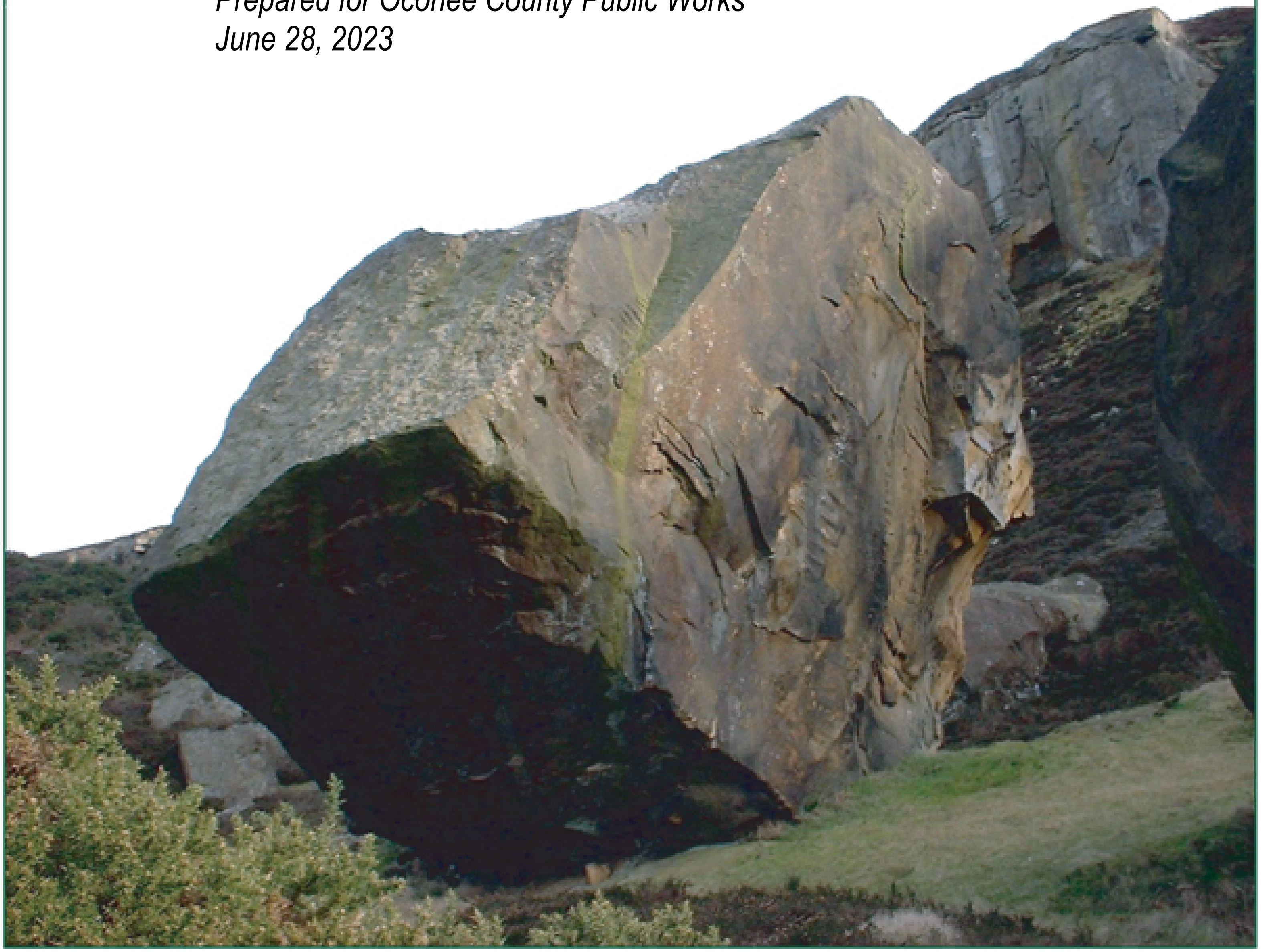
GEO HYDRO ENGINEERS

Report of Subsurface Exploration and Geotechnical Engineering Evaluation

Millers Lake Drive Culvert Replacement
Bogart, Georgia
Geo-Hydro Proposal Number 231601.20

Prepared for Oconee County Public Works
June 28, 2023



Mr. Jody Woodall, P.E.
Oconee County Public Works
1291 Greensboro Highway
Watkinsville, GA 30677

June 28, 2023

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Geo-Hydro Proposal Number 231601.20

Dear Mr. Woodall:

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 231601.P0 dated April 14, 2023.

