REPORT
For Gwinnett County Parks

Geotechnical Exploration Improvements to George Pierce Park Suwanee, Gwinnett County, Georgia
March 1, 2019

Mr. Rex Schuder  
**Gwinnett County Parks**  
75 Langley Drive  
Lawrenceville, GA 30046

Via Email: [Rex.Schuder@gwinnettcountry.com](mailto:Rex.Schuder@gwinnettcountry.com)

RE: Report of Geotechnical Exploration  
**Improvements to George Pierce Park**  
55 Buford Highway NE  
Suwanee, Georgia  
Project No. GCP&R-19-GA-03053-02

Dear Mr. Schuder:

United Consulting is pleased to submit this report of our Geotechnical Exploration for the above-referenced project. The work was completed in accordance with Proposal No. P2019.2455.01 dated February 4, 2019. We appreciate the opportunity to assist you with this project and look forward to our continued participation. Please contact us if you have any questions or if we can be of further assistance.

Sincerely,

**UNITED CONSULTING**

Tiffany Canan  
Project Geologist

Chris L. Roberds, P.G.  
Senior Executive Vice President

TC/TAT/CLR/nj

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1.0 EXECUTIVE SUMMARY

United Consulting has completed a Geotechnical Exploration of the George Pierce Park site located at 55 Buford Highway in Suwanee, Gwinnett County, Georgia. Please refer to the text of the report for a more detailed discussion of the items summarized below.

1. A complete geotechnical engineering service is performed through the observational method as an indivisible two-phase process. The first phase provides advice about project-specific risks and represents our firm's opinion of subsurface conditions with recommendations. Field observation during construction comprises the second phase of our service and provides us the opportunity to assess the reliability of the subsurface data and the appropriateness of our recommendations. Actual conditions may differ from those encountered in the exploration phase.

2. Partially weathered rock (PWR) and refusal were not encountered. Therefore, difficult excavation (ripping and/or blasting) is not anticipated for most of the mass excavation at this site and conventional excavation equipment should be sufficient to grade the site.

3. Groundwater was encountered in the three test locations near the creek. Boring B-1 encountered groundwater at 5 feet 24 hours after drilling, and the hand auger borings encountered groundwater at 7 and 8 feet at time of drilling. Given the anticipated maximum cut depths on the order of five feet, groundwater is not expected to significantly impact mass grading. However, it is possible that groundwater could be encountered in deeper utility excavations and in foundation excavations for the stair bridge. United Consulting can comment further on groundwater impacts on utilities and other deep excavations once the foundation and/or utility plans have been prepared.

4. After the site preparation as recommended, including remediation of the low consistency, near-surface soils, the abutment, restroom and boardwalk structures can be supported on shallow foundation systems. The shallow foundations may consist of shallow footings supported within and underlain by suitable bearing soils, and designed for a maximum bearing pressure of 2,000 pounds per square foot (psf).

5. The extent of the soft soils (B-1, 7 to 8 feet) and shallow groundwater may make the use of helical piers for support of the bridge over the creek a cost-effective alternative to undercutting soft soils or drilling concrete piers into firm soils. We anticipate that helical piers would be extended to depths of 10 to 15 feet in the area.
2.0 PROJECT INFORMATION

The George Pierce Park site is located at 55 Buford Highway in Suwanee, Gwinnett County, Georgia. The site includes the wooded area at the creek between the northwest fields and Lake Leaf View and the wooded region south of Lake Leaf View and west of the parking lot where the George Pierce Park private drive terminates. The Project Site is bound to the north by the Ruby Forest subdivision, to the east, south and west by the remaining portions of George Pierce Park. The park is located in mixed residential and commercial area.

At the time of our exploration the site was generally moderately wooded. There was a grassy field and asphalt parking lot to the east and the park’s access road to the south an existing in the eastern region of the Project Site, which was accessed off of the road. The western area of the Project Site included both sides of the creek where the creek was accessed off of the existing path to the south.

We understand that the construction will include a stair bridge at the creek and an abutment, play area, restroom building and boardwalk with associated driving and parking areas. Conceptual or grading plans with contours were provided by the client. Based on these plans, maximum cuts and fills at the Project Site are expected to be on the order of 5 feet or so.

