ITB 22-03 Spalding Drive Storm Drain Crossing

Addendum #1 **Posted, February 24, 2022**

Responses to Q&A received:

1. **DO YOU HAVE BORING LOGS FOR THE SITE?** ***Response: Yes, see attached report prepared by Nova dated, 10.07.2021.***

2. **IF THE CITY CANNOT SHUT THE WATER OFF TO EXISTING MAINS, WILL CONTRACTOR BE HELD LIABLE?** ***Response: The General Contractor is responsible for control of the jobsite during all phases of construction from mobilization to final acceptance. So generally speaking – YES. Ultimately, this would depend on the circumstances that necessitated isolation of the water mains. The contractor and any subs working on the project are responsible to take necessary precautions to prevent disruption of the mains. Valve crews from City of Atlanta Water will be standing by on site at all times when work is underway in the event we were to experience a breach of any of the mains.***

   The contractor will be responsible to coordinate with Atlanta Water to locate and exercise valves on all four mains prior to onset of construction. Representatives from Atlanta Water will be at the Pre-Construction Conference to plan and coordinate this effort.

   Finally, in the event we were to experience a water main break, the contractor would need to work with City of Atlanta crews to minimize and mitigate collateral damage within the construction zone and downstream from the break.

3. **WHEN IS THE ANTICIPATED START DATE FOR THE PROJECT?** ***Response: Subject to City of Dunwoody review and approval by Council, we anticipate an early to mid-April 2022 start.***

Other Bid Document Instructions:

Use the attached revised Bid Form dated 2.24.22

SPALDING DRIVE STORM CROSSING BID FORM

Revised 02.24.2022 (Addendum #1)

ITEM NO.	DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	MEAN EXTENSION	NOTES / COMMENTS
ROADWAY						
150-1000	TRAFFIC CONTROL	LS	LUMP			
210-0100	GRADING COMPLETE -	LS	LUMP			Ref. GDOT Standard 1401 / See notes below
522-1000	STRUCTURAL SHORING AND SUPPORT	LS	LUMP			Per Wade Trim structural design, 01.24.22
550-1180	STORM DRAIN PIPE, 18 IN, H 1-10	LF	8			MATERIAL IS ALREADY ON SITE - INSTALL ONLY
550-1241	STORM DRAIN PIPE, 24 IN, H 10-15	LF	70			Road crossing from A-1 to A-2, double row MATERIAL IS ALREADY ON SITE - INSTALL ONLY
550-1360	STORM DRAIN PIPE, 36 IN, H 1-10	LF	31			A-2 to A-3; MATERIAL IS ALREADY ON SITE - INSTALL ONLY
550-1481	STORM DRAIN PIPE, 48 IN, H 10-15 (INSTALLATION ONLY)	LF	8			MATERIAL IS ALREADY ON SITE - INSTALL ONLY
550-4218	FES, 18 IN	1	EA			MATERIAL IS ALREADY ON SITE - INSTALL ONLY
668-1205	CATCH BASIN, GP 2, SPCL DES	EA	2			Field construct Structures A-1 and A-2
668-1215	CATCH BASIN, GP 2, ADDL DEPTH, SPCL DES	LF	13			
668-5020	STORM SEWER BLIND JUNCTION BOX	EA	1			MODIFY EXISTING IN-PLACE STRUCTURE
668-4300	STORM SEWER MANHOLE, TP 1	EA	1			TIE TO EXISTING AT STR A-3
Submitted by / Contractor Name	______________________________			**MEAN TOTAL:**		

BID NOTES

1. Traffic Control: Provide all barricades, detour signage and personnel as necessary during all stages of the Work. (Per MUTCD)
2. Grading Complete includes, but is not limited to, the following items:
   ** Selective demolition, haul-off and disposal of all debris generated by the work (including but not limited to curbs, drainage structures and pavements)
   ** Sawcutting, coring, layout
   ** De-watering as necessary during all phases
   ** Remove, haul and dispose of all soils generated by excavation within the roadway
   ** Hydro-vac as necessary to safely expose water mains and utilities in the work zone
   ** Fine grading, temp grassing and erosion control
   ** constructing shoulders and subgrade
   ** Ensure that the completed grading work conforms to the horizontal and vertical alignment to match existing or as directed by the Engineer.
3. The double row of 24-inch RCP price shall include select backfill (#57 stone bedding, select fill/compaction, minimum 18 inches of GAB and 8-inch high-early strenth concrete cap) in the LF price

GEOTECHNICAL ENGINEERING REPORT

Spalding Drive Catch Basin Subgrade Exploration

Dunwoody, DeKalb County, Georgia

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

NOVA Project Number: 10103-1020086

October 7, 2021

PROFESSIONAL | PRACTICAL | PROVEN

October 7, 2021

CITY OF DUNWOODY
4800 Ashford Dunwoody Road
Dunwoody, Georgia

Attention: Mr. David Ayers
Construction Management

Subject: SUBGRADE EXPLORATION AND ENGINEERING EVALUATION
SPALDING DRIVE CATCH BASIN SUBGRADE EXPLORATION
Dunwoody, DeKalb County, Georgia
NOVA Project Number 10103-1020086

Dear Mr. Ayers:

NOVA Engineering and Environmental, LLC (NOVA) has completed the authorized Geotechnical Engineering Report for the Spalding Drive Catch Basin Subgrade Evaluation located along Spalding Drive in Dunwoody, Georgia. The work was performed in general accordance with NOVA Proposal Number 003-20216053, dated September 19, 2021. 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 REGISTERED No. 027809 PROFESSIONAL ENGINEER MARCUS D. JOHNSTON

Michael W. Mckenzie, P.E.
Project Engineer

Marc D. Johnston, P.E.
Regional Manager / Principal
GA P.E. License 027809

Copies Submitted: Addressee (electronic)

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1.0 INTRODUCTION

1.1 PROJECT INFORMATION

Our understanding of this project is based on discussions with Mr. David Ayers and Mr. Todd Meadows of City of Dunwoody, and our experience with similar projects.

The project site is within the area of the existing stormwater inlets along Spalding Drive in Dunwoody, Georgia, approximately 650 feet southwest of the intersection with Chamblee-Dunwoody Road. The City of Dunwoody is planning a catch basin installation project related to a stormwater pipe expansion. The proposed catch basin will cross Spalding Drive between the two existing stormwater inlets.

1.2 SCOPE OF WORK

City of Dunwoody engaged NOVA to provide geotechnical engineering consulting services for the proposed catch basin installation project. This report briefly discusses our understanding of the project, describes our exploratory procedures, and presents our findings, conclusions, and recommendations.

The primary objective of this study was to perform a preliminary geotechnical exploration within the area of the proposed construction and to assess these findings as they relate to geotechnical aspects of the proposed catch basin installation project. The authorized geotechnical engineering services included a site reconnaissance, a soil test boring and sampling program, engineering evaluation of the field and laboratory data, and the preparation of this report.

The services were performed as outlined in our proposal number 003-20216053, dated September 19, 2021, and in general accordance with industry standards.

As authorized per the above referenced proposal, the completed geotechnical report includes:

- A description of the site, fieldwork, laboratory testing and general soil conditions encountered, as well as a Boring Location Plan, and individual Boring Records.
- Assessment of existing subgrade conditions within the proposed catch basin excavation.

The assessment of the presence of wetlands, floodplains, or water classified as State Waters of Georgia is beyond the scope of this study. Additionally, the assessment of site environmental conditions, including the detection of pollutants in the soil, rock, or groundwater, at the site is also beyond the scope of this geotechnical study. If desired by the client, NOVA can provide these services.

2.0 SITE DESCRIPTION

2.1 LOCATION AND LEGAL DESCRIPTION

The project site is located in the area of the existing stormwater inlets along Spalding Drive in Dunwoody, Georgia, approximately 650 feet southwest of the intersection with Chamblee-Dunwoody Road. The project site lies within the right-of-way of Spalding Drive.

2.2 CURRENT USE OF THE PROPERTY

The project site is the existing Spalding Drive roadway along with its right-of-way, which includes sidewalks and adjacent landscaped areas.

3.0 FIELD AND LABORATORY PROCEDURES

3.1 FIELD EXPLORATION

Boring locations were established in the field by NOVA personnel during a site meeting with Mr. Todd Meadows of the City of Dunwoody. Boring elevations were estimated from available online topographic data. The approximate boring locations are shown on Figure 3 in Appendix A. If increased accuracy is desired by the client, NOVA recommends that the boring locations and elevations be surveyed.

Our field exploration was conducted on September 29, 2021 and included:

- Three (3) soil test borings (B-1 through B-3) drilled through the pavement to depths of up to 25 feet below the existing pavement surface.

Soil Test Borings: The soil test borings 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. These records represent our interpretation of the subsurface conditions based on the field exploration data, visual examination of the split-barrel samples, laboratory test data, and generally accepted geotechnical engineering practices. The stratification lines and depth designations represent approximate boundaries between various subsurface strata. Actual transitions between materials may be gradual.

Groundwater: The groundwater levels reported on the Test Boring Records represent measurements made at the completion of the soil test borings and 24 hours after, where feasible. The soil test borings were subsequently backfilled with the soil cuttings and the auger drill holes in the pavement patched with quick-setting concrete.

3.2 LABORATORY TESTING

Split-barrel samples were returned to our testing laboratory, where they were classified using visual/manual methods in accordance with the Unified Soil Classification System (USCS) and ASTM designations. The descriptions presented in the boring logs should be considered approximate. It should be noted that all soil samples would be properly disposed of 30 days following the submittal of this NOVA subsurface exploration report unless you request otherwise.