Project Information

The project involves the replacement of twin CMP pipes which function as a culvert beneath Millers Lake Drive in Bogart, Georgia. Figure 1 in the Appendix shows the approximate site location.

The culvert location is about 150 feet south of the intersection of Millers Lake Drive and Meriweather Drive. We understand that the new culvert will be a precast bottomless structure. The annotated aerial photo below left shows site conditions and the existing culverts. The photo below right shows the condition of the culverts at the time of our exploration.

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 as close to the existing culverts as possible. In general, the locations of the borings should be considered approximate.

Standard penetration testing, as provided for in ASTM D1586, 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 included in the Appendix.

Regional Geology

The project site is located in the Southern 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, which 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 which the crystalline rock has undergone. The degree of weathering is most advanced at the ground surface, where fine grained soil may be present. And 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.

Soil Test Boring Summary

Starting at the ground surface, borings B-1 and B-2 encountered approximately 4 inches of asphalt underlain by approximately 4 and 5 inches of crushed stone base, respectively. Surface material thicknesses at the site should be expected to vary, and measurements necessary for detailed quantity estimation were not performed for this report. For planning purposes, we suggest using a combined surface material thickness of 10 inches.

Beneath the surface materials, borings B-1 and B-2 encountered fill materials classified as silty sand and clayey sand extending to depths of about 6 and 12 feet, respectively. Standard penetration test resistances recorded in the fill ranged from 3 to 6 blows per foot.

Beneath the fill materials, borings B-1 and B-2 encountered alluvial, (water-deposited) soils extending to a depth of about 17 feet. The alluvial soils were classified as clayey sand and silty sand with standard penetration test resistances ranging from 1 to 10 blows per foot.

Beneath the alluvium, both borings encountered residual soils typical of the Piedmont region. The residual soils were classified as silty sand. Standard penetration test resistances recorded in the residual soils ranged from 10 to 24 blows per foot.

Both borings encountered partially weathered rock at a depth of about 32 feet. Partially weathered rock is locally defined as residual material having standard penetration resistance values greater than 100 blows per foot.

Materials causing auger refusal were encountered in borings B-1 and B-2 at depths of 37 and 36 feet, respectively. 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 borings B-1 and B-2 at depths of 12 and 8 feet, respectively. 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 lake 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.

Test Boring Summary

Boring	Bottom of Fill (feet)	Top of Alluvial (feet)	Top of PWR (feet)	Depth to Auger Refusal (feet)	Boring Termination Depth (feet)	Depth to Groundwater at Time of Drilling (feet)
B-1	6	6	32	37	37	12
B-2	12	12	32	36	36	8

All Depths 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 considered for planning and design:

- Fill materials were encountered in borings B-1 and B-2 extending to depths of about 6 and 12 feet, respectively. The standard penetration resistances recorded in the fill suggest little to no compactive effort at the time of fill placement.

- Alluvial (water-deposited) soils were encountered in both test borings extending to a depth of about 17 feet. Alluvial soils are likely present immediately adjacent to the culvert both above and below the lake level. It is likely that stabilization or improvement of soils within the foundation influence zone via excavation and replacement will be required during foundation construction for the new bottomless culvert.

- Both borings encountered partially weathered rock at a depth of about 32 feet. Borings B-1 and B-2 encountered materials causing auger refusal indicative of rock at depths of 37 and 36 feet, respectively. It is important to note that the depth to partially weathered rock and rock can vary drastically over relatively short distances.

