REPORT of SUBSURFACE EXPLORATION and GEOTECHNICAL ENGINEERING EVALUATION

Dunwoody Maintenance Facility
Dunwoody, DeKalb County, Georgia

PREPARED FOR:
City of Dunwoody
4800 Ashford Dunwoody Road
Dunwoody, Georgia 30338

NOVA Project Number: 10103-2024073

August 14, 2024

NOVA
ENGINEERING AND ENVIRONMENTAL

NOVA

August 14, 2024

City of Dunwoody
4800 Ashford Dunwoody Road
Dunwoody, Georgia 30338

Attention: Mr. Michael Smith

Subject: Report of Subsurface Exploration and
Geotechnical Engineering Evaluation
DUNWOODY MAINTENACE FACILITY
Dunwoody, DeKalb County, Georgia
NOVA Project Number 10103-2024073

Dear Mr. Smith:

NOVA Engineering and Environmental, LLC (NOVA) has completed the authorized Geotechnical Engineering Report for the Dunwoody Maintenance Facility project located in Dunwoody, Georgia. The work was performed in general accordance with NOVA Proposal Number 10103-2024073.3, revised July 19, 2024. This report briefly discusses our understanding of the project at the time of the subsurface exploration, describes the geotechnical consulting services provided by NOVA, and presents our findings, conclusions, and recommendations.

We appreciate your selection of NOVA and the opportunity to be of service on this project. If you have any questions, or if we may be of further assistance, please do not hesitate to contact us.

Sincerely,
NOVA Engineering and Environmental, LLC
Georgia Engineering License No. PEF005170

GEORGIA REGISTERED PROFESSIONAL ENGINEER No. 027809 MARCUS D. JOHNSTON

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Copies Submitted: Addressee (electronic)

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1.0 INTRODUCTION

This section provides information relating to our contract, the purpose of the services provided, and a summary of our understanding of the project.

1.1 PROJECT NAME AND LOCATION

The Dunwoody Maintenance Facility project is located at 4770 North Peachtree Road in Dunwoody, DeKalb County, Georgia. The Dekalb County Tax Assessor's office maps the site as 102.58 acres in one (1) parcel identified as parcel number 18 354 01 005. The location of the site is indicated in Figure 1 of Appendix A.

1.2 AUTHORIZATION AND SCOPE OF SERVICES

Our services on this project were as described in our Proposal Number 10103-2024073.3, revised July 19, 2024. These services were authorized on July 18, 2024, by Mr. Michael Smith from the City of Dunwoody Public Works with our confirming proposal delivered the following day.

The primary objective of these services was to perform a geotechnical exploration within the areas of the proposed construction and to assess the findings as they relate to the geotechnical aspects of the planned site development. The authorized geotechnical engineering services included site reconnaissance, soil test borings (STBs) and sampling, engineering evaluation of the field data and the preparation of this report.

The assessment of the presence of wetlands, floodplains, or water classified as state waters was beyond the scope of this exploration. Additionally, the assessment of site environmental conditions, including the detection of pollutants in the soil, rock, or groundwater, at the site was also beyond the scope of this geotechnical exploration and evaluation.

2.0 PROJECT INFORMATION

Our understanding of this project is based on email correspondence with Mr. Luke Norris of Vandermeer Management, LLC, review of the provided site plans, a site reconnaissance during boring layout, and our experience with similar projects.

2.1 SITE PLANS and DOCUMENTS

We were furnished with the following plan and document:

- New Maintenance Facility at Brook Run Park, prepared by Breedlove Land Planning dated March 18, 2024.

2.2 PROJECT SITE

The site is currently developed as Brook Run Park with a grassland area, baseball fields, soccer fields, and playground areas. Based on the provided documents and available aerial imagery, existing site grades on the site range from about 985 feet (MSL) in the western portion of the site to approximately 1,006 feet (MSL) in the central-southern portion of the site.

2.3 PROPOSED DEVELOPMENT

The proposed development will consist of demolishing the existing maintenance facility building to be replaced with a similar new 2-story brick maintenance facility. We understand that part of the re-development will include the retaining walls as planned:

- An approximately 80-foot-long modular block wall along the northern limits of the parking lot west of the new building. The wall height will be 1 to 4 feet.
- An approximately 180-foot-long cast-in-place retaining wall which will serve as foundation walls for 3 sides of the planned building. This will be about 13 feet tall.
- An approximately 50-foot-long modular block wall northeast of the planned building which will be 3 to 6 feet tall.
- An approximately 175-foot-long cast-in-place wall, which will be 2 to 12 feet tall, along the parking lot southwest of the planned building.

Structural loading information was not provided. Therefore, we have assumed that maximum wall and column loads will be on the order of 4 kips per linear foot (klf) and 150 kips, respectively.

Traffic loading for pavement design purposes was not provided. We have presumed the project's civil engineer will finalize the design of the asphalt and concrete pavements, incorporating the geotechnical recommendations from this exploration to ensure proper pavement design for the site based final design traffic loading. Below is the presumed traffic loading used for our recommended pavement designs:

- **For Standard-Duty Pavements** –50 automobiles and 2-3 delivery van/panel trucks per day, 7 days per week, for a 20-year pavement life.

- **For Heavy-Duty Pavements** – in addition to similar traffic as the standard-duty pavements, we have presumed an additional 3 garbage or similar vehicles for a 20-year pavement life.

If the above project information and/or presumptions are incorrect, NOVA should be afforded the opportunity to re-evaluate the recommendations detailed herein based on the correct information. Once the project design is complete, additional field and laboratory testing may be required to finalize the geotechnical exploration.

3.0 SUBSURFACE EXPLORATION

3.1 AREA GEOLOGY

The site is in the Piedmont Geologic Region, a broad northeasterly-trending geologic province underlain by crystalline rocks up to 600 million years old. The Piedmont is bounded on the northwest by the Blue Ridge Range of the Appalachian Mountains, and on the southeast by the leading edge of Coastal Plain sediments, commonly referred to as the “Fall Line.” Numerous episodes of crustal deformation have produced varying degrees of metamorphism, folding, and shearing in the underlying rock. The resulting metamorphic rock types in this area of the Piedmont are predominantly a series of Precambrian age schists and gneisses, with scattered granitic or quartzite intrusions.

According to "Geology of the Greater Atlanta Area" McConnell and Abrams (1984), the site is generally underlain by the Southern Piedmont Province and Brevard Fault Zone’s Ductilely Sheared Rocks, composed of undifferentiated ductilely sheared button schists (bz).

Residual soils in the region are primarily the product of in-situ chemical decomposition of the parent rock. The extent of the weathering is influenced by the mineral composition of the rock and defects such as fissures, faults and fractures. The residual profile can generally be divided into three zones:

- An upper zone near the ground surface consisting of red silty sands and sandy silts which have undergone the most advanced weathering,
- An intermediate zone of less weathered micaceous sandy silts and silty sands, frequently described as “saprolite”, whose mineralogy, texture, and banded appearance reflects the structure of the original rock, and
- A transitional zone between soil and rock termed partially weathered rock (PWR). Partially weathered rock is defined locally by standard penetration resistances exceeding 100 blows per foot.

The boundaries between zones of soil, partially weathered rock, and bedrock are erratic and poorly defined. Weathering is often more advanced next to fractures and joints that transmit water, and in mineral bands that are more susceptible to decomposition. Boulders and rock lenses are sometimes encountered within the overlying PWR or soil matrix. Consequently, significant fluctuations in depth to materials requiring difficult excavation techniques may occur over short horizontal distances.

3.2 FIELD EXPLORATION

Our field exploration was conducted on July 29th, 2024, and included:

- Eight (8) STBs drilled within the proposed building and pavement footprint areas to depths ranging from 10½ to 20½ feet below the existing ground surface at locations requested by the Client.

- Five (5) STBs drilled within the infiltration areas to a depth of 5 feet below existing grade at locations requested by the Client.

Test locations were established in the field by NOVA personnel using a handheld GPS device and estimating distances and angles from site landmarks. Prior to initiating field testing, underground utilities were marked by our subcontracted private utility locating firm. Underground utility related adjustments of the test locations were made at the time of the field exploration. The approximate test locations are shown on Figure 3 in Appendix A. If increased accuracy is desired, test locations and elevations should be surveyed.

3.2.1 Soil Test Borings

The STBs were performed using the guidelines of ASTM Designation D-1586, "Penetration Test and Split-Barrel Sampling of Soils". A hollow-stem auger was used to advance the borings. At regular intervals, soil samples were obtained with a standard 1.4-inch I.D., 2.0-inch O.D., split-tube sampler. The sampler was first seated six inches and then driven an additional foot with blows of a 140-pound hammer falling 30 inches. The number of hammer blows required to drive the sampler the final foot is designated the "Penetration Resistance". The penetration resistance, when properly interpreted, is an index to the soil strength and density. Representative portions of the soil samples, obtained from the sampler, were placed in glass jars and transported to our laboratory for further evaluation and laboratory testing.

Test Boring Records in Appendix B show the standard penetration test (SPT) resistances, or “N-values”, and present the soil conditions encountered in the borings.

3.2.2 Percolation Testing

Five (5) percolation infiltration tests were performed at the locations requested by the Client, as shown on Figure 3 in Appendix A. The percolation tests were drilled to an approximate depth of 5 feet below existing grade and approximately 4 to 6 inches of gravel was placed at the bottom of each borehole. The percolation tests were performed in general accordance with standard test procedures described in the US Public Health Servies's "Manual of Septic Tank Practice."

3.3 LABORATORY TESTING

Following completion of the field exploration, collected soil samples were returned to our office for visual classification. The soil samples will be discarded following the submittal of this report, unless you request otherwise in writing.

3.4 SITE SUBSURFACE CONDITIONS

The following paragraphs provide generalized descriptions of the subsurface conditions encountered by the borings conducted during this exploration.

The Test Boring Records in Appendix B should be reviewed to provide more detailed descriptions of the subsurface conditions encountered at each boring location. These records represent our interpretation of the subsurface conditions based on the field logs and visual observations of samples by an engineer. The lines designating the interface between various strata on the Boring Records represent the approximate interface locations and elevation. The actual transition between strata may be gradual. Groundwater levels shown on the Boring Records represent the conditions at the time of drilling. It should be understood conditions may vary between boring locations.