3.2.1 Soil Classification

Soil classification provides a general guide to the engineering properties of various soil types and enable the engineer to apply past experience to current problems. In our explorations, samples obtained during drilling operations are observed in our laboratory and visually classified by an engineer. The soils are classified according to consistency (based on number of blows from standard penetration tests), color and texture. These classification descriptions are included on our "Test Boring Logs". The classification system discussed above is primarily qualitative; laboratory testing is generally performed for detailed soil classification. Using the test results, the soils were classified using the Unified Soil Classification Systems nomenclature. This classification system and the in-place physical soil properties provide an index for estimating the soil's behavior and engineering characteristics. The soil classification and physical properties obtained are presented in this report.

3.2.2 Moisture Content

The moisture content is the ratio expressed as a percentage of the weight of water in a given mass of soil to the weight of the solid particles. This test was conducted in general accordance with ASTM D 2216. A total of five (5) moisture content tests were performed in this study on split-spoon samples obtained from Borings B-1, B-2, and B-3. Results are shown on the Boring Logs in Appendix B and in the table below.

Boring	Sample Number	Depth (ft)	Moisture Content (%)
B-1	2	3.5-5	18.3
B-2	2	3.5-5	5.4
B-2	3	6-7.5	43.6
B-3	1	1-2.5	25.8
B-3	5	13.5-15	60.6

3.2.3 Proctor Test

Two (2) Standard Proctor compaction tests were performed on two (2) bulk samples in accordance with ASTM D 698 – Standard Test Methods for Laboratory Compaction of Soil Using Standard Effort to determine the relationship between the soils' maximum dry unit weight and various moisture contents for use in controlling fill placement. The results are provided in Appendix C of this report, and in the table below.

3.2.4 Sieve Analysis

Two (2) sieve analysis tests were performed in this study on two (2) bulk samples. Sieve analysis consists of passing a soil sample through a series of standard sieve openings. The percentage of soil, by weight, passing the individual sieves is then recorded and generally presented in a graphical format. The percentage of fines passing through the No. 200 sieve is generally considered to represent the amount of silt and clay of the tested soil sample. The sieve analysis test was conducted in general accordance with ASTM Designation D 1140. Results are included on the Standard Proctor compaction test reports in Appendix C of this report, and in the table below.

3.2.5 Atterberg Limits

Two (2) Atterberg Limits tests were performed in this study. The Atterberg Limits are different descriptions of the moisture content of fine-grained soils as it transitions between a solid to a liquid-state. For classification purposes the two primary Atterberg Limits used are the plastic limit (PL) and the liquid limit (LL). The plastic index (PI) is also calculated for soil classification.

The plastic limit (PL) is the moisture content at which a soil transitions from being in a semisolid state to a plastic state. The liquid limit (LL) is defined as the moisture content at which a soil transitions from a plastic state to a liquid state. The tests were performed in this study in accordance with ASTM D4318. Results are included on the Liquid and Plastic Limits test reports in Appendix C of this report, and in the table below.

Bulk Sample	Natural Moisture Content (%)	Percent Passing Size 200 Sieve (%)	Optimum Moisture Content (%)	Maximum Dry Density (pcf)	Liquid Limit (LL)	Plastic Limit (PL)	USCS
B-1	32.4	42.5	14.5	110.6	34	29	SM
B-3	44.1	63.4	22.4	95.6	53	35	MH

4.0 SUBSURFACE CONDITIONS

4.1 GEOLOGY

The site is located in the Piedmont Geologic Region, a broad northeasterly trending 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 crystal 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 the "Geology of the Greater Atlanta Region" by McConnell and Abrams, (1984), the site is generally underlain by the Northern Piedmont Province and Brevard Fault Zone, Sandy Springs Group, Eastern Belt, Powers Ferry Formation (Higgins and McConnell 1978a), consisting of undifferentiated biotite-quart-plagioclase gneiss (metagraywacke), mica schist, and amphibolite (pfu).

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 clays and clayey 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 depths to materials requiring difficult excavation techniques may occur over short horizontal distances.

4.2 SOIL AND ROCK CONDITIONS

The following paragraphs provide generalized descriptions of the subsurface profiles and soil conditions encountered by the borings conducted during this study.

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 Logs represent the approximate interface locations and elevation. The actual transition between strata may be gradual. Groundwater levels shown on the Boring Logs represent the conditions at the time of drilling. It should be understood that soil conditions may vary between boring locations.

4.2.1 Surface Materials

Asphalt Pavement: Boring B-2 was performed within the existing pavement and encountered approximately 7 inches of asphalt. The pavement was underlain by graded aggregate base (GAB) with a thickness of approximately 4 inches.

Topsoil: Topsoil was encountered at boring locations B-1 and B-3 within landscape areas with a thickness of approximately 2 inches. Please note that topsoil thicknesses are frequently erratic and thicker zones of topsoil should be anticipated.

4.2.2 Fill

Existing fill was encountered in all the borings extending to depths of 3 to 8 feet below the existing ground surface. The fill was variable in composition and consistency but generally consisted of very loose to medium dense micaceous silty medium to fine SAND and very soft to very stiff medium to fine micaceous sandy SILT. Standard penetration resistances in the fill ranged from weight-of-hammer (WOH) to 16 blow per foot (bpf).

BORING	DEPTH (feet)	APPROXIMATE ELEVATION TO FILL BOTTOM (feet-MSL)
B-1	3.0	1037.0
B-2	8.0	1031.0
B-3	8.0	1031.0

4.2.3 Alluvial Soils

Alluvial (water deposited) soils were encountered beneath the fill materials in borings B-1 and B-2. The sampled alluvium generally consisted of loose to medium dense micaceous silty medium to fine SAND. Standard penetration resistance values ranged from 7 to 22 bpf.

BORING	DEPTH (feet)	APPROXIMATE ELEVATION TO ALLUVIUM BOTTOM (feet-MSL)
B-1	13.0	1027.0
B-2	13.0	1026.0

4.2.4 Residual Soils

Residual soils were encountered beneath the alluvial fill materials in each of the borings. The sampled residuum generally consisted of very loose to medium dense micaceous silty medium to fine SAND and very soft to firm micaceous medium to fine sandy SILT. Standard penetration resistance values ranged from weight-of-hammer (WOH) to 18 bpf.

4.2.5 Partially Weathered Rock

Partially weathered rock (PWR) is a transitional material between soil and the underlying parent rock that is defined locally as materials that exhibit a standard penetration resistance exceeding 100 bpf.

PWR was not encountered in any borings to the depths explored below the ground surface. PWR is typically observed immediately above auger refusal levels.

4.2.6 Auger Refusal

Auger refusal materials are any very hard or very dense material, frequently boulders or the upper surface of bedrock, which cannot be penetrated by a power auger. Auger refusal was not encountered in any borings to the depths explored below the existing ground surface.

4.3 GROUNDWATER CONDITIONS

4.3.1 General

Groundwater in the Piedmont 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. The groundwater table in the Piedmont is expected to be a subdued replica of the original surface topography.

Groundwater levels vary with changes in season and rainfall, development, construction activity, surface water runoff, and other site-specific factors. Groundwater levels in the Dunwoody area are typically 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 vary at times.

4.3.2 Soil Test Boring Groundwater Conditions

Groundwater was encountered in all of the borings performed during this study at depths ranging from 9.5 to 11.5 feet below the existing surface. Groundwater readings were made immediately upon completion of borings, and 24-hour readings in borings B-1 and B-3 (B-2 was backfilled on the day of drilling due to its location in an active roadway).

BORING	DEPTH (feet)	APPROXIMATE ELEVATION (feet-MSL)
B-1	9.5	1030.5
B-2	11.5*	1027.5
B-3	10.5	1028.5

* This reading was taken on the day of drilling due to its location in an active roadway.

5.0 CONCLUSIONS AND RECOMMENDATIONS

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

Subsurface conditions in unexplored locations or at other times may vary from those encountered at specific boring locations. 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.

As previously noted, boring locations were established by estimating distances and angles from site landmarks. If increased accuracy is desired by the client, we recommend that the boring locations and elevations be surveyed.

5.1 SITE PREPARATION

5.1.1 General

General: Prior to proceeding with construction, all structures, vegetation, root systems, topsoil, and other deleterious non-soil materials should be stripped from proposed construction areas. Clean topsoil may be stockpiled and subsequently re-used in landscaped areas. Debris-laden materials 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, which are at grade or will receive fill should be carefully evaluated by a NOVA geotechnical engineer. The engineer will require proofrolling of the subgrade with multiple passes of a 20 to 30 ton loaded truck, or other vehicle of similar size and weight.

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 proof-rolling operations should be undercut and replaced with structural fill or stabilized in-place by scarifying and re-densifying.

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.

The site should be graded during construction such that positive drainage is maintained 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 also be sealed and crowned with a smooth roller to minimize ponding water from storm events at the end of each day of work. The fine-grained soils encountered during our exploration are weather sensitive and will be susceptible to loss of strength and density if exposed to freeze/thaw and or wetting/drying cycles. Care must be exercised by the grading contractor to protect subgrades and new fills from inclement weather.

5.1.2 Trench Excavation

Weak and/or low-density soils were encountered in our borings at the project site. These weak soils are present at or near the existing invert elevation and extending several feet below in some borings. Stabilization of soft soils beneath the new catch basin may be required. The contractor should be prepared to undercut and place a minimum of 12” of bedding stone beneath the catch basin. Some areas requiring additional over excavation and stone replacement and/or the use of a stabilization fabric and surge stone should be anticipated. The actual extent and nature of the required remedial measures can best be determined in the field by the Geotechnical Engineer at the time of construction.

Groundwater was measured in all the borings in this study immediately upon completion of drilling; 24-hour groundwater readings were not performed in B-2 due to the location of the borings within an active roadway. Typically, we recommend that construction be scheduled for the drier late summer, early fall seasons when groundwater levels tend to be at their lowest. However, if the project schedule does not allow for this, the contractor should anticipate that the construction site will require dewatering.

Excavations below groundwater table will require the installation of a temporary dewatering system to facilitate excavation. During excavations, weak, wet alluvial soils will be unstable and highly susceptible to caving in. The contractor should be prepared to stabilize trench excavations during culvert construction in accordance with applicable OSHA regulations. Typically, excavated fill soils are re-used as backfill. ***However, it should be noted that the existing fill soils beneath the roadway have moisture contents well above their optimum moisture content and will likely require extensive aeration and drying before re-use as structural***

fill. Use of imported material for backfill of the new utilities will likely be a more efficient means of construction.

5.1.3 Existing / Old Fill

Previously placed fill materials were encountered during this exploration. 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.

5.1.4 Difficult Excavation

Very dense soils, PWR, and/or shallow auger refusal materials were not encountered above anticipated finished or utility grades for the project. However, as discussed in the geology section of this report, the weathering process is erratic and variations in the PWR or rock profile can occur in small lateral distances. Therefore, it is likely that very dense soils, PWR, and/or rock pinnacles or ledges requiring difficult excavation techniques may be encountered in site areas intermediate of our boring locations.

5.2 FILL PLACEMENT

5.2.1 Fill Suitability

Fill materials should be low plasticity soil (Plasticity Index less than 30), free of non-soil materials and rock fragments larger than 3 inches in any one dimension. Based on visual examination, the existing residual soils and much of the existing fill, which does not contain appreciable amounts of debris, rock organics or other deleterious materials, encountered during this exploration generally appear suitable for re-use as structural fill.

Natural moisture content testing indicates that most of the existing fill, alluvial, and residual soils are well above optimum water content. These soils would need extensive aeration and drying to bring them to within 3 percent of optimum moisture in order to be used as structural fill. Consequently, the use of imported material for backfill of the new utilities will likely be a more efficient means of construction.

Prior to construction, bulk samples of the proposed fill materials should be laboratory-tested to confirm their suitability.

Organic and/or debris-laden material is not suitable for re-use as 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.

5.2.2 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). 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. If soils are excavated near or below the groundwater table, they will require significant efforts to achieve acceptable moisture contents prior to re-use as fill.

Filling operations should be observed by a NOVA soils technician, who can confirm suitability of material used and uniformity and appropriateness of compaction efforts. He/she can also document compliance with the specifications by performing field density tests using nuclear or sand cone testing methods (ASTM D 6938, or D 1556, 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.

5.3 GROUNDWATER CONTROL

5.3.1 General

During the current study, groundwater was encountered in all of the soil test borings at depths ranging from 9.5 to 11.5 feet below existing grades (approximate elevations of 1027.5 to 1030.5 ft-MSL).

Based on the proposed depths of the catch basin excavation, we anticipate that temporary dewatering measures will be required during excavation and installation of the catch basin structures.

As previously noted, groundwater levels are subject to seasonal, climatic, and other variations and may be different at other times and locations. The extent and nature of any dewatering required during construction will be dependent on the actual groundwater conditions prevalent at the time of construction and the effectiveness of construction drainage to prevent run-off into open excavations.

5.3.2 Temporary Dewatering

Design of a temporary dewatering system is usually the responsibility of the contractor. However, based on our experience with similar conditions, we believe a conventional construction dewatering system of trenches, sumps, and pumps should be possible to control both groundwater and rainfall runoff.

At the time of construction, groundwater must be lowered and continuously maintained at a minimum depth of 3 feet below the working elevation to permit a stable platform for construction of the proposed box culvert. The dewatering system should be installed and operational prior to excavation beneath the water table. The dewatering system should remain in continuous operation until the culvert bottom and walls are complete and backfilled.

5.4 SLOPES

All new fill placed should be properly benched into the existing slopes and should be compacted in accordance with the requirements outlined in Section 5.2. It is good practice to over-build slopes and then back cut the slopes to the proper grade.

Slope stability analysis using laboratory shear strength data was beyond the scope of this study. However, based on our experience with similar subsurface conditions and construction, permanent slopes no steeper than 2.0(H): 1.0(V) should be stable long term, if limited in height to 25 feet, and are not inundated or subjected to rapid draw-down conditions, or subjected to groundwater seepage.

Adjacent to buildings, a top of slope set-back of 10 feet is recommended. In pavement areas, a minimum top of slope setback of 5 feet is acceptable.

Temporary slopes should be no steeper than OSHA guidelines. During construction, temporary slopes should be regularly inspected for signs of movement or unsafe condition. Soil slopes should be covered for protection from rain, and surface run-off should be diverted away from the slopes. For erosion protection, a protective cover of grass or other vegetation should be established on permanent soil slopes as soon as possible.

5.5 BELOW GRADE 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.

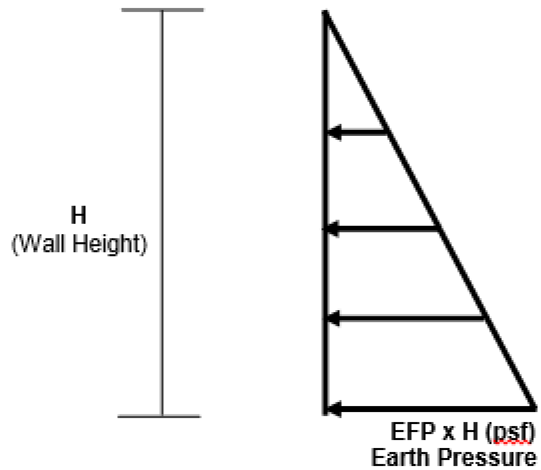
Free-standing retaining walls are usually designed for active earth pressures. Rigid basement walls are typically designed for at-rest earth pressures. If basement walls will be backfilled before they are braced by the floor slabs, they should also be designed to withstand active earth pressures as self-supporting 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, wall must move horizontally to mobilize resistance.

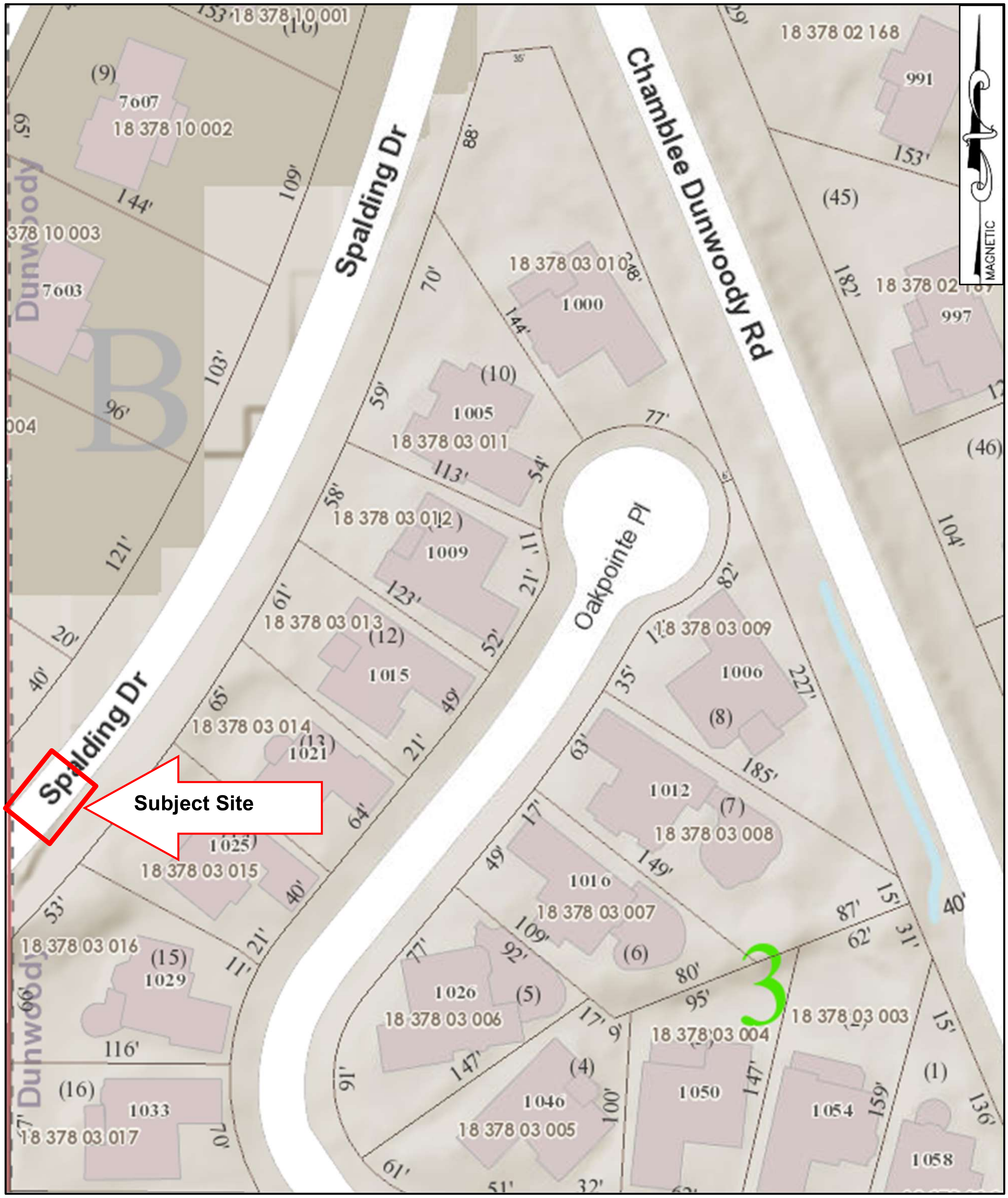
6.0 CONSTRUCTION OBSERVATIONS

6.1 SUBGRADE

Once site grading is completed, the subgrade may be exposed to adverse construction activities and weather conditions. The subgrade should be well-drained to prevent the accumulation of water. If the exposed subgrade becomes saturated or frozen, the NOVA geotechnical engineer should be consulted.

A final subgrade evaluation should be performed by the NOVA geotechnical engineer immediately prior to pavement construction. If practical, proof-rolling may be used to re-densify the surface and to detect any soil, which has become excessively wet or otherwise loosened.

APPENDIX A
Figures and Maps

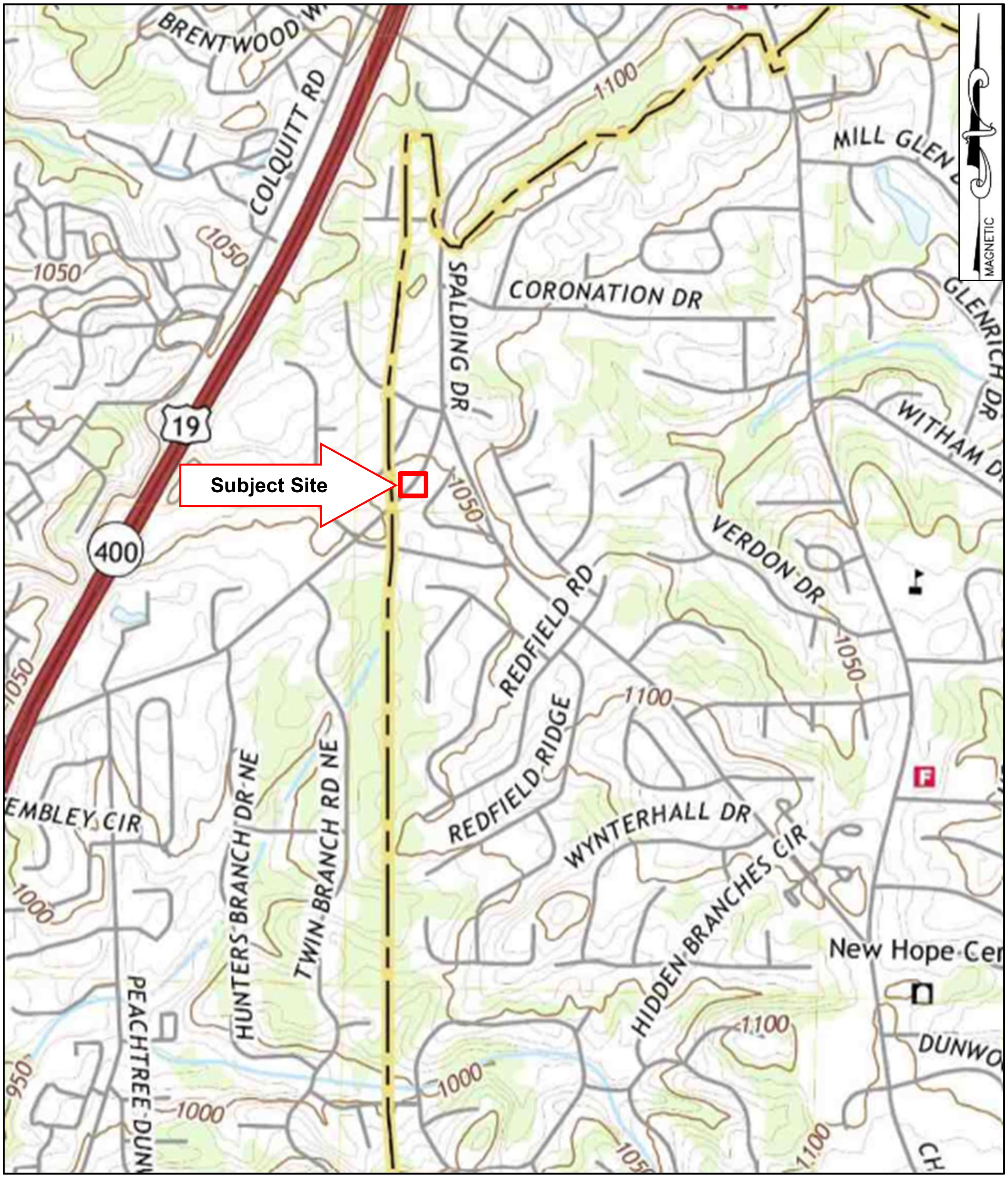


Approximate Subject Site Boundary

FIGURE 1
SITE LOCATION
Source: DeKalb GIS
Date Accessed: 10/6/2021
Scale: Not to Scale

NOVA
PROFESSIONAL | PRACTICAL | PROVEN

Spalding Drive Catch Basin
Subgrade Exploration
Dunwoody, DeKalb County, Georgia
NOVA Project Number 10103-1020086

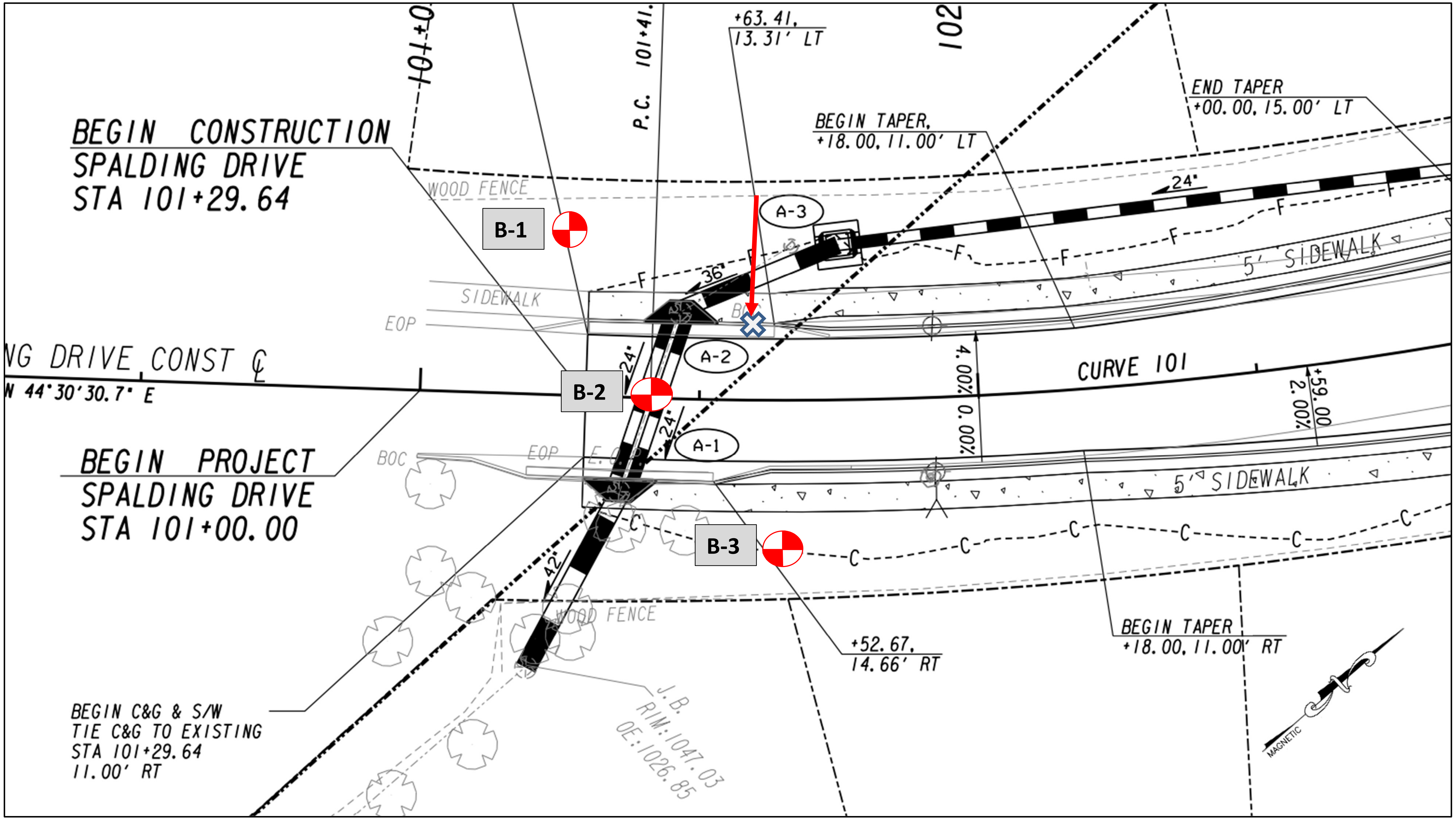


Approximate Subject Site Boundary

FIGURE 2
TOPOGRAPHIC MAP
Source: USGS Topoview
Chamblee, GA Quadrangle 2020
Scale: Not to Scale

NOVA
PROFESSIONAL | PRACTICAL | PROVEN

Spalding Drive Catch Basin
Subgrade Exploration
Dunwoody, DeKalb County, Georgia
NOVA Project Number 10103-1020086