- At the time of drilling, groundwater was encountered in borings B-1 and B-2 at depths of 12 and 8 feet, respectively. However, we expect that the stabilized groundwater level will be at the approximate lake elevation. The contractor must be prepared to implement temporary dewatering as necessary to advance the work. For this project, we assume that the normal pool elevation for the lakes will be lowered during construction of the new culvert. We expect that a temporary 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 our experience with similar projects, we expect the foundation loads for the bottomless culvert to be on the order of 15 kips per lineal foot. To maintain foundation settlement within tolerable limits, we recommend planning for excavation and replacement of weak fill and alluvial materials along the culvert foundations. The depth of excavation will depend on the foundation bearing elevation, which is currently unknown, and the width of the foundation. We suggest planning for excavation and replacement extending to a depth equal to the width of the foundation, which based on an allowable bearing pressure of 2,000 psf should be approximately 8 feet.

- Once weak bearing soils are excavated and removed, the resulting excavation should be backfilled with lean concrete or non-excavatable flowable fill to the foundation bearing elevation. A greater depth of excavation and replacement may be required depending on the soils and conditions encountered at the time of construction. The use of crushed stone materials in foundation excavations is not acceptable.

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

Construction Dewatering

At the time of drilling, groundwater was encountered in borings B-1 and B-2 at depths of 12 and 8 feet, respectively. However, we expect that the stabilized groundwater level will be at the approximate lake elevation.

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 for 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 2 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

Both borings encountered partially weathered rock at a depth of about 32 feet. Borings B-1 and B-2 encountered materials causing auger refusal at depths of 37 and 36 feet, respectively. Based on our understanding of the project we do not expect excavations to extend to these depths. It is important to note that the depth to partially weathered rock and rock can vary drastically. It would not be unusual for rock or partially weathered rock to occur at higher elevations between or around 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.

Reuse of Excavated Materials

Based on the results of test borings and our observations, excavated soils may be suitable for reuse as structural fill after moisture content adjustment. Geo-Hydro should observe the excavation of existing fill materials to evaluate their suitability for reuse. Some of the existing fill materials and alluvial soils may not be suitable 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.

Structural Fill – General Grading

Materials selected for use as structural fill should 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).

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.

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 floor slab subgrade soils should be compacted to at least 98 percent of the standard Proctor maximum dry density. 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.

Backfill Over the Culvert

Suppliers of prefabricated or modular culvert structures such as Contech have specific gradation requirements for backfill materials over the culvert structure within specified backfill zones. Based on the results of the test borings, it is unlikely that onsite soils will meet the typical gradation requirements for the backfill zone. For planning and budgeting, we recommend considering that an offsite borrow source or a quarry product such as M10 sand will be required as backfill for granular backfill zones over the culvert.

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. This dictates that temporary excavation slopes must 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.

Formal analysis of slope stability was beyond the scope of work for this project. Based on our experience, permanent cut or fill slopes should be no steeper than 2H:1V to maintain long term stability and to provide ease of maintenance. The crest or toe of cut or fill slopes should be no closer than 10 feet to any foundation or to the edge of any pavement that will support truck traffic. The crest or toe should be no closer than 5 feet to the edge of any pavements supporting cars or light truck traffic or parking. Erosion protection of slopes during construction and during establishment of vegetation should be considered an essential part of construction.

Earth Pressure (cast-in-place structures)

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 loading dock 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 partially saturated 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)	0.53
Active (K_a)	0.36
Passive (K_p)	2.8

The groundwater term, $W_w(Z\text{-}d)$, 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 66 pcf may be used. For the "active" state of stress an equivalent fluid density of 45 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 at least 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 cast-in-place concrete 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 that 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 0.1 to 0.3 inches.

Foundation Design

Based on our experience with similar projects, we expect that foundation loads for a bottomless arch culvert may be as high as 15 kips per lineal foot. To maintain foundation settlement within tolerable limits, we recommend planning for excavation and replacement of weak fill and alluvial materials along the culvert foundations. The depth of excavation will depend on the foundation bearing elevation, which is currently unknown, and the width of the foundation. We suggest planning for excavation and replacement extending to a depth equal to the width of the foundation, which based on an allowable bearing pressure of 2,000 psf should be approximately 8 feet.

Once weak bearing soils are excavated and removed, the resulting excavation should be backfilled with lean concrete or non-excavatable flowable fill to the foundation bearing elevation. A greater depth of excavation and replacement may be required depending on the soils and conditions encountered at the time of construction. The use of crushed stone materials in foundation excavations is not acceptable.

