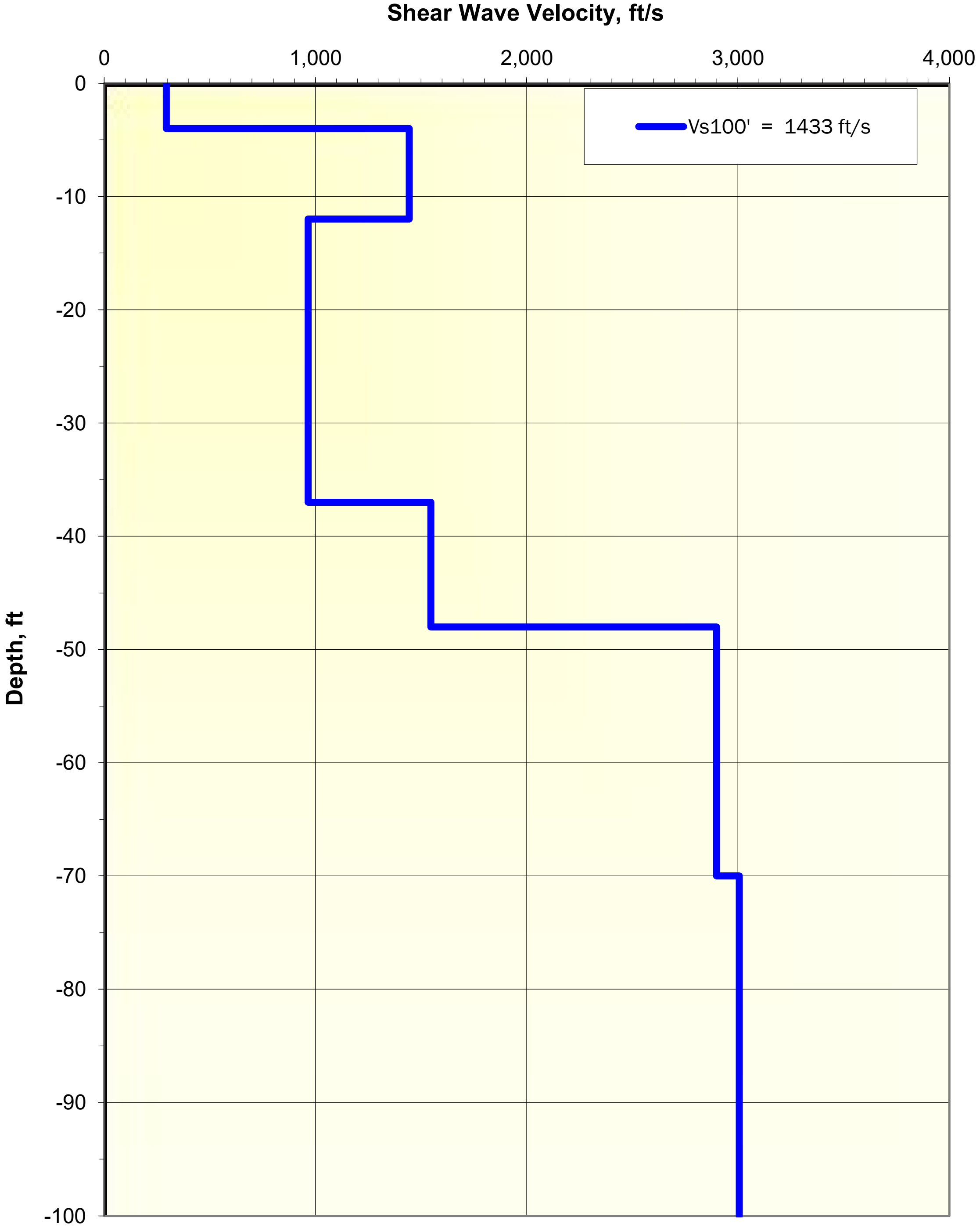
Dispersion Curve Showing Picks and Fit



p-f Image with Dispersion Modeling Picks



SHEAR WAVE VELOCITY MODEL
SOUTH 5TH STREET AND SUMMER PLACE
NASHVILLE, DAVIDSON COUNTY, TENNESSEE
TTL PROJECT NUMBER 000220804344.00