Structural loads were not provided. However, based on our past experience with similar projects for Gwinnett County Parks, we anticipate the restroom building to have a maximum continuous strip footing load of 1.2 kips per linear foot at the load-bearing walls, and a maximum spread footing load of 1.3 kips at the columns, and the new stair bridge is expected to have maximum loads on the order of 30 kips per bent. If the actual loads or grading schemes vary significantly from those stated above, United Consulting must be contacted to determine if our recommendations should be re-evaluated and/or revised.
3.0 PURPOSE

The purpose of this Geotechnical Exploration was to obtain information regarding soil types, fill availability and suitability, depth to rock and groundwater, and potential foundation types.
4.0 SCOPE

The scope of our preliminary geotechnical exploration included the following items:

1. A visual reconnaissance of the site from a geotechnical standpoint;

2. Drilling five (5) Standard Penetration Tests (SPT) borings and two (2) hand auger borings to determine the quality and consistency of the subsurface soils;

3. Visual evaluation of the soil samples obtained during our field testing program for further identification and classification;

4. Analyzing the existing soil conditions with respect to the proposed construction; and

5. Preparing this report to document the results of our field-testing program, engineering analysis, and to provide our findings and preliminary recommendations.
5.0 SUBSURFACE CONDITIONS

5.1 SPT Borings

A layer of topsoil was initially encountered in the SPT borings. Below the topsoil, residual soils typical of the Piedmont Physiographic Province were encountered. The residual soils encountered generally consisted of very soft to hard silt with varying amounts of sand, clay and mica and occasional traces of rock fragments. N-values within the residual silt ranged from 2 to 34 blows per foot (bpf). Low consistency (N-values ≤ 5 bpf) residual soils were encountered in borings B-1 and B-2 from the ground surface to 8 feet and in borings B-4 and B-5 from the ground surface to 3 feet.

Partially weathered rock (PWR) and or rock was not encountered.

Groundwater was encountered at the time of drilling in boring B-1 at 22 feet at the time of drilling and at 5 feet 24 hours after drilling. Groundwater levels should be anticipated to fluctuate with the change of seasons, during periods of very low or high precipitation, or due to changes in the floodplain or watershed upstream of the site. Soils at this site are also susceptible to development of perched water conditions during period of wet weather.

For a more detailed description of the subsurface conditions encountered, please refer to the boring logs in The Appendix.

5.2 Hand Auger Borings

A layer of topsoil was initially encountered in hand augers HA-1 and HA-2 at the surface to several inches deep. Below the topsoil, residual soils typical of the Piedmont Physiographic Province were encountered in the hand auger borings. The residual soils encountered generally consisted of silt with varying amounts of sand, clay and mica. Dynamic cone penetrometer resistance values in the residual soils ranged from 2 to 21 blows per increment (bpi). Low consistency (bpi ≤ 5) residual soils were encountered in hand auger HA-1 in the top 2 feet. Hand auger refusal was not encountered.

Groundwater was encountered at the time of drilling in hand auger borings HA-1 at 7 feet and HA-2 at 8 feet.

For a more detailed description of the subsurface conditions encountered, please refer to the hand auger boring logs in The Appendix.
6.0 DISCUSSION AND RECOMMENDATIONS

The following recommendations are based on our understanding of the proposed construction, the data obtained in our soil test borings, a site reconnaissance, and our experience with subsurface conditions similar to those encountered at the Project Site.

United Consulting requests the opportunity for a general review of final design documents and specifications in order to verify that earthwork and foundation recommendations have been properly interpreted and implemented in the design and specifications. We recommend that United Consulting, as the Geotechnical Engineer of Record, be consulted during construction to conduct Geotechnical Controls as the Owner’s Representative.

6.1 Existing Fill Consideration

Fill soils were not encountered at the test locations. However, fill may be present in unexplored areas of the site. If encountered, any low-consistency fills or fills containing deleterious materials should be excavated and replaced with suitable soils as needed.

6.2 Site Preparation

The site is moderately wooded and has not generally been graded with the exception of the existing, paved parking lot in the east portion of the site. As such, vegetation, topsoil, and trees including their root mats should be removed from the construction areas. Removal of trees should include removal of their root balls that may extend to several feet below grade. Any deleterious surficial materials should be removed from the construction areas.