An allowable bearing pressure of 2,000 psf can be used for properly prepared foundation excavations. Using the prescribed excavation and replacement approach, we estimate that total foundation settlement will be about 1 inch, with differential settlement between the two culvert foundation lines not exceeding about ½-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.

Scour Protection

Depending on the flow velocity along the bottomless culvert, it may be prudent to install scour protection Conceptually, scour protection should consist of properly placed riprap or stone gabions. For planning and budgeting, we suggest lining the base of the culvert structure, the base of the wingwalls, and the banks for a distance of 10 feet upstream and downstream of each wingwall using Type 3 rip rap as defined by Georgia DOT (section 805.2.01 of GDOT *Standard Specifications Construction of Transportation Systems* 2021 edition) and underlain by a non-woven, needle-punched filter fabric (Class I – AASHTO M288) such as Mirafi 180N or similar. The need for, footprint, and final design of scour protection for the project should be determined by the culvert designer.

Seismic Design

Based on the results of the test borings and following the calculation procedure in the 2018 International Building Code (Chapter 20, ASCE 7-16), the seismic *Site Class* for the site is *D.* The mapped and design spectral response accelerations are as follows: S_S=0.199, S_1=0.085, S_{DS}=0.213, S_{D1}=0.136.

Based on the information obtained from the soil test borings, it is our opinion that the potential for liquefaction of the 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.

John T. Redding, P.E.
Geotechnical Engineer
jredding@geohydro.com

Luis E. Babler, P.E.
Chief Engineer
luis@geohydro.com

JTR/LEB/231601.20 - Millers Lake Drive Culvert Replacement - Bogart GA leb

APPENDIX

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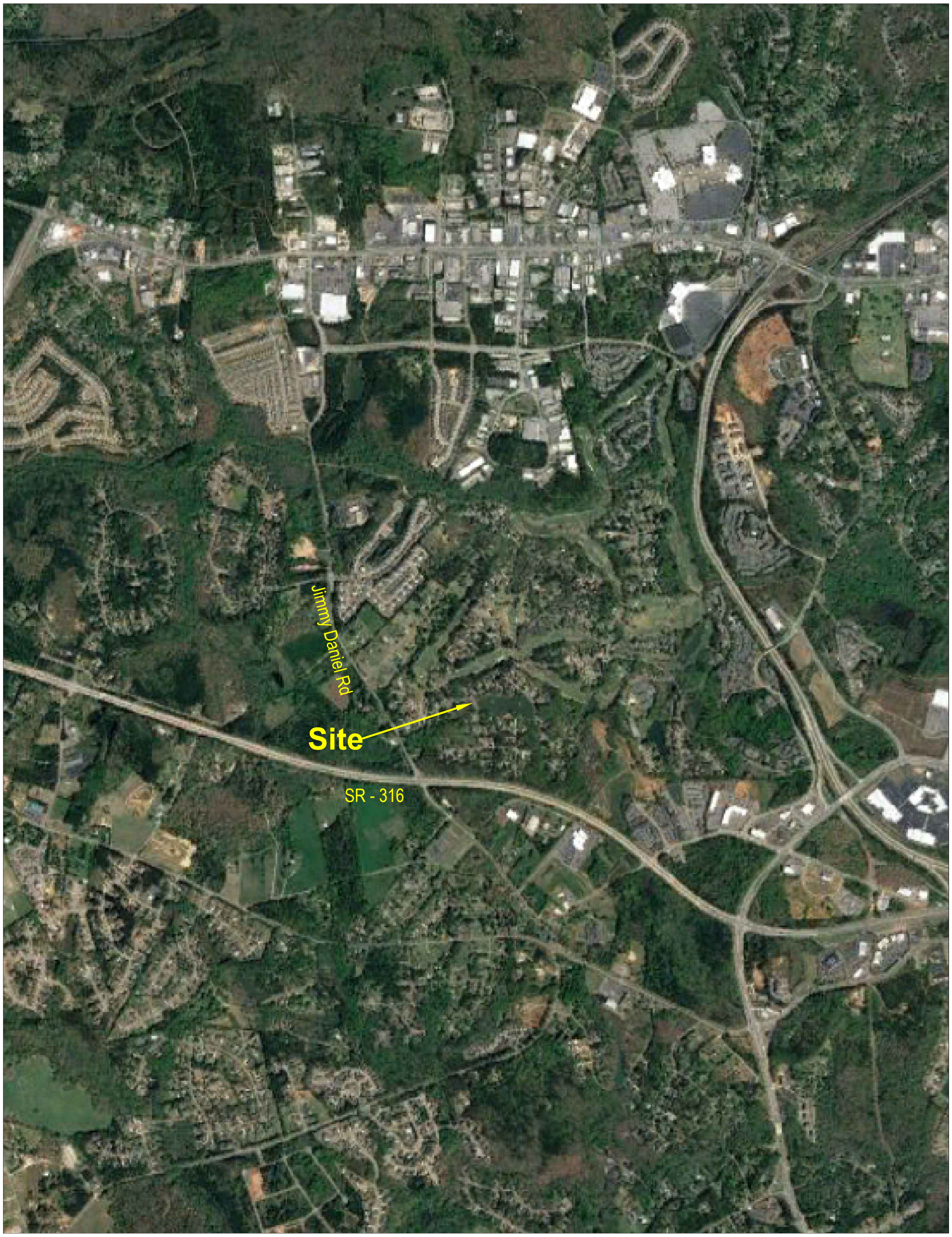


