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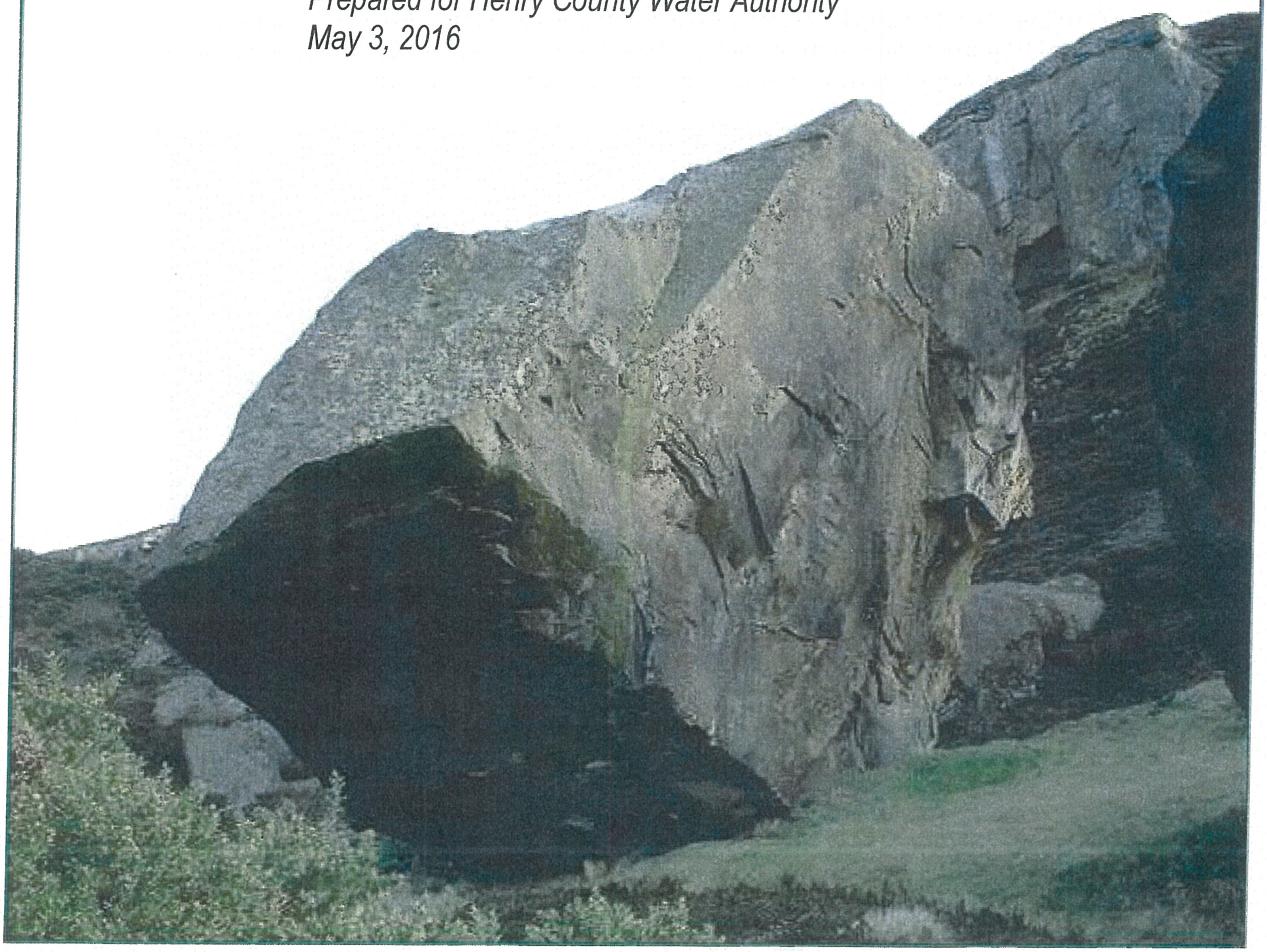
Report of Subsurface Exploration and Geotechnical Engineering Evaluation

Indian Creek Water Reclamation Facility Expansion
Locust Grove, Henry County, Georgia
Geo-Hydro Project Number 160205.20

Prepared for Henry County Water Authority
May 3, 2016



Mr. Patrick S. Hembree
Henry County Water Authority
199 Preservation Drive
Jackson, Georgia 30233

May 3, 2016

Report of Subsurface Exploration and
Geotechnical Engineering Evaluation
Indian Creek Water Reclamation Facility Expansion
Henry County Water Authority
Locust Grove, Henry County, Georgia
Geo-Hydro Project Number 160205.20

Dear Mr. Hembree:

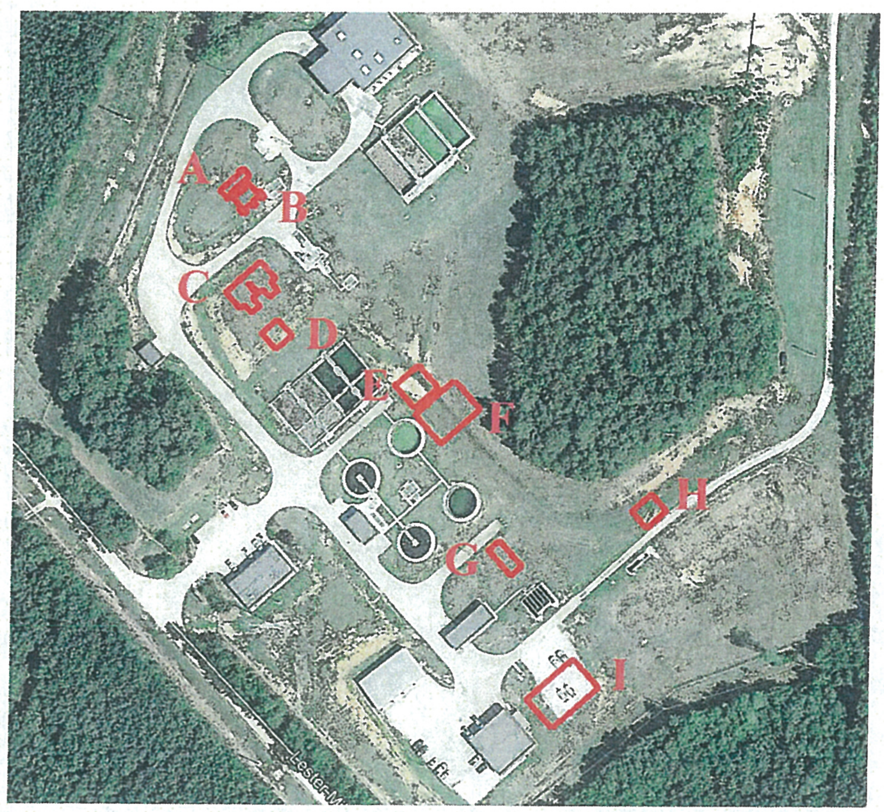
Geo-Hydro Engineers, Inc. has completed the authorized subsurface exploration for the above referenced project. The scope of services for this project was outlined in our proposal number 18863.2 dated March 14, 2016.

Project Information

The project involves the expansion of Henry County's existing Indian Creek Water Reclamation Facility on Lester Mill Road to a total capacity of 3 million gallons per day. The annotated aerial photograph below shows the primary components of the expansion superimposed on the existing plant.

The expansion at the existing facility includes:

- Influent metering flume (marked "A" in the photograph),
- Influent metering screens (B),
- NaOH/Alum Storage (C),
- MBR splitter box (D),
- MBR membrane tank and canopy (E),
- MBR Process Building (F),
- UV structure (G),
- Cascade aerator (H),
- Administration building (I).



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The following table summarizes foundation bearing elevation and structural loading information provided by Engineering Strategies at the time of our report.

Structure	Foundation Type	Existing Ground Elevation	Foundation Bearing Elevation	Unfactored Foundation Service Loads
Influent Metering Flume	Basin structure with mat foundation	805	806	1,000 psf or less
Influent Metering Screens	Basin structure with mat foundation	805	800	1,000 psf or less
NaOH/Alum Structure	Mat foundation	802	799	1,000 psf
MBR Splitter Box	Basin structure with mat foundation	798	793	1,000 psf or less
MBR Membrane Tank and Canopy	Basin structure with mat foundation and canopy column foundations	788	780	Tank: 1,500 psf Canopy Columns: 25 kips
MBR Process Building	Column and wall foundations	788	777	Perimeter Walls: 10½ kips/foot Interior Columns: 170 kips
UV Structure	Basin structure with mat foundation	782	780	1,000 psf or less
Cascade Aerator	Basin structure with sloping mat foundation	776	761	1,500 psf
Administration Building	Slab-on-grade	781	779	Perimeter Walls: 2 kips/foot Interior Columns: 13½ kips

All Depths and Elevations in this Summary Table are Approximate

The expansion also includes the construction of a new discharge pipe extending generally eastward from the plant to a discharge location in the creek immediately south of the Henry County Gardner Reservoir. The red line superimposed on the aerial photograph below indicates the approximate route for the new discharge pipe. Geo-Hydro understands that the pipe invert will generally be about 10 feet below the existing ground surface.

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Exploratory Procedures

Soil Test Borings

The subsurface exploration originally consisted of 16 machine-drilled soil test borings to be performed at the approximate locations shown on Figures 2 and 3 included in the Appendix. During the exploration, boring P-10 was deleted from the investigation to avoid potential conflict with an existing PVC force main pipe. The remaining 15 borings were performed as planned. The borings were located in the field by Geo-Hydro by measuring angles and distances from site reference points. The elevations shown on the test boring records were interpolated from the topographic site plan provided to us and were rounded to the nearest foot. In general, the boring locations and elevation should be considered approximate.

Standard penetration testing, as provided for in ASTM Dl586, was performed at select intervals in the 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 logs included in the Appendix.

Laboratory Testing

Samples of the soil cuttings from borings P-11 and P-15 were obtained and returned to the laboratory for reduction/oxidation testing (ASTM G200), pH testing (ASTM G51), and resistivity testing using the soil box method (ASTM G187).

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.

Overall geologic conditions at the site have been modified by previous development.

Soil Test Boring Summary

Brief summaries of the subsurface conditions encountered in each of the main areas of exploration are presented in the following sections. For more detailed descriptions of subsurface soil conditions, please refer to the Test Boring Records included in the Appendix.

Influent Metering Flume and Influent Metering Screens (Boring W-1)

Starting at the ground surface, boring W-1 encountered approximately 1 inch of topsoil. Beneath the topsoil, the boring encountered residual soil typical of the Piedmont region. The residual soils were generally classified as silty sand and sandy silt with varying mica content. Standard penetration resistances in the residual soils ranged from 10 to 17 blows per foot.

At least 24 hours after drilling, groundwater was encountered in boring W-1 at a depth of 16 feet. The boring was backfilled with soil cuttings after the 24-hour groundwater check. It should be noted that groundwater levels will fluctuate depending on yearly and seasonal rainfall variations and other factors, and may rise in the future.

Summary of Subsurface Conditions for the Influent Metering Flume and Metering Screens

Boring	Existing Ground Elevation	Foundation Bearing Elevation	Foundation Cut/Fill Depths	Top of PWR Elevation	Auger Refusal Elevation	Groundwater Elevation	Boring Termination Elevation
W-1	805	Flume 806	Fill 1 ft.	NE	NE	789	780
		Screens 801	Cut 4 ft.				

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

NaOH/AlUM Storage and MBR Splitter Box (Boring W-2)

Starting at the ground surface, boring W-2 encountered approximately 1 inch of topsoil. Beneath the topsoil, the boring encountered residual soil typical of the Piedmont region. The residual soils were generally classified as silty sand and sandy silt with varying mica content. Standard penetration resistances in the residual soils ranged from 10 to 27 blows per foot.

At least 24 hours after drilling, groundwater was encountered in boring W-2 at a depth of 15 feet. The boring was backfilled with soil cuttings after the 24-hour groundwater check. It should be noted that groundwater levels will fluctuate depending on yearly and seasonal rainfall variations and other factors, and may rise in the future.