Also, any underground utilities that may exist should be relocated to at least 10 feet outside the perimeter of the proposed building footprint. We do not recommend active or non-active utility lines located below the area of the proposed structures be left in place. Existing utility lines should be re-routed outside the proposed structure. The abandoned lines should then be excavated and removed from the area of the proposed construction. All excavations should be subsequently backfilled with properly compacted engineered fill. Any abandoned utility pipes, if left in place and outside of the proposed building footprint, should be filled-in under pressure with cement grout having a minimum 28-day compressive strength of 500 pounds per square inch (psi). This would prevent localized cave-in upon eventual deterioration and loss of structural integrity of the pipe. Also, septic tanks, septic fields, and associated underground structures, if present, should be properly removed. Any excavated trenches and pits associated with the removal of buried structures should be backfilled with engineered fill.

After clearing the site and lowering the site grade where planned and prior to placement of engineered fill or commencement of construction, areas to receive fill, foundations, slabs, and pavements, including the area of the proposed structures, should be proofrolled with a fully loaded tandem-axle dump truck. Proofrolling should be performed under the observation of the Geotechnical Engineer or his representatives so that areas which exhibit “pumping” (wave type displacement) during proofrolling, may be treated by a method recommended by the Geotechnical Engineer. This method may consist of undercutting and backfilling with suitable engineered fill, replacing with surge stone and a layer of crusher run, or some other method that is deemed suitable.
We anticipate that the low consistency, near-surface residual soils encountered in borings B-1, B-2, B-4, and B-5 and in the top two feet of hand auger boring HA-1, if not removed during grading, should be densified during proof rolling.

### 6.3 Difficult Excavation

Partially weathered rock (PWR) and/or rock refusal were not encountered. Therefore, difficult excavation (ripping and/or blasting) is not anticipated for most of the mass excavation at this site and conventional excavation equipment should be sufficient.

### 6.4 Groundwater Considerations

Groundwater was encountered at the time of drilling in boring B-1 at a depth of 22 feet at the time of drilling and at 5 feet 24 hours after drilling. Hand auger borings HA-1 and HA-2 encountered groundwater at 7 and 8 feet, respectively. Given the anticipated maximum cut depths on the order of five feet, groundwater is not expected to significantly impact mass grading. However, it is possible that groundwater could be encountered in foundation excavations for the stair bridge, and deeper utility excavations. United Consulting can comment further on groundwater impacts on utilities and other deep excavations once a utility plan has been prepared.

Shallower depths to groundwater should be anticipated near the creek and drainage gullies. Also, the site is susceptible to the formation of perched water conditions. Perched water is surface water that infiltrates the higher permeability sandy soils and is trapped above the less permeable silt, PWR, and rock layers. The contractor should be prepared to remove perched water and groundwater as needed.

### 6.5 Slopes

The topography at the site is moderate to steeply sloping terrain. We recommend that where fill is to be placed on existing slopes or gullies greater than 4(H):1(V), the slopes be benched to prevent sliding of the fill mass along the sloping surfaces. This can be achieved by notching the slope face by at least about two feet horizontally with the compactor blade as each lift is compacted. A typical benching detail is provided in The Appendix.

Permanent slopes should be constructed no steeper than 2(H):1(V). Fill slopes of up to 20 feet in total height constructed to 2(H):1(V) should be acceptable for this project, assuming proper benching, and placement and compaction of engineered fill. Fill slopes greater than 20 feet must be evaluated for global stability and should be designed by a licensed Geotechnical Engineer. Slopes higher than 35 feet should be benched. If less than desirable soils, such as topsoil or wet soils are to be wasted on slopes, or if an appropriate level of quality control and compaction testing under the supervision of the geotechnical engineer is not planned during slope construction, 2(H):1(V) slopes will not likely be adequate, and flatter slopes should be considered.
All slopes should be protected from erosion during construction and provided with appropriate permanent vegetation or another cover after construction. Slopes should be protected from concentrated runoff flow by means of berms and drainage ditches to direct runoff around slopes or through concrete channels. The appropriate vegetative cover should consist of fast-growing grasses that will rapidly create a dense root mat over the entire slope. Landscaping consisting of isolated shrubs and pine straw will not provide adequate slope protection.