0 0.25 0.5 1 1.5

Approximate Scale, Miles

Figure 1: Site Location Plan	Millers Lake Drive Culvert Replacemnt Bogart, Georgia Geo-Hydro Project Number 231601.20

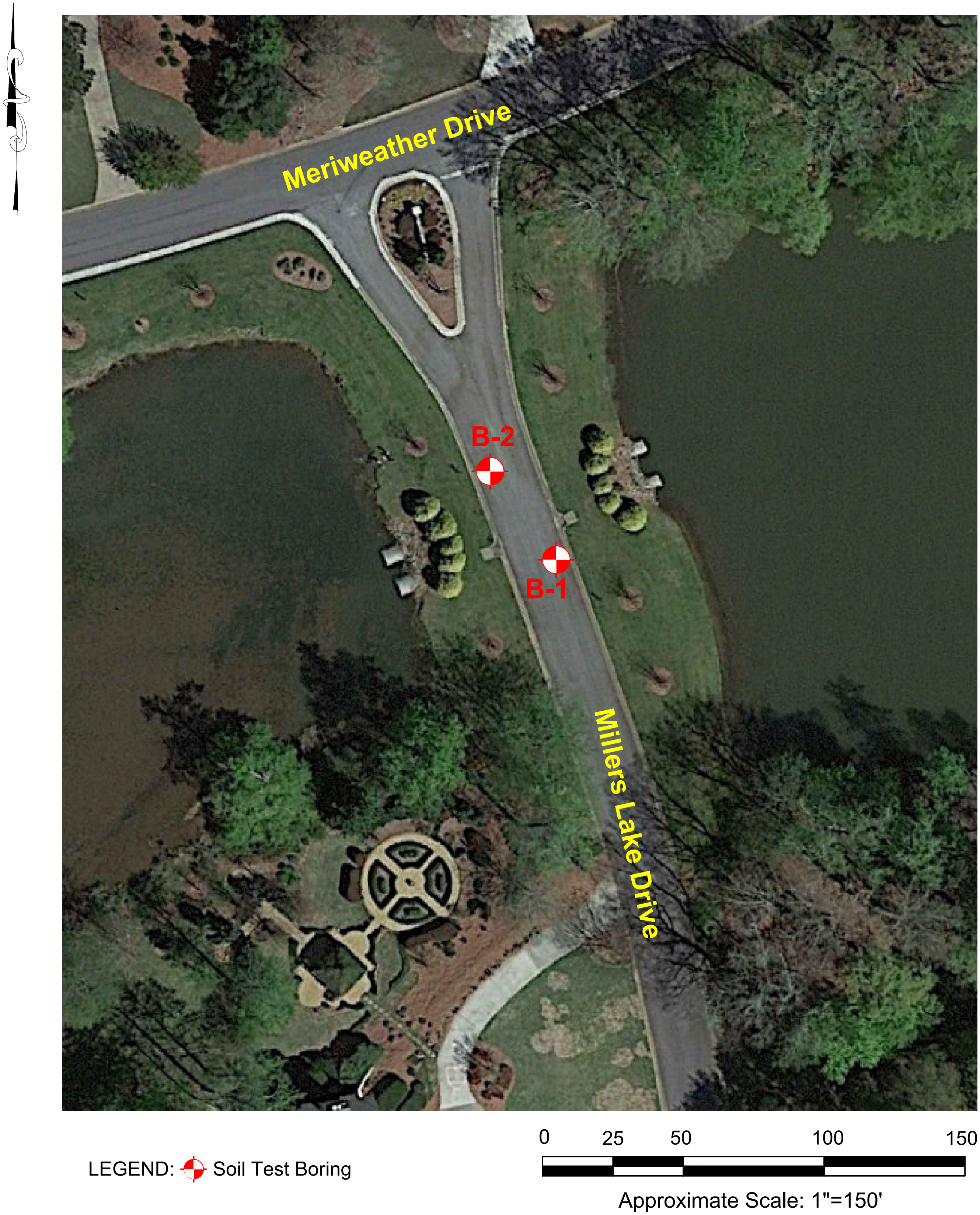


Figure 2: Boring Location Plan

Millers Lake Drive Culvert Replacemnt
Bogart, Georgia
Geo-Hydro Project Number 231601.20

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 Clays	0-1	very soft
	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.

B-1

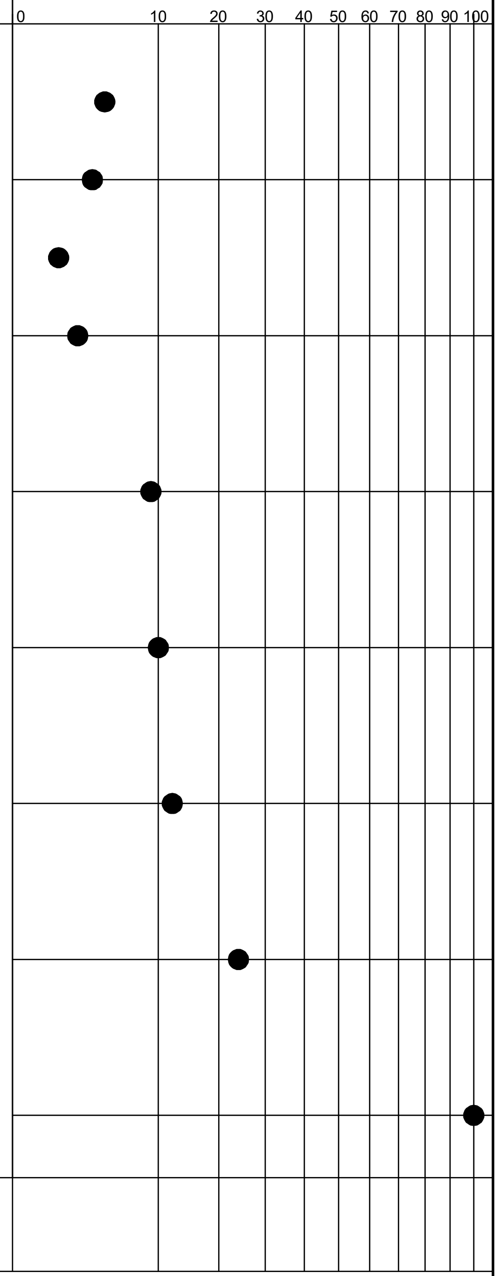
Test Boring Record

GEO HYDRO ENGINEERS

Project: **Millers Lake Drive Culvert Replacement**		Project No: **231601.20**
Location: **Bogart, Georgia**		Date: **6/8/23**
Method: **HSA- ASTM D1586**	GWT at Drilling: **12 feet**	G.S. Elev:
Driller: **GCD (Auto-Hammer)**	GWT at 24 hrs: **N/A (Boring Backfilled)**	Logged By: **AJK**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N
	0			Asphalt (Approximately 4 inches)	
				Crushed Stone Base (Approximately 4 inches)	4
				Very loose orange silty fine sand (SM) (FILL)	
	5			Very loose brown and gray clayey fine sand (SC) (FILL)	3
				Very loose brown and gray clayey fine to medium sand (SM) (ALLUVIAL)	1
	10				2
	12	▽		Loose light gray silty fine to medium sand (SM) (ALLUVIAL)	
	15				9
				Loose to firm orange and gray to brown, gray, and white micaceous silty fine sand (SM) (RESIDUUM)	
	20				10
	25				12
				Very firm gray-brown and white micaceous silty fine sand (SM)	
	30				24
				Partially weathered rock sampled as gray and white silty fine sand (SM)	
	35				50/2"
				Auger Refusal at 37 feet	
	40				