3.4.1 Surface Materials

Asphalt pavement was encountered at all boring locations. Asphalt thicknesses ranged from approximately 2 to 4 inches.

Concrete was encountered beneath the asphalt pavement at 3 boring locations numbered B-9, B-10, and I-5. Concrete thicknesses ranged from approximately 4 to 6½ inches.

3.4.2 Fill Soils

Fill soils were encountered in 14 STBs beneath the surface materials. The fill generally consisted of micaceous silty coarse to fine SANDs and coarse to fine sandy SILTs with isolated zones containing varying amounts of rock fragments, plant roots, and wood debris. Fill depths ranged from approximately 6 inches to 2 feet below the existing ground surface. Standard penetration resistance values ranged from 6 to 12 blows per foot (bpf).

3.4.3 Residual Soils

Residual soils were encountered in all STBs beneath the surface and fill materials. The residuum generally consisted of micaceous to very micaceous silty coarse to fine SANDs and coarse to fine sandy SILTs. Standard penetration resistance values ranged from 4 to 58 blows per foot (bpf), but more typically varied from 6 to 28 bpf.

3.5 GROUNDWATER CONDITIONS

Groundwater in the Piedmont geologic province typically occurs as an unconfined or semi-confined aquifer condition. Recharge is provided by the infiltration of rainfall and surface water through the soil overburden. More permeable zones in the soil matrix, as well as fractures, joints and discontinuities in the underlying bedrock can affect groundwater conditions. Groundwater was not encountered in any of the STB at the time of drilling.

Groundwater levels vary with changes in season and rainfall, construction activity, surface water runoff, and other site-specific factors. Groundwater levels in the Dekalb County area are generally lowest in the late summer-early fall and highest in the late winter-early spring, with annual groundwater fluctuations of 4 to 8 feet; consequently, the water table may be different than measured during this exploration at other times.

3.6 INFILTRATOIN TEST RESULTS

A total of 5 percolation tests (numbered I-1 through I-5) were conducted during our field exploration at locations depicted on Figure 3 in Appendix A. The below table summarizes the measured percolation rates which were converted into infiltration rates via the Porchet Method. It should be noted that actual infiltration rates may vary at other depths and locations. The Test Boring Records in Appendix B should be reviewed to provide more detailed descriptions of the subsurface conditions encountered at each infiltration testing location.

INFILTRATION TEST LOCATION	MEASURED PERCOLATION RATE (inches/hour)	INFILTRATION RATE (inches/hour)
I-1	0.49	0.17
I-2	0.50	0.10

INFILTRATION TEST LOCATION	MEASURED PERCOLATION RATE (inches/hour)	INFILTRATION RATE (inches/hour)
I-3	0.50	0.06
I-4	0.50	0.03
I-5	0.50	0.18

4.0 GEOTECHNICAL ASSESSMENT

The following assessment is based on our understanding of the proposed construction, site observations, our evaluation and interpretation of the field data obtained during this exploration, our experience with similar subsurface conditions, and generally accepted geotechnical engineering principles and local practices.

Based on our professional opinion, the site and subsurface conditions were deemed favorable for the planned construction. Based on our findings and site observations, there were no geologic hazards such as shallow bedrock or groundwater, and there were no identified light-weight silts or plastic clays encountered to the depths explored during this study. ***However, some existing fill soils encountered during our exploration may be unsuitable for re-use as structural fill due to the presence of wooden debris and plant roots. These soils, ranging from ½ feet to 2 feet below the existing surface, may need to be undercut and replaced with structural fill if they contain organics or other deleterious materials. See the following sections for our recommendations.***

It should be noted that subsurface conditions in unexplored locations may be different from those encountered at the test locations considered and discussed herein. If such variations are noted during construction, or if project development plans are changed, we request the opportunity to review the changes and amend our recommendations, if necessary.

The following sections present our recommendations for site preparation, grading and excavations, and foundation design.

5.0 RECOMMENDATIONS

5.1 SITE PREPARATION

5.1.1 General

Prior to proceeding with construction, all slabs, foundations, pavements, vegetation, root systems, topsoil, and other deleterious non-soil materials should be stripped from proposed construction areas. Topsoil may be stockpiled and subsequently re-used in landscaped areas. Debris-laden materials, if present, should be excavated, transported, and disposed of off-site in accordance with appropriate solid waste rules and regulations. All existing utility locations should be reviewed to assess their impact on the proposed construction and relocated/grouted in-place as appropriate.

After clearing and stripping, areas that are at grade or which will receive fill should be carefully evaluated by a NOVA geotechnical engineer. This evaluation should initially include observation of the materials exposed below the stripped subgrade, The exposed materials should be proofrolled with multiple passes of a 20- to 30-ton loaded truck, or other vehicle of similar size and weight under the observation of the geotechnical engineer. The purpose of the proofrolling is to locate soft, weak, or excessively wet fill or residual soils present at the time of construction. Unstable materials observed during the evaluation and proofrolling operations should be undercut and replaced with structural fill or stabilized in-place by scarifying and re-densifying.

Should low consistency/relative density and/or debris laden fill materials be encountered during construction, it may need to be excavated and replaced or stabilized in place. Actual remedial recommendations can best be determined by the geotechnical engineer in the field at the time of construction.

5.1.2 Existing / Old Fill

Previously placed fill materials were encountered during this exploration in the across most of the site of the site. Based on our experience, we anticipate fill materials likely exist at other locations between our borings. If low consistency and/or debris-laden fill materials are encountered during construction, typical recommendations would include undercutting and backfilling with structural fill and/or stabilizing in-place with fabric, stone, and/or other remedial techniques. Actual remedial recommendations can best be determined by the geotechnical engineer in the field at the time of construction

5.2 GROUNDWATER CONTROL

During the current exploration, groundwater was not encountered in any of the soil test borings. Based on the planned fill necessary to reach design subgrades, we do not anticipate significant groundwater control problems during mass grading, or foundation/utility excavation operations. However, if required, the design of a temporary dewatering system is usually the responsibility of the contractor.

At the time of constriction, groundwater, if encountered, must be lowered and continuously maintained at a minimum level depth of 3 feet below the working elevation to permit subgrade preparation and foundation excavation and construction. If required, design of a temporary dewatering system is usually conditions, and in consideration of the planned excavation depths, we believe a conventional construction dewatering system of trenches, sumps and pumps should be possible to control both groundwater and rainfall runoff. The need for a permanent dewatering system beneath the structure should be evaluated during the final subsurface exploration.

5.3 FILL PLACEMENT

5.3.1 FILL SUITABILITY

All materials to be used for backfill or structural fill should be evaluated and, if necessary, tested by NOVA prior to placement to determine if they are suitable for the intended use. In general, based upon the exploration results, a majority of the soils encountered across the site can be re-used as structural fill as well as general subgrade fill and backfill, provided that the fill material is free of glass, rubble, clay, rock, roots and organics. Any off-site materials used as fill should be approved by NOVA prior to acquisition.

Organic and/or debris-laden material is not suitable for re-use as structural fill. Note that zones of the existing fill included unsuitable materials and these soils should be segregated from soils to be used as backfill or structural fill. Topsoil, mulch and similar organic materials can be wasted in architectural areas. Debris-laden materials should be excavated, transported and disposed of off-site in accordance with appropriate solid waste rules and regulations.

5.3.2 SOIL COMPACTION

Fill should be placed in thin, horizontal loose lifts (maximum 8-inch) and compacted to at least 95 percent of the standard Proctor maximum dry density (ASTM D 698). The upper 8 inches of soil beneath pavements and slab-on-grade should be compacted to at least 98 percent of the maximum dry density. In confined areas, such as utility trenches or behind retaining walls, portable compaction equipment and thinner fill lifts (3 to 4 inches) may be necessary. Fill materials used in structural areas should have a target maximum dry density of at least 95 pounds per cubic foot (pcf). If lighter weight fill materials are used, the NOVA geotechnical engineer should be consulted to assess the impact on design recommendations.

Soil moisture content should be maintained within 3 percent of the optimum moisture content. We recommend that the grading contractor have equipment on site during earthwork for both drying and wetting fill soils. Moisture control may be difficult during rainy weather.

Filling operations should be observed by a NOVA soils technician, who can confirm suitability of material used and uniformity and appropriateness of compaction efforts. The technician can also document compliance with the specifications by performing field density tests using the drive cylinder, nuclear, or sand cone testing methods (ASTM D2937, D6938, or D1556, respectively). One test per 400 cubic yards and every 2 feet of placed fill is recommended, with test locations well distributed throughout the fill mass. When filling in small areas, at least one test per day per area should be performed.

The site should be graded during construction to maintain positive drainage away from the construction areas, to prevent ponding of storm water on the site during and shortly following significant rain events. The construction areas should be sealed and crowned with a smooth roller to minimize ponding water from storm events at the end of each day of work.

5.4 SHALLOW FOUNDATIONS

We understand that the proposed structure will replace the existing structure with a similar new 2-story brick maintenance facility. Structural loading information was not provided. Therefore, we have assumed that maximum wall and column loads will be on the order of 4 kips per linear foot (klf) and 150 kips, respectively

Based on the subsurface conditions encountered and the presumed structural loading, it is NOVA's professional opinion that the planned two-story structure can be supported on a conventional shallow foundation system subject to the recommendations contained herein.

Design: After the recommended site and subgrade preparation and fill placement, shallow foundation support of the proposed additions should be feasible. Foundations bearing on firm/stiff, undisturbed residual soils and/or properly compacted structural fill may be designed for a maximum allowable bearing pressure of 2,000 pounds per square foot (psf).

We recommend minimum foundation widths of 24 inches for ease of construction and to reduce the possibility of localized shear failures. Exterior foundation bottoms should be at least 18 inches below exterior grades for protection against frost damage.

Settlement: Settlements for shallow foundations were assessed based on the subsurface conditions encountered during this exploration using SPT values to estimate elastic modulus, based on published correlations and our previous experience. Based on the stated structural loads, the recommended soil bearing capacities and the presumed foundation elevations as discussed above, we expect post-construction, primary, total settlement beneath individual foundations will be less than 1 inch. We estimate differential settlement between adjacent foundations will be less than ½ inch. The final deflected shape of the structure will be dependent on actual foundation locations and loading.

To reduce differential settlement, if low consistency/relative density materials are encountered, a lower bearing capacity should be used, or the foundations should be extended to more competent materials. We anticipate that timely communication between the geotechnical engineer and the structural engineer, as well as other design and construction team members, will be required.

Please note that if actual design column loads differ significantly from the stated loads, NOVA should be notified immediately in order to re-evaluate the foundation recommendations and confirm anticipated settlements are still appropriate for the actual design loads.

Foundation excavations should be level and free of debris, ponded water, mud, and loose, frozen or water-softened soils. Concrete should be placed as soon as is practical after the foundation is excavated and the subgrade evaluated. Foundation concrete should not be placed on frozen or saturated soil. If a foundation excavation remains open overnight, or if rain or snow is imminent, a 3 to 4-inch thick "mud mat" of lean concrete should be placed in the bottom of the foundation to protect the bearing soils until reinforcing steel and concrete can be placed.

Foundation excavations should be evaluated by the NOVA geotechnical engineer prior to reinforcing steel placement to observe foundation subgrade preparation and confirm bearing pressure capacity.

5.5 SLABS-ON-GRADE

5.5.1 GENERAL

The conditions exposed at subgrade levels will vary across the site and may include structural fill or residual soils. Slabs-on-grade may be adequately supported on these subgrade conditions subject to the recommendations in this report. Slabs-on-grade should be jointed around columns and along walls to reduce cracking due to differential movement.

An underdrain system is not required. However, we recommend a minimum of 4-inches of graded aggregate base (GAB) beneath the slabs to:

- Reduce non-uniform support conditions,
- Provide a stable base to support construction traffic, and
- Provide a base that can be fine graded to design tolerances.

GAB should be compacted to 95 percent of the maximum dry density as determined by the modified Proctor compaction test (ASTM D 1557) and overlain by a conventional plastic vapor barrier.

Once grading is completed, the subgrade is usually exposed to adverse construction activities and weather conditions during the period of sub-slab utility installation. The subgrade should be well-drained to prevent the accumulation of water. If the exposed subgrade becomes saturated or frozen, the geotechnical engineer should be consulted.

After utilities have been installed and backfilled, a final subgrade evaluation should be performed by the geotechnical engineer immediately prior to slab-on-grade placement. If practical, proofrolling may be used to redensify the surface and to detect any soil that has become excessively wet or otherwise loosened.

5.5.2 Subgrade Modulus

A coefficient of subgrade reaction (***k***) of 125 pci may be used for conventional slab design where slabs bear upon subgrades prepared in accordance with previous recommendations.

Please note that this magnitude of ***k*** is intended to reflect the elastic response of soil beneath a typical floor slab under light loads with a small load contact area often measured in square inches, such as loads from forklifts, automobile/truck traffic or lightly loaded storage racks. The recommended coefficient of subgrade reaction (***k***) is not applicable for heavy slab loads caused by bulk storage or tall storage racks, or for mat foundation design.

Several design methods are applicable for conventional slab design. We have assumed that the slab designer will utilize the methods discussed in the American Concrete Institute (ACI) Committee 360 report, *"Guide to Design of Slabs-on-Ground, (ACI 360R-10)*.

5.6 PAVEMENT DESIGN RECOMMENDATIONS

Traffic loading for pavement design purposes was not provided. We have presumed the project's civil engineer will finalize the design of the asphalt and concrete pavements, incorporating the geotechnical recommendations from this exploration to ensure proper pavement design for the site based final design traffic loading. Below is the presumed traffic loading used to prepare this report:

- **For Standard-Duty Pavements** – 50 automobiles and 2 to 3 delivery van/panel trucks per day, 7 days per week, for a 20-year pavement life.

- **For Heavy-Duty Pavements** – in addition to similar traffic as the standard-duty pavements, we have presumed additional traffic loading of 3 garbage or similar vehicles per week for a 20-year pavement life.

If the above assumptions are incorrect, then NOVA should be afforded the opportunity to re-evaluate the following recommendations based on final design parameters. Additionally, if the planned pavements are to be constructed and utilized by construction traffic, the following pavement sections will likely prove insufficient for heavy truck traffic, such as concrete trucks or tractor-trailers used for construction delivery. Unexpected distress, reduced pavement life, and /or pre-mature failure of the pavement section could result if subjected to heavy construction traffic and the owner should be made aware of this risk. If the assumed traffic loading stated herein is not correct, NOVA should review actual pavement loading conditions to determine if revisions to these recommendations are warranted.

5.6.1 FLEXIBLE PAVEMENT

Based on the subsurface conditions encountered at this site, the recommended site preparation, and an estimated CBR of 4, our recommended flexible pavement design for this development is shown in the following table:

Pavement Section	Standard Duty	Heavy Duty
Asphaltic Surface Course (9.5 or 12.5 mm SuperPave, GDOT approved mix w/ anti-stripping agent)	2 inches of 12.5 mm	1½ inches of 12.5 mm
Asphaltic Base Course (19 mm SuperPave, GDOT approved mix)	n/a	2 inches

Graded Aggregate Base (GAB) or Concrete Base Course (from an approved GDOT source)	6 inches	8 inches
Stabilized Subgrade (compacted to a minimum 98% of the standard Proctor maximum dry density)	12 inches	12 inches

We recommend a minimum compaction of 98 percent of the maximum dry density for the Graded Aggregate Base (GAB as determined by the modified Proctor compaction test (ASTM D 1557, Method C). The crushed stone should conform to applicable sections of the current GDOT Standard Specifications. All asphalt material and paving operations should meet applicable specifications of the Asphalt Institute and GDOT. A NOVA technician should observe placement and perform density testing of the base course material and asphalt.

5.6.2 Rigid Pavement

A rigid pavement section is recommended in areas where heavy truck traffic, excessive braking, sharp wheel turning and/or point loads, like dumpsters and loading docks are planned.

Based on the subsurface conditions at the site, the recommended site preparation, presumed traffic loads and an estimated subgrade modulus (k) of 125 psi/inch for traffic or wheel loading, our recommended rigid pavement design is as follows:

Pavement Section	Heavy Duty
GDOT approved air-entrained concrete mix	4¼ inches
Graded Aggregate Base (GAB) or Concrete Base Course (from an approved GDOT source, compacted to a minimum 98% of the modified Proctor maximum dry density, per ASTM D1557)	6 inches
Stabilized Subgrade (compacted to a minimum 98% of the standard Proctor maximum dry density per ASTM D698)	12 inches

All concrete materials and placement should conform to applicable GDOT specifications. We recommend that a non-woven geotextile (about 3 feet wide) be placed beneath the construction joints to prevent upward "pumping" movement of soil fines through the joints.

We recommend using concrete with a minimum compressive strength of 4,000 psi and a minimum 28-day flexural strength (modulus of rupture) of at least 600 pounds per square inch, based on 3-point loading of concrete beam test samples. Layout of the saw-cut control joints should form square panels of about 10 feet, and the depth of saw-cut joint should be approximately ¼ of the concrete slab thickness. The joints should be sawed within 6 hours of concrete placement or as soon as the concrete has developed sufficient strength to support workers and equipment.

We recommend allowing NOVA to review and comment on the final concrete pavement design, including section and joint details (type of joints, joint spacing, etc.), prior to the start of construction. For further details on concrete pavement construction, please reference the "Building Quality Concrete Parking Areas", published by the Portland Cement Association.

5.7 RETAINING WALLS

5.7.1 Cast-in-Place Walls

The magnitude and distribution of earth pressures against below grade walls depends on the deformation condition (rotation) of the wall, soil properties and water conditions. When the soil behind the wall is prevented from lateral strain, the resulting force is known as the at-rest earth pressure (K_O). If the retaining structure moves away from the soil mass, the earth pressure decreases with the increasing lateral expansion until a minimum pressure, known as the active earth pressure (K_A), is reached. If the wall is forced into the soil mass, the earth pressure increases until a maximum pressure, known as the passive earth pressure (K_P), is obtained.

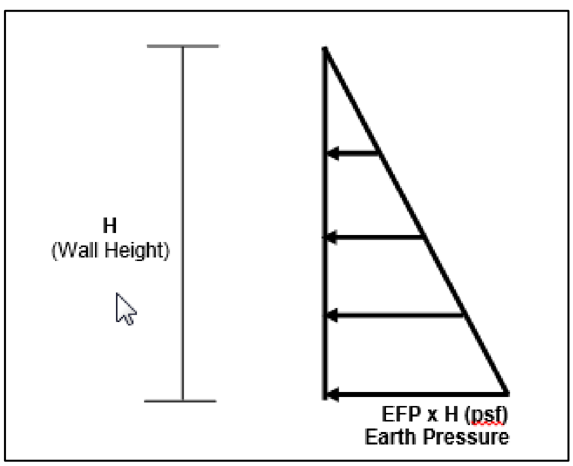
Free-standing retaining walls are usually designed for active earth pressures. Rigid basement walls are typically designed for at-rest earth pressures. If basement walls will be backfilled before they are braced by the floor slabs, they should also be designed to withstand active earth pressures as self-supporting cantilever walls. However, the earth pressures must be compatible with the wall rotation, which is limited by the wall rigidity, foundation support conditions and connections to adjoining structures. If active earth pressure development requires horizontal wall movements that cannot occur, or which are architecturally undesirable, walls should be designed for an intermediate pressure based on restraint conditions.

Laboratory analysis to determine actual soil shear strength properties was beyond the authorized scope of services. Based on our experience with similar soils and construction, we have provided the earth pressure estimates shown in the following table.