A minimum building or retaining wall setback (from the nearest edge of foundations) of at least 10 feet from the crest of slopes is recommended. A minimum setback of 5 feet is recommended for pavement and curbs.

6.6 Foundation Design and Construction

After the site preparation as recommended, including undercutting of the low consistency, near-surface soils or using helical piers (see below), the proposed restroom, abutment, boardwalk, and stair bridge structures can be supported on shallow foundation systems. The shallow foundations may consist of shallow strip footings supported within and underlain by suitable bearing soils, and designed for a maximum bearing pressure of 2,000 pounds per square foot (psf).

We recommend minimum footing dimensions of 20 inches for strip footings and 24 inches for square footings. Footings should bear at least 12 inches below lowest adjacent finished grades for frost protection.

The Geotechnical Engineer must evaluate each footing excavation prior to steel reinforcement or concrete placement. Conditions that are observed should be compared to the test boring data and design requirements. If unsuitable bearing materials are encountered, they should be excavated and replaced or otherwise treated as recommended by the Geotechnical Engineer at the time of construction.

Surface water control should be maintained to prevent accumulation of water in footing excavations. Standing water in footing excavations should be removed promptly. Soil softened by the water should be removed, and the Geotechnical Engineer or his representative should re-examine the area.

6.6.1 Helical Piers at Bridge

The extent of the soft soils (B-1, 7 to 8 feet) and shallow groundwater may make the use of helical piers for support of the bridge over the creek a cost-effective alternative to undercutting soft soils or drilling piers into firm soils. Helical piers are installed by rotating helical anchors through the upper overburden material to dense bearing strata. Depending on the manufacturer and the specific pier type, helical piers may be designed for a working capacity of up to 40 tons. High capacity helical piers may have an uplift capacity of up to 35 tons. Lateral resistance is typically provided by installing piers at a batter. We estimate that the piers will be installed to depths of about 8 to 15 feet. The helical pier manufacturer/installer typically provides detailed design and installation criteria.
During installation of the helical piers, detailed records should be maintained by a representative of our firm to verify pier type, location, length, installation conditions and estimated capacity. We request that we be allowed to review the contractor’s proposed equipment and installation procedure prior to mobilization and construction.

6.7 Ground Floor Slabs

A slab-on-grade may be utilized for the proposed timber frame structures. We recommend a subgrade modulus of 125 pounds per cubic inch (pci) be used for slab design. It has been our experience that the floor slab subgrade is often disturbed by weather, foundation and utility line installation, and other construction activities between completion of grading and slab construction. For this reason, our geotechnical engineer should evaluate the subgrade immediately prior to placing the concrete. Areas judged by the geotechnical engineer to be unstable should be densified or undercut and replaced with engineered soil fill compacted to at least 98 percent of its standard Proctor maximum dry density.

6.8 Retaining Walls

The following retaining wall recommendations pertain to cast-in-place building and site retaining walls within the areas explored and are not intended for modular blocks or MSE walls. If modular block walls are planned on the site, United Consulting should be notified because additional evaluation will be required to provide recommendations specific to the planned wall types and locations.

The design of retaining walls must include the determination of the lateral earth pressure that will act on the wall. The lateral earth pressure is a function of the soil properties, surcharge loads behind the wall, and amount of deformation that the wall can undergo. This deformation is basically dependent upon the relative rigidity of the wall system.

The active earth pressure condition develops when the wall moves away from the soil over a sufficient distance, such as for a freestanding cantilever wall. The at-rest condition exists when there is no lateral strain on the soil, such as walls, which are rigidly restrained like a basement or sub-foundation wall. The passive condition occurs when the wall moves into the soil.

The following equivalent fluid pressures are recommended for three earth pressure conditions.

**Table 1 - Lateral Earth Pressures**

<table>
<thead>
<tr>
<th>Earth Pressure Condition</th>
<th>Earth Pressure Coefficient</th>
<th>Recommended Equivalent Fluid Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Active</td>
<td>(K_A = 0.33)</td>
<td>40 psf/foot</td>
</tr>
<tr>
<td>At-Rest</td>
<td>(K_O = 0.50)</td>
<td>60 psf/foot</td>
</tr>
<tr>
<td>Passive</td>
<td>(K_P = 3.00)</td>
<td>360 psf/foot</td>
</tr>
</tbody>
</table>

We note that considerable horizontal deflections are required to mobilize the passive pressure; therefore, the designer should consider a safety factor of 2 to the stated ultimate passive earth pressure in design.
The recommended equivalent fluid pressures are based on an assumed soil density of 120 pcf, an internal friction angle of 30 degrees and cohesion of zero. A coefficient of friction of 0.36 for sliding may be used for the retaining wall design.

The parameters listed above are based on a level properly compacted backfill, no friction at the wall-soil interface, and no surcharge effects. For the design of retaining walls, which could be inundated, the buoyant unit weight of the inundated soil should be used to determine the lateral earth pressure. The hydrostatic pressure based on the maximum ponding elevation should be utilized in the analysis.

Heavy compaction equipment should not be used to compact backfill within 5 feet laterally behind any retaining wall unless the wall is designed for the increased pressure or temporarily braced. Therefore, light compaction equipment may be required in this zone. Retaining wall backfill should be compacted to 95 percent of the Standard Proctor maximum dry density. A permanent drainage system such as a footing drain, or a fabric drain such as Enka drain, Mira drain, etc., is recommended for any retaining walls which are more than 5 feet in height. A typical retaining wall drainage detail is included in The Appendix.

The retaining walls should be designed by a professional engineer familiar with retaining wall design and registered in Georgia. The designer should consider sloping backfill, surcharges and other factors affecting wall loadings. The wall designer should also consider the global stability of the wall.
6.9 Earthwork

Most of the soils encountered on the Project Site are generally expected to be suitable for re-use as engineered fill. The Geotechnical Engineer must evaluate excavated soils to assess their suitability for reuse as engineered fill. Typical restrictions on suitable fill are no organics, plasticity index less than 20, and maximum particle size of four inches, with not more than 30 percent greater than 3/4-inch. These restrictions should also be applied to the imported borrow soils if needed.

Positive drainage should be maintained at all times to prevent saturation of exposed soils in case of sudden rains. Sealing the surface of disturbed soils with a smooth-drum roller will also improve runoff and reduce the potential for construction delays due to undercutting and/or stabilization of saturated soils. The degree of soil stability problems will also be dependent upon the precautions taken by the contractor to help protect these moisture sensitive soils.

Moisture-density determinations should be performed for each soil type used, to provide data necessary for quality assurance testing. The natural moisture content at the time of compaction should be within moisture content limits, which will allow the required compaction to be obtained.

The fill should be placed in thin lifts and compacted. We recommend that fill be compacted to at least 98% of Standard Proctor (ASTM D 698) maximum dry density within two feet below pavement subgrade or floor slabs and at least 95% of the Standard Proctor maximum dry density elsewhere.

A Geotechnical Engineer should observe grading operations on a full-time basis. In-place density tests taken by that individual will assess the degree of compaction being obtained. The frequency of the testing should be determined by the Geotechnical Engineer.

6.10 Fill Placement

Moisture-density determinations should be performed for each soil type used, to provide data necessary for quality assurance testing. The natural moisture content at the time of compaction should be within moisture content limits, which will allow the required compaction to be obtained. This is generally within three percent of the optimum moisture. The contractor should be prepared to increase or decrease soil water content.

The fill should be placed in thin lifts (not to exceed 8-inch loose thickness) and compacted. We recommend that fill be compacted to at least 98% of Standard Proctor (ASTM D 698) maximum dry density within two feet below pavement subgrade or floor slabs and at least 95% of the Standard Proctor maximum dry density elsewhere.

A Geotechnical Engineer should observe grading operations on a full-time basis. In-place density tests taken by that individual will assess the degree of compaction being obtained. The frequency of the testing should be determined by the Geotechnical Engineer.
7.0 LIMITATIONS

This report is for the exclusive use of Gwinnett County Parks and the designers of the project described herein, and may only be applied to this specific project. Our conclusions and recommendations have been prepared using generally accepted standards of Geotechnical Engineering practice in the State of Georgia. No other warranty is expressed or implied. Our firm is not responsible for conclusions, opinions or recommendations of others.

The right to rely upon this report and the data within may not be assigned without UNITED CONSULTING’S written permission.

The scope of this evaluation was limited to an evaluation of the load-carrying capabilities and stability of the subsoils. Oil, hazardous waste, radioactivity, irritants, pollutants, molds, or other dangerous substance and conditions were not the subject of this study. Their presence and/or absence are not implied or suggested by this report, and should not be inferred.

Our conclusions and recommendations are based upon design information furnished to us, data obtained from the previously described exploration and testing program and our past experience. They do not reflect variations in subsurface conditions that may exist intermediate of our borings, and in unexplored areas of the site. Should such variations become apparent during construction, it will be necessary to re-evaluate our conclusions and recommendations based on “on-site” observations of the conditions.

If the design or location of the project is changed, the recommendations contained herein must be considered invalid, unless our firm reviews the changes and our recommendations are either verified or modified in writing. When the design is complete, we should be given the opportunity to review the foundation plan, grading plan, and applicable portions of the specifications to confirm that they are consistent with the intent of our recommendations.

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APPENDIX

General Notes/Narrative of Drilling Operations
Figure 1 – Boring and Hand Auger Location Plan
Exploration Procedures/Laboratory Procedures
   SPT Boring Logs (5)
   Hand Auger Boring Logs (2)
   Typical Benching Detail
   Typical Retaining Wall Drainage Detail
GENERAL NOTES

The soil classifications noted on the Boring Logs are visual classifications unless otherwise noted. Minor constituents of a soil sample are termed as follows:

<table>
<thead>
<tr>
<th>Trace</th>
<th>0 - 10%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Some</td>
<td>11 - 35%</td>
</tr>
<tr>
<td>Suffix &quot;y&quot; or &quot;ey&quot;</td>
<td>36 - 49%</td>
</tr>
</tbody>
</table>

LEGEND

Split Spoon Sample obtained during Standard Penetration Testing

Relatively Undisturbed Shelby Tube Sample

Groundwater Level at Time of Boring Completion

Groundwater Level at 24 hours (or as noted) after Termination of Boring

w Natural Moisture Content

LL Liquid Limit
PL Plastic Limit Atterberg Limits
PI Plasticity Index
PF Percent Fines (Percent Passing #200 Sieve)

yd Dry Unit Weight (Pounds per Cubic Foot or PCF)
vm Moist or In-Situ Unit Weight (PCF)
ysat Saturated Unit Weight (PCF)
The test borings were made by mechanically advancing helical hollow stem augers into the ground. Samples were collected at regular intervals in each of the borings following established procedures for performing the Standard Penetration Test in accordance with ASTM Specification D 1586. Soil samples were obtained with a standard 1.4” I.D. x 2.0” O.D. split barrel sampler. The sampler is first seated 6” to penetrate any loose cuttings and then driven an additional foot with the blows required of a 140-pound hammer freely falling a distance of 30 inches. The number of blows required to drive the sampler the final foot is designated the “standard penetration resistance.” The driving resistance, known as the “N” value, can be correlated with the relative density of granular soils and the consistency of cohesive deposits.

The following table describes soil consistency and relative densities based on standard penetration resistance values (N) determined by the Standard Penetration Test (SPT).

<table>
<thead>
<tr>
<th>&quot;N&quot;</th>
<th>Clay and Silt</th>
<th>Sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-2</td>
<td>Very Soft</td>
<td>Very Loose</td>
</tr>
<tr>
<td>3-4</td>
<td>Soft</td>
<td>Loose</td>
</tr>
<tr>
<td>5-8</td>
<td>Firm</td>
<td>Firm</td>
</tr>
<tr>
<td>9-15</td>
<td>Stiff</td>
<td>Medium Dense</td>
</tr>
<tr>
<td>16-30</td>
<td>Very Stiff</td>
<td>Dense</td>
</tr>
<tr>
<td>Over 31</td>
<td>Hard</td>
<td>Very Dense</td>
</tr>
</tbody>
</table>
EXPLORATION PROCEDURES

Five (5) Standard Penetration Tests (SPT) borings (designated B-1 through B-5) and two (2) hand auger borings (designated HA-1 and HA-2) were performed at the approximate locations indicated on the attached Boring and Hand Auger Location Plan (Figure 1).