Standard Penetration Test (Blows/Foot)



Remarks:

TEST BORING RECORD LOGS.GPJ GEO HYDRO.GDT 6/28/23

B-2

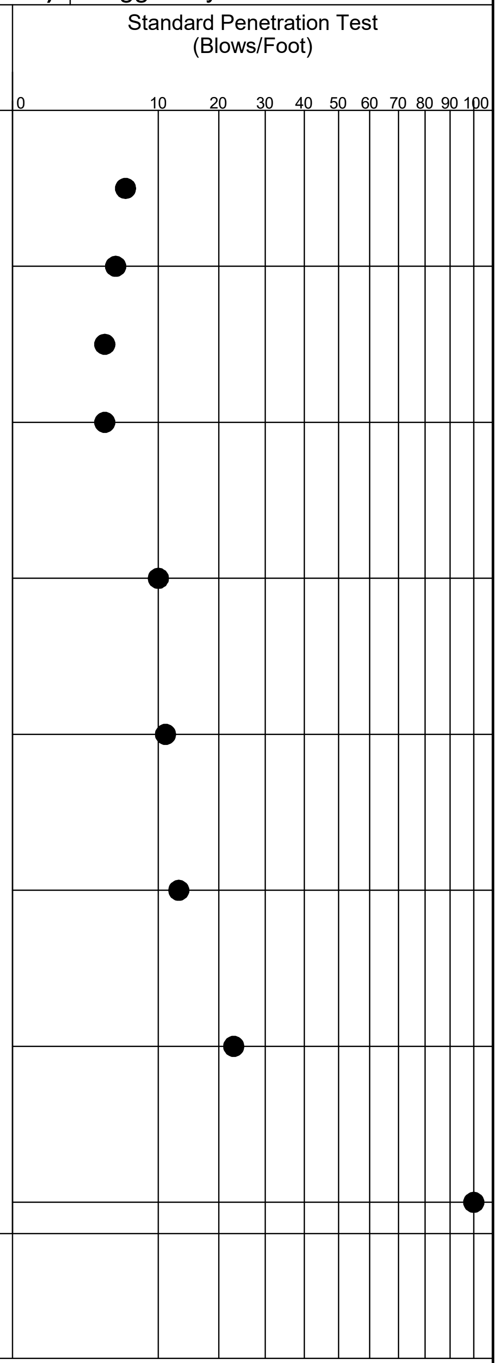
Test Boring Record

GEO HYDRO ENGINEERS

Project: **Millers Lake Drive Culvert Replacement**	Project No: **231601.20**
Location: **Bogart, Georgia**	Date: **6/8/23**
Method: **HSA- ASTM D1586** / GWT at Drilling: **8 feet**	G.S. Elev:
Driller: **GCD (Auto-Hammer)** / GWT at 24 hrs: **N/A (Boring Backfilled)**	Logged By: **AJK**

Elev. (Ft)	Depth (Ft)	GWT	Description	N
			Asphalt (Approximately 4 inches)	
			Crushed Stone Base (Approximately 5 inches)	6
	5		Loose brown silty fine sand (SM) (FILL)	5
		∇	Very loose clayey fine sand (SC) (FILL)	4
	10			4
	15		Loose gray silty fine sand (SM) (ALLUVIAL)	10
	20		Firm to very firm gray and brown micaceous silty fine sand (SM) (RESIDUUM)	11
	25			13
	30			23
	35		Partially weathered rock sampled as gray, brown, and white micaceous silty fine sand (SM)	50/1"
	40		Auger Refusal at 36 feet	

Standard Penetration Test (Blows/Foot)



Remarks: