GEOTECHNICAL ENGINEERING REPORT

For

Proposed Taco Bell #315462
NWC Maple Street and Common Boulevard
Carrollton, Georgia

PSI Project No. 0775-3218

PREPARED FOR:

Taco Bell Corporation
1 Glen Bell Way
Irvine, Georgia 92618

December 20, 2021

BY:

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December 20, 2021

Taco Bell Corporation
1 Glen Bell Way
Irvine, Georgia 92618

Attn: Mr. Chad Gomall
Associate Construction Manager
Mobile: 814.572.4800
Email: Chad.Gornall@yum.com

Re: Geotechnical Engineering Report
Proposed New Taco Bell #315462
NWC Maple Street and Common Boulevard
Carrollton, Georgia
PSI Project No. 0775-3218
Taco Entity No. 457424

Dear Mr. Gomall:

Professional Service Industries, Inc. (PSI), an Intertek Company, is pleased to submit our geotechnical engineering services report for the Proposed New Taco Bell Project to be located at NWC Maple Street and Common Boulevard Carrollton, Georgia. This report presents the results of our field exploration program and includes geotechnical recommendations to guide design and construction of the project.

Should there be any questions, please do not hesitate to contact our office at (813) 886-1075 at your convenience. PSI would be pleased to continue providing geotechnical services or construction materials testing throughout the implementation of the project. We look forward to working with you and your organization on this and future projects.

Respectfully submitted,

PROFESSIONAL SERVICE INDUSTRIES, INC.
Certificate of Authorization No: 3684

Muthanna Al Saadi

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JOSE N GOMEZ

This document has been signed and sealed by Jose N. Gomez, P.E. Printed copies of this document are not considered signed and sealed and the signature must be verified on any electronic copies.
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1.0 PROJECT INFORMATION

1.1 PROJECT AUTHORIZATION

Professional Service Industries, Inc. (PSI), an Intertek company, has completed a geotechnical exploration evaluation and corresponding report for the Proposed New Taco Bell Project to be located at NWC Maple Street and Common Boulevard Carrollton, Georgia. These services were performed in general accordance with the Project Agreement For Architectural / Engineering / Consultant Services Form between Taco Bell of America, Inc., and PSI.

1.2 PROJECT DESCRIPTION

Based on information provided by Taco Bell Corporation, PSI's review provided site sketch dated September 08, 2021, prepared by GPD Group Professional Corporation, we understand the project consists of constructing a new single-story structure with an approximate footprint area of 2,200 square feet with a corresponding sidewalk and paved area. The total developed area is approximately 28,314 square feet.

This report is based on loads for isolated column and continuous wall footings not exceeding 30 kips and three kips per linear foot, respectively. Traffic loading information was not provided; therefore, the recommendations are based on light daily consisting of 30,000 ESAL over 20 years and heavy-duty being 60,000 ESAL over 20 years.

At the time of this report, we were not provided with proposed grading information for the project. However, based on our site reconnaissance observations, we estimate relief within the proposed building footprint is on the order of one to two feet and overall relief across the proposed construction area on the order of three feet. This report is based on maximum cut and fill depths being on the order of three feet.

The following table summarizes general project characteristics.

Table 1.2-1: General Project Description

Project Items	Single Story ±2,200 Sq ft Structure with associated parking and drive areas.
Existing Grade Change within Building Pad Area	± Three feet estimate (Google Earth Pro)
Existing Grade Change within Project Site Area	± Seven feet estimate (Google Earth Pro)
Finished Floor Elevation	Not available at this time, considered within two feet ± of current grade
Anticipated Foundation Type	Shallow foundation and slab on grade
Estimated Maximum Column Loading	30 kips
Estimated Maximum Wall Loading	Three linear kips per foot

Pavement	Flexible Asphalt (HMAC) and/or Rigid Concrete Pavement

The geotechnical recommendations presented in this report are based on the available project information, structure locations, considered structure loads, and the subsurface materials encountered during the field evaluation. If the noted information or structural loads are incorrect, please inform PSI so that the recommendations presented in this report can be amended as necessary before project construction commences. PSI will not be responsible for the implementation of provided recommendations if not notified of changes in the project.

1.3 PURPOSE AND SCOPE OF SERVICES

The purpose of this study is to evaluate the subsurface conditions at the site and develop geotechnical engineering recommendations and guidelines for use in preparing the design and other related construction documents associated with structure foundations and pavement for the proposed project. The scope of services included drilling soil borings, performing laboratory testing, and preparing this geotechnical engineering report.

This report briefly outlines the available project information, describes the site and subsurface conditions, and presents the recommendations regarding the six Standard Penetration Testing (SPT) drilled to depths of 20 feet below existing grades within the proposed building (see **Figure 2**, **Appendix A**).

The scope of services for this geotechnical exploration did not include an environmental, mold nor detailed seismic/fault assessment for determining the presence or absence of wetlands, or hazardous or toxic materials in the soil, bedrock, surface water, groundwater, or air on or below, or around this site. Statements in this report or on the boring logs regarding odors, colors, and unusual or suspicious items or conditions are strictly for informational purposes.

2.0 SITE AND SUBSURFACE CONDITIONS

2.1 SITE DESCRIPTION

The project site is undeveloped land located at NWC Maple Street and Common Boulevard Carrollton, Georgia (west of Atlanta). At the time of our fieldwork, the site was observed to be relatively sloped to the north. The following table provides a generalized description of the existing site conditions based on visual observations during the field activities, as well as other available information.

Table 2.1-1: Site Description

Site Location	Latitude: 33.562687; Longitude: -85.111657, Carrollton, GA
Site History	Undeveloped land
Existing Site Ground Cover	Grass area.
Existing Grade/Elevation Changes	Approximately EL +1079 to +1087 (Google Earth Pro).
Description of Adjacent Property	North Boundary: Maple Crossing East Boundary: Common Boulevard West Boundary: Undeveloped land South boundary: Maple Street

2.2 SITE PHOTO

The following photograph shows the area where the new development. Exploration borings were conducted within the highlighted yellow frame.

Figure 2.2-1: Aerial photograph of the subject property in Carrollton, GA

2.3 FIELD EXPLORATION

Field exploration for the proposed Taco Bell project area consisted of drilling a total of six Standard Penetration Test (SPT) borings. The boring design element, boring labels, approximate depths, and drilling footage are provided in the following table.

Table 2.3-1: Field Exploration Summary

Design Element	Number of Borings	Boring depth (ft)	Drilling Footage (feet)
Building Area (B-1 through B-6)	6	20	120
		TOTAL:	**120**

All Borings were drilled to the planed depths as shown in the boring logs with soils descriptions included in **Appendix B** .

The boring locations were selected by PSI personnel based on boring layout provided, and located in the field using a recreational-grade GPS system. Elevations of the ground surface at the boring locations were not provided and should be surveyed by others prior to construction. The references to elevations of various subsurface strata are based on depths below existing grade at the time of drilling. The approximate boring locations are depicted on the Boring Location Plan provided in **Figure 2, Appendix A**. The following table summarizes the characteristics of the field exploration and drilling.

Table 2.3-2: Field Exploration Description

Drilling Equipment	Truck Mounted Drilling Rig
Drilling Method	Solid Flight Augers, Hollow Stem Augers
Drilling Procedure	Applicable ASTM and PSI Safety Manual
Field Testing	Standard Penetration Test (ASTM D1586)
Sampling Procedure	Soils: ASTM D1587/1586
Sampling Frequency	Two and a half -foot Intervals to a depth of 10 feet and at five-foot intervals thereafter
Frequency of Groundwater Level Measurements	During and after drilling
Boring Backfill Procedures	Soil cuttings, Bentonite pellets.

During field activities, the encountered subsurface conditions were observed, logged, and visually classified (in general accordance with ASTM D2487). Field notes were maintained to summarize soil types and descriptions, water levels, changes in subsurface conditions, and drilling conditions.

2.4 LABORATORY TESTING PROGRAM

The soil samples recovered from the borings were visually reviewed in the laboratory by a geotechnical engineer to confirm the field classifications. The samples were classified using the Unified Soil Classification System (USCS) in general accordance with the American Society of Testing and Materials (ASTM) test designation D2487. The soil classification was based on visual observations and laboratory testing including passing 200 sieve, moisture content and Atterberg Limits.

2.5 SITE GEOLOGY

The project site is located in Carroll County within the Piedmont Physiographic Province of Georgia. Native soils in upland areas within this province are formed from the in-place weathering and decomposition of native igneous and metamorphic bedrock materials. The residual soils near the ground surface are generally finer grained in texture due to advanced weathering. With depth, materials tend to be coarser grained and partially exhibit the former visual patterns of the underlying bedrock. Separating the residual soil from the underlying parent bedrock is typically a variable thickness transition zone of high consistency material referred to as partially weathered rock (PWR). PWR is defined as residual soil which regularly exhibits elevated standard penetration resistance (ASTM D1586) values approaching or exceeding “refusal” criteria defined as 50+ blows per sampling interval (six inches).