Earth Pressure Condition	Earth Pressure Coefficient	Equivalent Fluid Pressure (pcf)	
		Above Water Table	Below Water Table
Soil Backfill			
Active (K_a)	0.33	40	80
At-Rest (K_o)	0.50	60	89
Passive (K_p)	3.00	150*	TBD**
#57 Stone Backfill			
Active (K_a)	0.29	35	75
At-Rest (K_o)	0.46	55	84
Passive (K_p)	3.40	400*	TBD**

* *Passive earth pressure is frequently used in retaining wall design to resist active earth pressures. Wall movements required to develop full passive earth pressures are significantly greater than movements necessary for active earth pressures. Consequently, this passive pressure value has been reduced by at least 50% for wall design*

** *Passive earth pressure for submerged wall design shall be determined on a case-by-case basis.*



We recommend a value of 0.35 as the coefficient of friction (sliding resistance) between wall foundations and the underlying residual or fill soils. A coefficient of friction of 0.45 is recommended for foundations bearing on PWR. A coefficient of friction of 0.5 is recommended for foundations bearing on rock. These design values do not contain a safety factor.

Our lateral earth pressure recommendations assume that:

- The ground surface adjacent to the wall is level,
- Residual soils will be reused for wall backfill, compacted between 95% to 98% of the standard Proctor maximum dry density,
- Soil backfill weight is a maximum of 120 pcf,
- Heavy construction equipment does not operate within 5 feet of the walls,
- A constantly functioning drainage system is installed between the wall and the soil backfill to prevent hydrostatic pressures from acting on the wall,
- Foundations or other significant surcharge loads are located outside the wall a distance at least equal to the wall height,
- For active earth pressure, wall must rotate about base, with top lateral movements of about 0.002 H to 0.004 H, where H is wall height,
- For passive earth pressure to develop, the wall must move horizontally to mobilize resistance.

5.7.2 Alternative Walls – Fill Areas

We understand that a mechanically stabilized earth (MSE) wall system may be used. MSE wall systems consist of thin strips or grids made of metal or plastic that are placed horizontally between backfill layers at right angles to the wall face. The strips/grids provide tensile reinforcement within the fill, as well as tie the precast concrete wall facing to the soil mass. Because the system is a self-supporting soil mass, the “design bearing pressure” concept, typically used in conventional cast-in-place retaining wall design to size the wall foundations, is generally not applicable. The reinforced soil system is interpreted to behave as a flexible, mass gravity wall, consequently, the design usually considers the resistance to wall overturning and global slope stability, as well as the internal stability of the reinforced earth system. Wall system design must also consider any surcharges caused by sloping fill, the potential impact of leaks from water or sewer lines, and the proximity of adjacent buildings.

Typically, these walls are a design/build system that are the responsibility of the contractor and his specialty wall subcontractor. The specifications usually state that the wall supplier is to design, install, warrant and guarantee the MSE wall without reliance on other entities. This includes the determination and confirmation of foundation and fill parameters used in design, such as total and effective shear stress parameters, as well as settlement and deformation characteristics of the wall system.

Please note that NOVA has not performed a geotechnical exploration for MSE walls at this site. The bearing pressures and earth pressures presented in other sections of this report may not be appropriate for MSE wall design. Consequently, we recommend that the wall supplier confirm the parameters used in his MSE wall design.

6.0 LIMITATIONS

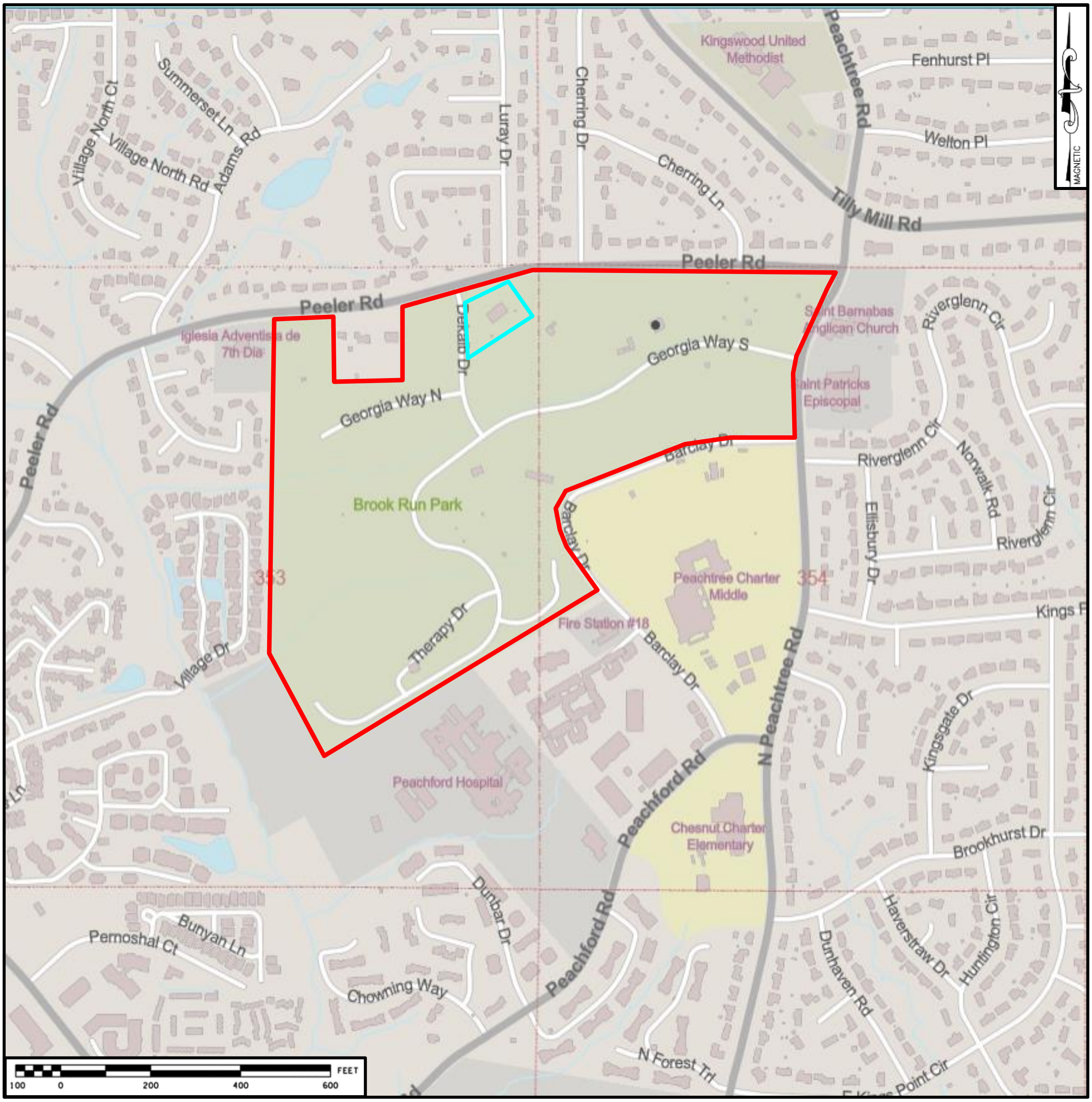
The findings, conclusions and recommendations presented in this report represent our professional opinions concerning subsurface conditions at the site. The opinions presented are relative to the dates of our exploration and should not be relied on to represent conditions at significantly later dates or at locations not explored. The opinions included herein are based on information provided to us, the data obtained at specific locations during the study and our experience. If additional information becomes available that might impact our geotechnical opinions, it will be necessary for NOVA to review the information, reassess the potential concerns, and re-evaluate our conclusions and recommendations.

Regardless of the thoroughness of a geotechnical exploration, there is the possibility that conditions between test locations will differ from those encountered at specific test locations, that conditions are not as anticipated by the designers and/or the contractors, or that either natural events or the construction process have altered the subsurface conditions. These variations are an inherent risk associated with subsurface conditions in this region and the approximate methods used to obtain the data. These variations may not be apparent until construction.

This report is intended for the sole use of the above-mentioned project. The scope of services performed during this study may not satisfy other users' requirements. Use of this report or the findings, conclusions or recommendations by others will be at the sole risk of the user. NOVA is not responsible or liable for the interpretation by others of the data in this report, nor their conclusions, recommendations, or opinions.

Our professional services have been performed, our findings obtained, our conclusions derived, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and local practices in the State of Georgia. This warranty is in lieu of all other statements or warranties, either expressed or implied.