APPENDIX B
REFERENCE MATERIALS

EXPLORATION PROCEDURES

Field Locating of Explorations

Exploratory borings and shear wave velocity tests were located in the field by pacing distances and estimating right-angles from on-site landmarks and should not be considered more accurate than implied by the methods used. Surveying the test locations for vertical and horizontal control was beyond the scope of this exploration.

The ground elevation at each boring was interpolated from 1-foot topographic contours shown on the provided grading plan based on the approximate boring location. The interpolated elevations should also be considered approximate.

Soil Borings

The borings were drilled using conventional hollow-stem auger or wash rotary drilling methods by a truck-mounted drill rig. Soil samples were obtained at selected depths in general accordance with the Standard Penetration Test (SPT) described in ASTM D1586. For this test, a split-barrel sampler is driven into the soil through three increments of 6 inches with blows from a 140-pound hammer falling 30 inches. The number of hammer blows required to advance the split-barrel sampler through each increment is recorded, and the sum of the final two blow counts is called the "N-value," with units of blows per foot (bpf). Where it was not possible to advance the sampler through a full 6-inch increment with 50 hammer blows, driving the sampler was terminated and the sampler penetration was measured. N-values for this condition are reported as "50/x," where x is the sampler penetration in inches. In some cases the sampler advanced through the full 18 inches of sampler penetration without driving the sampler with the hammer. In these cases the N-values are reported as Weight-of-Rod (WOR) or Weight-of-Hammer (WOH), depending on whether the sampler advance was caused by the weight of the hammer and rods or by the weight of the rods alone. The N-values recorded during the sampling process provide an index to the strength and compressibility of the soil.

Rock Coring

Some borings were extended below auger refusal depths by NQ-wireline rock coring methods in general accordance with ASTM D2113. The rock coring was typically performed in discrete advances, called "runs," of 5 feet or less. The rock core samples recovered from each run were placed in prepared rock core boxes that are designed to store and display up to 10 feet of recovered core. Each core was placed in the box from top to bottom, and the beginning and ending depths of each run were identified within the box.

Groundwater Measurements

Each borehole was checked for the presence of groundwater after removing the drill tools by lowering a measuring tape down the open borehole. Where rock coring was performed, the borehole was checked for the presence of groundwater through the hollow-stem auger or drill casing after reaching auger refusal but before the start of rock coring. The depth to groundwater was recorded, if present.

Backfilling Boreholes

Each borehole was backfilled to the ground surface with auger cuttings after making final groundwater measurements. Auger cuttings sometimes consolidate after backfilling causing the top of the backfill column to settle and leaving an open hole at the ground surface. Return trips to the site to top-off backfill that has settled were not part of our scope of services.

LABORATORY TESTING PROCEDURES

Soil Classification and Index Testing

The recovered soil samples were visually classified in the laboratory by a geoprofessional using the USCS as a guide. Samples were tested for the following properties in general accordance with the applicable ASTM standards:

- Moisture content (ASTM D2216);
- Atterberg Limits (ASTM D4318); and
- Percent Fines (ASTM D1140).

Results of tests are presented on individual boring logs in Appendix A and tabulated on the Summary of Laboratory Results sheet in Appendix A.

Rock Core Classification

Rock core samples were described for lithology, measured for Recovery (REC) and Rock Quality Designation (RQD) according to ASTM D2113, and photographed. The descriptions, REC, and RQD are presented on the boring logs in Appendix A. The REC and RQD are also tabulated below photographs of the rock core runs in Appendix A. The photograph pages include a designation of rock quality based on the measured RQD of each run. RQD was not reported for alluvial materials sampled by coring.

Rock core specimens were not tested.