The weathering processes that produce residual soils and partially weathered rock in the regional geology can be extremely variable. Differential weathering of the parent bedrock can result in highly variable subsurface conditions and can include abrupt changes in soil type and consistency over relatively short horizontal and vertical distances. Furthermore, depths to rock can also be highly variable; and suspended boulders, discontinuous rock layers/lenses, or rock pinnacles can be present within the residual soils and transitional zones of soft weathered rock.

Residual soils may be overlain by alluvium. Alluvium is material that has been transported and deposited by flowing water and is usually found adjacent to watercourses. Based on its method of depositions, it can vary considerably from the parent material and have extreme changes in material type and consistency in short horizontal and vertical distances. Alluvium can include organic materials. Because it is a relatively recent geologic deposit it can be soft or loose and normally consolidated. Soil survey mapping indicates alluvial deposits in strips of land adjacent to the creeks at the north and east sides of the site.

A review of the USGS geologic map of Georgia suggests Granitic Gneiss likely underlies the site. Figure below depicts a general geology map within the project location.

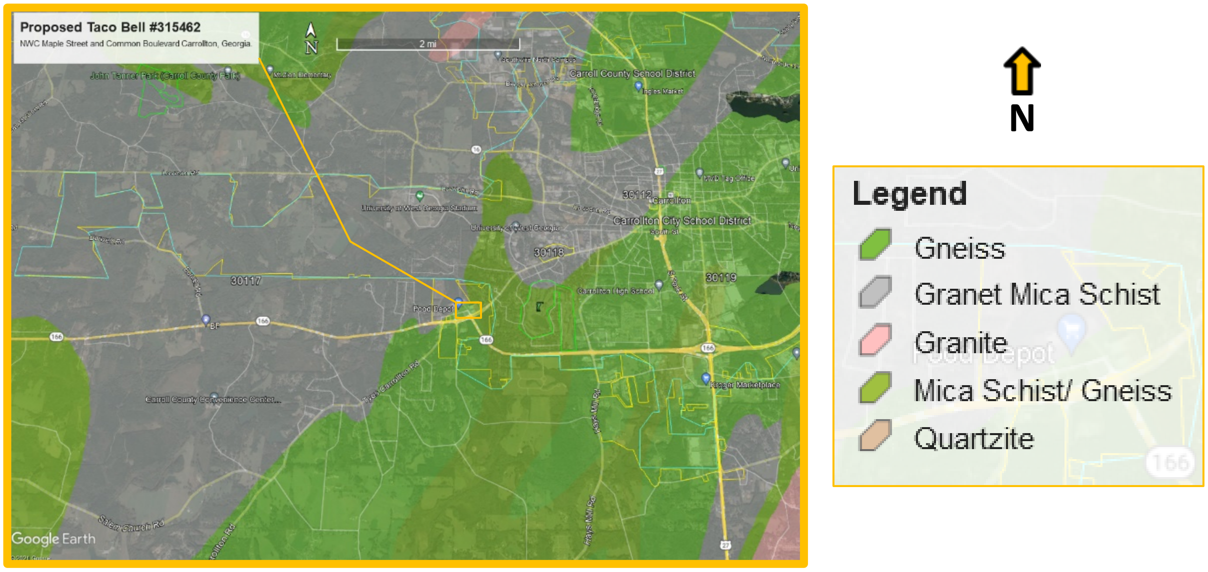


Figure 2.5-1: Carroll County Geology; project location in yellow frame.

2.6 SUBSURFACE CONDITIONS

The results of the field and laboratory evaluation have been used to generalize a subsurface profile at the project site. The following subsurface descriptions provide a highlighted generalization of the major subsurface stratification features and material characteristics within the proposed Taco Bell property footprint, based on the borings and samples visual description.

Table 2.6-1: Generalized Soil Profile Description

Stratum	Top (ft)	Bot. (ft)	Soil Type	N Range	N Average	Density
-	0.0	0.3	Topsoil	-	-	-
1	0.3	20	Sandy SILT (ML)	6- 23	14	Stiff
3	6	13.5	High Plasticity SILT (MH)	10-16	13	Stiff
4	13.5	20	Silty SAND (SM)	17-51	22	Medium Dense

*Where: N=Standard Penetration Test blow count (blows/foot)-See Key to Terms and Symbols Used on Logs, **Appendix B***

The boring logs and general soil profile are included at the end of the **Appendix B** should be reviewed for specific information at individual boring locations. Note that the existing ground elevations in the boring logs and in the soil profile are approximate and they need to be confirmed with a topographic survey as needed it. The boring logs include soil descriptions, stratifications, locations of the samples, and field and laboratory test data. The descriptions provided on the logs only represent the conditions at that actual boring location; the stratifications represent the approximate boundaries between subsurface materials. The actual transitions between strata may be more gradual and less distinct. Variations will occur and should be expected across the site. If variations in subsurface conditions from those described are noted during construction, recommendations in this report may need to be re-valuated.

2.6.1 GROUNDWATER INFORMATION

Groundwater wasn't encountered in the borings to a depth of 20 feet below the ground surface at the time that drilling was conducted in December 2021. It should be noted that groundwater levels tend to fluctuate during periods of prolonged drought and extended rainfall and may be affected by man-made influences. In addition, a seasonal effect will also occur in which higher groundwater levels are normally recorded during rainy seasons.

3.0 GEOTECHNICAL EVALUATION AND RECOMMENDATIONS

3.1 GEOTECHNICAL DISCUSSION

Below the surface materials, residual soils were generally encountered. Based on the laboratory testing results and interpretations, these soils appear to be partially saturated and most likely overconsolidated, and a groundwater table was not encountered in the upper 20 feet of soil explored. The residual soils generally consisted of sandy silt (i.e. ML material) underlined by high plasticity Silt (MH) and silty sand (SM). Based on this information and the structure and loads characteristics, it is our opinion that the proposed building structure can be supported on shallow foundation bearing on compacted granular soils or on existing natural soils, provide that our recommendations given in **Section 5.0** are met.

The following design recommendations have been developed based on the previously described project characteristics and subsurface conditions encountered. If there are changes in the project criteria and/or structural loads, PSI should be retained to determine if modifications in the recommendations will be required before project construction starts. The findings of such a review would be presented in a supplemental report and additional resources may be needed.

Once final design plans and specifications are available, a general review by PSI is recommended to observe that the conditions assumed in the project description are correct and to verify that the earthwork and foundation recommendations are properly interpreted and implemented within the construction documents; this scope will require additional PSI resources.

3.2 FOUNDATION RECOMMENDATIONS DISCUSSION

Based on information provided to PSI, information obtained during the field operations, results of the laboratory testing, the considered structural loads and PSI's experience with similar projects, recommendations for shallow foundation system consisting of individual columns and/or continuous wall footings supported on compacted natural soils are presented in this report. If an alternative foundation type is desired, PSI can provide alternative recommendations in a supplemental letter upon request.

3.3 SHALLOW FOOTING FOUNDATIONS RECOMMENDATIONS

As previously mentioned, the building can be supported by continuous wall footings and isolated spread footings bearing on compacted soils or natural soils as shown in the illustration and described below:

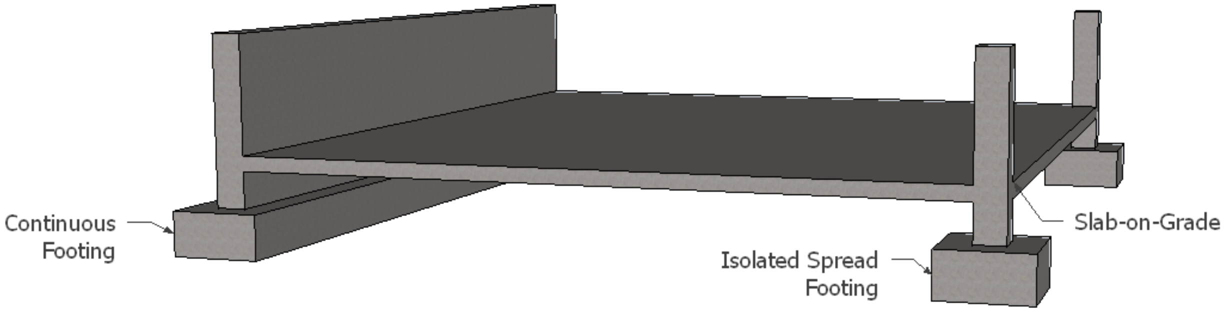


Figure 3.3-1: Spread footing systems

Bearing Capacity. Following site preparation procedures noted in **Section 5.0** of this report, and the proposed structures can be supported on shallow foundations that are designed using a maximum net allowable soil bearing capacity of 2,500 pounds per square foot (psf) bearing on the compacted granular soils. Depending upon grading and footing locations, potentially high plasticity Silt (MH) soils will have to be removed and replaced with imported structural fill to a depth of two feet below the foundation subgrade in some areas if founded (B-2, B-3, and B-6 encountered (MH) at six feet below ground surface).

We recommend a minimum footing embedment depth of 24 inches below the finished exterior grade to assure allowable bearing capacity. We further recommend that the individual column and continuous wall footings have a minimum width of 24 inches. The purpose of limiting the minimum footing size is to prevent a "punching" shear failure and to reduce the possibility of bearing on an isolated weak zone. Considering that the frost line depth in Carrollton, Georgia is 14 inches, any structure foundations or utilities must also be located below that frost line.

