Report of Subsurface Exploration and Geotechnical Engineering Evaluation

Pine Grove Road Culvert Replacement
Roswell, Georgia

Prepared for AECOM
August 10, 2017

Ms. Katherine McLeod Gurd, P.E.
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Report of Subsurface Exploration
and Geotechnical Engineering Evaluation
Pine Grove Road Culvert Replacement
Roswell, Georgia
Geo-Hydro Project Number 170674.20

Dear Ms. Gurd:

Geo-Hydro Engineers, Inc. has completed the authorized subsurface exploration for the above referenced project. The scope of services for this project was outlined in proposal number 20614.2 dated June 20, 2017.

Project Information

We understand that the City of Roswell is planning to replace the damaged culvert beneath Pine Grove Road just east of its intersection with Old Oak Trace in Roswell, Georgia. The annotated aerial photograph below shows the approximate existing culvert alignment. The existing culvert is an 8’x8’ composite precast concrete and stone masonry structure, and the City has requested design services to replace the existing structure with a bottomless culvert system.

We understand that the final replacement culvert type has not been determined at the time of this report. Options being considered include a reinforced concrete box culvert with dimensions of 12 feet by 10 feet, a ConSpan O-Series culvert, and a ConSpan bottomless aluminum box culvert by Contech. At the time of this report, we have not been provided structural loading information for the new culvert. Once structural loading information is available, please allow us the opportunity to revise our recommendations as necessary.

The existing culvert has an invert elevation of approximately 919, and we have assumed that the invert elevation of the new culvert will be similar.

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Exploratory Procedures

The subsurface exploration consisted of two machine-drilled soil test borings performed at the approximate locations shown on Figure 2 included in the Appendix. The test borings were located in the field by Geo-Hydro by measuring angles and distances from existing site features. Elevations shown on the test boring records were interpolated from the topographic site plan provided to us and have been rounded to the nearest foot. In general, the boring locations and elevations should be considered approximate.

Standard penetration testing, as provided for in ASTM Dl586, was performed at select depth intervals in the machine-drilled soil test borings. Soil samples obtained from the drilling operation were examined and classified in general accordance with ASTM D2488 (Visual-Manual Procedure for Description of Soils). Soil classifications include the use of the Unified Soil Classification System described in ASTM D2487 (Classification of Soils for Engineering Purposes). The soil classifications also include our evaluation of the geologic origin of the soils. Evaluations of geologic origin are based on our experience and interpretation and may be subject to some degree of error.

Descriptions of the soils encountered, groundwater conditions, standard penetration resistances, and other pertinent information are provided in the test boring records and hand auger log included in the Appendix.

Regional Geology

The project site is located in the Northern Piedmont Geologic Province of Georgia. Soils in this area have been formed by the in-place weathering of the underlying crystalline rock, which accounts for their classification as "residual" soils. Residual soils near the ground surface that have experienced advanced weathering frequently consist of red brown clayey silt (ML) or silty clay (CL). The thickness of this surficial clayey zone may range up to roughly 6 feet. For various reasons, such as erosion or local variation of mineralization, the upper clayey zone is not always present.

With increased depth, the soil becomes less weathered, coarser grained, and the structural character of the underlying parent rock becomes more evident. These residual soils are typically classified as sandy micaceous silt (ML) or silty micaceous sand (SM). With a further increase in depth, the soils eventually become quite hard and take on an increasing resemblance to the underlying parent rock. When these materials have a standard penetration resistance of l00 blows per foot or greater, they are referred to as partially weathered rock. The transition from soil to partially weathered rock is usually a gradual one, and may occur at a wide range of depths. Lenses or layers of partially weathered rock are not unusual in the soil profile.

Partially weathered rock represents the zone of transition between the soil and the indurated metamorphic rocks from which the soils are derived. The subsurface profile is, in fact, a history of the weathering process that the crystalline rock has undergone. The degree of weathering is most advanced at the ground surface, where fine-grained soil may be present. Conversely, the weathering process is in its early stages immediately above the surface of relatively sound rock, where partially weathered rock may be found.

The thickness of the zone of partially weathered rock and the depth to the rock surface have both been found to vary considerably over relatively short distances. The depth to the rock surface may frequently range from the ground surface to 80 feet or more. The thickness of partially weathered rock, which overlies the rock surface, may vary from only a few inches to as much as 40 feet or more.

Stream valleys in the Piedmont Region may contain alluvial (water-deposited) soils, depending on ground surface topography, stream flow characteristics, and other factors. By nature, alluvial soils can be highly variable depending upon the energy regime at the time of deposition. Coarse materials such as sand or gravel are deposited in higher energy environments, while fine grained materials such as silt and clay are deposited in low energy environments. Alluvial soils may also contain significant organic materials, and are frequently in a loose, saturated condition. In many cases, fine grained alluvial soils will be highly compressible and have relatively low shear strength.

Near surface geologic conditions at the site have been modified by previous roadway construction activities.

Soil Test Boring Summary

Both borings were performed on Pine Grove Road and initially encountered pavement materials consisting of about 6 to 8 inches of asphalt underlain by 3 to 6 inches of graded aggregate base.

Beneath the pavement structure, borings B-1 and B-2 encountered fill materials extending to depths of about 12 and 8 feet, respectively. The fill was classified as silty sand with varying amounts of mica and rock fragments. Standard penetration resistances recorded in the fill ranged from 9 to 30 blows per foot. It is important to note that rock fragments and other hard inclusions will typically amplify standard penetration resistances. Such artificially inflated values should not be considered representative of the consistency of fill materials.

Beneath fill materials, both borings encountered residual soils typical of the Piedmont region. The residual soils were classified as silty sand. Standard penetration resistances in the residual soils ranged from 3 to 27 blows per foot.

Both borings encountered partially weathered rock at a depth of about 18 feet. Partially weathered rock is locally defined as residual material having standard penetration resistance values greater than 100 blows per foot.

Boring B-1 encountered conditions causing auger refusal at a depth of 20 feet. An initial attempt to advance boring B-2 encountered conditions causing auger refusal at a depth of 10 feet. Boring B-2 was offset about 10 feet west and encountered auger refusal at a depth of 20 feet. Auger refusal is the condition that prevents further advancement of the boring using conventional soil drilling techniques. Auger refusal may be indicative of a boulder, a lens or layer of rock, a rock pinnacle, or a larger rock mass.

At the time of drilling, groundwater was encountered in boring B-1 at a depth of about 9 feet. Boring B-2 did not encounter groundwater. The borings were backfilled with soil cuttings after the groundwater check and patched with asphalt. It should be noted that groundwater levels will fluctuate depending on yearly and seasonal rainfall variations, the creek level, and other factors, and may rise in the future.

For more detailed descriptions of subsurface conditions, please refer to the test boring records and hand auger log included in the Appendix.

Soil Test Boring Summary

Boring	Approx. Ground Elevation	Approx. Invert Elevation	Groundwater		Bottom of Fill Materials		Top of PWR		Auger Refusal	
			Depth (feet)	Approx. Elevation	Depth (feet)	Approx. Elevation	Depth (feet)	Approx. Elevation	Depth (feet)	Approx. Elevation
B-1	930	919	9	921	12	918	18	912	20	910
B-2	930	919	NE	-	8	922	18	912	20	910

All Depths and Elevations in this Summary Table are Approximate
NE: Not Encountered
PWR: Partially Weathered Rock

Evaluations and Recommendations

The following evaluations and recommendations are based on the information available on the proposed construction, the data obtained from the test borings, and our experience with soils and subsurface conditions similar to those encountered at this site. Because the test borings represent a statistically small sampling of subsurface conditions, it is possible that conditions may be encountered during supplemental exploration or during construction that are substantially different from those indicated by the test borings. In these instances, adjustments to the design and construction may be necessary.

Geotechnical Considerations

The following geotechnical characteristics of the site should be taken into account for planning and design:

- Borings B-1 and B-2 encountered fill materials extending to depths of about 12 and 8 feet, respectively. Standard penetration resistances recorded in the fill indicate moderate compactive effort at the time of fill placement. We expect the majority of fill materials to be removed during excavation to reach the culvert foundation bearing elevation. However, the depth and quality of fill materials can vary drastically over relatively short distances and poor quality fill may be encountered intermediate of the areas directly explored.

- The test borings indicate generally favorable excavation conditions within about 18 feet of the existing ground surface for larger loaders and track-mounted excavators. Both borings encountered partially weathered rock at an approximate elevation of approximately 912 and materials causing auger refusal at an elevation of about 910. It is important to note that the depth to partially weathered rock and rock can vary drastically over relatively short distances. If encountered, removal of partially weathered rock will require the use of impact hammers.

- At the time of drilling, groundwater was encountered in borings B-1 at a depth of 9 feet. However, we expect that the stabilized groundwater level will be at the approximate creek bottom elevation. The contractor must be prepared to implement temporary dewatering as necessary to advance the work. We expect that a temporary creek diversion in conjunction with direct pumping from excavations and sumps may be sufficient to provide adequate temporary dewatering. However, temporary dewatering is typically a means-and-methods item left to the contractor. We recommend providing a performance specification for dewatering in the construction documents rather than any specific way to accomplish temporary dewatering.

- Based on the results of the test borings and contingent on proper site preparation and thorough evaluation of foundation excavations, it is our opinion that the proposed replacement culvert can be supported using conventional shallow foundations bearing at an elevation of approximately 915. The *Foundation Design* section of this report presents preliminary allowable bearing pressures for a range of structural loading conditions.

The following sections provide recommendations regarding these issues and other geotechnical aspects of the project.

Construction Dewatering

Dewatering should be performed to maintain the groundwater level at least 2 feet below the lowest prevailing excavation depth. We recommend that the project specifications require the use of dewatering as necessary, and dictate the result of the dewatering operation. The contractor may then implement a technique or combination of techniques appropriate for the actual field conditions encountered. The following represents a minimum guide specification for dewatering.

Minimum Guide Specification for Dewatering

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NOTE: The following specifications are for use as a guide for development of actual specifications. The guide is not intended for direct use as a construction specification without modifications to reflect specific project conditions.

****

Control of groundwater shall be accomplished in a manner that will preserve the strength of the foundation soils, will not cause instability of the excavation slopes, and will not result in damage to existing structures. Where necessary to these purposes, the water level shall be lowered in advance of excavation, utilizing trenches, sumps, wells, well points or similar methods. The water level, as measured in piezometers, shall be maintained a minimum of 3 feet below the prevailing excavation level. Open pumping from sumps and ditches, if it results in boils, loss of soil fines, softening of the ground or instability of slopes, will not be permitted. Wells and well points shall be installed with suitable screens and filters so that continuous pumping of soil fines does not occur. The discharge shall be arranged to facilitate collection of samples by the Engineer.

Adapted from Construction Dewatering - A Guide to Theory and Practice, John Wiley and Sons.

Excavation Characteristics

An initial attempt to advance boring B-2 encountered conditions causing auger refusal at a depth of 10 feet. Boring B-1 and the B-2 offset encountered partially weathered rock at a depth of about 18 feet and conditions causing auger refusal at a depth of 20 feet. We expect ripping of partially weathered rock to be impractical for this project due to the size of equipment necessary to operate within the relatively small site. We expect the use of impact hammers to be necessary to remove partially weathered rock where encountered.

For planning purposes, we recommend assuming that blasting will be necessary to remove material below the depth of auger refusal. The depth to auger refusal encountered in boring B-2 and its offset location suggest that the depth to rock will vary drastically at the project site. It would not be unusual for rock or partially weathered rock to occur at higher elevations between or around some of the soil test borings.

For construction bidding and field verification purposes it is common to provide a verifiable definition of rock in the project specifications. The following are typical definitions of mass rock and trench rock:

- Mass Rock: Material that cannot be excavated with a single-tooth ripper drawn by a crawler tractor having a minimum draw bar pull rated at 56,000 pounds (Caterpillar D-8K or equivalent), and occupying an original volume of at least one cubic yard.

- Trench Rock: Material occupying an original volume of at least one-half cubic yard which cannot be excavated with a hydraulic excavator having a minimum flywheel power rating of 123 kW (165 hp); such as a Caterpillar 322C L, John Deere 230C LC, or a Komatsu PC220LC-7; equipped with a short tip radius bucket not wider than 42 inches.

Blasting

In most cases rock excavation is performed by blasting. Standard blasting procedures include drilling through the materials to be blasted to introduce the explosives and covering up the area to be blasted to prevent flying debris. The area to be blasted is typically covered with several feet of soil or a blast mat. Alternatively, the existing soil overburden can be left in place, which in most cases will eliminate the need for a soil cover or a blast mat.

Blasting generates ground vibrations that can be detrimental to adjacent structures. Research by the United States Bureau of Mines and other organizations provides limits for safeguarding adjacent structures during blasting operations. A peak particle velocity of 2 inches per second is generally recognized as a conservative limit, and is the maximum peak particle velocity allowed by the Georgia Blasting Standards Act of 1978.

State and local laws require that precondition surveys of neighboring properties be performed prior to conducting blasting activities. Typical requirements are to conduct a precondition survey of structures and facilities within a 1,000-foot radius of the blast site. Vibration monitoring is also required in all four compass directions at the nearest structure not owned by the developer/owner. Some municipalities have variations of these requirements, and the local requirements should be reviewed prior to beginning blasting activities.

Reuse of Excavated Materials

Based on the results of test borings and our observations, residual soils and existing fill materials appear to be suitable for reuse as structural fill after moisture adjustment. Geo-Hydro should observe the excavation of existing fill materials to evaluate their suitability for reuse.

It is important to establish as part of the construction contract whether soils having elevated moisture content will be considered suitable for reuse. We often find this issue to be a point of contention and a source of delays and change orders. From a technical standpoint, soils with moisture contents wet of optimum as determined by the standard Proctor test (ASTM D698) can be reused provided that the moisture is properly adjusted to within the workable range. From a practical standpoint, wet soils can be very difficult to dry in small or congested sites and such difficulties should be considered during planning and budgeting. A clear understanding by the general contractor and grading subcontractor regarding the reuse of excavated soils will be important to avoid delays and unexpected cost overruns.

Partially weathered rock materials will be suitable for reuse as structural fill only if they break down into a reasonably well-graded material that can be satisfactorily compacted. The presence of cobble size or boulder size material, which does not break down under the action of compaction equipment, will limit the suitability of partially weathered rock materials. Engineering judgment will be required in the field to evaluate the acceptability of partially weathered rock materials for reuse as structural fill.

For planning purposes, blast rock should be considered unsuitable for reuse as structural fill.

Structural Fill

We recommend that materials selected for use as structural fill be free of organic debris, waste construction debris, and other deleterious materials. The material should not contain rocks having a diameter over 4 inches. It is our opinion that the following soils represented by their USCS group symbols will typically be suitable for use as structural fill and are usually found in abundance in the Piedmont: (SM), (ML), and (CL). The following soil types are typically suitable but are not abundant in the Piedmont: (SW), (SP), (SC), (SP-SM), and (SP-SC). The following soil types are considered unsuitable: (MH), (CH), (OL), (OH), and (Pt). Special or more stringent requirements regarding the nature and gradation of fill and backfill materials may prevail if a Contech structure is selected.

Laboratory Proctor compaction tests and classification tests should be performed on representative samples obtained from the proposed borrow material to provide data necessary to determine acceptability and for quality control. The moisture content of suitable borrow soils should generally be no more than 3 percentage points below or above optimum at the time of compaction. Tighter moisture limits may be necessary with certain soils.

It is possible that highly micaceous soils could be utilized as structural fill material. The use of such materials will require very close attention to quality control of moisture content and density. Additionally, it is our experience that highly micaceous soils tend to rut under rubber-tired vehicle traffic. Continuous maintenance of areas subjected to construction traffic is typically required until construction is completed.

Suitable fill material should be placed in thin lifts. Lift thickness depends on the type of compaction equipment, but a maximum loose-lift thickness of 8 inches is generally recommended. The soil should be compacted by a self-propelled sheepsfoot roller. Within small excavations such as in utility trenches, around manholes, above foundations, or behind retaining walls, we recommend the use of “wacker packers” or “Rammax” compactors to achieve the specified compaction. Loose lift thicknesses of 4 to 6 inches are recommended in small area fills.

We recommend that structural fill be compacted to at least 95 percent of the standard Proctor maximum dry density (ASTM D698). The upper 12 inches of pavement subgrades should be compacted in accordance with Georgia DOT requirements to at least 100 percent of the standard Proctor maximum dry density (ASTM D698). Additionally, the maximum dry density of structural fill should be no less than 90 pcf. Geo-Hydro should perform density tests during fill placement.

Earth Slopes

Temporary construction slopes should be designed in strict compliance with OSHA regulations. The exploratory borings indicate that most soils at the site are Type C as defined in 29 CFR 1926 Subpart P (1994 Edition). This dictates that temporary construction slopes be no steeper than 1.5H:1V for excavation depths of 20 feet or less. Temporary construction slopes should be closely observed on a daily basis by the contractor’s “competent person” for signs of mass movement: tension cracks near the crest, bulging at the toe of the slope, etc. The responsibility for excavation safety and stability of construction slopes should lie solely with the contractor.

We recommend that extreme caution be observed in trench excavations. Several cases of loss of life due to trench collapses in Georgia point out the lack of attention given to excavation safety on some projects. We recommend that applicable local and federal regulations regarding temporary slopes, and shoring and bracing of trench excavations be closely followed.

Earth Pressure – Cast-In-Place Walls

Three earth pressure conditions are generally considered for retaining wall design: "at rest", "active", and "passive" stress conditions. Retaining walls which are rigidly restrained at the top and will be essentially unable to rotate under the action of earth pressure (such as basement or foundation walls) should be designed for "at rest" conditions. Retaining walls which can move outward at the top as much as 0.5 percent of the wall height (such as free-standing walls) should be designed for "active" conditions. For the evaluation of the resistance of soil to lateral loads the "passive" earth pressure must be calculated. It should be noted that full development of passive pressure requires deflections toward the soil mass on the order of l.0 percent to 4.0 percent of total wall height.

Earth pressure may be evaluated using the following equation:

$$p_h = K\,(D_w Z + q_s) + W_w(Z-d)$$

where: p_h = horizontal earth pressure at any depth below the ground surface (Z).
W_w = unit weight of water
Z = depth to any point below the ground surface
d = depth to groundwater surface
D_w = wet unit weight of the soil backfill (depending on borrow sources). The wet unit weight of most residual soils may be expected to range from approximately 115 to 125 pcf. Below the groundwater level, D_w must be the buoyant weight.
q_s = uniform surcharge load (add equivalent uniform surcharge to account for construction equipment loads)
K = earth pressure coefficient as follows:

Earth Pressure Condition	Coefficient
At Rest (K_o)	0.5
Active (K_a)	0.33
Passive (K_p)	3.0

The groundwater term, $W_w(Z-d)$, should be used if no drainage system is incorporated behind retaining walls. If a drainage system is included which will not allow the development of any water pressure behind the wall, then the groundwater term may be omitted. The development of excessive water pressure is a common cause of retaining wall failures. Drainage systems should be carefully designed to ensure that long term permanent drainage is accomplished.

The above design recommendations are based on the following assumptions:

- Horizontal backfill
- 95 percent standard Proctor compactive effort on backfill (ASTM D698)
- No safety factor is included

For convenience, equivalent fluid densities are frequently used for the calculation of lateral earth pressures. For "at rest" stress conditions, an equivalent fluid density of 63 pcf may be used. For the "active" state of stress an equivalent fluid density of 42 pcf may be used. These equivalent fluid densities are based on the assumptions that drainage behind the retaining wall will allow *no* development of hydrostatic pressure; that native sandy silts or silty sands will be used as backfill; that the backfill soils will be compacted to 95 percent of standard Proctor maximum dry density; that backfill will be horizontal; and that no surcharge loads will be applied.

For analysis of sliding resistance of the base of a retaining wall, the coefficient of friction may be taken as 0.4 for the soils at the project site. This is an ultimate value, and an adequate factor of safety should be used in design. The force which resists base sliding is calculated by multiplying the normal force on the base by the coefficient of friction. Full development of the frictional force could require deflection of the base of roughly 0.l to 0.3 inches.

Foundation Design

Based on the results of the test borings, we expect that the proposed replacement culvert will be underlain by residual soil. Typically, the construction of a bottomless culvert in an existing creek channel requires the temporary diversion of the creek and the removal of some of the existing loose materials located at culvert bearing elevation.

Based on our experience with similar projects and the results of the test borings, we expect conventional shallow foundations to be suitable for support of the new culvert. The following table presents recommended allowable bearing pressures based on the maximum structural load for the replacement culvert. If the maximum structural load exceeds the values presented in the table, additional measures will be necessary to reduce settlement of the proposed culvert under the heavier structural loading.

Maximum Vertical Load	Allowable Bearing Pressure
Less than 8 kips per lineal foot	3,000 psf
8 to 12 kips per lineal foot	2,500 psf
12 to 15 kips per lineal foot	2,000 psf

In addition, we recommend a minimum width of 18 inches for culvert wall footings to prevent general bearing capacity failure. Footings should bear at a minimum depth of 18 inches below the prevailing exterior ground surface elevation to help avoid potential problems due to frost heave.

The recommended allowable soil bearing pressure is based on an estimated maximum total foundation settlement no greater than approximately 1 inch. Most of the expected settlement will occur as the culvert is constructed and backfilled. We do not anticipate the need for a waiting period to allow consolidation and settlement to occur prior to paving. If the structural engineer determines that the estimated settlement cannot be accommodated by the proposed structure, please contact us.

Seismic Design

Based on the results of the test borings and following the calculation procedure in the 2012 International Building Code (Chapter 20, ASCE 7-10), the *Site Class* for the site is *C.* The mapped and design spectral response accelerations are as follows: S_S=0.210, S_1=0.095, S_{DS}=0.168, S_{D1}=0.107.

Based on the information obtained from the soil test borings, it is our opinion that the potential for liquefaction of the residual soils at the site due to earthquake activity is relatively low.

* * * * *

We appreciate the opportunity to serve as your geotechnical consultant for this project, and are prepared to provide any additional services you may require. If you have any questions concerning this report or any of our services, please call us.

Sincerely,

GEO-HYDRO ENGINEERS, INC.

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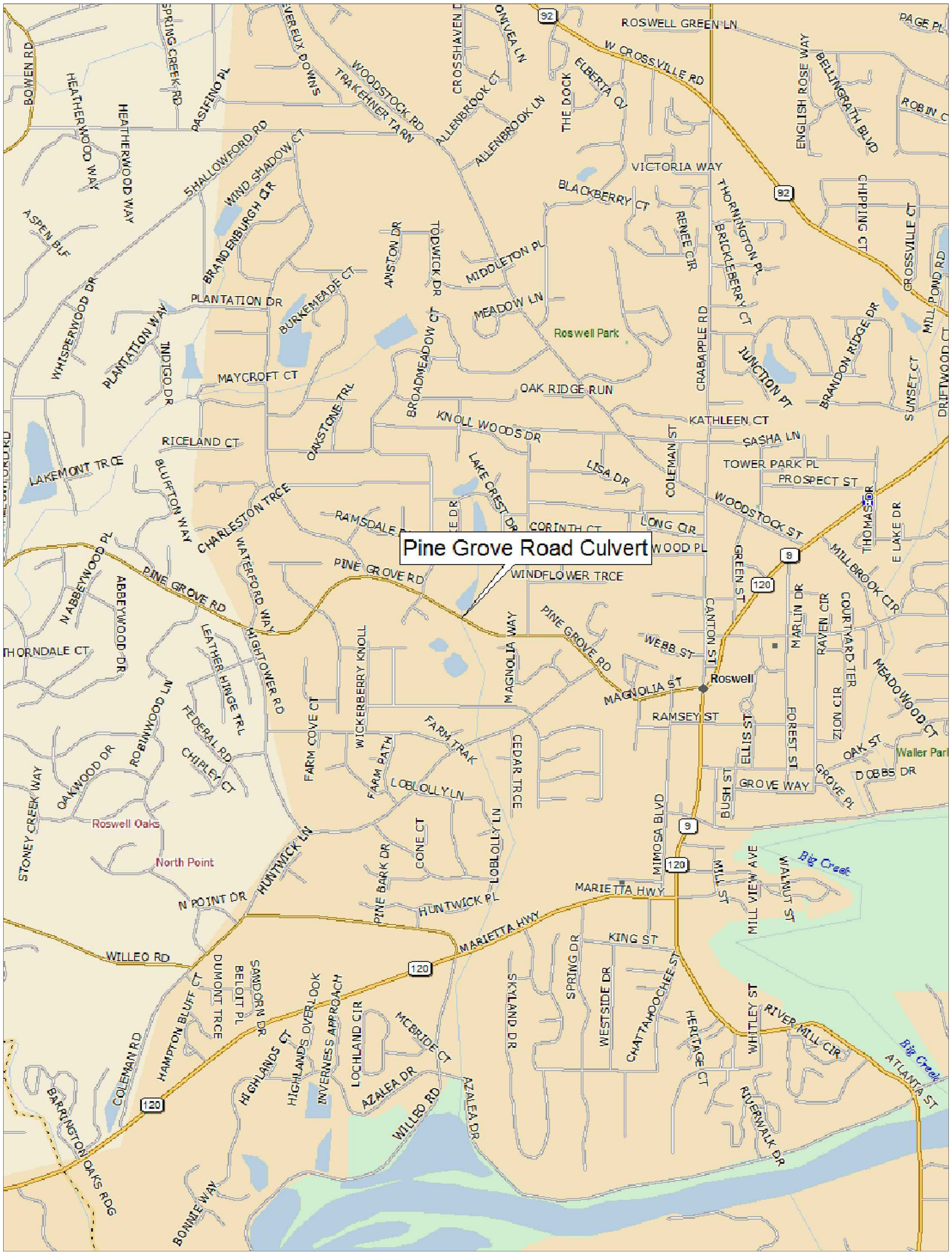
AMP/LEB/170674.2 Pine Grove Culvert Replacement - Geotechnical

APPENDIX

GEO HYDRO ENGINEERS

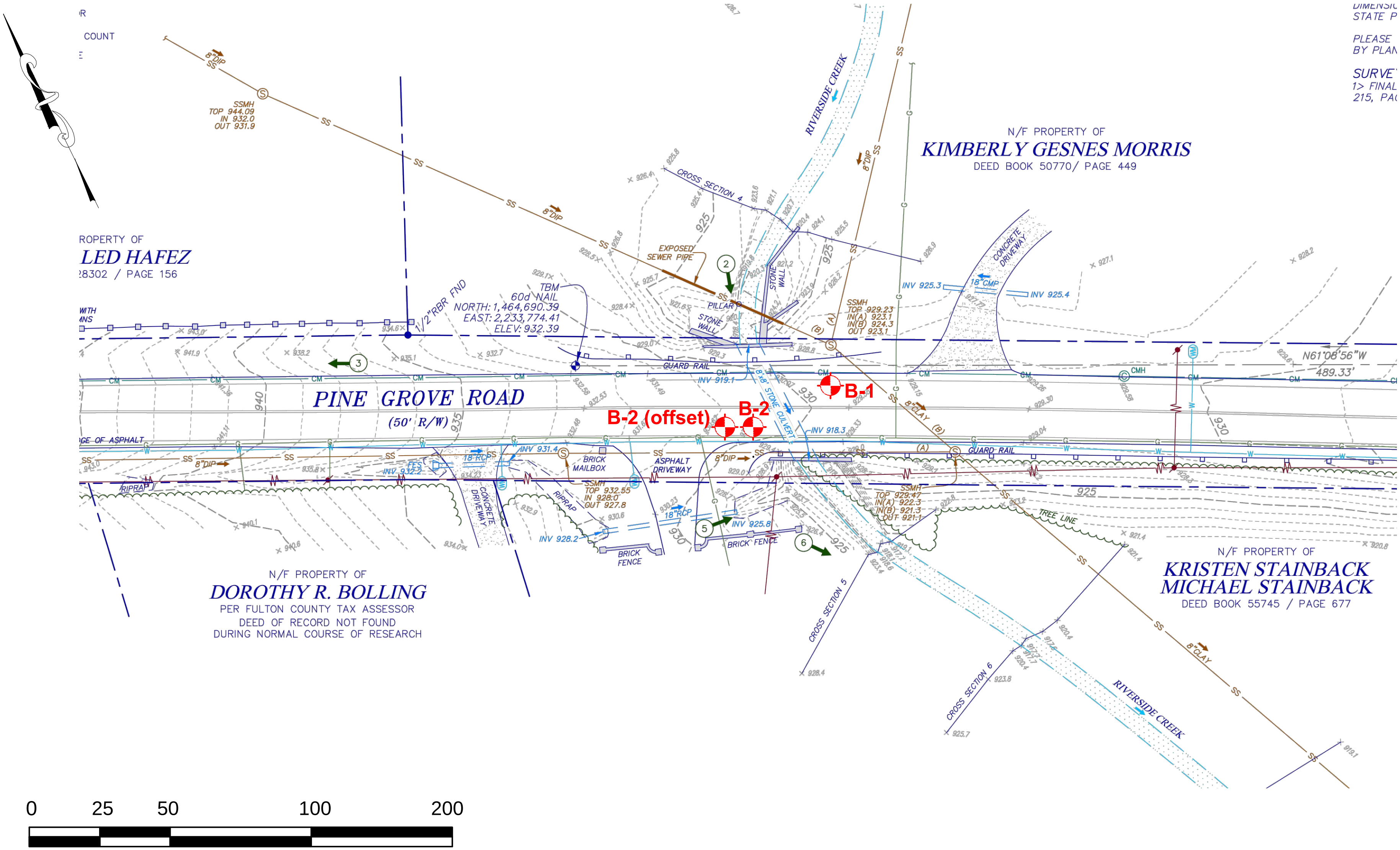


0 0.25 0.5 1 1.5

Approximate Scale, Miles

Figure 1: Site Location Plan	Pine Grove Culvert Replacement Roswell, Georgia Geo-Hydro Project Number 170674.20

GEO HYDRO ENGINEERS



Approximate Scale: 1"=50'

LEGEND: Soil Test Boring

Figure 2: Boring Location Plan

Pine Grove Culvert Replacement
Roswell, Georgia
Geo-Hydro Project Number 170674.20

Symbols and Nomenclature

Symbols

▌	Thin-walled tube (TWT) sample recovered
▯	Thin-walled tube (TWT) sample not recovered
●	Standard penetration resistance (ASTM D1586)
50/2”	Number of blows (50) to drive the split-spoon a number of inches (2)
65%	Percentage of rock core recovered
RQD	Rock quality designation - % of recovered core sample which is 4 or more inches long
GW	Groundwater
▼	Water level at least 24 hours after drilling
▽	Water level one hour or less after drilling
ALLUV	Alluvium
TOP	Topsoil
PM	Pavement Materials
CONC	Concrete
FILL	Fill Material
RES	Residual Soil
PWR	Partially Weathered Rock
SPT	Standard Penetration Testing

Penetration Resistance Results

	Number of Blows, N	Approximate Relative Density
Sands	0-4	very loose
	5-10	loose
	11-20	firm
	21-30	very firm
	31-50	dense
	Over 50	very dense

	Number of Blows, N	Approximate Consistency
Silts and	0-1	very soft
Clays	2-4	soft
	5-8	firm
	9-15	stiff
	16-30	very stiff
	31-50	hard
	Over 50	very hard

Drilling Procedures

Soil sampling and standard penetration testing performed in accordance with ASTM D 1586. The standard penetration resistance is the number of blows of a 140-pound hammer falling 30 inches to drive a 2-inch O.D., 1.4-inch I.D. split-spoon sampler one foot. Rock coring is performed in accordance with ASTM D 2113. Thin-walled tube sampling is performed in accordance with ASTM D 1587.

GEO HYDRO ENGINEERS

B-1

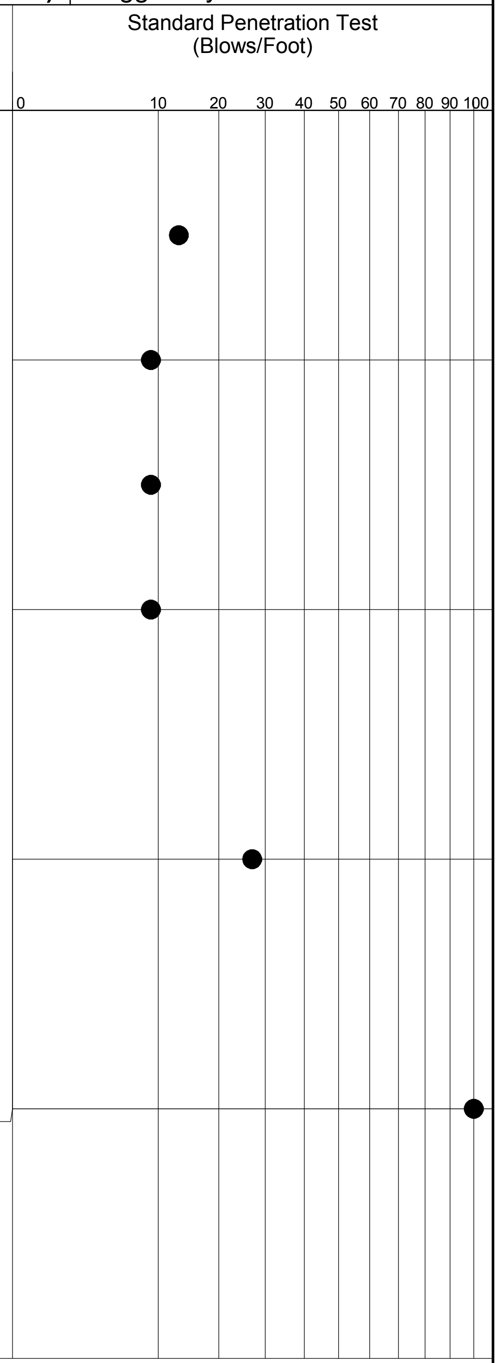
Test Boring Record

GEO HYDRO ENGINEERS

Project: **Pine Grove Road Culvert Replacement**	Project No: **170674.20**
Location: **Roswell, Georgia**	Date: **7/24/17**
Method: **HSA- ASTM D1586** / GWT at Drilling: **9 feet**	G.S. Elev: **930**
Driller: **GCD (Rope& Cathead)** / GWT at 24 hrs: **N/A (Boring Backfilled)**	Logged By: **EM**

Elev. (Ft)	Depth (Ft)	GWT	Description	N
			Asphalt (Approximately 6 inches)	
			Graded Aggregate Base (Approximately 3 inches)	
			Loose to firm brown to tan slightly micaceous silty fine sand (SM) with rock fragments (FILL)	13
925	5			9
				9
		▽		
920	10			9
			Very firm brown silty fine sand (SM) (RESIDUUM)	
915	15			27
			Partially weathered rock - No sample recovered	
910	20			50/1"
			Auger Refusal at 20 feet	
905	25			

Standard Penetration Test (Blows/Foot)



Remarks:

B-2

Test Boring Record

GEO HYDRO ENGINEERS

Project: **Pine Grove Road Culvert Replacement**		Project No: **170674.20**
Location: **Roswell, Georgia**		Date: **7/24/17**
Method: **HSA- ASTM D1586**	GWT at Drilling: **Not Encountered**	G.S. Elev: **930**
Driller: **GCD (Rope& Cathead)**	GWT at 24 hrs: **N/A (Boring Backfilled)**	Logged By: **EM**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N	Standard Penetration Test (Blows/Foot) 0 10 20 30 40 50 60 70 80 90 100
				Asphalt (Approximately 8 inches)		
				Graded Aggregate Base (Approximately 6 inches)		
				Firm brown fine sand (SM) (FILL)	17	
				Tan to gray silty fine sand (SM) with abundant rock fragments (FILL)*		
925	5				30	
					50/2"	
				Very loose silty fine sand (SM) (RESIDUUM)		
920	10				3	
				Firm black silty fine sand (SM)		
915	15				15	
				Partially weathered rock - No sample recovered		
910	20				50/1"	
				Auger Refusal at 20 feet**		
				**Initial boring attempt encountered auger refusal at a depth of 10 feet. The boring was offset 10 feet west of its original location.		
905	25					

Remarks: *Standard penetration resistances not representativedue to rock fragments in the fill