SPT borings were performed in general accordance with ASTM D 1586. Soil samples obtained during testing were visually evaluated by the Project Engineer and classified according to the visual-manual procedure described in ASTM D 2488. A narrative of field operations is included in The Appendix. The boring locations in the field were determined by the Project Engineer by using the client-provided boring locations, Google Earth, and a Trimble Geo7X GPS Unit. Elevations were obtained using Gwinnett County GIS contours and an interpolation method. The test locations and elevations should, therefore, be considered very approximate.
## Boring Log

**Contracted With:** Gwinnett County Parks  
**Project Name:** Improvements to George Pierce Park  
**Job No.:** GCP&R-19-GA-03053-02  
**Driller:** Big Dog  
**Rig:** Diedrich D-50  
**Logged By:** JJ  
**Date:** 2/18/19

<table>
<thead>
<tr>
<th>Elev. (ft)</th>
<th>Description</th>
<th>Depth in Feet</th>
<th>No.</th>
<th>Type</th>
<th>Blows/6&quot;</th>
<th>Recovery W</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>970</td>
<td>Leaves and topsoil</td>
<td>0</td>
<td>1</td>
<td>1-1-1</td>
<td>14</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Silt - trace clay, mica and rock fragments; very soft; red-brown (Residual)</td>
<td></td>
<td>2</td>
<td>2-2-1</td>
<td>8</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>- soft; dark brown</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>965</td>
<td>- very stiff; tan</td>
<td>10</td>
<td>3</td>
<td>2-6-18</td>
<td>18</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>- trace sand</td>
<td>15</td>
<td>4</td>
<td>5-13-15</td>
<td>16</td>
<td></td>
<td></td>
</tr>
<tr>
<td>955</td>
<td>- hard</td>
<td>20</td>
<td>5</td>
<td>4-6-16</td>
<td>16</td>
<td></td>
<td></td>
</tr>
<tr>
<td>950</td>
<td>Boring terminated at 25 feet</td>
<td>25</td>
<td>6</td>
<td>4-9-25</td>
<td>18</td>
<td></td>
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**Notes:**
- Automatic hammer used with an energy transfer ratio of 86 to 98%.
- Groundwater was encountered at 5 feet 24 hours after drilling.
- Groundwater was encountered at 22 feet at time of drilling.
<table>
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<tr>
<th>ELEV.</th>
<th>DESCRIPTION</th>
<th>DEPTH in FEET</th>
<th>SAMPLES</th>
<th>NO. TYPE</th>
<th>BLOWS/&quot;&quot;</th>
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<th>NOTES</th>
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<tbody>
<tr>
<td></td>
<td>Leaves and topsoil</td>
<td>0</td>
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<td>16</td>
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<td></td>
<td>Silt - trace clay and mica; soft; red-brown (Residual)</td>
<td>5</td>
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<td>2</td>
<td>2-2-2</td>
<td>12</td>
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<tr>
<td></td>
<td>- very stiff</td>
<td>10</td>
<td></td>
<td>3</td>
<td>3-7-11</td>
<td>14</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- firm</td>
<td>15</td>
<td></td>
<td>4</td>
<td>2-2-6</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- red-tan</td>
<td>20</td>
<td></td>
<td>5</td>
<td>2-3-5</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Boring terminated at 25 feet</td>
<td>25</td>
<td></td>
<td>6</td>
<td>3-3-3</td>
<td>18</td>
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</tr>
</tbody>
</table>

Groundwater was not encountered at the time of drilling.

Automatic hammer used with an energy transfer ratio of 86 to 98%.
## BORING LOG

**CONTRACTED WITH:** Gwinnett County Parks  
**PROJECT NAME:** Improvements to George Pierce Park  
**JOB NO.:** GCP&R-19-GA-03053-02  
**DRILLER:** Big Dog  
**RIG:** Diedrich D-50  
**LOGGED BY:** JJ  
**DATE:** 2/18/19

<table>
<thead>
<tr>
<th>ELEV.</th>
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<th>NOTES</th>
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<td>3-4-5</td>
</tr>
<tr>
<td></td>
<td>- stiff</td>
<td>10</td>
<td>3</td>
<td>3-4-6</td>
</tr>
<tr>
<td>965</td>
<td>- firm</td>
<td>15</td>
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<td>3-5-6</td>
</tr>
<tr>
<td>960</td>
<td>- some mica; tan</td>
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<td>5</td>
<td>2-3-4</td>
</tr>
<tr>
<td>955</td>
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<td>25</td>
<td>6</td>
<td>7-11-14</td>
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<tr>
<td>950</td>
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<td></td>
<td></td>
</tr>
</tbody>
</table>

FFE=Finished Floor Elevation
# Boring Log

**CONTRACTED WITH:** Gwinnett County Parks  
**PROJECT NAME:** Improvements to George Pierce Park  
**JOB NO.:** GCP&R-19-GA-03053-02  
**DRILLER:** Big Dog  
**RIG:** Diedrich D-50  
**LOGGED BY:** JJ

<table>
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<tr>
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<td>2</td>
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<td>4-5-14</td>
<td>18</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- trace rock fragments; very stiff</td>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>960</td>
<td>- stiff; tan</td>
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<td>3</td>
<td></td>
<td>3-4-5</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>- some rock fragments; firm</td>
<td>15</td>
<td>4</td>
<td></td>
<td>2-3-5</td>
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<td></td>
</tr>
<tr>
<td>955</td>
<td>- stiff</td>
<td>20</td>
<td>5</td>
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<td>3-7-6</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- trace rock fragments; firm</td>
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<td>6</td>
<td></td>
<td>2-3-4</td>
<td>18</td>
<td>Groundwater was not encountered at the time of drilling</td>
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Boring terminated at 25 feet
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<th>SAMPLES</th>
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<tr>
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<td>3-4-6</td>
<td>3</td>
<td>16</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>- stiff; red-tan</td>
<td>15</td>
<td>3-5-6</td>
<td>4</td>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
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<td></td>
<td>20</td>
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<td>- very stiff</td>
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<td>DESCRIPTION</td>
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<td>NOTES</td>
<td></td>
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<td>-------</td>
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<td>2 2 2</td>
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</tr>
<tr>
<td></td>
<td>- some sand; dark brown-red</td>
<td>4</td>
<td>3 8 7</td>
<td></td>
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<td></td>
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</tr>
<tr>
<td></td>
<td>- tan</td>
<td>6</td>
<td>4 4 3</td>
<td></td>
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<td></td>
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<td>5 3 4</td>
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<td>10</td>
<td>6 4 6</td>
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</table>

Groundwater encountered at 7 feet
<table>
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<th>PENETROMETER TESTS</th>
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<td>970</td>
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<td>5</td>
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<tr>
<td>968</td>
<td>- some sand; dark brown-red</td>
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<td>4</td>
<td>12</td>
</tr>
<tr>
<td>966</td>
<td>- tan</td>
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<td>4</td>
<td>12</td>
</tr>
<tr>
<td>964</td>
<td>- some clay</td>
<td>12</td>
<td>5</td>
<td>21</td>
</tr>
<tr>
<td>962</td>
<td></td>
<td></td>
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<td>6</td>
</tr>
<tr>
<td>960</td>
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</tr>
<tr>
<td>958</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
1. The above diagram illustrates a typical benching for placement of fill on a sloping surface.

2. The diagram shows that before fill is placed, the first step is cut into the slope a maximum distance of about 8 feet (about \( \frac{3}{4} \) the width of usual D-8 bulldozer blade). Successive layers of fill are then placed. Before final layer is placed, the second step is cut 8 feet into the slope and successive layers are again placed.

3. Select fill material should be placed in 8 inch lifts and compacted to the specified density ("B").
AGGREGATE SYSTEM

MIRADRAIN SYSTEM

RETAINING WALL DRAIN DETAIL
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 NotCovered
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.