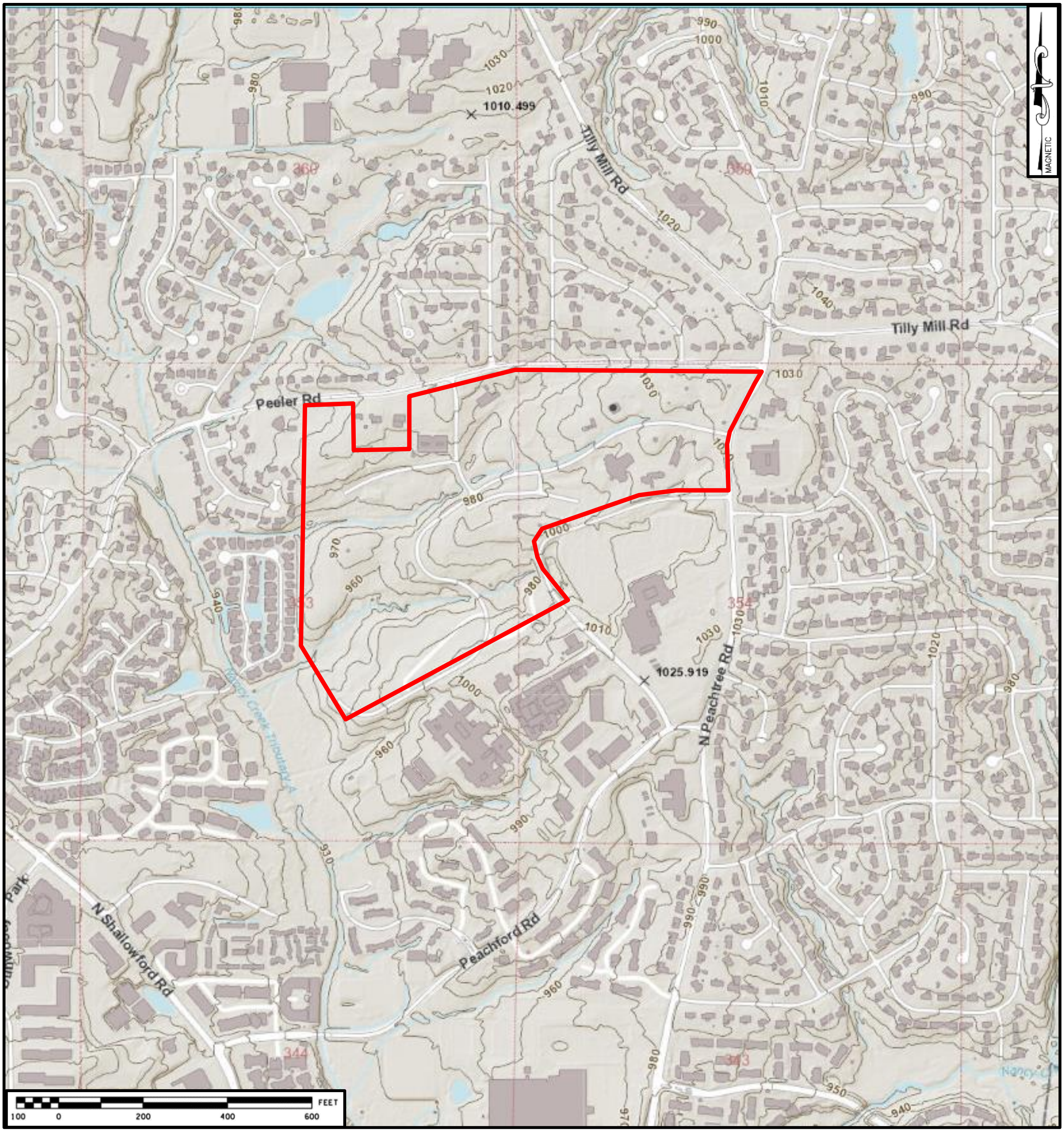
APPENDIX A
Figures and Maps



APPROXIMATE BOUNDARY OF BROOK RUN PARK

APPROXIMATE BOUNDARY OF MAINTENANCE FACILITY

FIGURE 1 SITE LOCATION Source: Dekalb County Geographic Information System (GIS) map viewer Scale: Graphic as shown above	NOVA ENGINEERING AND ENVIRONMENTAL	DUNWOODY MAINTENANCE FACILITY City of Dunwoody Dunwoody, Dekalb County, Georgia NOVA Project Number 10103-2024073

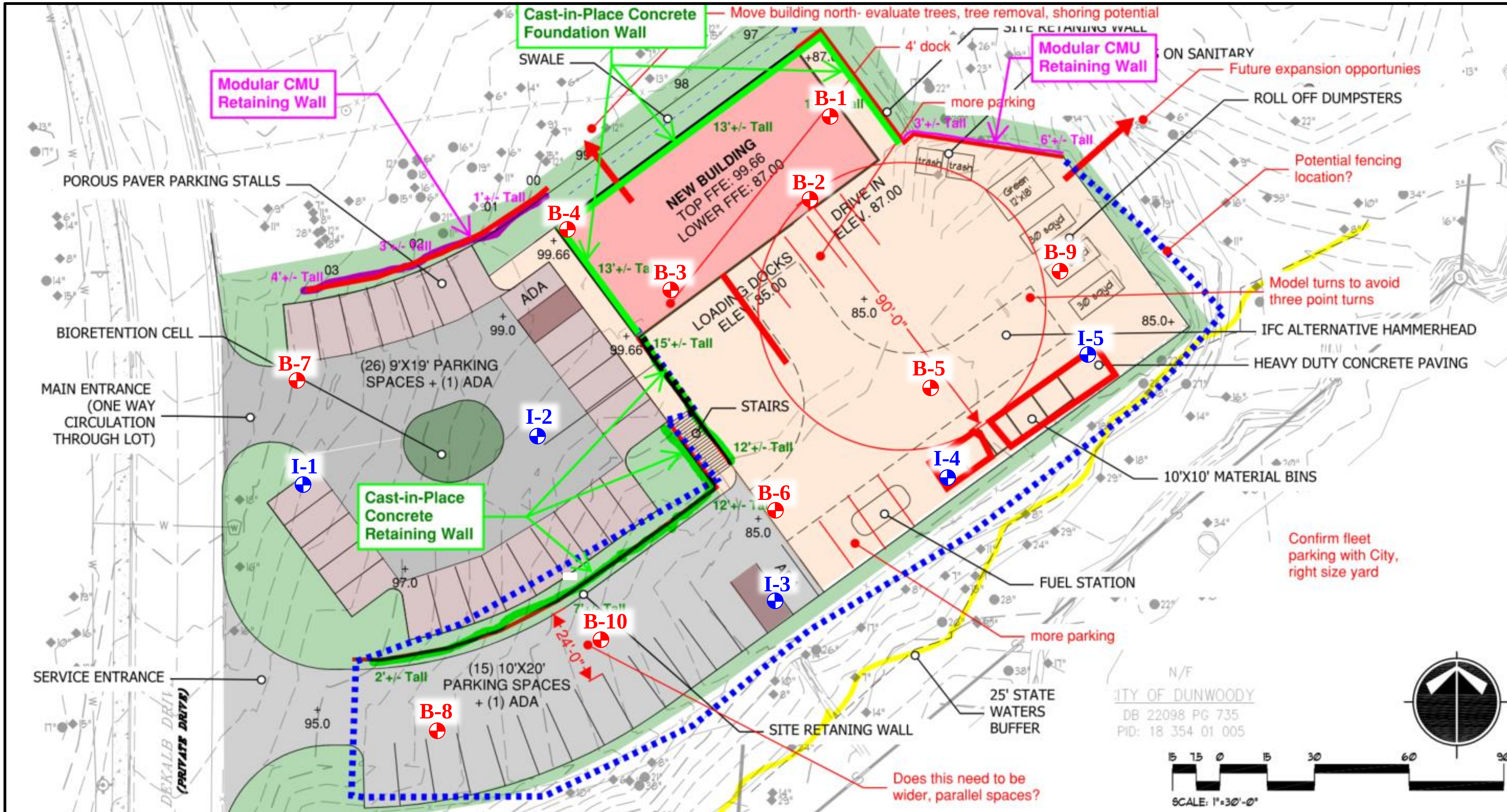


APPROXIMATE BOUNDARY OF SUBJECT PROPERTY

FIGURE 2
TOPOGRAPHIC MAP
Source: Dekalb County Geographic Information System (GIS) map viewer
Scale: Graphic as shown above

NOVA
ENGINEERING AND ENVIRONMENTAL

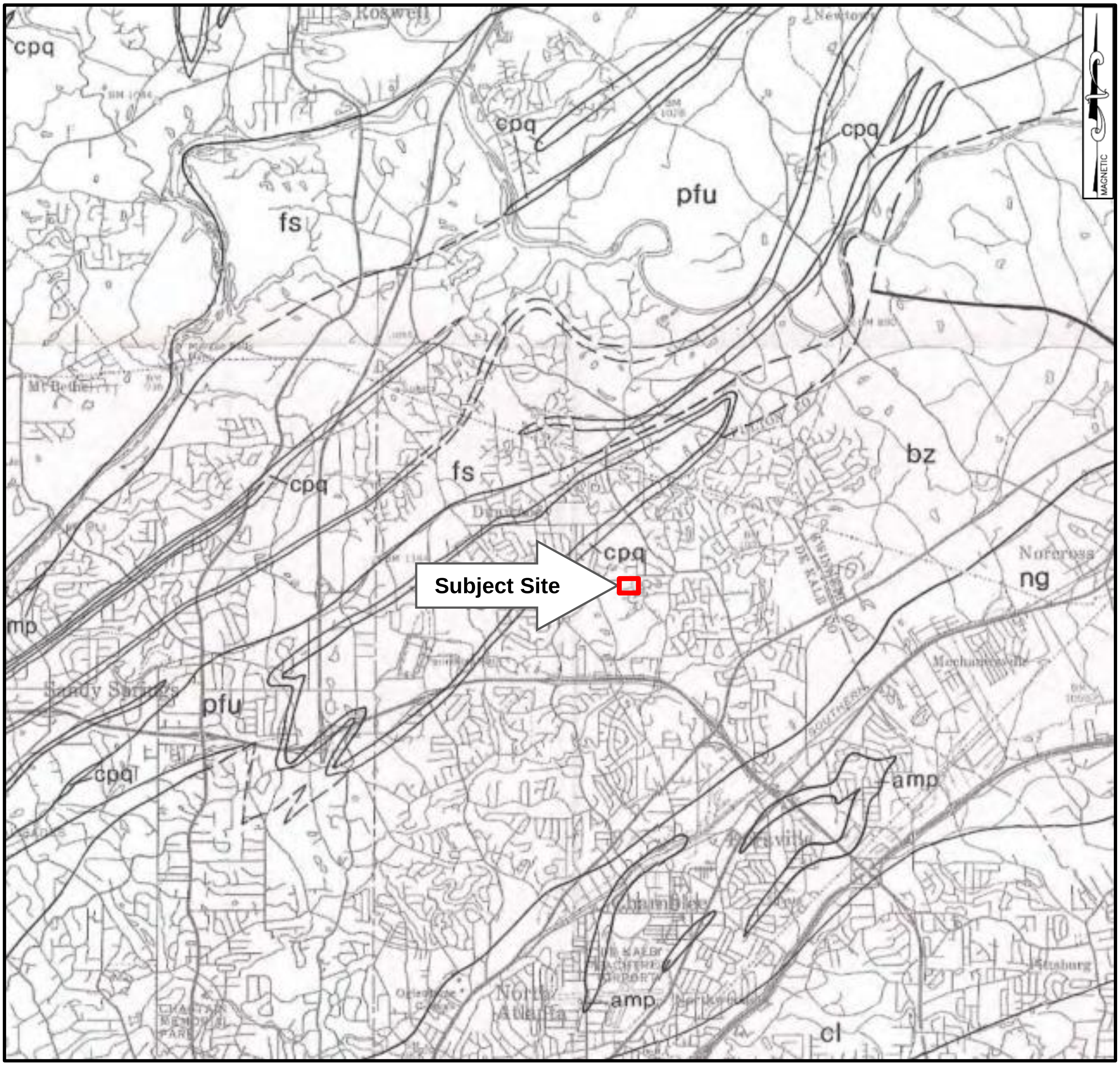
DUNWOODY MAINTENANCE FACILITY
City of Dunwoody
Dunwoody, DeKalb County, Georgia
NOVA Project Number 10103-2024073



APPROXIMATE LOCATION OF NOVA SOIL TEST BORINGS

APPROXIMATE LOCATION OF NOVA PERCOLATION TEST BORINGS

PROPOSED BORING LOCATION PLAN SCALE: Graphic SOURCE: New Maintenance Facility at Brook Run Park, prepared by Breedlove Land Planning, dated March 18, 2024.	NOVA ENGINEERING AND ENVIRONMENTAL	**DUNWOODY MAINTENANCE FACILITY** City of Dunwoody Dunwoody, DeKalb County, Georgia **NOVA Project Number 10103-2024073**



Southern Piedmont Province and Brevard Fault Zone Ductilely Sheared Rocks:
Undifferentiated ductilely sheared button schists (bz).

APPROXIMATE SITE BOUNDARY

FIGURE 4
REGIONAL GEOLOGY
Source: Geology of the Greater Atlanta Region, McConnell and Abrams, DNR, EPD, GGS, Bulletin 96 – 1984.
Scale: Not To Scale

NOVA ENGINEERING AND ENVIRONMENTAL

DUNWOODY MAINTENANCE FACILITY
City of Dunwoody
Dunwoody, DeKalb County, Georgia
NOVA Project Number 10103-2024073

APPENDIX B
Subsurface Data

KEY TO SYMBOLS AND CLASSIFICATIONS

DRILLING SYMBOLS

	Standard Penetration Testing Sample
	Undisturbed Sample (UD)
	Auger without Sampling
	Rock Core Sample
	Standard Penetration Resistance (ASTM D1586)
	Dynamic Cone Penetrometer (DCP) Resistance
	Water Table at least 24 Hours after drilling
	Water Table 1 Hour or less after drilling
50/2"	Number of Blows (50) to Drive the Spoon a Number of Inches (2)
NX, NQ	Core Barrel Sizes: 2⅛- and 2-Inch Diameter Rock Core, Respectively
REC	Percentage of Rock Core Recovered
RQD	Rock Quality Designation - Percentage of Recovered Core Segments 4 or more Inches Long
	Loss of Drilling Fluid
N/E	Not Encountered
N/M	Not Measured
C	Boring Cave-in Depth
WOH	Weight of Hammer

DRILLING PROCEDURES

Soil sampling and standard penetration testing performed in general accordance with ASTM D1586-18$^{\varepsilon 1}$. The standard penetration resistance (N-value) is the number of blows of a 140-pound hammer falling 30 inches to drive a 2-inch O.D., 1.375-inch I.D. split-barrel sampler one foot. Core drilling performed in general accordance with ASTM D2113-14. The undisturbed sampling procedure is described by ASTM D1587-15. Unless other arrangements are made, NOVA will dispose of all soil and rock samples at the time of report submission.

Paving

Gravel / Graded Aggregate Base

Fill

Topsoil

Alluvium

Poorly Graded Sand - SP

Well Graded Sand - SW

Silty Sand - SM

Clayey Sand - SC

Poorly graded silty, clayey sand - SM/SC

Clayey Sand and Gravel - SC/GC

Silty Sand and Gravel - SM/GM

Silt - ML

Elastic Silt - MH

Low Plasticity Clay - CL

High Plasticity Clay - CH

Partially Weathered Rock (PWR)

Rock

NOVA

CORRELATION OF PENETRATION RESISTANCE WITH RELATIVE DENSITY AND CONSISTENCY

	Number of Blows, "N"	Approximate Relative Density
	0 – 4	Very Loose
	5 – 10	Loose
SANDS	11 – 30	Medium Dense
	31 – 50	Dense
	Over 50	Very Dense

	Number of Blows, "N"	Approximate Consistency
	0 – 2	Very Soft
	3 – 4	Soft
SILTS	5 – 8	Firm
and	9 – 15	Stiff
CLAYS	16 – 30	Very Stiff
	31 – 50	Hard
	Over 50	Very Hard

SOIL CLASSIFICATION CHART

COARSE GRAINED SOILS	**GRAVELS**	Clean Gravel less than 5% fines	GW	Well graded gravel
			GP	Poorly graded gravel
		Gravels with Fines more than 12% fines	GM	Silty gravel
			GC	Clayey gravel
	SANDS	Clean Sand less than 5% fines	SW	Well graded sand
			SP	Poorly graded sand
		Sands with Fines more than 12% fines	SM	Silty sand
			SC	Clayey sand
FINE GRAINED SOILS	**SILTS AND CLAYS** Liquid Limit less than 50	Inorganic	CL	Lean clay
			ML	Silt
		Organic	OL	Organic clay and silt
	SILTS AND CLAYS Liquid Limit 50 or more	Inorganic	CH	Fat clay
			MH	Elastic silt
		Organic	OH	Organic clay and silt
HIGHLY ORGANIC SOILS		Organic matter, dark color, organic odor	PT	Peat

PARTICLE SIZE IDENTIFICATION

GRAVELS	Coarse	¾ inch to 3 inches
	Fine	No. 4 to ¾ inch
SANDS	Coarse	No. 10 to No. 4
	Medium	No. 40 to No. 10
	Fine	No. 200 to No. 40
SILTS AND CLAYS		Passing No. 200

NOVA

TEST BORING RECORD
B-1

PROJECT: Dunwoody Maintenance Facility | PROJECT NO.: 10103-2024073
CLIENT: City of Dunwoody
PROJECT LOCATION: Dunwoody, DeKalb County, Georgia
LOCATION: 33.935382° N, -84.300289° W | ELEVATION: 993 feet-MSL
DRILLER: Piedmont Environmental Drilling | LOGGED BY: AMH
DRILLING METHOD: Hollow Stem Auger | DATE: 07/29/2024
DEPTH TO - WATER> INITIAL: NE | AFTER 24 HOURS: NM | CAVING> 11 feet

Depth (feet)	Elevation (ft-MSL)	Description	N-Value
0	993	ASPHALT: 3.5 inches	
		FILL: Firm tan orange and gray coarse to fine sandy SILT	7
	990	RESIDUUM: Loose to medium dense tan orange and brown silty corase to fine SAND with quartz fragments	15
5			9
	985		13
10			22
	980	Dense tan orange and brown silty coarse to fine SAND with quartz fragments	
15			47
		Boring Terminated at 15.5 ft.	

Graphic Depiction: ● BLOW COUNT; ▲ NATURAL MOISTURE; PLASTIC LIMIT |——| LIQUID LIMIT; scale 10, 20, 30, 40, 60, 100

Boring elevations were estimated from the Existing Conditions Plan.
Where quartz and rock fragments were encountered, N-Value(s) may have been exaggerated.

This information pertains only to this boring and should not be interpreted as being indicative of the site.

NOVA

TEST BORING RECORD B-2

PROJECT: Dunwoody Maintenance Facility PROJECT NO.: 10103-2024073
CLIENT: City of Dunwoody
PROJECT LOCATION: Dunwoody, DeKalb County, Georgia
LOCATION: 33.935361° N, -84.300253° W ELEVATION: 993 feet-MSL
DRILLER: Piedmont Environmental Drilling LOGGED BY: AMH
DRILLING METHOD: Hollow Stem Auger DATE: 07/29/2024
DEPTH TO - WATER> INITIAL: NE AFTER 24 HOURS: NM CAVING> 7 feet

Depth (feet)	Elevation (ft-MSL)	Description	Groundwater	N-Value
0	993	ASPHALT: 3 inches		
		FILL: Firm orange brown and gray micaceous coarse to fine sandy SILT		6
				13
	990	RESIDUUM: Loose to medium dense orange brown and gray micaceous silty coarse to fine SAND		
5				9
		Medium dense white tan and black very micaceous silty coarse to fine SAND	Caving	
				11
10				13
15				24
15.5		Boring Terminated at 15.5 ft.		

Graphic Depiction: BLOW COUNT; NATURAL MOISTURE; PLASTIC LIMIT — LIQUID LIMIT; scale 10, 20, 30, 40, 60, 100

This information pertains only to this boring and should not be interpreted as being indicative of the site.

Boring elevations were estimated from the Existing Conditions Plan.

NOVA

TEST BORING RECORD
B-3

PROJECT: Dunwoody Maintenance Facility PROJECT NO.: 10103-2024073
CLIENT: City of Dunwoody
PROJECT LOCATION: Dunwoody, DeKalb County, Georgia
LOCATION: 33.935244° N, -84.300436° W ELEVATION: 993 feet-MSL
DRILLER: Piedmont Environmental Drilling LOGGED BY: AMH
DRILLING METHOD: Hollow Stem Auger DATE: 07/29/2024
DEPTH TO - WATER> INITIAL: NE AFTER 24 HOURS: NM CAVING> 7 feet

Depth (feet)	Elevation (ft-MSL)	Description	Sample Type	N-Value
0	993	ASPHALT: 2 inches		
1		FILL: Stiff brown and gray micaceous coarse to fine sandy SILT	SPT	9
3	990	RESIDUUM: Medium dense red orange and tan micaceous silty coarse to fine SAND	SPT	22
4.5		Loose tan orange and brown micaceous silty coarse to fine SAND	SPT	7
8	985		SPT	7
10			SPT	6
15			SPT	7
15.5		Boring Terminated at 15.5 ft.		

Graphic Depiction: BLOW COUNT; NATURAL MOISTURE; PLASTIC LIMIT |—| LIQUID LIMIT; scale 10, 20, 30, 40, 60, 100

This information pertains only to this boring and should not be interpreted as being indicative of the site.

Boring elevations were estimated from the Existing Conditions Plan.

TEST BORING RECORD B-4

PROJECT: Dunwoody Maintenance Facility	PROJECT NO.: 10103-2024073
CLIENT: City of Dunwoody	
PROJECT LOCATION: Dunwoody, DeKalb County, Georgia	
LOCATION: 33.935289° N, -84.300569° W	ELEVATION: 993 feet-MSL
DRILLER: Piedmont Environmental Drilling	LOGGED BY: AMH
DRILLING METHOD: Hollow Stem Auger	DATE: 07/29/2024
DEPTH TO - WATER> INITIAL: NE	AFTER 24 HOURS: NM
CAVING>: 11 feet	

Depth (feet)	Elevation (ft-MSL)	Description	Sample Type	N-Value
0	993	ASPHALT: 3.5 inches		
		FILL: Loose brown gray very micaceous silty coarse to fine SAND	SPT	7
			SPT	13
	990	RESIDUUM: Loose orange tan and brown very micaceous silty coarse to fine SAND		
5		Loose to medium dense white tan and orange very micaceous silty coarse to fine SAND	SPT	5
	985		SPT	7
10			SPT	13
	980			
15			SPT	15
		Boring Terminated at 15.5 ft.		

Graphic Depiction: ● BLOW COUNT ▲ NATURAL MOISTURE PLASTIC LIMIT ├──┤ LIQUID LIMIT (10, 20, 30, 40, 60, 100)

Boring elevations were estimated from the Existing Conditions Plan.

NOVA

TEST BORING RECORD B-5

PROJECT:	Dunwoody Maintenance Facility	PROJECT NO.:	10103-2024073
CLIENT:	City of Dunwoody		
PROJECT LOCATION:	Dunwoody, DeKalb County, Georgia		
LOCATION:	33.933167° N, -84.300192° W	ELEVATION:	984 feet-MSL
DRILLER:	Piedmont Environmental Drilling	LOGGED BY:	AMH
DRILLING METHOD:	Hollow Stem Auger	DATE:	07/29/2024

DEPTH TO - WATER> INITIAL: NE AFTER 24 HOURS: NM CAVING> 6.5 feet

Depth (feet)	Elevation (ft-MSL)	Description	Sample Type	N-Value
0		ASPHALT: 3.75 inches		
		FILL: Firm gray red and brown micaceous coarse to fine sandy SILT with rock fragments		6
		RESIDUUM: Medium dense tan gray and orange micaceous silty coarse to fine SAND with quartz fragments		28
5	980			25
				16
10	975			23
		Boring Terminated at 10.5 ft.		

Graphic Depiction: BLOW COUNT; NATURAL MOISTURE; PLASTIC LIMIT — LIQUID LIMIT; 10 20 30 40 60 100

Depth scale: 0, 5, 10, 15, 20, 25, 30, 35; Elevation scale: 980, 975, 970, 965, 960, 955, 950

This information pertains only to this boring and should not be interpreted as being indicative of the site.

Boring elevations were estimated from the Existing Conditions Plan. Where quartz and rock fragments were encountered, N-Value(s) may have been exaggerated.

TEST BORING RECORD B-6

NOVA

PROJECT: Dunwoody Maintenance Facility
PROJECT NO.: 10103-2024073
CLIENT: City of Dunwoody
PROJECT LOCATION: Dunwoody, DeKalb County, Georgia
LOCATION: 33.935058° N, -84.30035° W
ELEVATION: 985 feet-MSL
DRILLER: Piedmont Environmental Drilling
LOGGED BY: AMH
DRILLING METHOD: Hollow Stem Auger
DATE: 07/29/2024
DEPTH TO - WATER> INITIAL: NE   AFTER 24 HOURS: NM   CAVING> 6.5 feet

Depth (feet)	Elevation (ft-MSL)	Description	Sample Type	N-Value
0	985	ASPHALT: 3.5 inches		
		FILL: Firm brown gray micaceous medium to fine sandy SILT with rock fragments	SPT	7
		RESIDUUM: Firm gray red and brown micaceous coarse to fine sandy SILT with rock fragments	SPT	22
5	980	Medium dense orange tan and brown micaceous silty coarse to fine SAND	SPT	20
		Loose to medium dense orange black and tan very micaceous silty coarse to fine SAND	SPT	6
10	975		SPT	18
		Boring Terminated at 10.5 ft.		

Depth scale: 0, 5, 10, 15, 20, 25, 30, 35; Elevation scale: 985, 980, 975, 970, 965, 960, 955, 950.

Graphic Depiction: ● BLOW COUNT ▲ NATURAL MOISTURE; PLASTIC LIMIT |—| LIQUID LIMIT; scale 10, 20, 30, 40, 60, 100

This information pertains only to this boring and should not be interpreted as being indicative of the site.

Boring elevations were estimated from the Existing Conditions Plan.
Where quartz and rock fragments were encountered, N-Value(s) may have been exaggerated.

NOVA

TEST BORING RECORD B-7

PROJECT:	Dunwoody Maintenance Facility
PROJECT NO.:	10103-2024073
CLIENT:	City of Dunwoody
PROJECT LOCATION:	Dunwoody, DeKalb County, Georgia
LOCATION:	33.935169° N, -84.300828° W
ELEVATION:	1003 feet-MSL
DRILLER:	Piedmont Environmental Drilling
LOGGED BY:	AMH
DRILLING METHOD:	Hollow Stem Auger
DATE:	07/29/2024
DEPTH TO - WATER> INITIAL:	NE
AFTER 24 HOURS:	NM
CAVING>	6.5 feet

Graphic Depiction: ● BLOW COUNT; ▲ NATURAL MOISTURE; PLASTIC LIMIT |——| LIQUID LIMIT; scale 10, 20, 30, 40, 60, 100

Depth (feet)	Elevation (ft-MSL)	Description	Groundwater	N-Value
0	1003	ASPHALT: 3.5 inches		
		FILL: Medium dense orange gray and brown micaceous silty coarse to fine SAND with rock fragments		12
				13
	1000	RESIDUUM: Medium dense red orange and tan very micaceous silty coarse to fine SAND		
5		Medium dense white tan and orange very micaceous silty coarse to fine SAND		10
			Caving	
	995			13
10				9
		Boring Terminated at 10.5 ft.		

This information pertains only to this boring and should not be interpreted as being indicative of the site.

Boring elevations were estimated from the Existing Conditions Plan.
Where quartz and rock fragments were encountered, N-Value(s) may have been exaggerated.

NOVA TEST BORING RECORD B-8	PROJECT: Dunwoody Maintenance Facility — PROJECT NO.: 10103-2024073 CLIENT: City of Dunwoody PROJECT LOCATION: Dunwoody, DeKalb County, Georgia LOCATION: 33.934872° N, -84.300686° W — ELEVATION: 993 feet-MSL DRILLER: Piedmont Environmental Drilling — LOGGED BY: AMH DRILLING METHOD: Hollow Stem Auger — DATE: 07/29/2024 DEPTH TO - WATER> INITIAL: NE — AFTER 24 HOURS: NM — CAVING> 6.5 feet

This information pertains only to this boring and should not be interpreted as being indicative of the site.

Depth (feet)	Elevation (ft-MSL)	Description	N-Value
0		ASPHALT: 3 inches	
		FILL: Firm orange gray and brown micaceous medium to fine sandy SILT with rock fragments and wood debris	6
	990	RESIDUUM: Medium dense orange red and tan very micaceous silty coarse to fine SAND	15
5			12
	985		11
10		Loose orange tan and black very micaceous silty coarse to fine SAND	6
		Boring Terminated at 10.5 ft.	

Graphic Depiction: BLOW COUNT; NATURAL MOISTURE; PLASTIC LIMIT — LIQUID LIMIT; 10 20 30 40 60 100

Depth scale: 0, 5, 10, 15, 20, 25, 30, 35. Elevation scale: 990, 985, 980, 975, 970, 965, 960.

Boring elevations were estimated from the Existing Conditions Plan.
Where quartz and rock fragments were encountered, N-Value(s) may have been exaggerated.

NOVA

TEST BORING RECORD
B-9

PROJECT: Dunwoody Maintenance Facility PROJECT NO.: 10103-2024073
CLIENT: City of Dunwoody
PROJECT LOCATION: Dunwoody, DeKalb County, Georgia
LOCATION: 33.935356° N, -84.300092° W ELEVATION: 986 feet-MSL
DRILLER: Piedmont Environmental Drilling LOGGED BY: AMH
DRILLING METHOD: Hollow Stem Auger DATE: 07/29/2024
DEPTH TO - WATER> INITIAL: NE AFTER 24 HOURS: NM CAVING> 7 feet

Depth (feet)	Elevation (ft-MSL)	Description	Graphic	Groundwater	Sample Type	N-Value
0		ASPHALT: 3 inches				
	985	CONCRETE: 4 inches				11
		FILL: Stiff red orange and gray micceous coarse to fine sandy SILT with rock fragments and root hairs				36
5		RESIDUUM: Dense tan gray and orange very micaceous silty coarse to fine SAND with quartz fragments				31
	980	Medium dense tan gray and orange very micaceous silty coarse to fine SAND				23
10		Very dense tan gray and orange very micaceous silty coarse to fine SAND				58
	975	Boring Terminated at 10.5 ft.				
15	970					
20	965					
25	960					
30	955					
35						

Graphic Depiction

BLOW COUNT
NATURAL MOISTURE
PLASTIC LIMIT LIQUID LIMIT
10 20 30 40 60 100

This information pertains only to this boring and should not be interpreted as being indicative of the site.

Boring elevations were estimated from the Existing Conditions Plan.
Where quartz and rock fragments were encountered, N-Value(s) may have been exaggerated.

TEST BORING RECORD B-10

NOVA

PROJECT: Dunwoody Maintenance Facility	PROJECT NO.: 10103-2024073
CLIENT: City of Dunwoody	
PROJECT LOCATION: Dunwoody, DeKalb County, Georgia	
LOCATION: 33.934944° N, -84.300528° W	ELEVATION: 991 feet-MSL
DRILLER: Piedmont Environmental Drilling	LOGGED BY: AMH
DRILLING METHOD: Hollow Stem Auger	DATE: 07/29/2024
DEPTH TO - WATER> INITIAL: NE	AFTER 24 HOURS: NM CAVING> 7 feet

Depth (feet)	Elevation (ft-MSL)	Description	N-Value
0–0.2	991	ASPHALT: 2 inches	
0.2–0.6		CONCRETE: 5.5 inches	
0.6–3.5	990	RESIDUUM: Loose to medium dense orange red and tan very micaceous silty coarse to fine SAND	8, 20
3.5–8.5	985	Loose to very loose orange red and tan very micaceous silty coarse to fine SAND	6, 4
8.5–10.5		Medium dense tan brown and orange very micaceous silty coarse to fine SAND	13
10.5	980	Boring Terminated at 10.5 ft.	

Graphic Depiction: BLOW COUNT; NATURAL MOISTURE; PLASTIC LIMIT — LIQUID LIMIT; 10 20 30 40 60 100

This information pertains only to this boring and should not be interpreted as being indicative of the site.

Boring elevations were estimated from the Existing Conditions Plan.
Where quartz and rock fragments were encountered, N-Value(s) may have been exaggerated.

NOVA

TEST BORING RECORD
I-1

PROJECT: Dunwoody Maintenance Facility PROJECT NO.: 10103-2024073
CLIENT: City of Dunwoody
PROJECT LOCATION: Dunwoody, DeKalb County, Georgia
LOCATION: 33.935091° N, -84.300833° W ELEVATION: 1003 feet-MSL
DRILLER: Piedmont Environmental Drilling LOGGED BY: AMH
DRILLING METHOD: Hollow Stem Auger DATE: 07/29/2024
DEPTH TO - WATER> INITIAL: NE AFTER 24 HOURS: NM CAVING> NE

Depth (feet)	Elevation (ft-MSL)	Description	Graphic	Groundwater	Sample Type	N-Value	Graphic Depiction
0		ASPHALT: 3.5 inches					
		FILL: Orange gray and brown micaceous silty coarse to fine SAND with rock fragments					
	1000	RESIDUUM: Red orange and tan very micaceous silty coarse to fine SAND					
5		Boring Terminated at 5 ft.					
10	995						
15	990						
20	985						
25	980						
30	975						
35	970						

Graphic Depiction: ● BLOW COUNT ▲ NATURAL MOISTURE PLASTIC LIMIT |———| LIQUID LIMIT
10 20 30 40 60 100

This information pertains only to this boring and should not be interpreted as being indicative of the site.

Boring elevations were estimated from the Existing Conditions Plan.
Infiltration testing locations were each augered to 5 feet.

NOVA

TEST BORING RECORD I-2

PROJECT:	Dunwoody Maintenance Facility	PROJECT NO.:	10103-2024073
CLIENT:	City of Dunwoody		
PROJECT LOCATION:	Dunwoody, DeKalb County, Georgia		
LOCATION:	33.935122° N, -84.300589° W	ELEVATION:	995 feet-MSL
DRILLER:	Piedmont Environmental Drilling	LOGGED BY:	AMH
DRILLING METHOD:	Hollow Stem Auger	DATE:	07/29/2024

DEPTH TO - WATER> INITIAL: NE AFTER 24 HOURS: NM CAVING> NE

Depth (feet)	Elevation (ft-MSL)	Description	Graphic	Groundwater	Sample Type	N-Value
0	995	ASPHALT: 3 inches				
		FILL: Orange gray and brown micaceous silty coarse to fine SAND with rock fragments				
		RESIDUUM: Red orange and tan very micaceous silty coarse to fine SAND				
5	990	White tan and orange very micaceous silty coarse to fine SAND				
		Boring Terminated at 5 ft.				
10	985					
15	980					
20	975					
25	970					
30	965					
35	960					

Graphic Depiction

● BLOW COUNT
▲ NATURAL MOISTURE
PLASTIC LIMIT |———| LIQUID LIMIT
10 20 30 40 60 100

This information pertains only to this boring and should not be interpreted as being indicative of the site.

Boring elevations were estimated from the Existing Conditions Plan.
Infiltration testing locations were each augered to 5 feet.

NOVA

TEST BORING RECORD
I-3

PROJECT: Dunwoody Maintenance Facility PROJECT NO.: 10103-2024073
CLIENT: City of Dunwoody
PROJECT LOCATION: Dunwoody, DeKalb County, Georgia
LOCATION: 33.934977° N, -84.300335° W ELEVATION: 985 feet-MSL
DRILLER: Piedmont Environmental Drilling LOGGED BY: AMH
DRILLING METHOD: Hollow Stem Auger DATE: 07/29/2024
DEPTH TO - WATER> INITIAL: NE AFTER 24 HOURS: NM CAVING> NE

Depth (feet)	Elevation (ft-MSL)	Description	Graphic	Groundwater	Sample Type	N-Value
0	985	ASPHALT: 3 inches				
		FILL: Orange gray and brown micaceous coarse to fine sandy SILT with rock fragments				
		RESIDUUM: Orange and tan very micaceous silty coarse to fine SAND				
5	980	Boring Terminated at 5 ft.				
10	975					
15	970					
20	965					
25	960					
30	955					
35	950					

Graphic Depiction
- ● BLOW COUNT
- ▲ NATURAL MOISTURE
- PLASTIC LIMIT |——| LIQUID LIMIT

10 20 30 40 60 100

This information pertains only to this boring and should not be interpreted as being indicative of the site.

Boring elevations were estimated from the Existing Conditions Plan.
Infiltration testing locations were each augered to 5 feet.

NOVA

TEST BORING RECORD
I-4

PROJECT:	Dunwoody Maintenance Facility
PROJECT NO.:	10103-2024073
CLIENT:	City of Dunwoody
PROJECT LOCATION:	Dunwoody, DeKalb County, Georgia
LOCATION:	33.935074° N, -84.300178° W
ELEVATION:	985 feet-MSL
DRILLER:	Piedmont Environmental Drilling
LOGGED BY:	AMH
DRILLING METHOD:	Hollow Stem Auger
DATE:	07/29/2024
DEPTH TO - WATER> INITIAL:	NE
AFTER 24 HOURS:	NM
CAVING>	NE

Depth (feet)	Elevation (ft-MSL)	Description	Graphic	Groundwater	Sample Type	N-Value
0	985	ASPHALT: 3.5 inches				
		FILL: Gray red and brown micaceous silty coarse to fine SAND with rock fragments				
		RESIDUUM: Brown orange and tan very micaceous silty coarse to fine SAND				
5	980	Boring Terminated at 5 ft.				
10	975					
15	970					
20	965					
25	960					
30	955					
35	950					

Graphic Depiction

● BLOW COUNT
▲ NATURAL MOISTURE
PLASTIC LIMIT |———| LIQUID LIMIT
10 20 30 40 60 100

This information pertains only to this boring and should not be interpreted as being indicative of the site.

Boring elevations were estimated from the Existing Conditions Plan.
Infiltration testing locations were each augered to 5 feet.

NOVA

TEST BORING RECORD
I-5

PROJECT: Dunwoody Maintenance Facility PROJECT NO.: 10103-2024073
CLIENT: City of Dunwoody
PROJECT LOCATION: Dunwoody, DeKalb County, Georgia
LOCATION: 33.935176° N, -84.300025° W ELEVATION: 987 feet-MSL
DRILLER: Piedmont Environmental Drilling LOGGED BY: AMH
DRILLING METHOD: Hollow Stem Auger DATE: 07/29/2024
DEPTH TO - WATER> INITIAL: NE AFTER 24 HOURS: NM CAVING> NE

Depth (feet)	Elevation (ft-MSL)	Description	Graphic	Groundwater	Sample Type	N-Value
0		ASPHALT: 4 inches				
		CONCRETE: 6.5 inches				
	985	FILL: Gray red and orange micaceous coarse to fine sandy SILT with rock fragments				
5		RESIDUUM: Brown orange and tan very micaceous silty coarse to fine SAND with quartz fragments				
		Boring Terminated at 5 ft.				

Depth scale: 0, 5, 10, 15, 20, 25, 30, 35. Elevation scale: 985, 980, 975, 970, 965, 960, 955.

Graphic Depiction

● BLOW COUNT
▲ NATURAL MOISTURE
PLASTIC LIMIT ├──┤ LIQUID LIMIT
10 20 30 40 60 100

This information pertains only to this boring and should not be interpreted as being indicative of the site.

Boring elevations were estimated from the Existing Conditions Plan.
Infiltration testing locations were each augered to 5 feet.

APPENDIX C
Qualifications of Recommendations

QUALIFICATIONS OF RECOMMENDATIONS

The findings, conclusions and recommendations presented in this report represent our professional opinions concerning subsurface conditions at the site. The opinions presented are relative to the dates of our on-site services and should not be relied on to represent conditions at later dates or at locations not explored. The opinions included herein are based on information provided to us, the data obtained at specific locations during the exploration and our past experience. If additional information becomes available that might impact our geotechnical opinions, it will be necessary for NOVA to review the information, reassess the potential concerns, and re-evaluate our conclusions and recommendations.

Regardless of the thoroughness of a geotechnical exploration, there is the possibility that conditions between borings will differ from those encountered at specific boring locations, that conditions are not as anticipated by the designers and/or the contractors, or that either natural events or the construction process have altered the subsurface conditions. These variations are an inherent risk associated with subsurface conditions in this region and the approximate methods used to obtain the data. These variations may not be apparent until construction.

The professional opinions presented in this geotechnical report are not final. Field observations and foundation installation monitoring by the geotechnical engineer, as well as soil density testing and other quality assurance functions associated with site earthwork and foundation construction, are an extension of this report. Therefore, NOVA should be retained by the owner to observe all earthwork and foundation construction to document that the conditions anticipated in this exploration actually exist, and to finalize or amend our conclusions and recommendations. NOVA is not responsible or liable for the conclusions and recommendations presented in this report if NOVA does not perform these observation and testing services.

This report is intended for the sole use of CLIENT only. The scope of services performed during this exploration was developed for purposes specifically intended by CLIENT and may not satisfy other users' requirements. Use of this report or the findings, conclusions or recommendations by others will be at the sole risk of the user. NOVA is not responsible or liable for the interpretation by others of the data in this report, nor their conclusions, recommendations or opinions.

Our professional services have been performed, our findings obtained, our conclusions derived and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices in the State of Georgia. This warranty is in lieu of all other statements or warranties, either expressed or implied.

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 not 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 not be adequate to develop geotechnical design recommendations for the project.

Do not 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 not rely on an executive summary. Do not 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 <u>not</u> 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 <u>Not</u> 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 <u>not</u> 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 <u>not</u> building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

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