Foundations subject to transient lateral loads will resist these forces through a combination of base shearing resistance mobilized at the footing-subgrade interface and earth pressure acting on the vertical faces of the footings at right angles to the direction of applied load. Base shearing resistance may be determined using a friction factor of 0.5.

Passive earth pressure resistance should be computed using an equivalent fluid pressure of 330 pounds per square foot per foot of depth (330H), for granular backfill material. Resistance to sliding determined in accordance with the noted parameters should be considered ultimate resistance. Accordingly, the design for sliding resistance should include a factor of safety of at least 1.5 be used.

Settlements. Provided that all vegetation, topsoil, organic, deleterious materials or unsuitable fill encountered during site preparation have been removed, and the site preparation recommendations stated in **Section 5.0** of this report are implemented, the potential total settlement of wall and isolated column footings from the applied loads is expected to be one inch or less considering that encountered soils appear to be overconsolidated. Maximum differential settlement between adjacent columns or across approximately 30 feet of continuous footing length is expected to be half the total settlement.

Total and differential settlements of the noted magnitudes are usually considered tolerable for most construction; however, the tolerance of the proposed structure to the predicted total and differential settlements should be confirmed by the structural engineer/architect. Additionally, our settlement estimates are based on the anticipated foundation loads on the order of magnitude noted in **Section 1.0**; any changes on these loads will modify the calculated settlements, as such, PSI must be notified to provide the required report update.

3.3.1 FLOOR SLAB

We recommend that the procedures described in **Section 5.0** of this report be used to prepare the floor slab subgrade. Ground floor slabs can bear directly on top of compacted structural fill material. A modulus of subgrade reaction value of 150 pounds per cubic inch (pci) (based on expected results of a one-foot square plate load test) may be used for design assuring that subgrade was properly compacted. The value should be geometrically modified and reduced for larger areas.

High plasticity Silt (MH) soils were encountered at six feet below ground surface; depending upon grading or cuts required for slab on grade subgrade, if these high plasticity soils are encountered, they will have to be removed and replaced with imported structural fill to a depth of one foot below the slab subgrade in some slab areas.

The floor slabs should be adequately reinforced to reduce the risk of cracking due to differential settlement. Floor slabs should not be rigidly connected to wall footings. An impervious membrane should be installed between the soil subgrade and bottom of floor slabs to avoid slab moisture problems. An ultimate friction factor of 0.21 should be used for the vapor barrier-soil interface with an appropriate factor of safety. Floor slab design should conform to American Concrete Institute (ACI) design standards and practices.

3.4 SIDEWALKS AND FLATWORK RECOMMENDATIONS

For sidewalks or other flatwork located adjacent to grade-supported foundations, the undercutting and select fill placement operations for the building should extend beyond the perimeter of the building and pavements to at least the width of the adjacent sidewalk or flatwork.

Any other sidewalks or flatwork not adjacent to buildings should be placed on an improved subgrade meeting or exceeding the pavement subgrade improvement methods previously recommended. A 12-inch-thick layer of material satisfying the requirements of select fill provided in **Section 5.0** must be placed below the sidewalk. The material should be compacted to 95 percent or greater than the maximum dry unit weight and contain a moisture content between -one and +three percent optimum moisture content.

Proper drainage around grade-supported sidewalks and flatwork is also very important to reduce potential movements. Elevating the sidewalks where possible and providing rapid, positive drainage away from them will reduce moisture variations within the underlying soils and will therefore provide valuable benefit in reducing the full magnitude of potential movements from being realized.

3.5 LATERAL EARTH PRESSURES

Retaining walls should be designed to withstand the lateral earth pressures exerted by the compacted backfill. The pressure diagram is triangular. It is anticipated that retaining walls associated with the building structure, will be rigid walls restrained from rotation. For rigid walls, the "At Rest" (k_o) soil condition should be used in the wall design and evaluation. For walls that are free to deflect at their tops, the "Active" (k_a) soil condition could be used in the wall design and evaluation. In the design of these retaining wall structures, the following soil parameters included in the following table can be utilized. These parameters assume that compacted Granular Soils meeting the requirements recommended herein for Retaining Wall Backfill will comprise the backfill in the Critical Zone. The Critical Zone is defined as the area between the back of the retaining wall structure and an imaginary line projected upward and rearward from the bottom back edge of the wall footing at a 45-degree angle.

Table 3.5-2: Soil Parameters

Retaining Wall Backfill in Critical Zone	Coefficient of Earth Pressure at Rest (K_o)	0.50
	Coefficient of Active Earth Pressure (K_a)	0.33
	Retained Soil Moist Unit Weight (γ)	110 pcf
	Cohesion (C)	0 psf
	Angle of Internal Friction (ϕ)	30°
	Friction Coefficient [Concrete on Soil] (μ)	0.40
	At-rest Equivalent Fluid Pressure	59H (psf)
	Active Equivalent Fluid Pressure	39H (psf)

Site retaining walls are often constructed from the "bottom-up" and therefore the type of soil used to backfill the wall is chosen or specified by contract. The lateral earth pressures developed behind site retaining walls is a function of the backfill soil type within an approximate 45-degree angle from the base of the wall upward.

Retaining Wall Backfill: All soils used as backfill within the Critical Zone behind retaining walls should have USCS classifications of SAND (SM) or more granular with a maximum of 10 percent fines (i.e., % passing No. 200 Sieve size) and minimum angle of internal friction of 30 degrees when compacted to a minimum of 95 percent of its maximum dry density per ASTM D 1557. Any existing soils not meeting these criteria should be removed from the Critical Zone of the walls, as determined by PSI personnel at the time of construction.

Foundation Drains: Retaining walls should be provided with a foundation drainage system to relieve hydrostatic pressures which may develop in the wall backfill. This system should consist of weepholes through the wall and/or a four-inch perforated, closed joint drain line located along the backside of the walls above the top of the footing. The drain line should be surrounded by a minimum of six inches of Size No. 57 Stone wrapped with an approved non-woven filter fabric, such as Mirafi 140-N or equivalent.

Wall Drains: All site retaining walls should be drained so that hydrostatic pressures do not build up behind the walls. Wall drains can consist of a 12-inch wide zone of free draining Gravel, such as No. 57 Stone, employed directly behind the wall and separated from the soils beyond with a non-woven filter fabric. Alternatively, the wall drain can consist of a suitable geocomposite drainage board material. The wall drain should be hydraulically connected to the foundation drain.

3.6 SEISMIC CONSIDERATIONS

The project site is located within a municipality that employs the International Building Code, 2018 edition. As part of this code, the design of structures must consider dynamic forces resulting from seismic events. These forces are dependent upon the magnitude of the earthquake event as well as the properties of the soils that underlie the site. As part of the procedure to evaluate seismic forces, the code requires the evaluation of the Seismic Site Class, which categorizes the site based upon the characteristics of the subsurface profile within the upper 100 feet of the ground surface. To define the Site Class for this project, we have interpreted the results of soil test borings drilled within the project site and estimated appropriate soil properties below the base of the borings to a depth of 100 feet based upon data available in published geologic reports as well as our experience with subsurface conditions in the general site area.

Based upon our evaluation, it is our opinion that the subsurface conditions within site are consistent with the characteristics of Site Class "D" (N<15, Stiff Silty Sand). The National Earthquake Hazards Reduction Program

(NEHRP) probabilistic ground motion values for coordinates 33.562687, -85.111657 Georgia. Obtained from the US Seismic Design Maps Tool from the USGS website are as follows:

Table No. 3.6-1 – Seismic Design Parameters

Period (seconds)	2% Probability of Event in 50 years (g)	Site Coefficients	Max. Spectral Acceleration Parameters	Design Spectral Acceleration Parameters
PGA	PGA = 0.095	F_{PGA} = 1.6	PGA_M = 0.152	---
0.517 (S_S)	S_S = 0.186	F_a = 1.6	S_{MS} = 0.298	S_{DS}= 0.198
0.122 (S_1)	S_1 = 0.084	F_v = 2.4	S_{M1} = 0.202	S_{D1}= 0.135

For category "D" site classifications, IBC code requires an assessment of slope stability, liquefaction potential, and surface rupture due to faulting or lateral spreading. Detailed assessments of these factors were beyond the scope of this study. However, the following presents a qualitative assessment of these issues considering the site class, the subsurface soil properties, the groundwater elevation, and probabilistic ground motions:

Table No. 3.6-2 Seismic Hazards

Hazard	Relative Risk	Comments
Liquefaction	Low	The soils within the upper 20 feet contain adequate measured fines to limit liquefaction potential and water is not present.
Slope Stability	Low	The probabilistic ground accelerations are low and site grades are slightly sloped to the north.
Surface Rupture	Low	At the time of our subsurface exploration in August 2021 and a review in google earth available map, no known active faults underlie the site.

4.0 PAVEMENT DESIGN CONSIDERATIONS

4.1 PAVEMENT DESIGN PARAMETERS

We recommend that the procedures described in **Section 5.0** of this report be used to prepare the subgrade of pavements. Flexible pavement sections in this geographic area typically consist of an asphaltic concrete wearing course, we estimate soils like those typically encountered in the borings which are primarily silty sands, will have a CBR value of about four or greater. Based on the expected traffic loading and our experience in the area, the minimum pavement section thicknesses noted in **Table 4.1.1** should be acceptable.