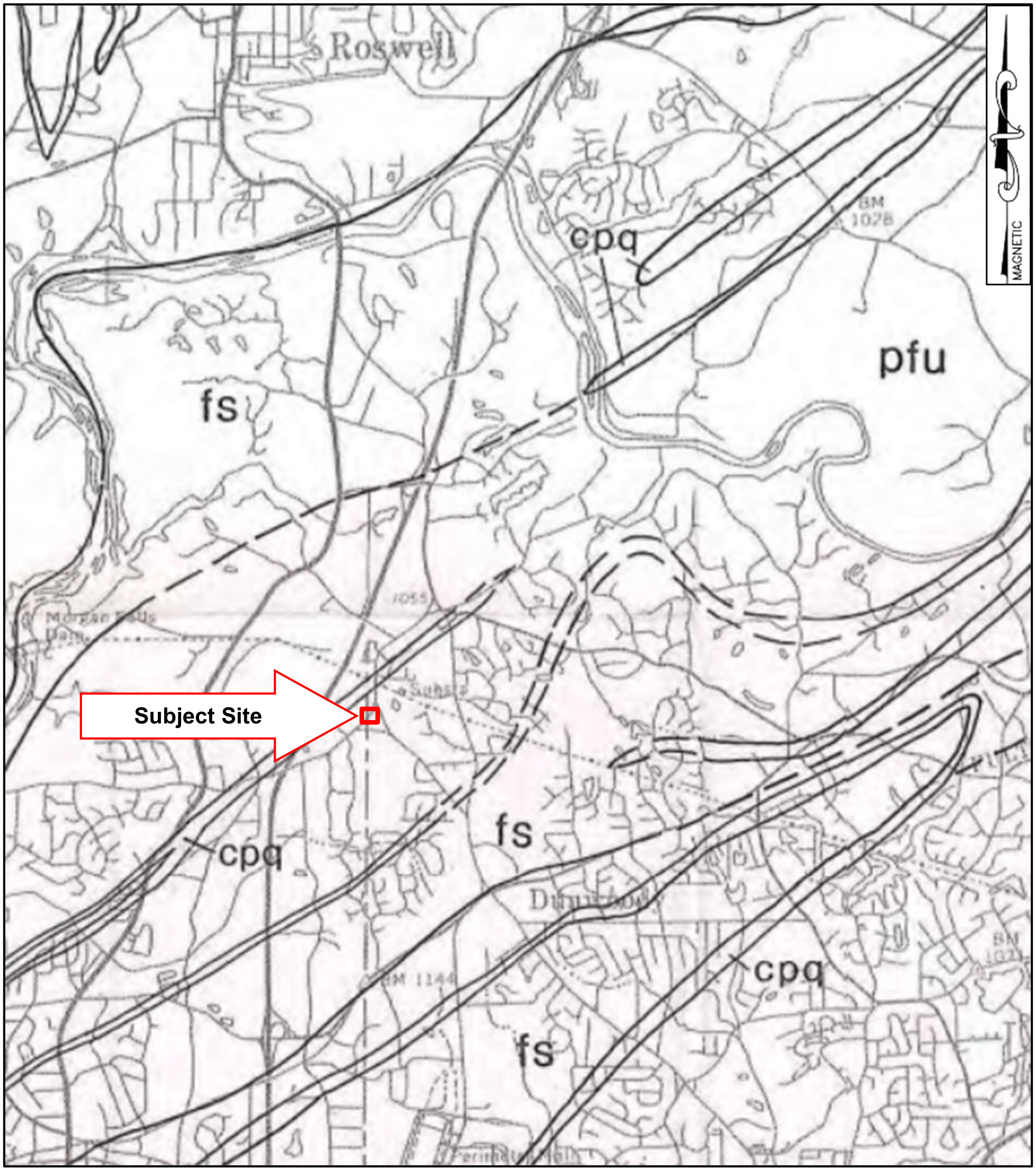


APPROXIMATE LOCATION OF NOVA SOIL TEST BORINGS

FIGURE 3
BORING LOCATION PLAN
Source: Mainline Plan Drawing No. 13-001
Prepared By: Southeastern Engineering, Inc.
Dated: 7-26-2021

NOVA
PROFESSIONAL | PRACTICAL | PROVEN

Spalding Drive Catch Basin
Subgrade Exploration
Dunwoody, DeKalb County, Georgia
NOVA Project Number 10103-1020086



Northern Piedmont Province and Brevard Fault Zone, Sandy Springs Group, Eastern Belt, Powers Ferry Formation (Higgins and McConnell 1978a): undifferentiated biotite-quartz-plagioclase gneiss (metagraywacke), mica schist, and amphibolite (pfu).

Approximate Subject Site Boundary

FIGURE 4
REGIONAL GEOLOGY
SOURCE: Geology of The Greater Atlanta Region, GA DNR/EPD/GGS, McConnell and Abrams, Bulletin 96, 1984
SCALE: Not to Scale

NOVA
PROFESSIONAL | PRACTICAL | PROVEN

Spalding Drive Catch Basin
Subgrade Exploration
Dunwoody, DeKalb County, Georgia
NOVA Project Number 10103-1020086

APPENDIX B
Subsurface Data

KEY TO SYMBOLS AND CLASSIFICATIONS

DRILLING SYMBOLS

◩	Split Spoon Sample
▯	Undisturbed Sample (UD)
●	Standard Penetration Resistance (ASTM D1586-67)
▼	Water Table at least 24 Hours after Drilling
▽	Water Table 1 Hour or less after Drilling
100/2”	Number of Blows (100) 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 Water
MC	Moisture Content Test Performed

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

DRILLING PROCEDURES

Soil sampling and standard penetration testing performed in accordance with ASTM D1586-67. The standard penetration resistance is the number of blows of a 140 pound hammer falling 30 inches to drive a 2-inch O.D., 1⅜-inch I.D. split spoon sampler one foot. Core drilling performed in accordance with ASTM D2113-08. The undisturbed sampling procedure is described by ASTM D1587-08 (2012). Soil and rock samples will be discarded 60 days after the date of the final report unless otherwise directed.

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

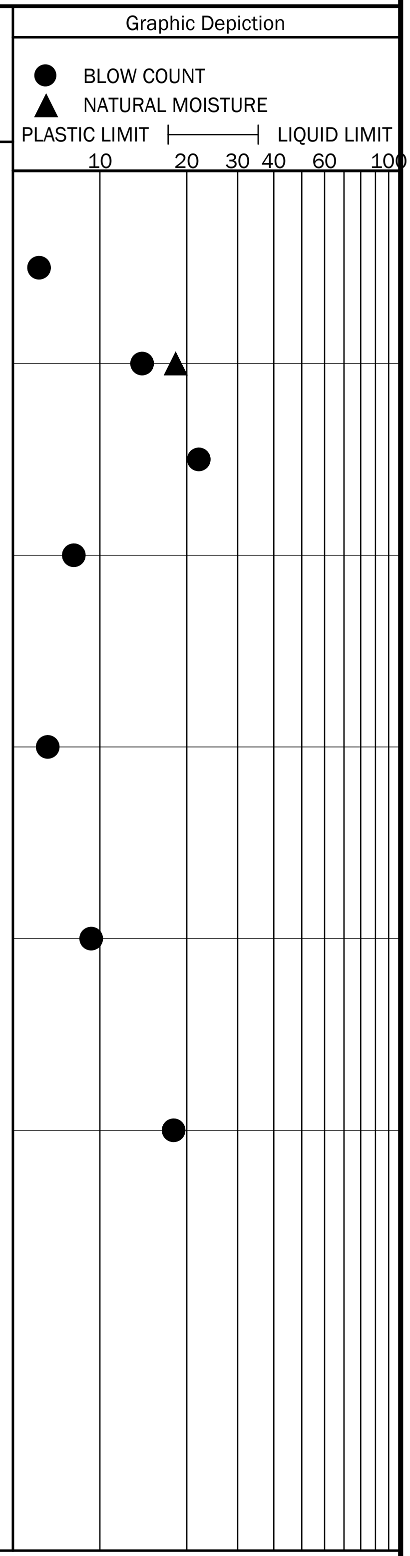
GRAVELS	Coarse	¾ inch to 3 inches
	Fine	No. 4 to ¾ inch
SANDS	Coarse	No. 10 to No. 4
	Medium	No. 40 to No. 10
	Fine	No. 200 to No. 40
SILTS AND CLAYS		Passing No. 200

TEST BORING RECORD
B-1

NOVA

PROJECT: Spalding Drive at Chamblee Dunwoody
PROJECT NO.: 10103-1020086
CLIENT: City of Dunwoody
PROJECT LOCATION: Dunwoody, GA
LOCATION: Proposed Storm Sewer Catch Basin
ELEVATION: 1040 FT-MSL
DRILLER: Betts Environmental
LOGGED BY: CS
DRILLING METHOD: Hollow Stem Auger
DATE: 09/29/2021
DEPTH TO - WATER> INITIAL: 22 AFTER 24 HOURS: 9.5 CAVING>

Depth (feet)	Elevation (ft-MSL)	Description	N-Value
0	1040	TOPSOIL: 2 Inches	
		FILL: Very loose orange-brown, slightly micaceous, silty fine SAND	3
		ALLUVIUM: Medium dense orange-gray, slightly micaceous, silty medium to fine SAND	
5	1035		14
		Medium dense orange-gray silty medium to fine SAND	22
		Loose orange-gray silty medium to fine SAND	
10	1030		7
		RESIDUUM: Very loose brown-orange silty medium to fine SAND, with quartz fragments	
15	1025		4
		Loose orange-brown, micaceous, silty medium to fine SAND	
20	1020		9
		Medium dense orange-brown, micaceous, silty medium to fine SAND	
25	1015	Boring Terminated at 25 ft.	18
30	1010		
35	1005		



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: Spalding Drive at Chamblee Dunwoody	PROJECT NO.: 10103-1020086
CLIENT: City of Dunwoody	
PROJECT LOCATION: Dunwoody, GA	
LOCATION: Proposed Storm Sewer Catch Basin	ELEVATION: 1039 FT-MSL
DRILLER: Betts Environmental	LOGGED BY: CS
DRILLING METHOD: Hollow Stem Auger	DATE: 09/29/2021
DEPTH TO - WATER> INITIAL: 11.5	AFTER 24 HOURS:
CAVING>	

Depth (feet)	Elevation (ft-MSL)	Description	N-Value
0		ASPHALT: 7 Inches	
		Aggregate Base: 4 Inches	
		FILL: Medium dense brown-red, slightly micaceous, silty fine SAND	14
5	1035	Loose orange-gray silty fine SAND	9
		Very stiff brown-gray medium to fine sandy SILT	16
10	1030	ALLUVIUM: Loose green-gray, micaceous, silty fine SAND	9
15	1025	RESIDUUM: Soft orange-brown, slightly micaceous, fine sandy SILT	3
20	1020	Firm orange-brown, micaceous, fine sandy SILT	6
25	1015	Medium dense orange-brown, micaceous, silty medium to fine SAND	5
		Boring Terminated at 25 ft.	
30	1010		
35	1005		

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.

TEST BORING RECORD B-3

PROJECT: Spalding Drive at Chamblee Dunwoody	PROJECT NO.: 10103-1020086
CLIENT: City of Dunwoody	
PROJECT LOCATION: Dunwoody, GA	
LOCATION: Proposed Storm Sewer Catch Basin	ELEVATION: 1039 FT-MSL
DRILLER: Betts Environmental	LOGGED BY: CS
DRILLING METHOD: Hollow Stem Auger	DATE: 09/29/2021
DEPTH TO - WATER> INITIAL: 24.5	AFTER 24 HOURS: 10.5 · CAVING>

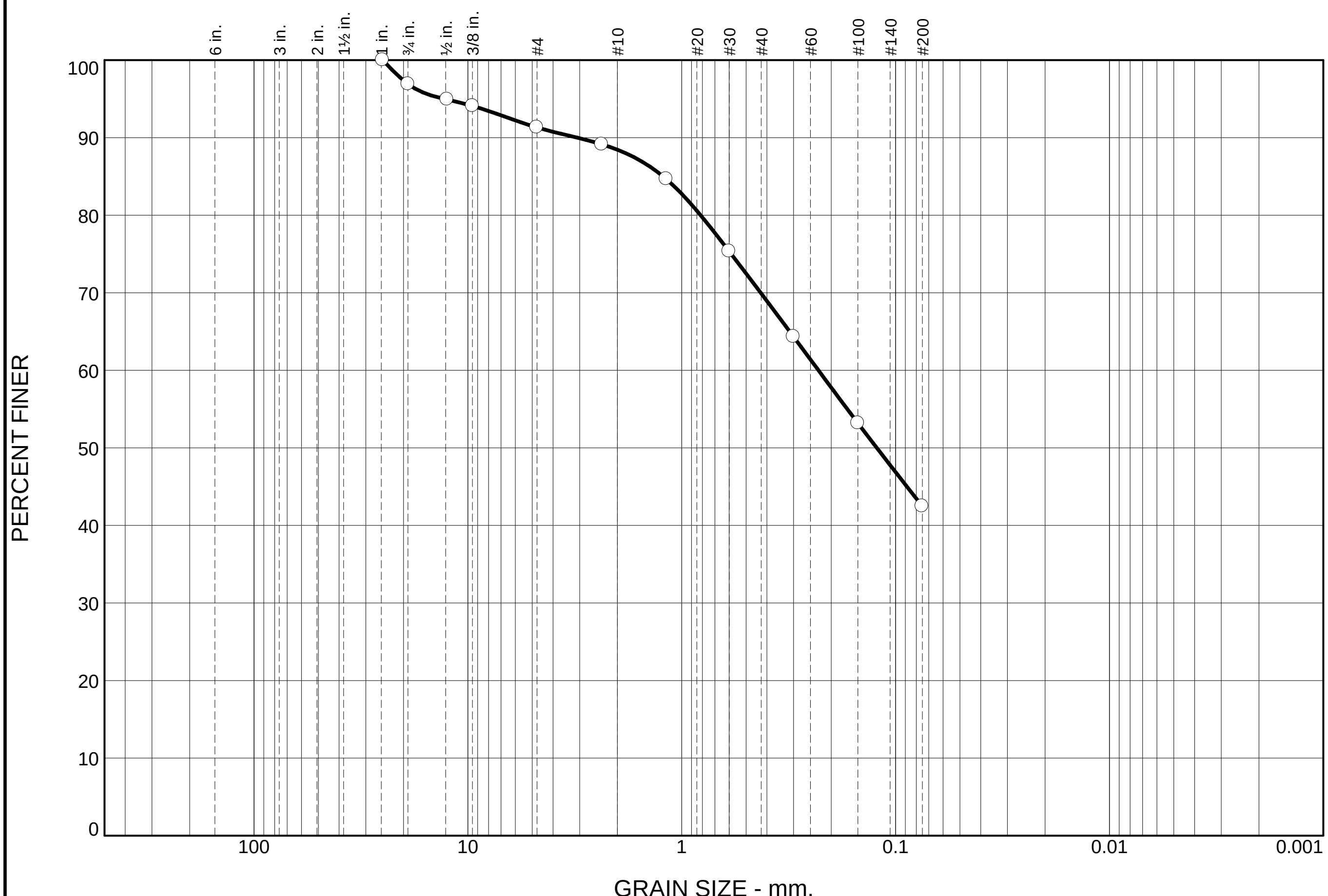
Graphic Depiction: ● BLOW COUNT; ▲ NATURAL MOISTURE; PLASTIC LIMIT |——| LIQUID LIMIT; scale 10, 20, 30, 40, 60, 100

Depth (feet)	Elevation (ft-MSL)	Description	N-Value
0	1039	TOPSOIL: 2 Inches	
0–3		FILL: Soft brown-red medium to fine sandy SILT	4
3–8	1035	Very soft brown-red medium to fine sandy SILT	WOH
8–13	1030	RESIDUUM: Loose gray-orange silty fine SAND	6
13–18	1025	Very soft orange-brown, slightly micaceous, fine sandy SILT	WOH
18–23	1020	Loose brown-orange, slightly micaceous, silty fine SAND	6
23–25	1015	Loose orange-brown, micaceous, silty fine SAND	7
25		Boring Terminated at 25 ft.	

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

APPENDIX C
Laboratory Reports

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	3.1	5.6	2.9	18.5	27.4	42.5	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			1.2214	0.2295	0.1224					

MATERIAL DESCRIPTION	TEST DATE	USCS	NM
○ Orange-gray silty medium to fine SAND		SM	32.4%

Project No. 10103-1020086 **Client:** City of Dunwoody

Project: Spalding Drive Catch Basin Subgrade Exploration

○ **Source of Sample:** B-1 **Depth:** 11-15' **Sample Number:** B-1

Remarks:

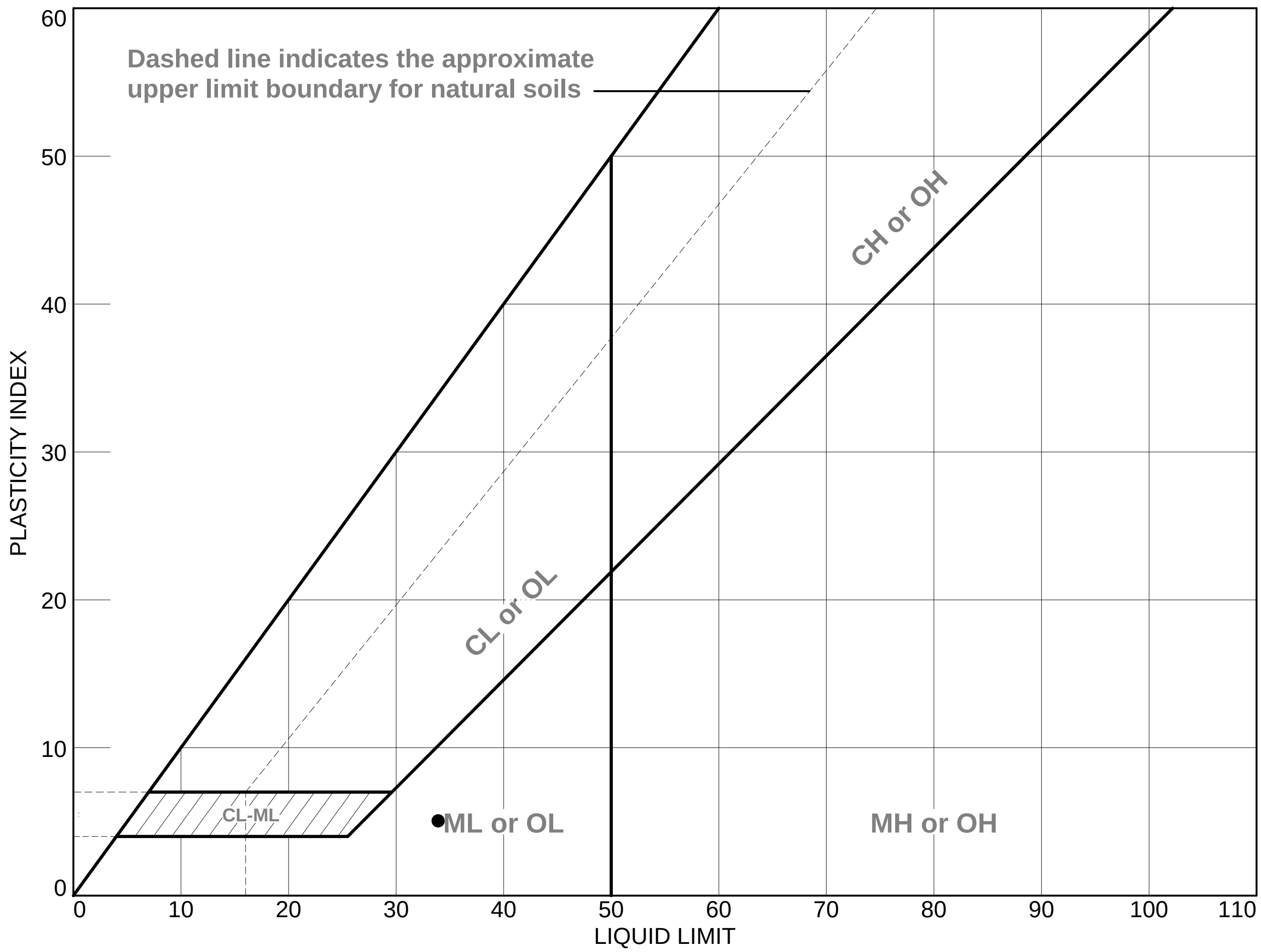
○Alluvium

Nova Engineering & Environmental

Norcross, GA

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



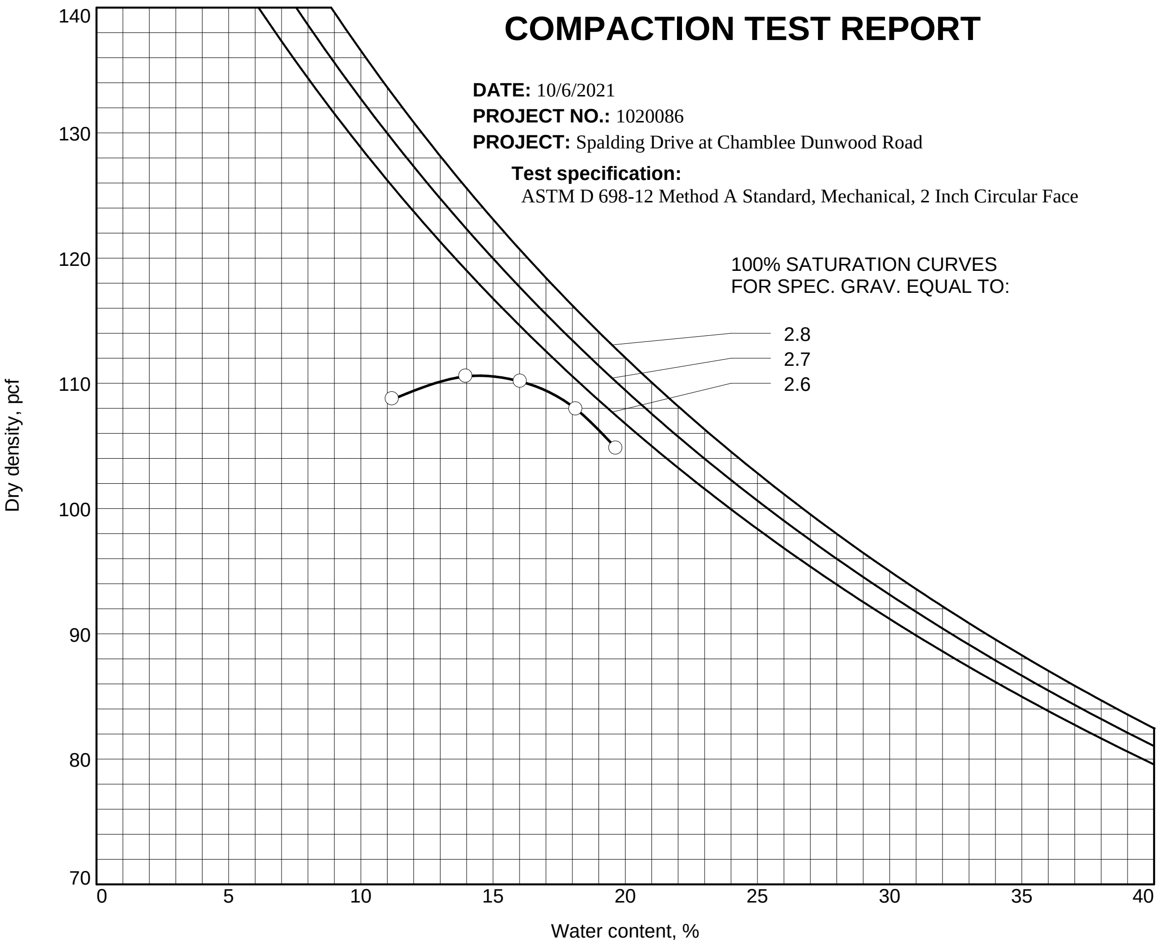
SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	B-1	B-1	11-15'	32.4	29	34	5	SM

Nova Engineering & Environmental Norcross, GA	**Client:** City of Dunwoody **Project:** Spalding Drive Catch Basin Subgrade Exploration **Project No.:** 10103-1020086 **Figure**

COMPACTION TEST REPORT

DATE: 10/6/2021
PROJECT NO.: 1020086
PROJECT: Spalding Drive at Chamblee Dunwood Road

Test specification:
ASTM D 698-12 Method A Standard, Mechanical, 2 Inch Circular Face



Sample No.	Elev. or Depth	Material Description	Specific Gravity	LL	PL	Oversize	% < #200
B-1	11-15'	Orange-gray silty medium to fine SAND		34	29	%>#4=8.7	42.5

Sample No.	B-1			
Natural water content, percent	32.4			
Optimum water content, percent	14.5			
Max dry density, pcf	110.6			

Remarks: Alluvium

Project: Spalding Drive Catch Basin Subgrade Evaluation
Project No.: 10103-1020086

Location:

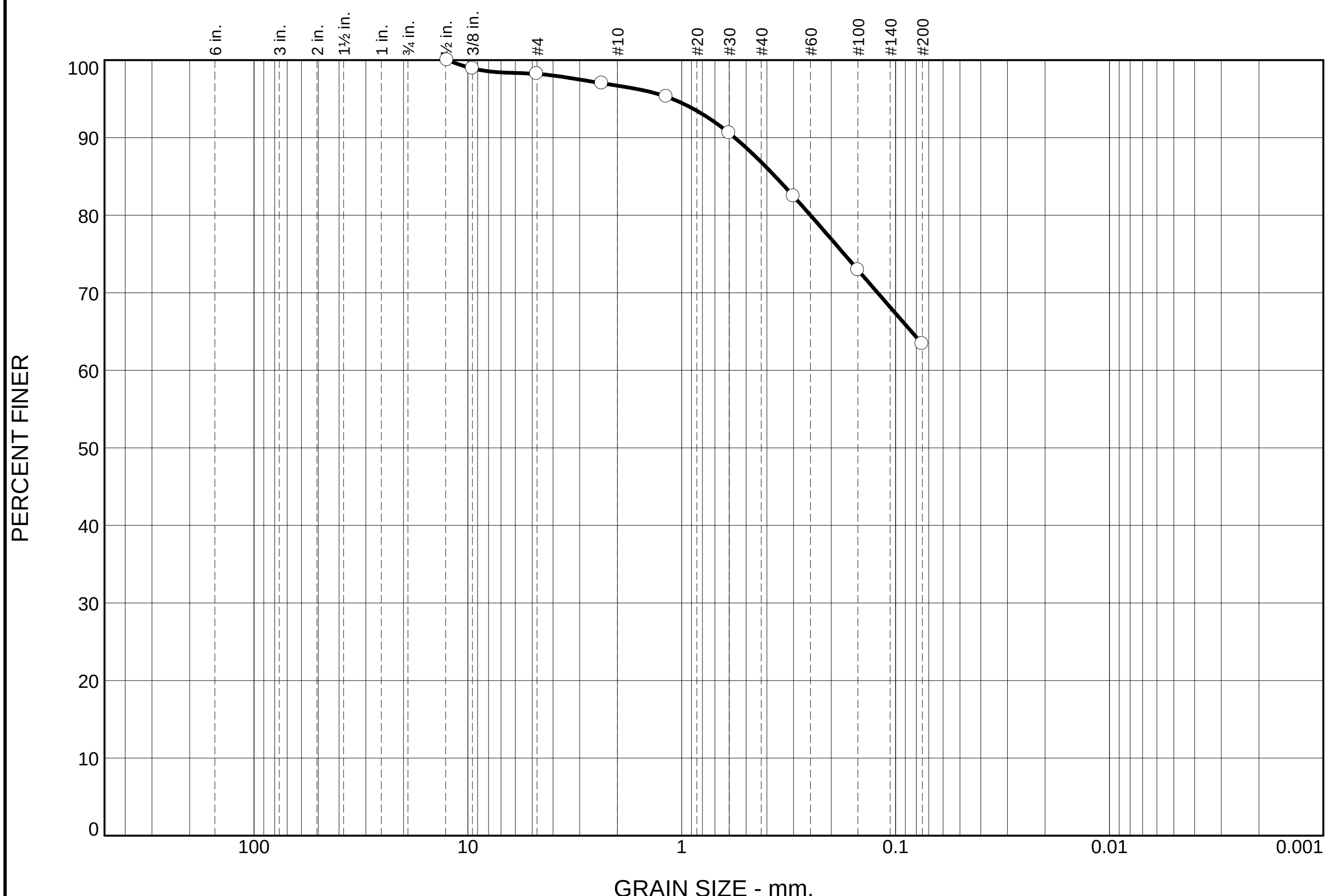
Source: B-1

Figure

Nova Engineering & Environmental

Norcross, GA

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	1.8	1.5	9.8	23.5	63.4	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			0.3656							

MATERIAL DESCRIPTION	TEST DATE	USCS	NM
○ Brown-red medium to fine sandy SILT		MH	44.1%

Project No. 10103-1020086 **Client:** City of Dunwoody

Project: Spalding Drive Catch Basin Subgrade Exploration

○ **Source of Sample:** B-3 **Depth:** 0-10' **Sample Number:** B-3

Remarks:

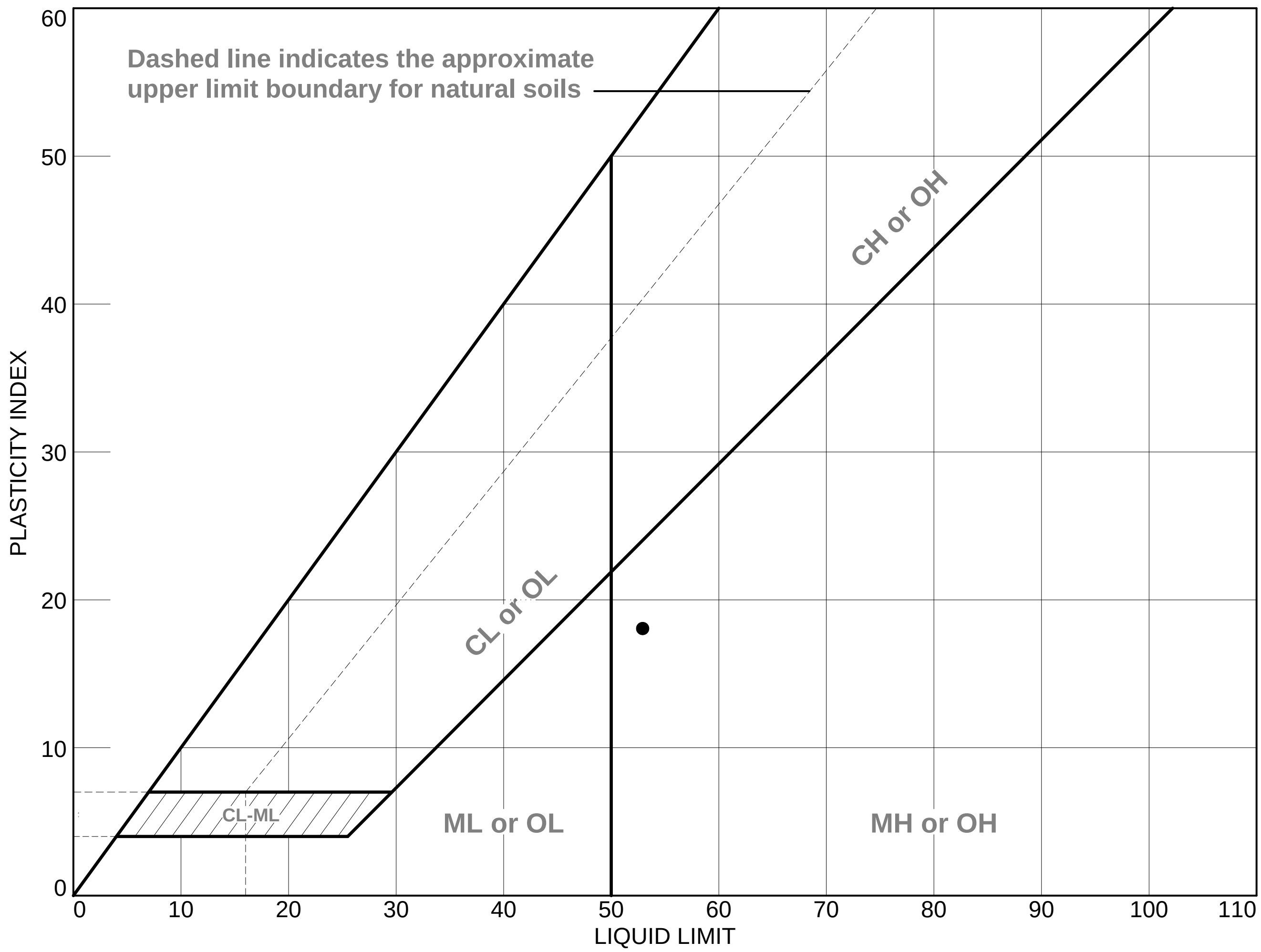
○Residuum

Nova Engineering & Environmental

Norcross, GA

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT



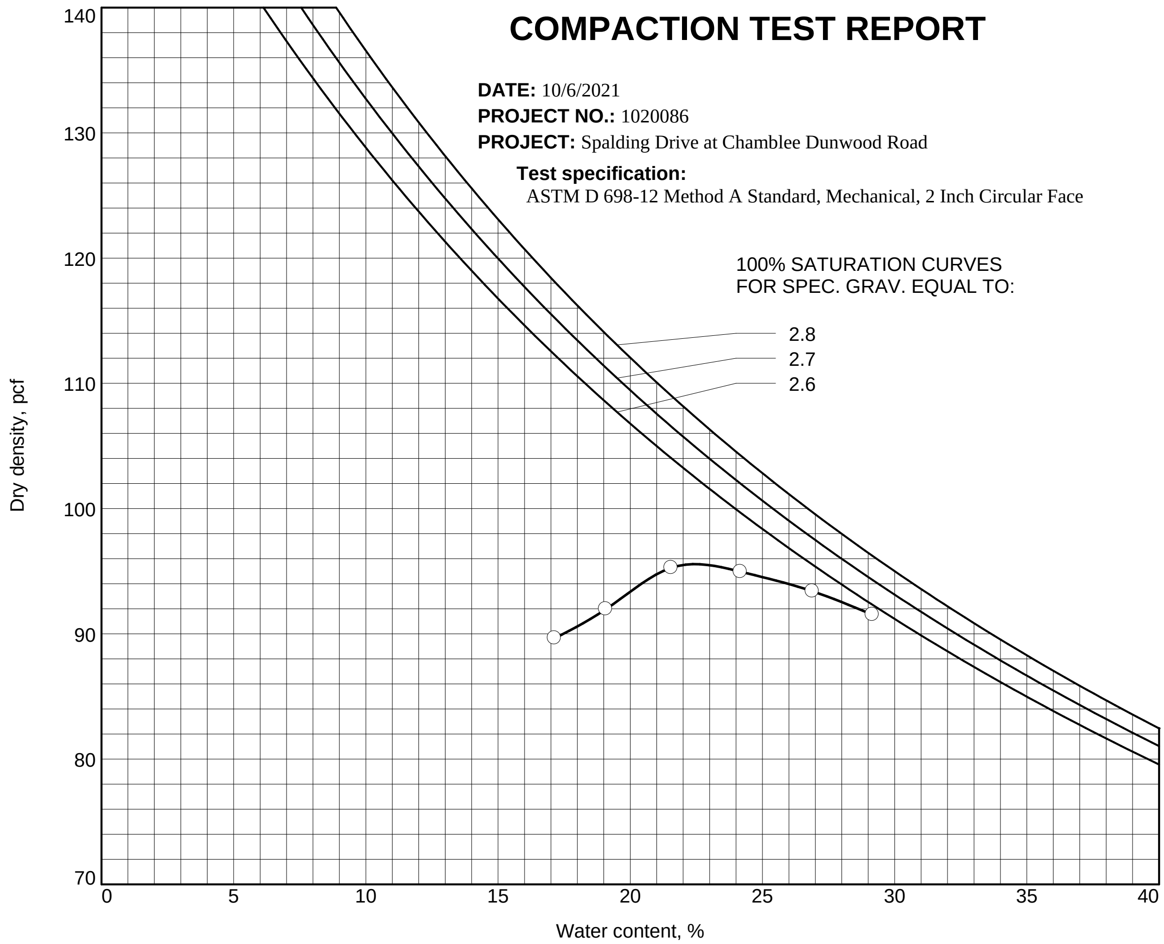
SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	B-3	B-3	0-10'	44.1	35	53	18	MH

Nova Engineering & Environmental Norcross, GA	**Client:** City of Dunwoody **Project:** Spalding Drive Catch Basin Subgrade Exploration **Project No.:** 10103-1020086 **Figure**

COMPACTION TEST REPORT

DATE: 10/6/2021
PROJECT NO.: 1020086
PROJECT: Spalding Drive at Chamblee Dunwood Road

Test specification:
ASTM D 698-12 Method A Standard, Mechanical, 2 Inch Circular Face



	Sample No.	Elev. or Depth	Material Description	Specific Gravity	LL	PL	Oversize	% < #200
○	B-3	0-10'	Brown-red medium to fine sandy SILT		53	35	%>#4=1.8	63.4

Sample No.	B-3			
Natural water content, percent	44.1			
Optimum water content, percent	22.4			
Max dry density, pcf	95.6			

Remarks: Residuum

Project: Spalding Drive Catch Basin Subgrade Exploration **Project No.:** 10103-1020086

Location:

Source: B-3

Nova Engineering & Environmental

Norcross, GA

Figure

APPENDIX D
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 site work 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 study 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 study 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 work performed during this study 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.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a civil engineer may not fulfill the needs of a constructor — a construction contractor — or even another civil engineer. Because each geotechnical- engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. No one except you should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply this report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical-engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

Geotechnical Engineers Base Each Report on a Unique Set of Project-Specific Factors

Geotechnical engineers consider many unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk-management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical-engineering report that was:

- not prepared for you;
- not prepared for your project;
- not prepared for the specific site explored; or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical-engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical-engineering report is based on conditions that existed at the time the geotechnical engineer performed the study. *Do not rely on a geotechnical-engineering report whose adequacy may have been affected by*: the passage of time; man-made events, such as construction on or adjacent to the site; or natural events, such as floods, droughts, earthquakes, or groundwater fluctuations. *Contact the geotechnical engineer before applying this report to determine if it is still reliable.* A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ — sometimes significantly — from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide geotechnical-construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are Not Final

Do not overrely on the confirmation-dependent recommendations included in your report. *Confirmation-dependent recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations *only* by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's confirmation-dependent recommendations if that engineer does not perform the geotechnical-construction observation required to confirm the recommendations' applicability.*

A Geotechnical-Engineering Report Is Subject to Misinterpretation

Other design-team members' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Constructors can also misinterpret a geotechnical-engineering report. Confront that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing geotechnical construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical-engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make constructors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give constructors the complete geotechnical-engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise constructors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure constructors have sufficient time* to perform additional study. Only then might you be in a position to give constructors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and constructors fail to recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of 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.

Environmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform an *environmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold-prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, many mold- prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical- engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; *none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.*

Rely, on Your GBC-Member Geotechnical Engineer for Additional Assistance

Membership in the Geotechnical Business Council of the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you GBC-Member geotechnical engineer for more information.

GBC GEOTECHNICAL BUSINESS COUNCIL
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