Where dumpsters are to be located on the pavement, so that considerable load is transferred from relatively small steel supports, it is recommended that rigid concrete pavement be constructed. In addition, the area utilized for unloading the dumpsters by heavy duty-trucks should also be provided with a rigid pavement. A minimum Portland cement concrete pavement thickness of five inches should be used in parking areas (light duty) and six inches in loading areas (heavy duty), if rigid pavements are to be employed. The subgrade soils below concrete pavements should be well-draining granular materials compacted to a minimum density of 98 percent of the modified Proctor maximum dry density (ASTM D-1557). Fill that may be required to raise grades in pavement areas should be compacted to at least 95 percent of the material's maximum dry density (ASTM D-1557).

A conservative California Bearing Ratio (CBR) value of five was considered for the on-site SILTS and SANDS, or newly placed structural fill, at compaction levels of about 98 percent of the standard Proctor maximum dry density within about three percent of optimum moisture.

Actual pavement section thicknesses and the reinforcement details for the rigid pavement section should be provided by the Design Civil Engineer based on traffic loads, volume, and the owner's design life requirements. The noted sections represent minimum thicknesses for typical local construction practices and, as such, periodic maintenance should be anticipated. All pavement materials and construction procedures should conform to GDOT, American Concrete Institute (ACI), or appropriate city/county requirements.

Table -1: Mimimum Pavement Section Recommendations

Pavement Type	**Layer**	**Material Description**	**Recommended Minimum Layer Thickness (Inches)**	
			Light Duty	**Heavy Duty**
Flexible	(A)	Georgia DOT Asphalt Type SP	2	2
	(A)	Asphalt Binder Layer	2	3
	(B)	Graded Aggregate Base (GAB)	8	8
Rigid	(C)	Georgia DOT Portland Cement Concrete	6	7
	(B)	Graded Aggregate Base (GAB)	8	8
(A) = Asphaltic Concrete, (B) = Base Course,(C) = Concrete				

Notes: 1) Light Duty Areas calculated based on traffic loading of 30,000 ESALS or less over 20 years with the reliability of 80%.
Parking stalls only with no through traffic. Prime coat required between ABC and asphalt.
2) Heavy Duty Areas calculated based on traffic loading of 60,000 ESALS or less over 20 years.

Pavement materials selection and placement procedures and specifications should be in accordance with the Georgia Department of Transportation Specifications. Actual pavement sections should be based upon the anticipated traffic and performance expectations and should meet all applicable local and state standards as well as client specific requirements.

The concrete should have a 28-day compressive strength of at least 4,000 psi and should be air entrained. The recommendations provided are for non-reinforced concrete. ACI guidance is recommended for selection and spacing of jointing, dowels, and sealing of pavement joints.

All pavements should be sloped a minimum of one percent to provide rapid surface drainage. Water allowed to pond on or adjacent to the pavement could saturate the subgrade and cause premature pavement deterioration.

We recommend that the pavement subgrade moisture content be adjusted to be within two percentage points of optimum moisture and the subgrade surface should be recompacted prior to pavement construction in accordance with the "Site Preparation/Structural Fill" section of this report.

The subgrade should be evaluated by a representative of PSI immediately prior to placing GAB. The evaluation should include proof rolling of the final subgrade with a loaded tandem axle dump truck. If low consistency or yielding soils are encountered which cannot be adequately densified in place, such soils should be removed and replaced with well-compacted soil fill or crushed stone materials.

The subsurface soils encountered in the borings were sandy silt. These soils typically behave like fine grained silty soils during subgrade preparation, particularly when the upper most soils with some cohesion are removed or buried. Where the silty and micaceous soils are exposed at subgrade levels it if is often difficult to prepare a final unyielding subgrade in these materials that is satisfactory for placement of base materials. If these soils cannot be made stable by scarification and compaction, it may be necessary to selectively undercut unyielding soils and thicken the crushed stone base layer.

5.0 CONSTRUCTION CONSIDERATIONS

Having a Geotechnical Engineer retained to review the earthwork recommendations in the Contract Documents and be an active participant in team meetings near the time of construction can often result in project cost savings. Therefore, PSI recommends that an AASHTO accredited 3rd party laboratory with qualified professional engineers who specialize in geotechnical engineering be retained to provide observation and testing of construction activities involved in the foundations, earthwork, pavements and related activities of this project. As the Geotechnical Engineer of Record, PSI's services can be retained as the 3rd party laboratory. PSI's participation would be advantageous to the project flow and value engineering during construction since we are most familiar with the existing soil conditions at the site.

The geotechnical engineer (GER) often does not have available all design information at the time of writing the original report since the report is done very early in the design process. The GER can be of great benefit immediately prior to construction since definitive information regarding the location of the building, surrounding flatwork, planned landscaping, and drainage features is available. The GER can then write supplement letters to the original geotechnical report often resulting in less risk and project cost savings.

PSI cannot accept responsibility for conditions which deviate from those described in this report, nor for the performance of the foundations or pavements if not engaged to also provide construction observation and materials testing for this project. The PSI geotechnical engineer of record must also be engaged by the Design Team, even if periodic on-call testing is contracted with PSI Construction Services.

5.1 INITIAL SITE PREPARATION CONSIDERATIONS

5.1.1 GENERAL

Based on the results of our field exploration, we anticipate site preparation procedures to include the steps listed below. All work should be carried out in accordance with current regulatory criteria. The earthwork, testing, and foundation inspection required herein should be performed under the supervision of PSI personnel.

1. Site preparation for the proposed development should include all unwanted ground cover or other unsuitable materials should be completely removed from the site and properly disposed of.
2. The location of any existing conflicting underground utility lines within the construction area should be established. Provisions should be made to relocate any interfering utility lines within the construction area. Abandoned utilities should be removed or grouted to reduce the possibility of subsurface erosion that could result in future settlement.
3. The cleared exposed subgrade should be densified as specified in **Section 5.1.2.** Densification of the soils should be performed within the proposed development areas plus a five-foot wide perimeter extending beyond the outside edges, where practical. Densification operations should continue until the subgrade soils are firm and unyielding.
4. Any fill required to raise grades should conform to the recommendations in **Section 5.1.4** of the report.
5. The exposed subgrade should be proofrolled. Proof rolling should be performed by traversing the construction areas with a fully loaded dump truck or similar rubber-tired equipment with a minimum weight of 15 tons. Proof rolling operations should be observed by a representative of PSI. Unstable soft or wet soils which are revealed by proof rolling should be further evaluated or removed. If they cannot be adequately densified in place. Following proofrolling, exposed soils should be lightly scarified, and moisture conditioned to bring the surface soils to near optimum

moisture and compacted to at least 98 percent of the standard Proctor maximum dry density. All areas to receive fill or at grade construction should be evaluated by a PSI representative. The evaluation should include density testing of near surface soils and proof rolling.

6. It is mandated by federal regulations that all excavations, whether they be utility trenches, footings/pile caps excavations, be constructed in accordance with the OSHA guidelines. It is our understanding that these regulations are being strictly enforced and if they are not closely followed, the owner and the contractor could be liable for substantial penalties.

5.1.2 IN-SITU DENSIFICATION

Following initial site preparation and clearing activities, in-situ densification with compaction of the resulting subgrade soils should be performed in the proposed development areas plus a five-foot-wide perimeter extending beyond the outside edges of the construction areas, where practical. Densification should be accomplished with a self-propelled vibratory roller which imparts a dynamic force of not less than 20 tons.

To minimize the effects of compaction induced vibrations on adjacent existing structures, the compaction operations should be limited to a distance not closer than 25 feet from existing structures (subject to field adjustment as necessary). The maximum drum roller weight to be used between five to 25 feet from existing structures should be limited to four tons. For distances of less than five feet, a walk behind vibratory sled or roller should be used. Compaction of the bearing surface using this equipment should continue until no further vertical settlement of that surface is visually discernible. Any area of the exposed surface that deflects excessively under the weight of the compaction equipment should be excavated approximately 24 inches and replaced with compacted structural fill.

Density control should be exercised in the upper 12 inches of the compacted subgrade. Soils in this interval should be compacted to at least 95 percent of the Modified Proctor maximum dry density determined per ASTM D-1557. Frequent wetting of the subgrade may be necessary during the rolling operations to prevent drying and loosening of the upper six to 12 inches of soil.

5.1.3 ON-SITE BORROW FILL

We anticipated the site didn't need a fill materiel. However, the material on site appears to be suitable to be used as a fill for the site, in such a case, classification and compaction tests must be run first to determine the quality of the soils. Due to the limited amount that can be obtained from site, imported fill must be used.

5.1.4 STRUCTURAL FILL AND BACKFILL

The existing on-site sandy silt (ML) can be used for structural fill provided it is free of organic or deleterious materials, and required amount is available Fill soils should be tested and approved by PSI prior to import and placement. Imported fill should consist of material satisfactory for structural fill within the building footprint should include clean soil or sand and gravel (GW, GM, SP, SW and SM). The contractor may use CL and ML materials satisfying the requirements and limitations below with the approval of the Geotechnical Engineer of Record. GC and SC material can be used in structural fills, subject to the following limitations:

Maximum Dry Density (per ASTM D698)	≥95 pcf
Liquid Limit	≤50
Plasticity Index	≤30

Highly plastic soils (MH, CH) should not be used as engineered fill. The fill materials should be free from topsoil, organics and rock fragments having a major dimension greater than three inches.

The moisture content of fill soils at the time of placement and compaction should be within two percent of optimum moisture content. We recommend that the structural fill be compacted to a minimum of 95 percent of the Standard Proctor maximum dry density (ASTM D-698). The upper 12 inches beneath pavement and grade slab areas should further compacted to 98 percent of the maximum dry density. It is recommended that the fill be placed in lifts not exceeding eight inches in loose thickness.

Prior to beginning compaction, soil moisture contents may need to be controlled in order to facilitate proper compaction. If additional moisture is necessary to achieve compaction objectives, then water should be applied in such a way that it will not cause erosion or removal of the subgrade soils. Moisture content within the percentage range needed to achieve compaction (typically +/- two percent) is recommended prior to compaction of the natural ground and fill.

Density testing should be performed by a soils technician working under the supervision of a geotechnical engineer to determine the degree of compaction. Areas that do not meet the compaction requirement should be further compacted and retested. The frequency of testing will depend on the area of fill placement and the rate at which the fill is placed. As a guidance we recommend one tests be performed for every 5000 square feet of fill in underfloor areas for everyone to two vertical feet of fill placement (typically every two or three lifts). Testing frequency should be increased in confined areas such as pipe trenches or wall backfills.

A representative of PSI should be retained to provide full time, on-site observation of earthwork and excavation activities. It is important that PSI be retained to observe that the subsurface conditions are as we have discussed herein, and that fill placement is in accordance with our recommendations.

5.1.5 GROUNDWATER CONTROL

Groundwater table was not encountered in our four borings that extend to depth of 20 ft below ground surface and drilled during December 2021. Rainy standing water control may be required for construction excavations at this site for either excavation dewatering or removal of temporarily perched water from a rain event, such water can be controlled by pumping from sumps located in ditches or pits.

5.2 EXCAVATION OBSERVATIONS

It is unknown if a classified or unclassified excavation contract will be sought. For a classified excavation contract, we recommend a definition of rock be based on heavy grading equipment used in the project area on similar large land development projects. The following language has been used on other projects and is provided herein.

Excavations should be observed by a representative of PSI prior to continuing construction activities in those areas. PSI needs to assess the encountered materials and confirm that site conditions are consistent with those discussed in this report. This is especially important to identify the condition and acceptability of the exposed subgrades under foundations and other structures that are sensitive to movement. Soft or loose soil zones encountered at the bottom of the excavations should be removed to the level of competent soils as directed by the Geotechnical Engineer or their representative. Cavities formed as a result of excavation of soft or loose soil zones should be backfilled with compacted select fill or lean concrete.

After opening, excavations should be observed, and concrete should be placed as quickly as possible to avoid exposure to wetting and drying. Surface run-off water should be drained away from the excavations and not be allowed to pond. Excavations left open for an extended period of time (greater than 24 hours) should be protected to reduce evaporation or entry of moisture.

5.3 DRAINAGE CONSIDERATIONS

Water should not be allowed to collect in foundation excavations, on foundation surfaces, or on prepared subgrades within the construction area during or after construction. Proper drainage around grade supported sidewalks and flatwork is important to reduce potential movements. Excavated areas should be sloped toward one corner to facilitate removal of collected rainwater, groundwater, or surface runoff. Providing rapid, positive drainage away from the building reduces moisture variations within the underlying soils and will aid in reducing the magnitude of potential movements.

5.4 EXCAVATIONS AND TRENCHES

Any material which cannot be excavated with a single-tooth ripper drawn by a crawler tractor having a draw bar pull rated at not less than 56,000 pounds (Caterpillar D8K or equivalent) or excavated by a front-end loader with a minimum bucket breakout force of 25,600 pounds (Caterpillar 977 or equivalent) would require blasting excavation methods.

Trench Excavation. Any material which cannot be excavated with a backhoe having a bucket curling force rated at not less than 33,010 pounds (Caterpillar 225B or equivalent), would require blasting excavation methods.

In foundation and slab areas where grade is established by blasting, removal of all blast damaged rock will be required. Slab areas may be backfilled to grade with structural fill after verification by geotechnical evaluation that the damaged blast rock has been satisfactorily removed.

The Occupational Safety and Health Administration (OSHA) Safety and Health Standards (29 CFR Part 1926, Revised October 1989), require that excavations be constructed in accordance with the current OSHA guidelines. Furthermore, the State of FL requires that detailed plans and specifications meeting OSHA standards be prepared for trench and excavation retention systems used during construction. PSI understands that these regulations are being strictly enforced, and if they are not closely followed, the owner and the contractor could be liable for substantial penalties.

The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor's "responsible person", as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, State, and Federal safety regulations.

PSI is providing this information as a service to the client. PSI does not assume responsibility for construction site safety or the contractor's or other parties' compliance with local, State, and Federal safety or other regulations. A trench safety plan was beyond the scope of our services for this project.

6.0 REPORT LIMITATIONS

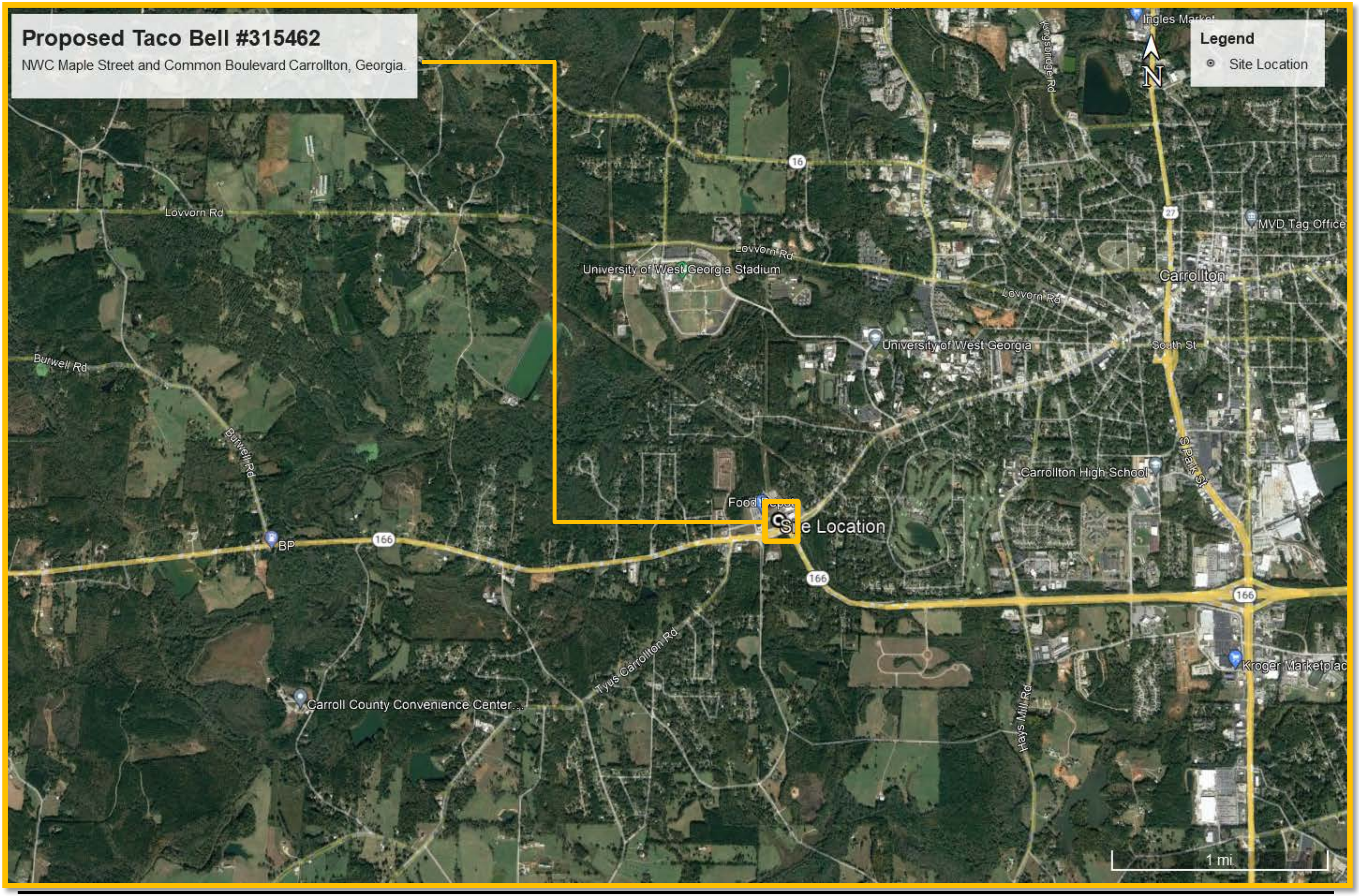
The recommendations submitted in this report are based on the available subsurface information obtained by PSI and design details furnished by Taco Bell Corporation for the proposed project. If there are revisions to the plans for this project, changes of the structural loads or if deviations from the subsurface conditions noted in this report are encountered during construction, PSI should be notified immediately to determine if changes in the foundation recommendations are required or need to be re-evaluated before construction starts. If PSI is not notified of such changes, PSI will not be responsible for the impact of those changes on the project.

The Geotechnical Engineer warrants that the findings, recommendations, specifications, or professional advice contained herein have been made in accordance with generally accepted professional Geotechnical Engineering practices in the local area. No other warranties are implied or expressed. This report may not be copied without the expressed written permission of PSI.

After the plans and specifications are more complete, the Geotechnical Engineer should be retained and provided the opportunity to review the final design plans and specifications to check that the engineering recommendations have been properly incorporated in the design documents. At this time, it may be necessary to submit supplementary recommendations. If PSI is not retained to perform these functions, PSI will not be responsible for the impact of those conditions on the project.

This report has been prepared for the exclusive use of Taco Bell Corporation for specific application to the Proposed new Taco Bel Project to be located at NWC Maple Street and Common Boulevard Carrollton, Georgia.

APPENDIX A



intertek psi

`5801 Benjamin Center Drive
Suite 112 Tampa, Florida
(813) 886-1075

Figure 1
Site Vicinity
PSI Project No.: 0775-3218

Proposed New Taco Bell #315462
NWC Maple Street and Common
Boulevard Carrollton, Georgia

5801 Benjamin Center Drive
Suite 112 Tampa, Florida
(813) 886-1075

Figure 2
Boring Location Plan
PSI Project No.: 07753218

Proposed New Taco Bell #315462
NWC Maple Street and Common
Boulevard Carrollton, Georgia

Appendix B

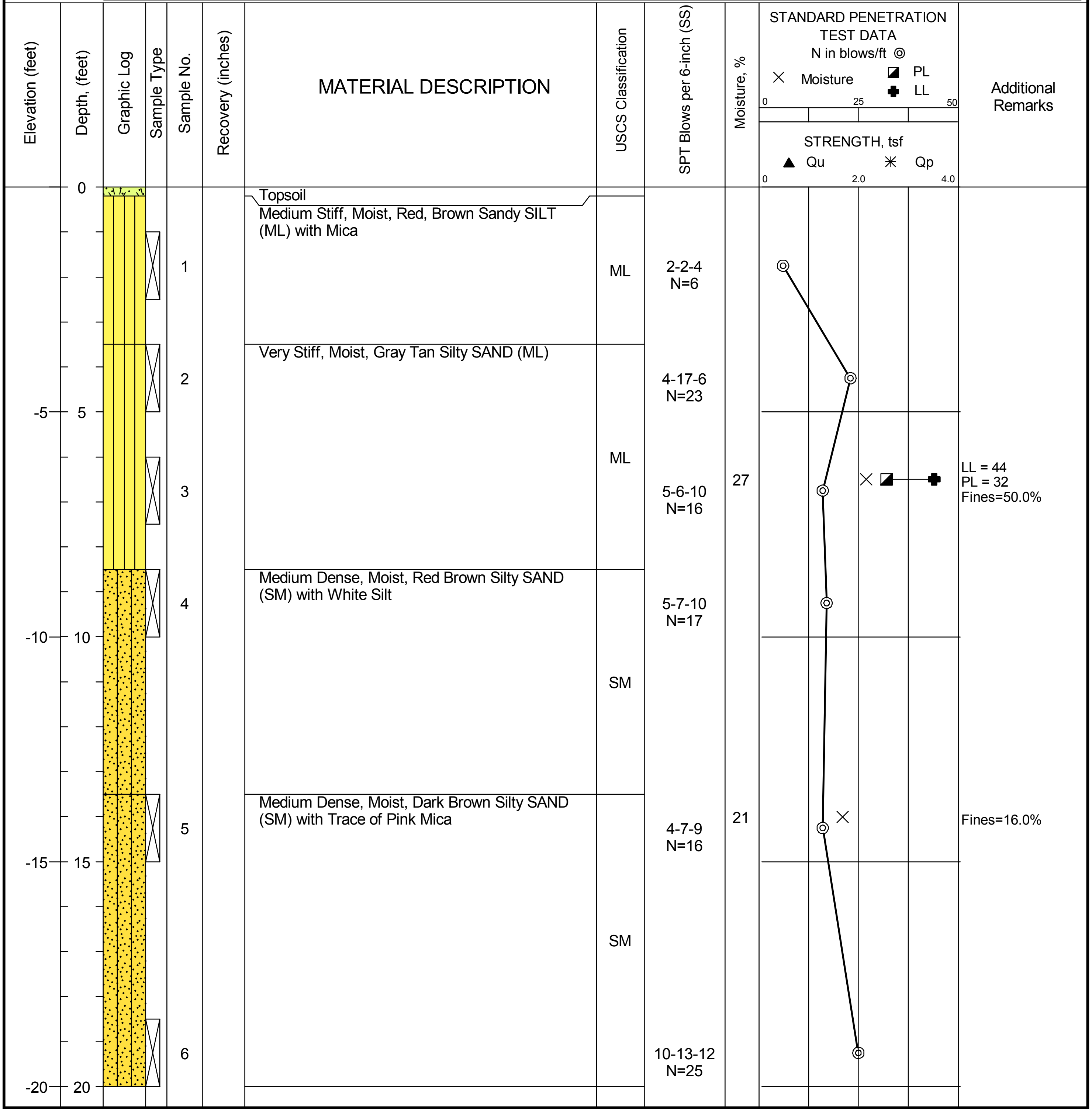
BORING B-1

DATE STARTED: 12/3/21
DATE COMPLETED: 12/3/21
COMPLETION DEPTH 20.0 ft
BENCHMARK: N/A
ELEVATION: N/A
LATITUDE:
LONGITUDE:
STATION: N/A **OFFSET:** N/A
REMARKS:

DRILL COMPANY: Drilling Solutions
DRILLER: **LOGGED BY:** Tony T.
DRILL RIG:
DRILLING METHOD: Mud Rotary
SAMPLING METHOD: SS
HAMMER TYPE: Automatic
EFFICIENCY N/A
REVIEWED BY:

Water		
▽	While Drilling	N.E. feet
▼	Upon Completion	feet
▽	Delay	N/A

BORING LOCATION:



Sample No.	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS)	Moisture, %	Additional Remarks
	Topsoil				
1	Medium Stiff, Moist, Red, Brown Sandy SILT (ML) with Mica	ML	2-2-4 N=6		
2	Very Stiff, Moist, Gray Tan Silty SAND (ML)		4-17-6 N=23		
3		ML	5-6-10 N=16	27	LL = 44 PL = 32 Fines=50.0%
4	Medium Dense, Moist, Red Brown Silty SAND (SM) with White Silt	SM	5-7-10 N=17		
5	Medium Dense, Moist, Dark Brown Silty SAND (SM) with Trace of Pink Mica	SM	4-7-9 N=16	21	Fines=16.0%
6			10-13-12 N=25		

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5801 Benjamin Center Dr, Suite 112
Tampa, FL 33634
Telephone: (813) 886-1075

PROJECT NO.: 0775-3218
PROJECT: Taco Bell #315462
LOCATION: Maple St & Commons Blvd
Carrollton, GA

The stratification lines represent approximate boundaries. The transition may be gradual.

DATE STARTED: 12/3/21
DATE COMPLETED: 12/3/21
COMPLETION DEPTH 20.0 ft
BENCHMARK: N/A
ELEVATION: N/A
LATITUDE:
LONGITUDE:
STATION: N/A **OFFSET:** N/A
REMARKS:

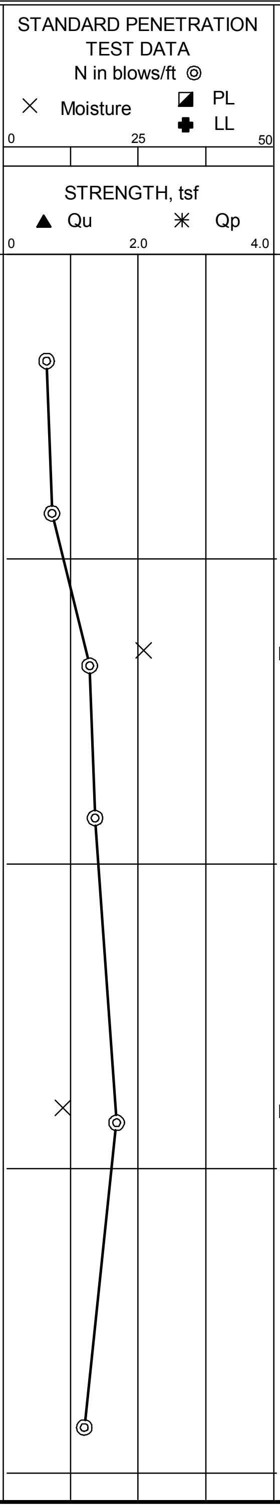
DRILL COMPANY: Drilling Solutions
DRILLER: **LOGGED BY:** Tony T.
DRILL RIG:
DRILLING METHOD: Mud Rotary
SAMPLING METHOD: SS
HAMMER TYPE: Automatic
EFFICIENCY N/A
REVIEWED BY:

BORING B-2

Water		
▽	While Drilling	N.E. feet
▼	Upon Completion	feet
▽	Delay	N/A

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS)	Moisture, %	Additional Remarks
	0			Topsoil				
				Medium Stiff to Stiff , Moist, Red/Brown Sandy SILT (ML) with Trace of Mica	ML			
		1				2-4-4 N=8		
		2				4-4-5 N=9		
-5	5							
				Very Stiff, Moist, Red Brown High Plasticity SILT (MH) with Sand	MH			
		3				4-6-10 N=16	26	Fines=68.0%
		4				4-7-10 N=17		
-10	10							
				Medium Dense, Moist, Red Brown Silty SAND (SM)	SM			
		5				5-7-14 N=21	11	Fines=19.0%
-15	15							
		6				5-7-8 N=15		
-20	20							



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DATE STARTED: 12/3/21
DATE COMPLETED: 12/3/21
COMPLETION DEPTH 20.0 ft
BENCHMARK: N/A
ELEVATION: N/A
LATITUDE:
LONGITUDE:
STATION: N/A **OFFSET:** N/A
REMARKS:

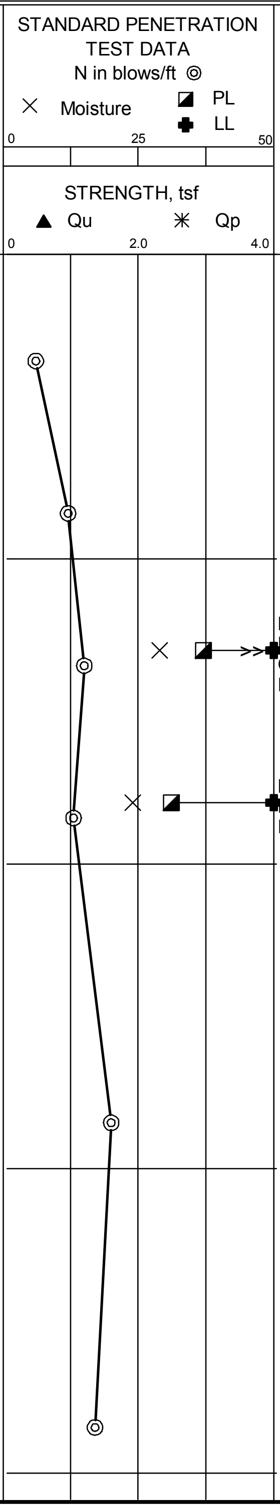
DRILL COMPANY: Drilling Solutions
DRILLER: **LOGGED BY:** Tony T.
DRILL RIG:
DRILLING METHOD: Mud Rotary
SAMPLING METHOD: SS
HAMMER TYPE: Automatic
EFFICIENCY N/A
REVIEWED BY:

BORING B-3

Water
▽ While Drilling N.E. feet
▼ Upon Completion feet
▽ Delay N/A

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS)	Moisture, %	Additional Remarks
	0					Topsoil				
				1		Medium Stiff, Moist, Red, Brown Sandy SILT (ML)	ML	2-3-3 N=6		
				2		Stiff, Moist, Red Gray Sandy SILT (ML)	ML	3-5-7 N=12		
-5	5									
				3		Stiff, Moist, Red High Plasticity SILT (MH) with Sand and Trace of Organic and Roots	MH	5-6-9 N=15	29	LL = 57 PL = 37 Organic=4% Fines=69.0%
				4				4-5-8 N=13	24	LL = 50 PL = 31 Fines=64.0%
-10	10									
				5		Very Stiff, Moist, Red Brown Sandy SILT (ML)	ML	7-11-9 N=20		
-15	15									
				6				6-7-10 N=17		
-20	20									



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PROJECT NO.: 0775-3218
PROJECT: Taco Bell #315462
LOCATION: Maple St & Commons Blvd
Carrollton, GA

The stratification lines represent approximate boundaries. The transition may be gradual.

DATE STARTED: 12/3/21
DATE COMPLETED: 12/3/21
COMPLETION DEPTH 20.0 ft
BENCHMARK: N/A
ELEVATION: N/A
LATITUDE:
LONGITUDE:
STATION: N/A **OFFSET:** N/A
REMARKS:

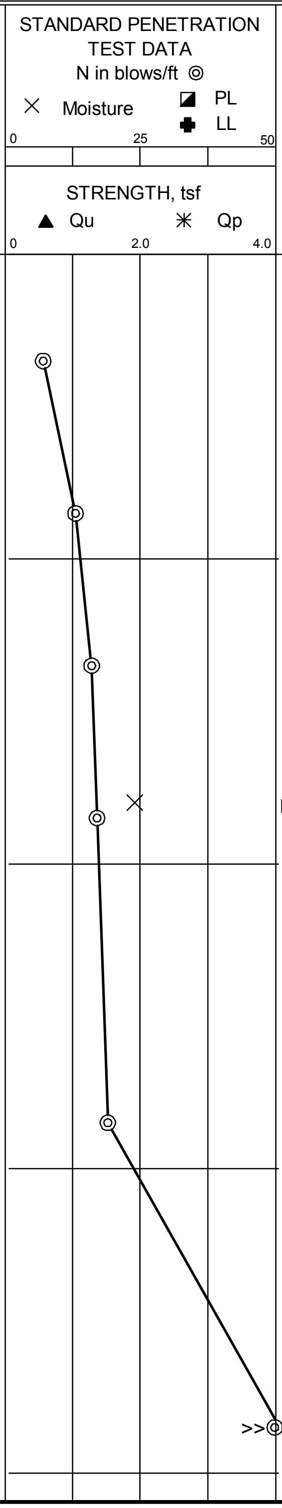
DRILL COMPANY: Drilling Solutions
DRILLER: **LOGGED BY:** Tony T.
DRILL RIG:
DRILLING METHOD: Mud Rotary
SAMPLING METHOD: SS
HAMMER TYPE: Automatic
EFFICIENCY N/A
REVIEWED BY:

BORING B-4

Water
▽ While Drilling N.E. feet
▼ Upon Completion feet
▽ Delay N/A

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS)	Moisture, %	Additional Remarks
	0					Topsoil				
						Stiff, Moist, Red Brown Sandy SILT (ML)				
				1				3-2-5 N=7		
				2			ML	5-6-7 N=13		
-5	5									
				3				6-7-9 N=16		
						Medium Dense, Moist, Red Brown Silty SAND (SM) with Roots				
				4				4-6-11 N=17	24	Fines=38.0%
-10	10									
							SM			
						Very Dense , Moist, Red Brown Sandy SILT (ML)				
				5				7-7-12 N=19		
-15	15									
							SM			
				6				14-26-25 N=51		
-20	20									



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PROJECT NO.: 0775-3218
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LOCATION: Maple St & Commons Blvd
Carrollton, GA

The stratification lines represent approximate boundaries. The transition may be gradual.

BORING B-5

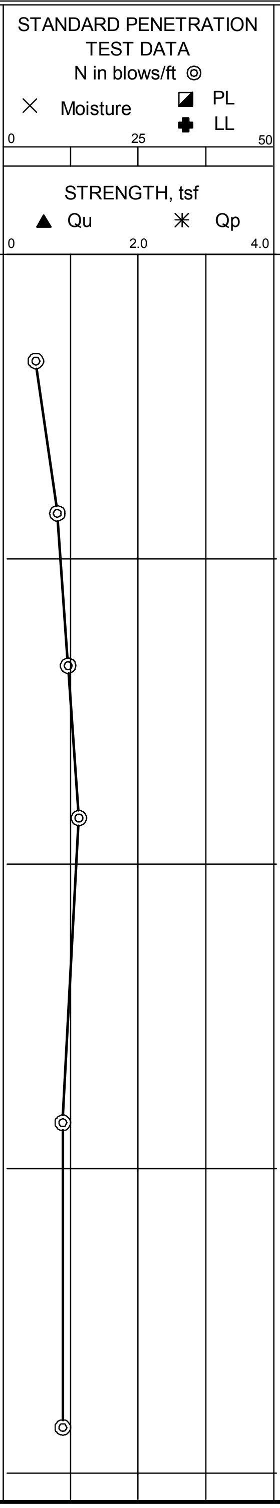
DATE STARTED: 12/3/21
DATE COMPLETED: 12/3/21
COMPLETION DEPTH 20.0 ft
BENCHMARK: N/A
ELEVATION: N/A
LATITUDE:
LONGITUDE:
STATION: N/A **OFFSET:** N/A
REMARKS:

DRILL COMPANY: Drilling Solutions
DRILLER: **LOGGED BY:** Tony T.
DRILL RIG:
DRILLING METHOD: Mud Rotary
SAMPLING METHOD: SS
HAMMER TYPE: Automatic
EFFICIENCY N/A
REVIEWED BY:

Water		
▽	While Drilling	N.E. feet
▼	Upon Completion	feet
▽	Delay	N/A

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS)	Moisture, %	Additional Remarks
	0				Topsoil				
		☒	1		Stiff, Moist, Red Sandy SILT (ML)	ML	3-3-3 N=6		
		☒	2		Stiff, Moist, Red Brown Sandy SILT (ML) with Trace of Rock Fragment		3-5-5 N=10		
-5	5								
		☒	3			ML	3-5-7 N=12		
		☒	4		Stiff, Moist, Red Brown Sandy SILT (ML)		4-7-7 N=14		
-10	10								
		☒	5			ML	4-5-6 N=11		
-15	15								
		☒	6				5-5-6 N=11		
-20	20								



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DATE STARTED: 12/3/21
DATE COMPLETED: 12/3/21
COMPLETION DEPTH 20.0 ft
BENCHMARK: N/A
ELEVATION: N/A
LATITUDE:
LONGITUDE:
STATION: N/A **OFFSET:** N/A
REMARKS:

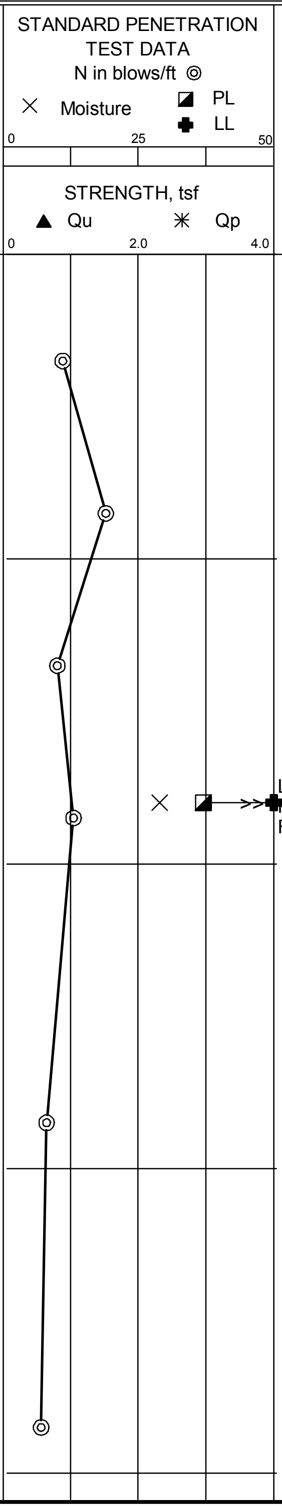
DRILL COMPANY: Drilling Solutions
DRILLER: **LOGGED BY:** Tony T.
DRILL RIG:
DRILLING METHOD: Mud Rotary
SAMPLING METHOD: SS
HAMMER TYPE: Automatic
EFFICIENCY N/A
REVIEWED BY:

BORING B-6

Water		
∇	While Drilling	N.E. feet
▼	Upon Completion	feet
∇	Delay	N/A

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS)	Moisture, %	Additional Remarks
	0				Topsoil				
					Stiff, Moist, Red Brown Sandy SILT (ML)	ML			
		☒	1				3-5-6 N=11		
		☒	2				6-8-11 N=19		
-5	5								
					Stiff, Moist, Red, Brown High Plasticity SILT (MH) with Sand	MH			
		☒	3				4-6-4 N=10		
		☒	4				3-5-8 N=13	29	LL = 54 PL = 37 Fines=60.0%
-10	10								
					Stiff, Moist, Brown Sandy SILT (ML)	ML			
		☒	5				3-4-4 N=8		
-15	15								
		☒	6				3-3-4 N=7		
-20	20								



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The stratification lines represent approximate boundaries. The transition may be gradual.

KEY TO TERMS AND SYMBOLS USED ON LOGS

ROCK CLASSIFICATION

RECOVERY

DESCRIPTION OF RECOVERY	% CORE RECOVERY
Incompetent	< 40
Competent	40 TO 70
Fairly Continuous	70 TO 90
Continuous	90 TO 100

ROCK QUALITY DESIGNATION (RQD)

DESCRIPTION OF ROCK QUALITY	RQD
Very Poor (VPo)	0 TO 25
Poor (Po)	25 TO 50
Fair (F)	50 TO 75
Good (Gd)	75 TO 90
Excellent (ExInt)	90 TO 100

CONSISTENCY OF COHESIVE SOILS

CONSISTENCY	N-VALUE (Blows/Foot)	SHEAR STRENGTH (tsf)	HAND PEN VALUE (tsf)
Very Soft	0 TO 2	0 TO 0.125	0 TO 0.25
Soft	2 TO 4	0.125 TO 0.25	0.25 TO 0.5
Firm	4 TO 8	0.25 TO 0.5	0.5 TO 1.0
Stiff	8 TO 15	0.5 TO 1.0	1.0 TO 2.0
Very Stiff	15 TO 30	1.0 TO 2.0	2.0 TO 4.0
Hard	>30	>2.0 OR 2.0+	>4.0 OR 4.0+

SOIL DENSITY OR CONSISTENCY

DENSITY (GRANULAR)	CONSISTENCY (COHESIVE)	THD (BLOWS/FT)	FIELD IDENTIFICATION
Very Loose (VLo)	Very Soft (VSo)	0 TO 8	Core (height twice diameter) sags under own weight
Loose (Lo)	Soft (So)	8 TO 20	Core can be pinched or imprinted easily with finger
Slightly Compact (SICmpt)	Stiff (St)	20 TO 40	Core can be imprinted with considerable pressure
Compact (Cmpt)	Very Stiff (VSt)	40 TO 80	Core can only be imprinted slightly with fingers
Dense (De)	Hard (H)	80 TO 5"/100	Core cannot be imprinted with fingers but can be penetrated with pencil
Very Dense (VDe)	Very Hard (VH)	5"/100 to 0"/100	Core cannot be penetrated with pencil

DEGREE OF PLASTICITY OF COHESIVE SOILS

DEGREE OF PLASTICITY	PLASTICITY INDEX (PI)	SWELL POTENTIAL
None or Slight	0 to 4	None
Low	4 to 20	Low
Medium	20 to 30	Medium
High	30 to 40	High
Very High	>40	Very High

BEDROCK HARDNESS

MORHS' SCALE	CHARACTERISTICS	EXAMPLES	APPROXIMATE THD PEN TEST	
5.5 to 10	Rock will scratch knife	Sandstone, Chert, Schist, Granite, Gneiss, some Limestone	Very Hard (VH)	0" to 2"/100
3 to 5.5	Rock can be scratched with knife blade	Siltstone, Shale, Iron Deposits, most Limestone	Hard (H)	1" to 5"/100
1 to 3	Rock can be scratched with fingernail	Gypsum, Calcite, Evaporites, Chalk, some Shale	Soft (So)	4" to 6"/100

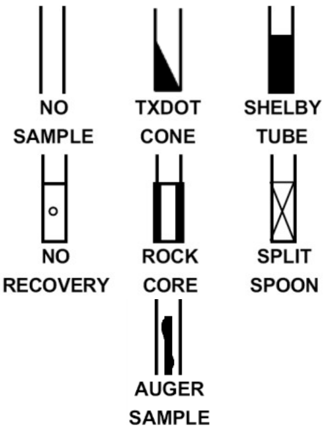
MOISTURE CONDITION OF COHESIVE SOILS

DESCRIPTION	CONDITION
Absence of moisture, dusty, dry to touch	DRY
Damp but no visible water	MOIST
Visible free water	WET

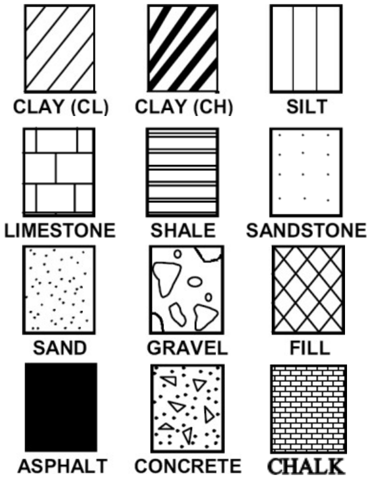
RELATIVE DENSITY FOR GRANULAR SOILS

APPARENT DESNITY	SPT (BLOWS/FT)	CALIFORNIA SAMPLER (BLOWS/FT)	MODIFIED CA. SMAPLER (BLOWS/FT)	RELATIVE DENSITY (%)
Very Loose	0 to 4	0 to 5	0 to 4	0 to 15
Loose	4 to 10	5 to 15	5 to 12	15 to 35
Medium Dense	10 to 30	15 to 40	12 to 35	35 to 65
Dense	30 to 50	40 to 70	35 to 60	65 to 85
Very Dense	>50	>70	>60	85 to 100

SAMPLER TYPES



SOIL TYPES



ABBREVIATIONS

PL – Plastic Limit
LL – Liquid Limit
WC – Percent Moisture

Q_P – Hand Penetrometer
Q_U – Unconfined Compression Test
UU – Unconsolidated Undrained Triaxial

Note: Plot Indicates Shear Strength as Obtained By Above Tests

▽ **WATER SEEPAGE**

▼ **WATER LEVEL AT END OF DRILLING**

CLASSIFICATION OF GRANULAR SOILS

U.S. STANDARD SIEVE SIZE(S)

6" 3" 3/4" 4 10 40 200

BOULDERS	COBBLES	GRAVEL		SAND			SILT OR CLAY	CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE		

152 76.2 19.1 4.76 2.0 0.42 0.074 0.002