Summary of Subsurface Conditions for the NaOH/ALUM Storage and MBR Splitter Box

Boring	Existing Ground Elevation	Foundation Bearing Elevation	Foundation Cut/Fill Depths	Top of PWR Elevation	Auger Refusal Elevation	Groundwater Elevation	Boring Termination Elevation
W-2	Alum 802	ALUM 799	Cut 3 ft.	NE	NE	787	777
	Splitter 798	Splitter 793	Cut 5 ft.				

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

MBR Membrane Tank and Canopy (Borings W-3 and W-4)

Starting at the ground surface, borings W-3 and W-4 encountered approximately 1 inch of topsoil. Beneath the topsoil, boring W-4 encountered fill materials extending to a depth of about 6 feet. The fill material was classified as silty sand. Standard penetration resistances of 11 and 12 blows per foot were recorded in the fill.

Beneath the topsoil or fill materials, boring W-3 and W-4 encountered residual soil typical of the Piedmont region. The residual soils were generally classified as silty sand with varying mica content. Standard penetration resistances in the residual soils ranged from 3 to 56 blows per foot.

A lens of partially weathered rock was encountered in boring W-4 from about 12 to 28 feet beneath the surface. Boring W-4 also encountered partially weathered rock at an approximate depth of 38 feet. Partially weathered rock is locally defined as residual material having a standard penetration resistance greater than 100 blows per foot.

Auger refusal was encountered in boring W-4 at a depth of 47 feet beneath the surface. Auger refusal is the condition that prevents advancement of the boring using conventional soil drilling techniques. The material causing auger refusal may consist of a boulder, a lens or layer of rock, the upper surface of relatively massive rock, or other hard material.

At least 24 hours after drilling, groundwater was encountered in borings W-3 and W-4 at depths of 7 to 8 feet. The borings were backfilled with soil cuttings after the 24-hour groundwater check. It should be noted that groundwater levels will fluctuate depending on yearly and seasonal rainfall variations and other factors, and may rise in the future.

Summary of Subsurface Conditions for the MBR Membrane Tank and Canopy

Boring	Existing Ground Elevation	Foundation Bearing Elevation	Foundation Cut/Fill Depths	Top of PWR Elevation	Auger Refusal Elevation	Groundwater Elevation	Boring Termination Elevation
W-3	788	780	Cut 8 ft.	NE	NE	**782***	763
W-4	790	780	Cut 10 ft.	778	743	**783***	743

All Depths and Elevations in this Summary Table are Approximate
***Bold font** indicates elevations above the foundation mat or footing bearing elevation.
PWR: Partially Weathered Rock
NE: Not Encountered

MBR Process Building (Borings W-3, W-4, and W-5)

Starting at the ground surface, borings W-3 through W-5 encountered approximately 1 inch of topsoil. Beneath the topsoil, borings W-4 and W-5 encountered fill materials extending to depths of about 6 and 3 feet, respectively. The fill material was classified as silty sand. Standard penetration resistances ranging from 11 to 20 blows per foot were recorded in the fill.

Beneath the topsoil or fill materials, all 3 of the borings encountered residual soil typical of the Piedmont region. The residual soils were generally classified as silty sand with varying mica content. Standard penetration resistances in the residual soils ranged from 3 to 56 blows per foot.

A lens of partially weathered rock was encountered in boring W-4 from about 12 to 28 feet beneath the surface. Boring W-4 also encountered partially weathered rock at an approximate depth of 38 feet. Partially weathered rock is locally defined as residual material having a standard penetration resistance greater than 100 blows per foot.

Auger refusal was encountered in boring W-4 at a depth of 47 feet beneath the surface. Auger refusal is the condition that prevents advancement of the boring using conventional soil drilling techniques. The material causing auger refusal may consist of a boulder, a lens or layer of rock, the upper surface of relatively massive rock, or other hard material.

At least 24 hours after drilling, groundwater was encountered in borings W-3 through W-5 at depths of 6 to 8 feet. The borings were backfilled with soil cuttings after the 24-hour groundwater check. It should be noted that groundwater levels will fluctuate depending on yearly and seasonal rainfall variations and other factors, and may rise in the future.

Summary of Subsurface Conditions for the MBR Process Building

Boring	Existing Ground Elevation	Foundation Bearing Elevation	Foundation Cut/Fill Depths	Top of PWR Elevation	Auger Refusal Elevation	Groundwater Elevation	Boring Termination Elevation
W-3	788	777	Cut 11 ft.	NE	NE	**782***	763
W-4	790	777	Cut 13 ft.	**778***	743	**783***	743
W-5	788	777	Cut 11 ft.	NE	NE	**780***	763

All Depths and Elevations in this Summary Table are Approximate
***Bold font** indicates elevations above the foundation mat or footing bearing elevation.
PWR: Partially Weathered Rock
NE: Not Encountered

UV Structure (Boring W-6)

Starting at the ground surface, boring W-6 encountered approximately 1 inch of topsoil. Beneath the topsoil, boring W-6 encountered fill materials extending to a depth of about 3 feet. The fill was classified as silty sand. A standard penetration resistance of 10 blows per foot was recorded in the fill.

Beneath the fill materials, boring W-10 encountered residual soil typical of the Piedmont region. The residual soils were generally classified as silty sand and sandy silt with varying mica content. Standard penetration resistances in the residual soils ranged from 9 to 18 blows per foot.

At least 24 hours after drilling, groundwater was encountered in boring W-6 at a depth of 8 feet. The boring was backfilled with soil cuttings after the 24-hour groundwater check. It should be noted that groundwater levels will fluctuate depending on yearly and seasonal rainfall variations and other factors, and may rise in the future.

Summary of Subsurface Conditions for the UV Structure

Boring	Existing Ground Elevation	Foundation Bearing Elevation	Foundation Cut/Fill Depths	Top of PWR Elevation	Auger Refusal Elevation	Groundwater Elevation	Boring Termination Elevation
W-6	782	780	Cut 2 ft.	NE	NE	774	757

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

Cascade Aerator (Boring W-7)

Starting at the ground surface, boring W-7 encountered approximately 1 inch of topsoil. Beneath the topsoil, boring W-7 encountered residual soil typical of the Piedmont region. The residual soils were generally classified as silty sand with varying mica content. Standard penetration resistances in the residual soils ranged from 8 to 19 blows per foot.

At least 24 hours after drilling, groundwater was encountered in boring W-7 at a depth of 22 feet. The boring was backfilled with soil cuttings after the 24-hour groundwater check. It should be noted that groundwater levels will fluctuate depending on yearly and seasonal rainfall variations and other factors, and may rise in the future.

Summary of Subsurface Conditions for the Cascade Aerator

Boring	Existing Ground Elevation	Foundation Bearing Elevation	Foundation Cut/Fill Depths	Top of PWR Elevation	Auger Refusal Elevation	Groundwater Elevation	Boring Termination Elevation
W-7	776	761	Cut 15 ft.	NE	NE	754	751

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

Administration Building (Borings W-8 and W-9)

Starting at the ground surface, boring W-9 encountered approximately 1 inch of topsoil. Beneath the topsoil or ground surface, borings W-8 and W-9 encountered fill materials extending to depths of about 12 and 6 feet, respectively. The fill material was classified as silty sand. Standard penetration resistances ranging from 6 to 21 blows per foot were recorded in the fill.

Beneath the fill materials, boring W-8 and W-9 encountered residual soil typical of the Piedmont region. The residual soils were generally classified as silty sand and sandy silt with varying mica content. Standard penetration resistances in the residual soils ranged from 8 to 19 blows per foot.

At least 24 hours after drilling, groundwater was encountered in borings W-8 and W-9 at depths of 15 and 13 feet, respectively. The borings were backfilled with soil cuttings after the 24-hour groundwater check. It should be noted that groundwater levels will fluctuate depending on yearly and seasonal rainfall variations and other factors, and may rise in the future.

Summary of Subsurface Conditions for the Administration Building

Boring	Existing Ground Elevation	Foundation Bearing Elevation	Foundation Cut/Fill Depths	Top of PWR Elevation	Auger Refusal Elevation	Groundwater Elevation	Boring Termination Elevation
W-8	781	779	Cut 2 ft.	NE	NE	769	756
W-9	782	779	Cut 3 ft.	NE	NE	769	757

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

New Discharge Pipe (Borings P-11 through P-16)

Starting at the ground surface, borings P-11 through P-16 encountered about 1 to 2 inch of topsoil. The borings were generally located in the dirt "road" running around the perimeter of the reservoir where most of the topsoil has been removed by vehicles. In the wooded areas adjacent to the borings, it would not be unusual for the grading contractor to report an average topsoil depth of 8 to 10 inches following the intermixing of topsoil, leaves, and branches during tree removal.

Beneath the topsoil, borings P-11, P-12, and P-16 encountered fill materials extending to a depth of about 3 feet. The fill material was classified as silty sand and sandy silt. Standard penetration resistances ranging from 5 to 11 blows per foot were recorded in the fill.

Beneath the topsoil or fill materials, boring P-11 through P-16 encountered residual soil typical of the Piedmont region. The residual soils were generally classified as sandy silt and silty sand with varying mica content. Standard penetration resistances in the residual soils ranged from 7 to 37 blows per foot.

Auger refusal was encountered in borings P-13 and P-14 at a depth of 13 feet beneath the surface. Auger refusal is the condition that prevents advancement of the boring using conventional soil drilling techniques. The material causing auger refusal may consist of a boulder, a lens or layer of rock, the upper surface of relatively massive rock, or other hard material.

At least 24 hours after drilling, groundwater was encountered in borings P-11 through P-15 at depths of 5 to 14 feet. These borings were backfilled with soil cuttings after the 24-hour groundwater check. Boring P-16 did not encounter groundwater at the time of drilling and it was backfilled with bentonite chips immediately after drilling. It should be noted that groundwater levels will fluctuate depending on yearly and seasonal rainfall variations and other factors, and may rise in the future.

Summary of Subsurface Conditions for the New Discharge Pipe

Boring	Anticipated Depth to Pipe's Invert	Depth to Top of PWR	Depth to Auger Refusal	Depth to Groundwater	Depth of Boring Termination
P-11	10 feet	NE	NE	**5 feet***	15 feet
P-12	10 feet	NE	NE	**9 feet***	15 fee
P-13	10 feet	NE	13 feet	**8 feet***	13 feet
P-14	10 feet	NE	13 feet	**5 feet***	13 feet
P-15	10 feet	NE	NE	14 feet	15 feet
P-16	10 feet	NE	NE	NE	15 feet

All Depths in this Summary Table are Approximate
***Bold font** indicates depths shallower that the pipe's anticipated invert depth.
PWR: Partially Weathered Rock
NE: Not Encountered

LABORATORY TESTING SUMMARY

The laboratory test results for the reduction/oxidation, pH, and soil box resistivity are summarized in the following table.

Summary of Laboratory Testing Results

Boring	Sample Description	pH (ASTM G51)	Resistivity (ASTM G57)	Oxidation Reduction Potential (ASTM G200)
P-11	Cuttings from 5 to 15 feet	4.9	49,300 ohm-cm	352 mV
P-15	Cuttings from 5 to 15 feet	5.3	40,200 ohm-cm	402 mV

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EVALUATIONS AND RECOMMENDATIONS – SPECIFIC STRUCTURES

The following evaluations and recommendations are based on the information available regarding the proposed construction, our observations, 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 very small statistical sampling of subsurface conditions, it is possible that conditions may be encountered during construction that are substantially different from those indicated by the borings. In these instances, adjustments to the design and construction may be necessary.

The following sections present discussions of subsurface conditions and specific design recommendations for each of the proposed structures. Following the individual sections, we present general evaluations and recommendations that typically apply to all construction.

Influent Metering Flume, Influent Metering Screens, & NaOH/ALUM Storage (Borings W-1 and W-2)

Following clearing and relocation of any existing utility lines, site preparation for the influent metering flume, influent metering screens, and NaOH/ALUM Storage area will require up to 6 feet of excavation and about 2 feet of structural fill placement to achieve the design bearing elevations for the mat foundations supporting these structures. All excavation necessary for minor grading and mat foundation excavation should be feasible with typical earth moving equipment such as backhoes.

We do not expect groundwater to be a concern for design or construction of the influent metering flume, influent metering screens, and the NaOH/ALUM Storage area.

Based on the results of the test borings, subsurface conditions are suitable for support of the influent metering flume, influent metering screens, and the NaOH/ALUM Storage area using conventional soil-supported mat foundations. An allowable bearing pressure of 1,000 psf is recommended and should be available without special bearing surface preparation requirements. The allowable soil bearing pressure is based on an anticipated total foundation settlement no greater than approximately 1 inch, with anticipated differential settlement between the influent metering flumes and the adjacent screens, or planar tilt of any of the mat foundations, not exceeding about ½ inch.

Because of natural variation, it is possible that some of the soils may have an allowable bearing pressure less than the recommended design value. Therefore, foundation bearing surface evaluations will be critical to aid in the identification and remediation of these situations. Prior to placing crushed stone in the area of the mat foundations, the soil subgrade should be evaluated. If weak areas are detected, remedial measures may be required. Generally, subgrade evaluation involves proofrolling, hand probing, and visual observations.

Remedial measures should be based on actual field conditions. However, in most cases we expect the use of the stone replacement technique to be the primary remedial measure. Stone replacement involves the removal of soft or loose soils, followed by replacement with well-compacted graded aggregate base (GAB) meeting Georgia DOT specifications for gradation.

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MBR Splitter Box (Boring W-2)

Following clearing and relocation of any existing utility lines, site preparation for the MBR splitter box will require about 5 feet of excavation to achieve the design bearing elevation for the mat foundation supporting the box. All excavation necessary for minor grading and mat foundation excavation should be feasible with typical earth moving equipment such as backhoes.

We do not expect groundwater to be a concern for design or construction of the MBR splitter box.

Based on the results of the test borings, subsurface conditions are suitable for support of the MBR splitter box using conventional soil-supported mat foundations. We recommend an allowable bearing pressure of 1,000 psf or less for the mat foundations. This allowable soil bearing pressure is based on an anticipated total foundation settlement no greater than approximately 1 inch, with anticipated planar tilt of the mat foundation not exceeding about ½ inch. The acceptability of these anticipated settlements should be reviewed by the design team.

Because of natural variation, it is possible that some of the soils may have an allowable bearing pressure less than the recommended design value. Therefore, foundation bearing surface evaluations will be critical to aid in the identification and remediation of these situations. Prior to placing crushed stone in the area of the mat foundation, the soil subgrade should be evaluated. If weak areas are detected, remedial measures may be required. Generally, subgrade evaluation involves proofrolling, hand probing, and visual observations.

Remedial measures should be based on actual field conditions. However, in most cases we expect the use of the stone replacement technique to be the primary remedial measure. Stone replacement involves the removal of soft or loose soils, followed by replacement with well-compacted graded aggregate base (GAB) meeting Georgia DOT specifications for gradation.

MBR Membrane Tank and Canopy and the MBR Process Building (Borings W-3, W-4, and W-5)

Following clearing and relocation of any existing utility lines, site preparation for the MBR membrane tank and the MBR process building will involve mass excavation extending to depths of about 8 to 15 feet below prevailing site grades.

The test borings suggest that most of the excavation for the MBR membrane tank and MBR process building can be performed using conventional earth moving equipment such as backhoes and loaders. However, difficult excavation conditions requiring ripping and blasting may be encountered in the vicinity of boring W-4 at elevations above the target grade. The construction budget should include a modest allowance for rock excavation within the MBR membrane tank footprint and in foundation and sub-slab utility excavations in the western portion of the MBR process building.

Temporary excavation slopes should have a gradient no steeper than 1H:1V for the portion of the excavation above the groundwater level. Parts of the excavation face extending below the groundwater level should have slopes no steeper than 1.5H:1V. Temporary shoring and bracing may also be required to protect existing underground utilities and the existing tanks immediately adjacent to the excavation. If excavation bracing is required, an internally braced system or tied-back system may be appropriate. Typically, design of temporary shoring and bracing is left to the contractor.

Groundwater was encountered borings W-3, W-4, and W-5 at elevations 2 to 5 feet higher than the planned foundation elevations of the MBR membrane tank and the MBR process building. The contractor must be prepared to implement temporary dewatering as necessary to allow excavation and construction. In our opinion, if an excavation bracing system such as sheet piling is used, it may be possible to dewater the excavation by direct pumping. We recommend that temporary dewatering and excavation shoring be treated as one operation by a specialty geotechnical contractor. Improper dewatering may have adverse effects on the temporary shoring system and therefore, it would be advisable to have the same specialty contractor perform both activities (shoring and dewatering). As a general rule, groundwater should be maintained approximately 2 to 3 feet below the prevailing excavation level. We recommend that the construction documents include a minimum *performance* specification for dewatering. The specification should require specific results from dewatering rather than dictate a dewatering method. A sample guide specification is included in the Appendix as Exhibit "A". In general, we feel that Exhibit "A" represents the minimum specification for a project of this scope.

More aggressive long-term dewatering measures such as blanket drains, closely spaced trench drains, and sumps will be required unless the deadweight of the structures, or the deadweight of the structures plus the additional weight of soil mobilized by extending mat foundations beyond the walls of the structures, is sufficient to resist buoyant uplift forces.

It is important to note that temporary and permanent dewatering for the MBR Membrane Tank and MBR Building will depress the groundwater level under the immediately adjacent existing basins. Depending on the foundation details and operating conditions of the existing basins, detrimental settlement of the existing basins could be induced by dewatering associated with the new MBR structures.

Estimating the ground subsidence that may be induced by dewatering involves calculating the compression of the soils within the groundwater fluctuation zone when the conditions change from buoyant to saturated or partially saturated. The magnitude of settlement induced by dewatering depends on the change in groundwater level and the location of adjacent structures within the drawdown profile zone. Additionally, dewatering-induced settlement is time dependent as settlement will occur both as the groundwater is lowered and after the new groundwater regime has stabilized. Additional subsurface exploration and laboratory testing will be required prior to estimating the potential magnitude of settlement associated with dewatering.

Prior to placing crushed stone, the soil subgrade should be evaluated. If weak areas are detected, remedial measures may be required. Generally, subgrade evaluation involves proofrolling, hand probing, and visual observations.

The MBR membrane tank and process building should benefit from overburden relief associated mass excavation. Once the new structures are constructed and in operation, there should be a negligible net increase in pressure applied at bearing elevation. A properly prepared bearing surface should result in little or no net settlement.

Contingent upon the proper preparation of the tank and building footprints, it is our opinion that monolithic reinforced concrete mats can be supported on a properly prepared subgrade including a 6- to 12-inch thick course of open-graded stone such as #57 stone meeting Georgia DOT specifications for gradation. An allowable bearing pressure of 2,000 psf can be used for design.

UV Structure (Boring W-6)

Following clearing and relocation of any existing utility lines, site preparation for the UV structure will require about 2 feet of excavation to achieve the design bearing elevation for the mat foundation supporting the structure. All excavation necessary for minor grading and mat foundation excavation should be feasible with typical earth moving equipment such as backhoes.

We do not expect groundwater to be a concern for design or construction of the UV structure.

Based on the results of the test borings, subsurface conditions are suitable for support of the UV structure using a conventional soil-supported mat foundation. We recommend an allowable bearing pressure of 1,000 psf or less for the mat foundation. This allowable soil bearing pressure is based on an anticipated total foundation settlement no greater than approximately 1 inch, with anticipated planar tilt of the mat foundation not exceeding about ½ inch side-to-side and about ¾ inch along the length of the structure. The acceptability of these anticipated settlements should be reviewed by the design team.

Because of natural variation, it is possible that some of the soils may have an allowable bearing pressure less than the recommended design value. Therefore, foundation bearing surface evaluations will be critical to aid in the identification and remediation of these situations. Prior to placing crushed stone in the area of the mat foundation, the soil subgrade should be evaluated. If weak areas are detected, remedial measures may be required. Generally, subgrade evaluation involves proofrolling, hand probing, and visual observations.

Remedial measures should be based on actual field conditions. However, in most cases we expect the use of the stone replacement technique to be the primary remedial measure. Stone replacement involves the removal of soft or loose soils, followed by replacement with well-compacted graded aggregate base (GAB) meeting Georgia DOT specifications for gradation.

Cascade Aerator (Boring W-7)

Following clearing and relocation of any existing utility lines, site preparation for the cascade aerator will require up to 15 feet of excavation to achieve the design bearing elevations for the mat foundations supporting these structures. All excavation necessary for minor grading and mat foundation excavation should be feasible with typical earth moving equipment such as backhoes.

We do not expect groundwater to be a concern for design or construction of the cascade aerator.

Based on the results of the test borings, subsurface conditions are suitable for support of the cascade aerator using a conventional soil-supported mat foundation. We recommend an allowable bearing pressure of 1,500 psf or less for the mat foundation. The recommended allowable soil bearing pressure is based on an anticipated total foundation settlement no greater than approximately 1 inch, with anticipated planar tilt of the mat foundation not exceeding about ½ inch.

Because of natural variation, it is possible that some of the soils may have an allowable bearing pressure less than the recommended design value. Therefore, foundation bearing surface evaluations will be critical to aid in the identification and remediation of these situations. Prior to placing crushed stone in the area of the mat foundations, the soil subgrade should be evaluated. If weak areas are detected, remedial measures may be required. Generally, subgrade evaluation involves proofrolling, hand probing, and visual observations.

Remedial measures should be based on actual field conditions. However, in most cases we expect the use of the stone replacement technique to be the primary remedial measure. Stone replacement involves the removal of soft or loose soils, followed by replacement with well-compacted graded aggregate base (GAB) meeting Georgia DOT specifications for gradation.

Administration Building (Borings W-8 and W-9)

Following clearing and relocation of any existing utility lines, minimal grading will be required to achieve design subgrade elevation for the administration building. All excavation necessary for minor grading and mat foundation excavation should be feasible with typical earth moving equipment such as backhoes.

We do not expect groundwater to be a concern for design or construction of the administration building.

Based on the results of the test borings, subsurface conditions are suitable for support of the administration building using conventional shallow foundations. Provided that maximum column loads are no greater than 100 kips and wall loads do not exceed 3 kips per lineal foot, we recommend an allowable bearing pressure of 2,000 psf or less. The recommended allowable bearing pressure is appropriate for the assumed maximum foundation loads. We estimate that total foundation settlement will be no greater than approximately 1 inch, with anticipated differential settlement between adjacent columns not exceeding about ½ inch.

Because of natural variation, it is possible that some of the soils at the project site may have an allowable bearing pressure less than the recommended design value. Therefore, foundation bearing surface evaluations will be critical to aid in the identification and remediation of these situations. Foundation bearing surface evaluations should be performed in all footing excavations prior to placement of reinforcing steel. These evaluations should be performed by Geo-Hydro to confirm that the design allowable soil bearing pressure is available. Foundation bearing surface evaluations should be performed using a combination of visual observation, hand augering, and portable dynamic cone penetrometer testing (ASTM STP-399).

Remedial measures should be based on actual field conditions. However, in most cases we expect the use of the stone replacement technique to be the primary remedial measure. Stone replacement involves the removal of soft or loose soils, followed by replacement with well-compacted graded aggregate base (GAB) meeting Georgia DOT specifications for gradation. Stone replacement is generally performed to depths ranging from a few inches to as much as 2 times the footing width, depending on the actual conditions.

New Discharge Pipe (Borings P-11 through P-16)

Excavation to depths of about 10 to 12 feet below the existing ground surface along the new discharge pipe alignment should be generally feasible with typical earth moving equipment such as backhoes. However, it is important to note that depths to partially weathered rock and rock are inherently variable in the Piedmont Region, and difficult excavation may be encountered during installation of the pipe intermediate of the boring locations.

Based on the groundwater levels in the borings, groundwater will generally be encountered throughout most of the discharge pipe alignment adjacent to the reservoir. Dewatering should be performed to maintain the groundwater level approximately 2 to 3 feet below the lowest prevailing excavation depth. In most cases we expect that direct pumping from the excavation will provide satisfactory temporary construction dewatering. However, the actual dewatering approach will be dictated by conditions at the time of excavation. Sand layers or other more permeable soil layers may significantly increase the amount of water inflow into open excavations.

The intensity of temporary dewatering required during construction is related not only to the prevailing weather conditions, but also the contractor's sequencing of construction activities. Construction specifications should include performance guidelines for temporary dewatering. Performance guidelines allow the contractor to select the actual means and methods of construction dewatering. The following sample specification[1] could be used as a guide for development of actual specifications.

> *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.*

[1] The sample specification was adapted from Construction Dewatering - A Guide to Theory and Practice, John Wiley and Sons, and is not intended for direct use as a construction specification without modifications to reflect specific project conditions.

We recommend that pipe bedding be used where groundwater is encountered. This will provide a level, stable base for pipe installation. We suggest #57 or #78 crushed stone meeting Georgia DOT specifications as pipe bedding.

EVALUATIONS AND RECOMMENDATIONS - GENERAL

General Site Preparation

Trees, underbrush, topsoil, roots, existing above-ground and underground structures, and other deleterious materials should be removed from the proposed construction areas. All existing utilities should be excavated and removed unless they are to be incorporated into the new construction. Additionally, site clearing, grubbing, and stripping should be performed only during dry weather conditions. Operation of heavy equipment on the site during wet conditions could result in excessive subgrade degradation. All excavations resulting from rerouting of underground utilities should be backfilled in accordance with the *Structural Fill* section of this report.

We recommend that areas to receive structural fill be proofrolled prior to placement of structural fill. Areas of proposed excavation should be proofrolled after rough finished subgrade is achieved. Proofrolling should be performed with multiple passes in at least two directions using a fully loaded tandem axle dump truck weighing at least 18 tons. Proofrolling must be avoided within 10 feet of existing structures. If low consistency soils are encountered that cannot be adequately densified in place, such soils should be removed and replaced with well compacted fill material placed in accordance with the *Structural Fill* section of this report. Proofrolling should be observed by Geo-Hydro to determine if remedial measures are necessary.

During site preparation, burn pits or trash pits may be encountered. On sites located in or near developed areas, this is not an unusual occurrence. All too frequently such buried material occurs in isolated areas which are not detected by the soil test borings. Any buried debris or trash found during the construction operation should be thoroughly excavated and removed from the site.

Existing Fill Materials

Existing fill materials were encountered in several of the borings. There are several important facts that should be considered regarding existing fill materials and the limitations of subsurface exploration.

- The quality of existing fill materials can be highly variable, and test borings are often not able to detect all of the zones or layers of poor quality fill materials.
- Layers of poor quality fill materials that are less than about 2.5 to 5 feet thick may often remain undetected by soil test borings due to the discrete-interval sampling method used in this exploration.
- The interface between existing fill materials and the original ground surface may include a layer of organic material that was not properly stripped off during the original grading. Depending on its relationship to the foundation and floor slab bearing surfaces, an organic layer might adversely affect

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support of footings and floor slabs. If such organic layers are encountered during construction, it may be necessary to "chase out" the organic layer by excavating the layer along with overlying soils.

- The construction budget should include funds for management of poor quality existing fill materials.
- Subsurface exploration is simply not capable of disclosing all conditions that may require remediation.

Temporary Dewatering

We recommend that the construction documents include a minimum *performance* specification for dewatering. The specification should require specific results from dewatering rather than dictate a dewatering method. A sample guide specification is included in the Appendix as Exhibit "A". In general, we feel that Exhibit "A" represents the minimum specification for a project of this scope.

Excavation

The soil test borings encountered a variety of subsurface conditions which will require different excavation methods. It is important to note that the elevation of partially weathered rock and rock can vary drastically of relatively short distances. A more general discussion of potential excavation conditions is provided herein to address potential excavation conditions not revealed by the exploratory borings.

For discussion of excavation characteristics the subsurface materials at the site may be placed into three broad categories: soil (fill and residuum), weathered rock, and rock. These categories, anticipated methods of excavation, and their occurrence in the soil test borings are presented as follows:

Material Category	*Excavation Method*	*Soil Test Borings*
Soil	Conventional Soil Excavators (Backhoes, front-end loaders)	Soil (SM), (SP), (SP-SM), (ML), (MH), (CL), (CH), etc.
Weathered Rock	Ripping (Single tooth ripper on D-8 bulldozer, heavy backhoes capable of ripping)	Partially weathered rock (Blow counts over 100 blows per foot, but less than 50 blows per inch)
Rock	Blasting	Material below auger refusal (Blow counts over 50 blows per inch)

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 which 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, a 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

Fill materials containing deleterious materials, organics, or debris cannot be used as structural fill and should be removed from the area of construction. Based on the results of the test borings, the residual soils and any debris-free fill materials on site appear suitable for reuse as structural fill. Depending on rainfall levels near the time of construction, the existing fill materials may have moisture contents above or below optimum as determined by the standard Proctor test (ASTM D698). Adding water or drying the soil may be necessary to achieve proper compaction.

Based on our experience, we expect that soils at the water reclamation facility will have moisture contents above optimum as determined by the standard Proctor test. Drying soil can be time consuming and the impact of drying soil on the schedule and budget should be carefully evaluated. In some cases wet soils may need to be disposed of offsite to maintain the schedule or expedite construction.

We expect that a significant portion of the soils excavated below the groundwater level during installation of the new discharge pipe around the perimeter of the reservoir will have moisture contents too high to allow proper compaction. While most of the discharge pipe alignment will be located in wooded areas, the pipe will be under (or immediately adjacent to) the existing dirt "road" and any reduction of the compaction criteria to allow the reuse of soils with higher moisture contents than is typical for structural fill may render the "road" impassable to vehicles.

Air-drying can be performed in the warmer, drier periods of the year but drying soil is typically only practical on larger grading sites. One or more staging areas near the discharge pipe alignment could be used to dry wet soils. The contractor should be prepared to dry soils excavated during installation of the gravity sewer line. We can provide further guidance concerning the supplemental use of chemical admixtures such as lime once a contractor is selected and a plan for addressing wet backfill soils is developed. Budget planning should consider the need to dry or replace wet soils.

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 (approximately +/- 3 percent of the optimum moisture content). From a practical standpoint, wet soils can be very difficult to dry in small or congested sites, particularly in the winter and spring, 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 as unsuitable for reuse as structural fill.

Structural Fill

We provide the following recommendations for any structural fill that may be required at the site.

Materials selected for use as structural fill should be free of organic matter, 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 Region: (SM), (ML), and (CL). The following soil types are typically suitable but are not abundant in the Piedmont Region: (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. Soils having a standard Proctor maximum dry density of less than 90 pcf should be considered unsuitable, unless laboratory evaluations of their stress-strain characteristics indicate that they will perform acceptably. The moisture content of suitable borrow soils should generally not be more than

3 percentage points above or below 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 for 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 type of compaction equipment, but in general, lifts of 8 inches loose measurement are recommended. The soil should be compacted by heavy compaction equipment such as a self-propelled sheepsfoot roller. Within small excavations such as in utility trenches 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 or mat foundation subgrade soils should be compacted to at least 98 percent of the standard Proctor maximum dry density (ASTM D698). 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 soils to be excavated consist primarily of Type B or Type C materials as defined in 29 CFR 1926.650 (1994 Edition). Temporary construction slope gradients in fill soils should be no steeper than 1.5H:1V. Temporary slopes in residual soils should be no steeper than 1H:1V. All temporary slopes in soil (fill or residual) below the groundwater level should be no steeper than 1.5H:1V and may need to be flatter for some soils. 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. The crest or toe should be no closer than 5 feet to the edge of any pavements. Erosion protection of slopes during construction and during establishment of vegetation should be considered an essential part of construction.

Temporary Excavation Bracing

Temporary shoring and bracing may be required to protect existing underground utilities and existing structures immediately adjacent to the excavations associated with the plant expansion – particularly the deeper excavations associated with MBR membrane tank and canopy, the MBR process building, and the cascade aerator. If excavation bracing is required, an internally braced system or tied-back system may be appropriate. Typically, design of temporary shoring and bracing is left to the contractor.

General Foundation Design

Recommendations for allowable soil bearing pressures for specific structures have been provided in earlier sections of this report. The following paragraphs supplement, but do not supersede, foundation recommendations outlined for specific structures.

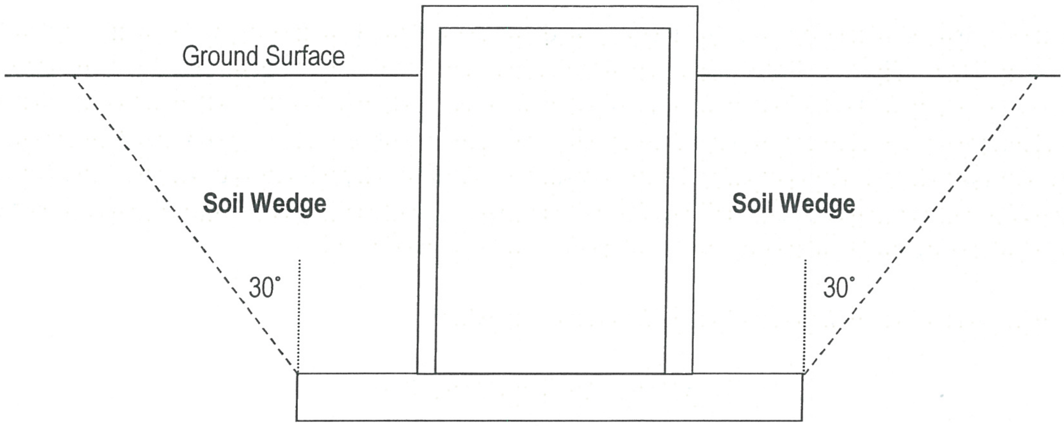
Some of the existing soils at the project site will have an allowable bearing pressure less than the recommended design value. Therefore, foundation bearing surface evaluations will be critical to aid in the identification of such soils and to enable the development of remedial measures.

Foundation bearing surface evaluations should be performed in each foundation excavation prior to placement of any crushed stone or reinforcing steel. These evaluations should be performed by Geo-Hydro to confirm that the design allowable soil bearing pressure is available. For mat foundations the subgrade evaluation will serve to confirm that the mat foundation subgrade is properly prepared and does not include any loose materials.

Remedial measures should be based on actual field conditions. However, in most cases we expect the use of the stone replacement technique to be the primary remedial measure. Improving subgrade conditions in mat foundation excavations is generally limited to removing unstable soils from the mat area and replacing the unstable soils with crushed stone materials. Usually, an open-graded stone course is placed on the prepared soil subgrade to provide a uniform, clean working surface and to preserve the underlying subgrade. We suggest a 6- to 12-inch thick course of #57 located immediately beneath concrete mats. Concrete mat areas should be evaluated by Geo-Hydro prior to placement of crushed stone.

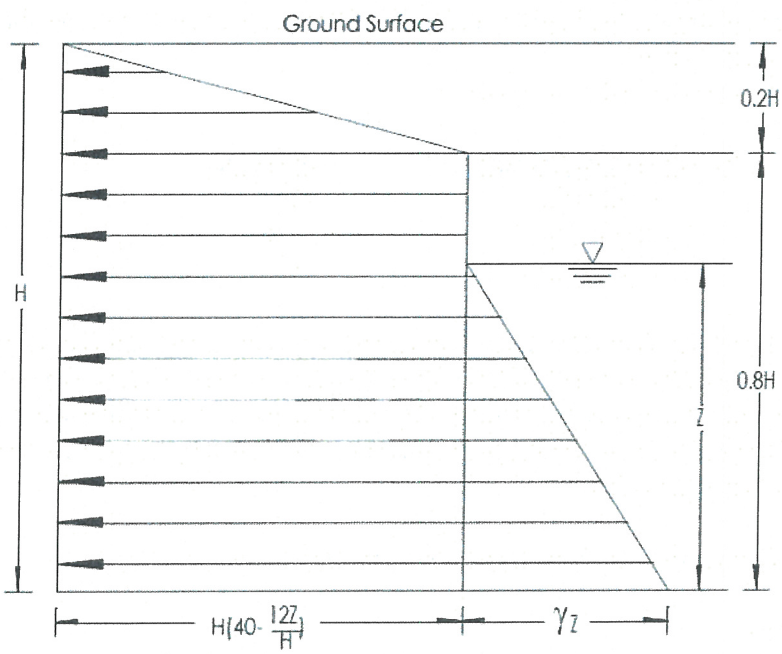
Uplift Design

In general, structures which will have bottom elevations below current or future groundwater levels or are in flood-prone areas should be designed to resist potential buoyant uplift forces. In some instances, uplift forces may be resisted by the dead weight of the structure itself. Where necessary, the dead weight of the structure can be effectively increased by extending the reinforced concrete mat foundation beyond the walls of the structure. This mobilizes additional weight of soil to increase the effective dead weight of the structure. The effective weight of a "wedge" of soil backfill as depicted in the following sketch can be used to calculate additional uplift resistance from soil backfill.



Earth Pressure – Small Below-Grade Structures

Based on our experience, the lateral earth pressure distribution for the walls of relatively small and rigid below-grade structures, such as vaults, wets wells, etc., can be approximated by a braced wall configuration. The walls of these structures may be designed for the pressure distribution shown on the following diagram:



Where: H = Total depth of wall below the ground surface, feet
Z = Height of groundwater level above the bottom of the wall, feet
γ= Unit weight of water (62.4 pcf)
pressure units are psf

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Earth Pressure – Large Basins and Conventional Retaining 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 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 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 1.0 percent to 4.0 percent of total wall height.

Earth pressure may be evaluated using the following equation:

$$p_h = K\ (D_w Z + q_s) + W_w(Z\text{-}d)$$

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 insure 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 silty sands or sandy silts 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 concrete cast-in-place retaining wall or foundation, the coefficient of friction may be taken as 0.4 for the residual 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 0.1 to 0.3 inches.

Seismic Design

Based on the results of the test borings and following the calculation procedure in the 2012 International Building Code, the following Seismic Site Class and mapped and design spectral response accelerations can be used for the planned structures:

Influent Metering Flume, Influent Metering Screens, and MBR Splitter Box

Site Class D; S_S=0.162, S_1=0.084, S_{DS}=0.173, S_{D1}=0.135.

MBR Membrane Tank and Canopy, and MBR Rocess Building

Site Class C; S_S=0.162, S_1=0.084, S_{DS}=0.130, S_{D1}=0.095.

UV Structure and Cascade Aerator

Site Class D; S_S=0.162, S_1=0.084, S_{DS}=0.173, S_{D1}=0.135.

Administration Building

Site Class D; S_S=0.162, S_1=0.084, S_{DS}=0.173, S_{D1}=0.135.

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.

Floor Slab Subgrade Preparation

The soil subgrade in the area of concrete slab-on-grade support is often disturbed during foundation excavation, plumbing installation, and superstructure construction. We recommend that the floor slab subgrade be evaluated by Geo-Hydro immediately prior to beginning floor slab construction. If low consistency soils are encountered that cannot be adequately densified in place, such soils should be removed and replaced with well-compacted fill material placed in accordance with the *Structural Fill* section of this report or with well-compacted graded aggregate base (GAB).

Assuming that the top 12 inches of floor slab subgrade soils are compacted to at least 98 percent of the standard Proctor maximum dry density, we recommend that a modulus of subgrade reaction of 120 pci be used for design.

Moisture Control for Concrete Slabs (Offices and Dry Storage)

To prevent the capillary rise of groundwater from adversely affecting the concrete slab-on-grade floor, we recommend that slab-on-grade floors be underlain by a minimum 4-inch thickness of open-graded stone. Use of #57 stone meeting Georgia DOT specifications for gradation is suggested. The stone must be covered by a vapor retarder. We suggest polyethylene sheeting at least 10 mils thick as a minimum vapor retarder.

Flexible Pavement Design

Based on our experience with similar projects, assuming standard pavement design parameters, and contingent upon proper pavement subgrade preparation, we recommend the following pavement sections:

Entrance/Exit Driveways and Truck Traffic Areas

Material	Thickness (inches)
Asphaltic Concrete 9.5mm Superpave	2
Asphaltic Concrete 19mm Superpave	2
Graded Aggregate Base (GAB) (Base Course)	6
Subgrade compacted to at least 100% standard Proctor maximum dry density (ASTM D698)	12

Automobile Parking and Automobile Traffic Only

Material	Thickness (inches)
Asphaltic Concrete 9.5mm Superpave	2
Graded Aggregate Base (GAB) (Base Course)	6
Subgrade compacted to at least 100% standard Proctor maximum dry density (ASTM D698)	12

A concrete thickness of 7 inches is recommended for the approach and collection zone in front of the dumpster. Please refer to the *Concrete Pavement* section of this report for concrete pavement recommendations.

The top 12 inches of pavement subgrade soils should be compacted to at least 100 percent of the standard Proctor maximum dry density (ASTM D698). Scarification and moisture adjustment will likely be required to achieve the recommended subgrade compaction level. Allowances for pavement subgrade preparation should be considered for budgeting and scheduling.

GAB must be compacted to at least 100 percent of the modified Proctor maximum dry density (ASTM D1557).

All pavement construction should be performed in general accordance with Georgia DOT specifications. Proper subgrade compaction, adherence to Georgia DOT specifications, and compliance with project plans and specifications, will be critical to the performance of the constructed pavement.

Concrete Pavement

A rigid Portland cement concrete pavement may be considered. Although usually more costly, a Portland cement concrete pavement is typically more durable and requires less maintenance throughout the life cycle of the facility. Concrete thicknesses of 5 inches in automobile parking areas and 6 inches in driveways and truck traffic areas are recommended. A concrete thickness of 7 inches is recommended for the approach and collection zone in front of the dumpster. A 650-psi flexural strength concrete mix with 3.5 to 5.5 percent air entrainment should be used. The concrete pavement should be underlain by no less than 4 inches of compacted graded aggregate base (GAB). GAB should be compacted to at least 100 percent of the modified Proctor maximum dry density (ASTM D1557). The top 12 inches of soil subgrade should be compacted to at least 100 percent of the standard Proctor maximum dry density (ASTM D698).

The concrete pavement may be designed as a "plain concrete pavement" with no reinforcing steel, or reinforcing steel may be used at joints. Construction joints and other design details should be in accordance with guidelines provided by the Portland Cement Association and the American Concrete Institute.

In general, all pavement construction should be in accordance with Georgia DOT specifications. Proper subgrade compaction, adherence to Georgia DOT specifications, and compliance with project plans and specifications will be critical to the performance of the constructed pavement.

Pavement Design Limitations

The pavement sections discussed above are based on our experience with similar facilities. After traffic information has been developed, we recommend that you allow us to review the traffic data and revise our recommendations as necessary.

Pavement Materials Testing

In order to aid in verifying that the pavement system is installed in general accordance with the design considerations, the following materials testing services are recommended:

- Density testing of subgrade materials.
- Proofrolling of pavement subgrade materials immediately prior to placement of graded aggregate base (GAB). This proofrolling should be performed the same day GAB is installed.
- Density testing of GAB and verification of GAB thickness. In-place density should be verified using the sand cone method (ASTM D1556).
- Coring of the pavement to verify thickness and density (asphalt pavement only). Two or three cores should suffice to evaluate the finished pavement.
- Preparation and testing of beams and cylinders for flexural and compressive strength testing (Portland cement concrete only). The total number of test specimens required will depend on the number of concrete placement events necessary to construct the pavement.

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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PWF/LEB/160205.20 Indian Creek WRF Expansion - Locust Grove, Georgia

cc: Mr. Scott Hennessey, P.E.; Engineering Strategies, Inc.

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Appendix

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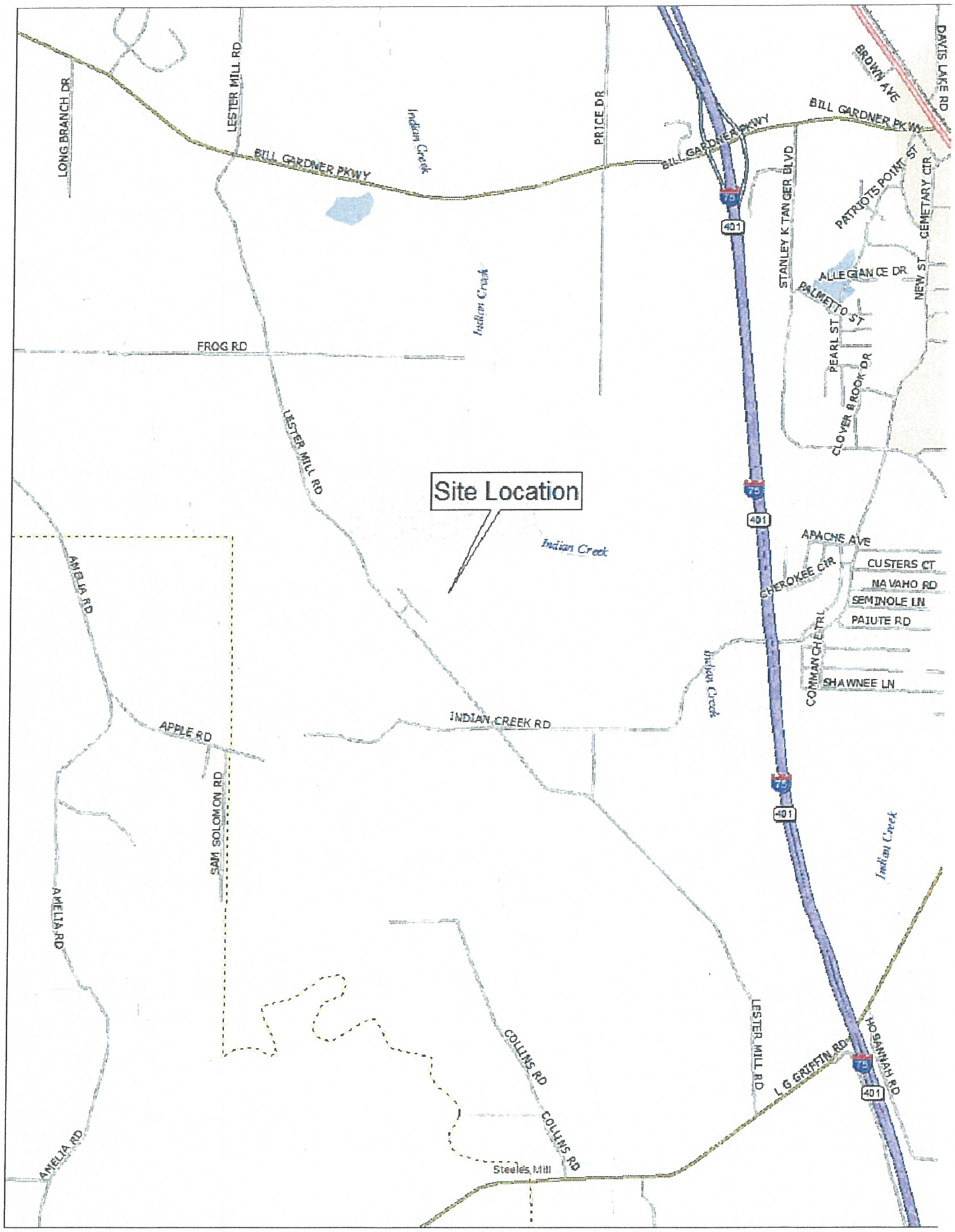

0 0.25 0.5 1 1.5

Approximate Scale, Miles

Figure 1: Site Location Plan	Indian Creek Water Reclamation Facility Expansion Locust Grove, Henry County, Georgia Geo-Hydro Project Number 160205.20

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LEGEND: Soil Test Boring

Figure 2: Water Plant Boring Location Plan	Indian Creek Water Reclamation Facility Expansion Locust Grove, Henry County, Georgia Geo-Hydro Project Number 160205.20

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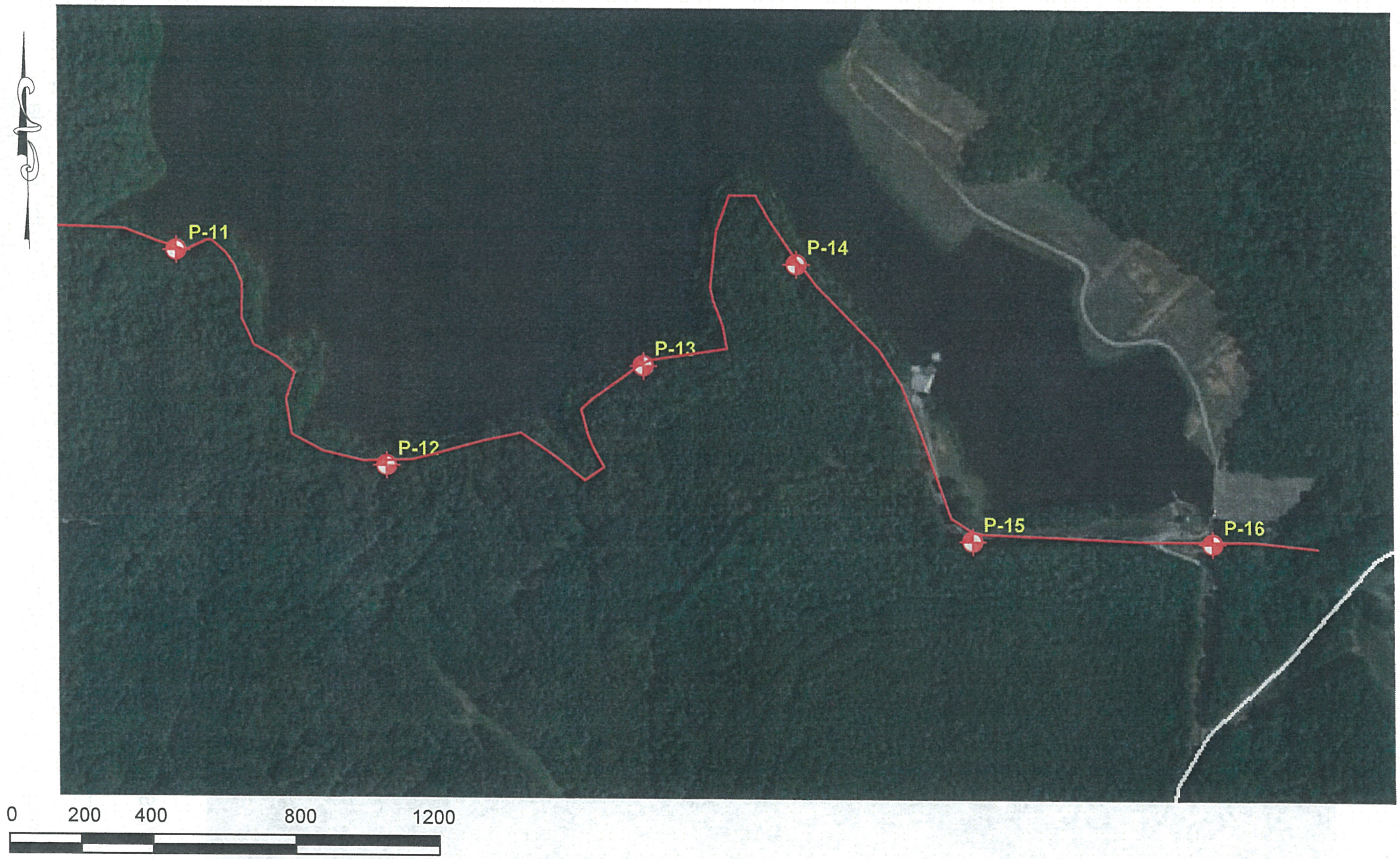

LEGEND: Soil Test Boring

Figure 3: Pipeline Boring Location Plan

Indian Creek Water Reclamation Facility Expansion
Locust Grove, Henry County, Georgia
Geo-Hydro Project Number 160205.20

Water Reclamation Facility - Summary of Subsurface Conditions

Structure	Boring	Existing Ground Elevation	Foundation Bearing Elevation	Foundation Cut/Fill Depths	Top of PWR Elevation	Auger Refusal Elevation	Groundwater Elevation	Boring Termination Elevation
Influent Metering Flume	W-1	805	806	Fill 1 ft.	NE	NE	789	780
Influent Metering Screens	W-1	805	801	Cut 4 ft.	NE	NE	789	780
NaOH/ALUM Storage	W-2	802	799	Cut 3 ft.	NE	NE	787	777
MBR Splitter Box	W-2	798	793	Cut 5 ft.	NE	NE	787	777
MBR Membrane Tank & Canopy	W-3	788	780	Cut 8 ft.	NE	NE	**782***	763
	W-4	790	780	Cut 10 ft.	778	743	**783***	743
MBR Process Building	W-3	788	777	Cut 11 ft.	NE	NE	**782***	763
	W-4	790	777	Cut 13 ft.	**778***	743	**783***	743
	W-5	788	777	Cut 11 ft.	NE	NE	**780***	763
UV Structure	W-6	782	780	Cut 2 ft.	NE	NE	774	757
Cascade Aerator	W-7	776	761	Cut 15 ft.	NE	NE	754	751
Administration Building	W-8	781	779	Cut 2 ft.	NE	NE	769	756
	W-9	782	779	Cut 2 ft.	NE	NE	769	757

All Depths and Elevations in this Summary Table are Approximate
***Bold font** indicates elevations above the foundation mat or footing bearing elevation.
PWR: Partially Weathered Rock
NA: Not Available
NE: Not Encountered

New Discharge Pipe - Summary of Subsurface Conditions

Boring	Anticipated Depth to Pipe's Invert	Depth to Top of PWR	Depth to Auger Refusal	Depth to Groundwater	Depth of Boring Termination
P-11	10 feet	NE	NE	**5 feet***	15 feet
P-12	10 feet	NE	NE	**9 feet***	15 fee
P-13	10 feet	NE	13 FEET	**8 feet***	13 feet
P-14	10 feet	NE	13 FEET	**5 feet***	13 feet
P-15	10 feet	NE	NE	14 feet	15 feet
P-16	10 feet	NE	NE	NE	15 feet

All Depths in this Summary Table are Approximate
***Bold font** indicates depths shallower that the pipe's anticipated invert depth.
PWR: Partially Weathered Rock
NE: Not Encountered

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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.

W-1

Test Boring Record

GEO HYDRO ENGINEERS

Project: **Indian Creek Water Reclamation Facility Expansion**		Project No: **160205.20**
Location: **Locust Grove, Henry County, Georgia**		Date: **3/29/16**
Method: **HSA- ASTM D1586**	GWT at Drilling: **Not Encountered**	G.S. Elev: **805**
Driller: **B&C (Auto Hammer)**	GWT at 24 hrs: **16 feet**	Logged By: **LEH**

Elev. (Ft)	Depth (Ft)	GWT	Description	N
805	0		Topsoil (Approximately 1 inch)	
			Firm brown slightly micaceous silty fine sand (SM) (RESIDUUM)	13
800	5		Stiff tan slighty micaceous fine sandy silt (ML)	14
				14
			Loose to firm tan silty fine sand (SM)	
795	10			10
790	15	▼ (16 ft)		17
785	20			10
			Very stiff brown fine sandy silt (ML)	
780	25			16
			Boring Terminated at 25 feet	
775	30			
770	35			
765	40			
760	45			
755	50			

Standard Penetration Test (Blows/Foot): 0, 10, 20, 30, 40, 50, 60, 70, 80, 90, 100

Remarks:

W-2

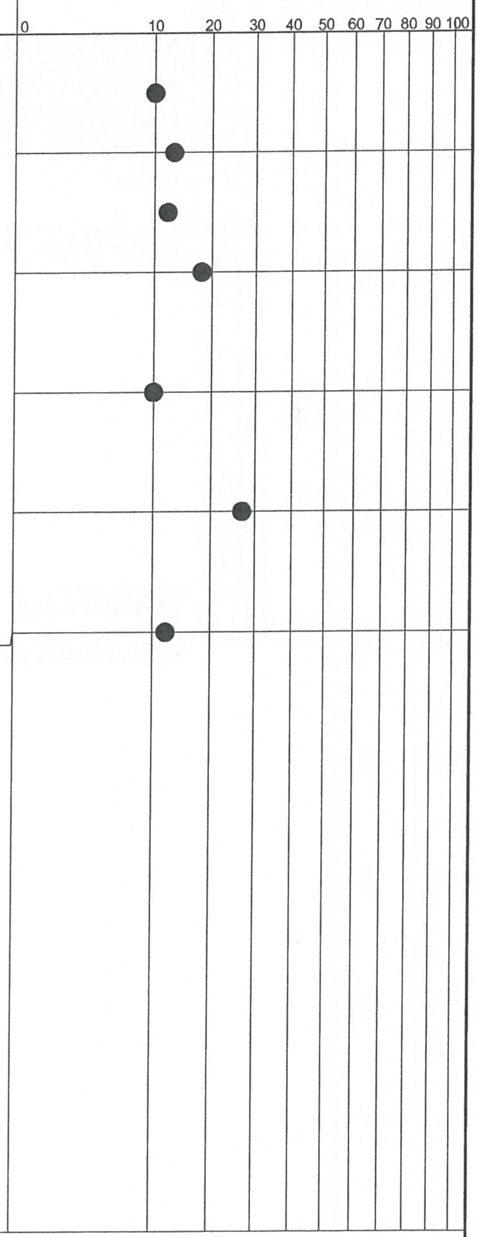
Test Boring Record

GEO HYDRO ENGINEERS

Project: **Indian Creek Water Reclamation Facility Expansion**		Project No: **160205.20**
Location: **Locust Grove, Henry County, Georgia**		Date: **3/29/16**
Method: **HSA- ASTM D1586**	GWT at Drilling: **21 feet**	G.S. Elev: **802**
Driller: **B&C (Auto Hammer)**	GWT at 24 hrs: **15 feet**	Logged By: **LEH**

Elev. (Ft)	Depth (Ft)	GWT	Description	N
802	0		Topsoil (Approximately 1 inch)	
			Loose to firm red micaceous silty fine sand (SM) (RESIDUUM)	10
				13
			Firm white and tan silty fine sand (SM)	12
			Loose to firm red-brown silty fine sand (SM)	18
	15	▼		10
			Firm to very firm brown slighty micaceous silty fine sand (SM)	27
	21	▽		
	25			12
			Boring Terminated at 25 feet	

Standard Penetration Test (Blows/Foot)



Remarks:

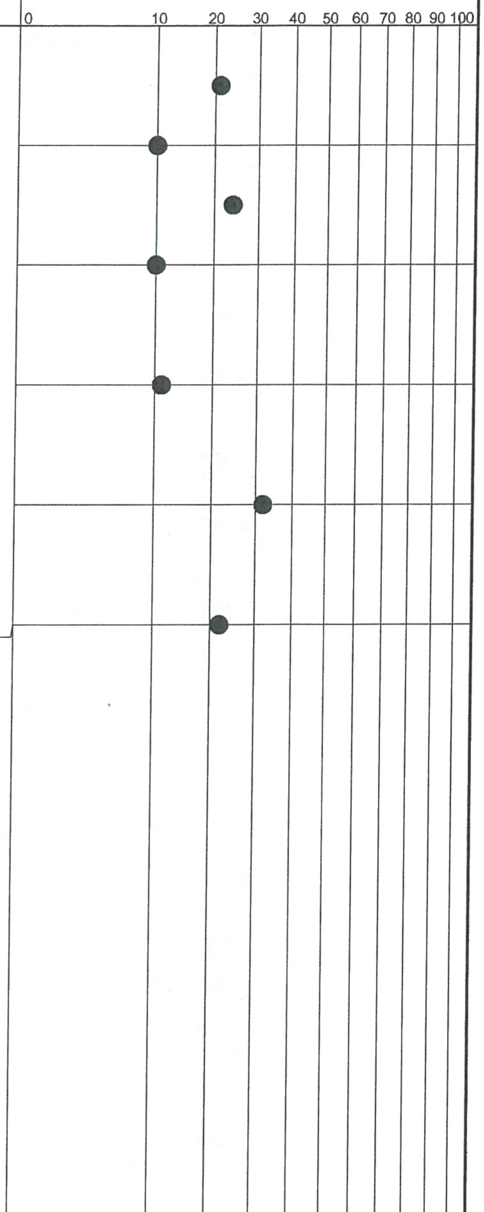
W-3 Test Boring Record

GEO HYDRO ENGINEERS

Project: **Indian Creek Water Reclamation Facility Expansion**		Project No: **160205.20**
Location: **Locust Grove, Henry County, Georgia**		Date: **3/29/16**
Method: **HSA- ASTM D1586**	GWT at Drilling: **9 feet**	G.S. Elev: **788**
Driller: **B&C (Auto Hammer)**	GWT at 24 hrs: **6 feet**	Logged By: **LEH**

Depth (Ft)	Description	N
0	Topsoil (Approximately 1 inch)	
0–3	Very firm red-brown silty fine sand (SM) (RESIDUUM)	21
3–6	Loose tan silty fine to medium sand (SM)	10
6–8	Very firm white and tan silty fine sand (SM)	24
8–18	Loose to firm brown slightly micaceous silty fine sand (SM)	10, 11
18–25	Very firm to dense brown slightly micaceous silty fine sand (SM)	32, 22
25	Boring Terminated at 25 feet	

Standard Penetration Test (Blows/Foot)



Remarks:

W-4

Test Boring Record

GEO HYDRO ENGINEERS

Project: **Indian Creek Water Reclamation Facility Expansion**			Project No: **160205.20**
Location: **Locust Grove, Henry County, Georgia**			Date: **3/29/16**
Method: **HSA- ASTM D1586**	GWT at Drilling: **12 feet**		G.S. Elev: **790**
Driller: **B&C (Auto Hammer)**	GWT at 24 hrs: **7 feet**		Logged By: **LEH**

Elev. (Ft)	Depth (Ft)	GWT	Description	N
			Topsoil (Approximately 1 inch)	
			Firm dark brown silty fine sand (SM) (FILL)	12
785	5			11
		▼	Very loose brown silty fine to medium sand (SM) (RESIDUUM)	3
780	10			4
		▽	Partially weathered rock sampled as brown micaceous silty fine to medium sand (SM)	
775	15			50/6"
770	20			50/6"
765	25			50/5"
760	30		Very dense dark brown silty fine sand (SM)	56
755	35		Dense light gray silty fine to medium sand (SM)	41
750	40		Partially weathered rock sampled as light gray silty fine to medium sand (SM)	50/4"
745	45			50/5"
			Auger Refusal at 47 feet	
740	50			

Standard Penetration Test (Blows/Foot)

0 10 20 30 40 50 60 70 80 90 100

Remarks:

W-5 Test Boring Record

GEO HYDRO ENGINEERS

Project: **Indian Creek Water Reclamation Facility Expansion**		Project No: **160205.20**
Location: **Locust Grove, Henry County, Georgia**		Date: **3/30/16**
Method: **HSA- ASTM D1586**	GWT at Drilling: **14 feet**	G.S. Elev: **788**
Driller: **B&C (Auto Hammer)**	GWT at 24 hrs: **8 feet**	Logged By: **LEH**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N	Standard Penetration Test (Blows/Foot)
788	0			Topsoil (Approximately 1 inch)		
				Firm red-brown silty fine sand (SM) (FILL)	20	20
785	5			Firm red-brown micaceous silty fine to medium sand (SM) (RESIDUUM)	17	17
		▼			17	17
780	10			Very firm black slightly micaceous silty fine sand (SM)	21	21
775	15	∇		Dense brown micaceous silty fine to medium sand (SM)	35	35
770	20			Firm brown micaceous silty fine to medium sand (SM)	14	14
765	25				17	17
				Boring Terminated at 25 feet		

Elevation scale: 785, 780, 775, 770, 765, 760, 755, 750, 745, 740. Depth scale: 5, 10, 15, 20, 25, 30, 35, 40, 45, 50. SPT scale: 0, 10, 20, 30, 40, 50, 60, 70, 80, 90, 100.

Remarks:

W-6

Test Boring Record

GEO HYDRO ENGINEERS

Project: **Indian Creek Water Reclamation Facility Expansion**		Project No: **160205.20**	
Location: **Locust Grove, Henry County, Georgia**		Date: **3/30/16**	
Method: **HSA- ASTM D1586**	GWT at Drilling: **15 feet**	G.S. Elev: **782**	
Driller: **B&C (Auto Hammer)**	GWT at 24 hrs: **8 feet**	Logged By: **LEH**	

Depth (Ft)	Elev. (Ft)	GWT	Description	N
0 – 1 inch	782		Topsoil (Approximately 1 inch)	
0 – 3			Loose red-brown silty fine sand (SM) (FILL)	10
3 – 8		▼ (8 ft, 24 hrs)	Stiff to very stiff red fine sandy silt (ML) (RESIDUUM)	17, 14
8 – 12			Firm red silty fine sand (SM)	13
12 – 18		▽ (15 ft, at drilling)	Loose red and tan slightly micaceous silty fine sand (SM)	9
18 – 25			Firm brown micaceous silty fine to medium sand (SM)	18, 17
25	757		Boring Terminated at 25 feet	

Standard Penetration Test (Blows/Foot): 0, 10, 20, 30, 40, 50, 60, 70, 80, 90, 100

Elev. (Ft): 780, 775, 770, 765, 760, 755, 750, 745, 740, 735

Depth (Ft): 5, 10, 15, 20, 25, 30, 35, 40, 45, 50

Remarks:

W-7 Test Boring Record

GEOHYDRO ENGINEERS

Project: **Indian Creek Water Reclamation Facility Expansion**	Project No: **160205.20**
Location: **Locust Grove, Henry County, Georgia**	Date: **3/30/16**
Method: **HSA- ASTM D1586** — GWT at Drilling: **19 feet**	G.S. Elev: **776**
Driller: **B&C (Auto Hammer)** — GWT at 24 hrs: **22 feet**	Logged By: **LEH**

Elev. (Ft)	Depth (Ft)	Description	N
776	0	Topsoil (Approximately 1 inch)	
		Loose to firm red-brown silty fine sand (SM) (RESIDUUM)	12
	5		10
			17
	10		19
	12	Loose brown slightly micaceous silty fine sand (SM)	
	15		8
	20		8
	23	Firm dark brown micaceous silty fine to medium sand (SM)	
	25		17
		Boring Terminated at 25 feet	

GWT: ▽ 19 ft (at drilling); ▼ 22 ft (at 24 hrs)

Standard Penetration Test (Blows/Foot): 0, 10, 20, 30, 40, 50, 60, 70, 80, 90, 100

Remarks:

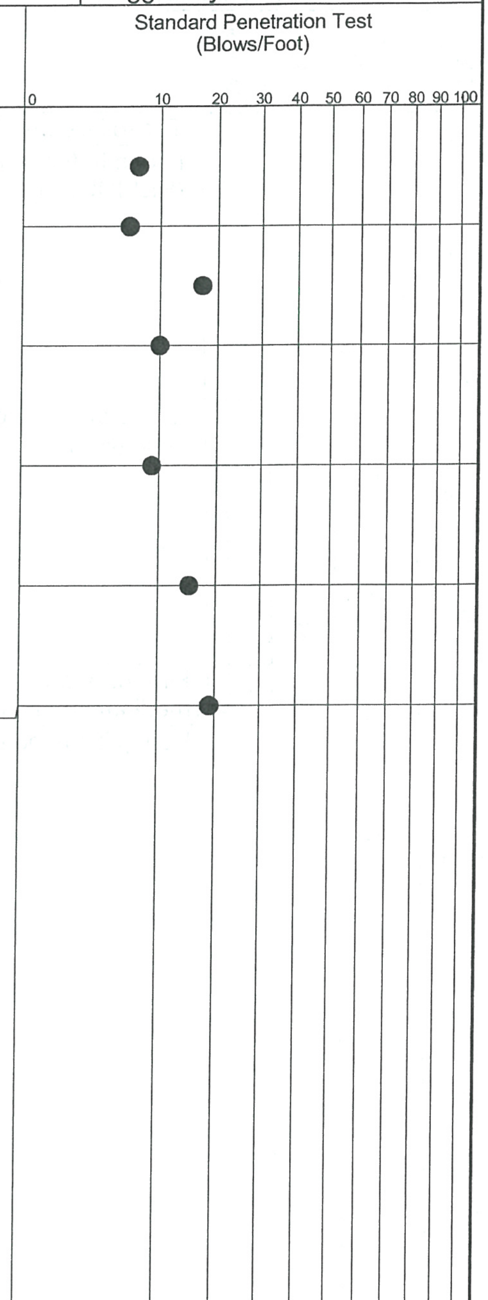
W-8

Test Boring Record

GEO HYDRO ENGINEERS

Project: **Indian Creek Water Reclamation Facility Expansion**		Project No: **160205.20**
Location: **Locust Grove, Henry County, Georgia**		Date: **3/30/16**
Method: **HSA- ASTM D1586**	GWT at Drilling: **17 feet**	G.S. Elev: **781**
Driller: **B&C (Auto Hammer)**	GWT at 24 hrs: **15 feet**	Logged By: **LEH**

Elev. (Ft)	Depth (Ft)	GWT	Description	N
781–780.75	0–0.25		Graded Aggregate Base (Approximately 3 inches)	
			Firm dark red fine sandy silt (ML) (FILL)	7
			Loose to firm red-brown silty fine sand (SM) (FILL)	6
				17
				10
	~12		Stiff red-brown slightly micaceous fine sandy silt (ML) (RESIDUUM)	
	15	▼		9
	17	▽		
	20			15
	~23		Firm brown slightly micaceous silty fine sand (SM)	19
756	25		Boring Terminated at 25 feet	



Remarks:

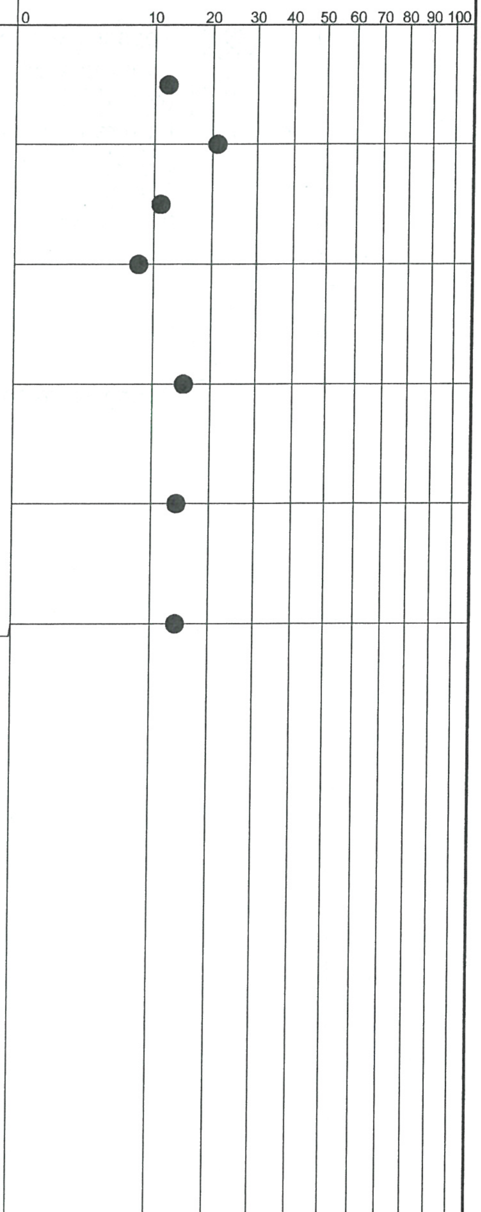
W-9 Test Boring Record

GEO HYDRO ENGINEERS

Project: **Indian Creek Water Reclamation Facility Expansion**		Project No: **160205.20**
Location: **Locust Grove, Henry County, Georgia**		Date: **3/30/16**
Method: **HSA- ASTM D1586**	GWT at Drilling: **14 feet**	G.S. Elev: **782**
Driller: **B&C (Auto Hammer)**	GWT at 24 hrs: **13 feet**	Logged By: **LEH**

Elev. (Ft)	Depth (Ft)	Description	N
782	0	Topsoil (Approximately 1 inch)	
		Firm dark brown silty fine sand (SM) (FILL)	12
		Very firm red-brown silty fine to medium sand (SM) (FILL)	21
		Loose to firm red-brown silty fine sand (SM) (RESIDUUM)	11
			8
		Firm dark red-brown micaceous silty fine to medium sand (SM)	15
			14
		Firm white and tan silty fine to coarse sand (SM)	14
757	25	Boring Terminated at 25 feet	

Standard Penetration Test (Blows/Foot)



Remarks:

P-11

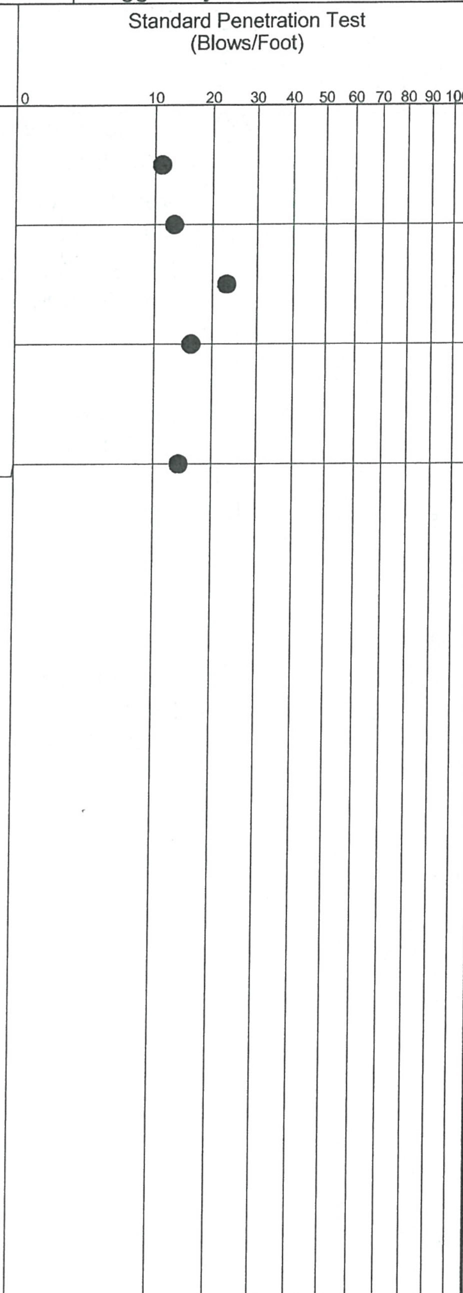
Test Boring Record

GEO HYDRO ENGINEERS

Project: **Indian Creek Water Reclamation Facility Expansion**	Project No: **160205.20**
Location: **Locust Grove, Henry County, Georgia**	Date: **3/31/16**
Method: **HSA- ASTM D1586** / GWT at Drilling: **7 feet**	G.S. Elev:
Driller: **B&C (Auto Hammer)** / GWT at 24 hrs: **5 feet**	Logged By: **LEH**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N
	0			Topsoil (Approximately 2 inches)	
				Firm red slightly micaceous silty fine sand (SM) (FILL)	11
	5	▼ (5 ft, 24 hrs)		Firm red-brown silty fine sand (SM) (RESIDUUM)	13
		▽ (7 ft, at drilling)		Firm to very firm dark brown slightly micaceous silty fine sand (SM)	23
	10				16
				Firm gray silty fine sand (SM)	
	15				14
				Boring Terminated at 15 feet	

Standard Penetration Test (Blows/Foot)



Remarks:

TEST BORING RECORD SOIL TEST BORINGS.GPJ GEO HYDRO.GDT 5/3/16

P-12

Test Boring Record

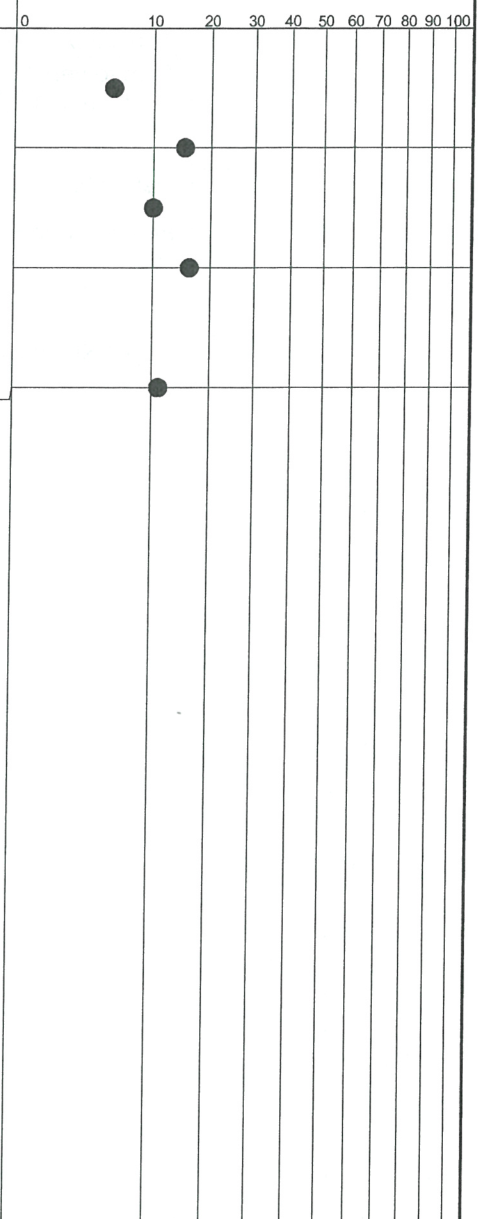
GEO HYDRO ENGINEERS

Project: **Indian Creek Water Reclamation Facility Expansion**		Project No: **160205.20**	
Location: **Locust Grove, Henry County, Georgia**		Date: **3/31/16**	
Method: **HSA- ASTM D1586**	GWT at Drilling: **11 feet**	G.S. Elev:	
Driller: **B&C (Auto Hammer)**	GWT at 24 hrs: **9 feet**	Logged By: **LEH**	

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N
	0			Topsoil (Approximately 2 inches)	
				Firm red fine sandy silt (ML) (FILL)	5
	5			Firm red-brown silty fine to medium sand (SM) (RESIDUUM)	15
				Loose red-brown silty fine sand (SM)	10
	10	▼ (9 ft) / ▽ (11 ft)		Firm brown silty fine sand (SM)	16
	15			Firm dark brown micaceous silty fine sand (SM)	11
				Boring Terminated at 15 feet	

Depth scale: 5, 10, 15, 20, 25, 30, 35, 40, 45, 50

Standard Penetration Test (Blows/Foot)



Remarks:

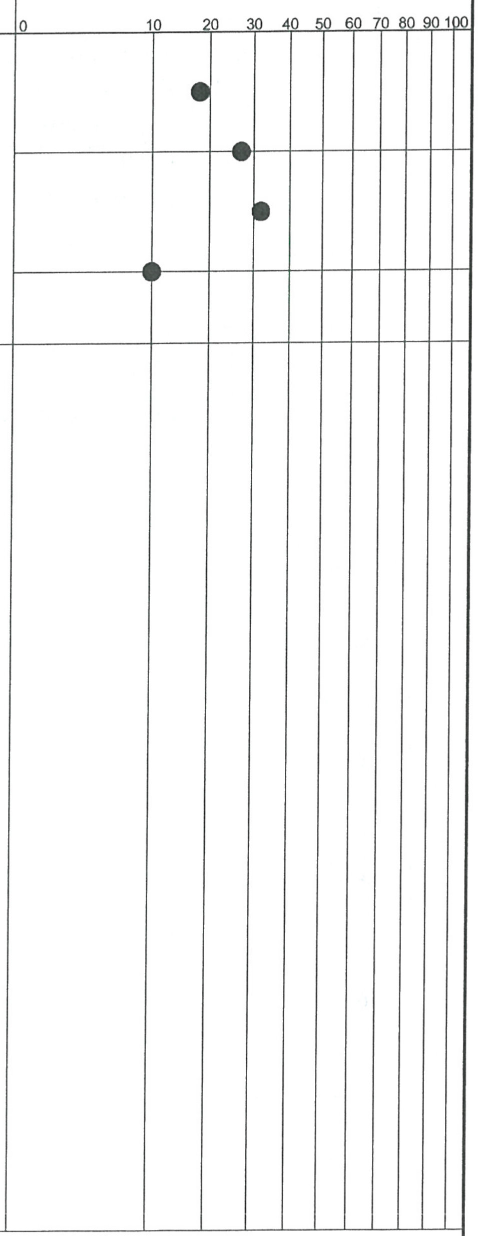
P-13 Test Boring Record

GEO HYDRO ENGINEERS

Project: **Indian Creek Water Reclamation Facility Expansion**	Project No: **160205.20**
Location: **Locust Grove, Henry County, Georgia**	Date: **3/31/16**
Method: **HSA- ASTM D1586** / GWT at Drilling: **10 feet**	G.S. Elev:
Driller: **B&C (Auto Hammer)** / GWT at 24 hrs: **8 feet**	Logged By: **LEH**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N
	0			Topsoil (Approximately 1 inch)	
				Firm to very firm red-brown silty fine to medium sand (SM) (RESIDUUM)	18
	5				27
		▼ (8 ft)		Dense brown silty fine sand (SM)	32
	10	▽ (10 ft)		Firm dark brown silty fine to medium sand (SM)	10
	13			Auger Refusal at 13 feet	
	15				
	20				
	25				
	30				
	35				
	40				
	45				
	50				

Standard Penetration Test (Blows/Foot)



Remarks:

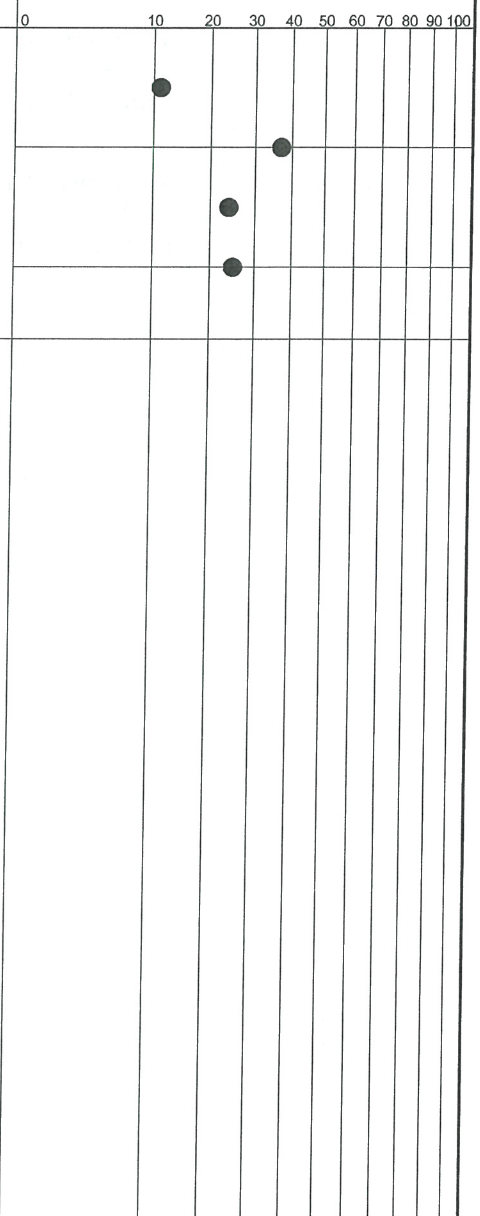
P-14 Test Boring Record

GEO HYDRO ENGINEERS

Project: **Indian Creek Water Reclamation Facility Expansion**	Project No: **160205.20**
Location: **Locust Grove, Henry County, Georgia**	Date: **3/31/16**
Method: **HSA- ASTM D1586** / GWT at Drilling: **6 feet**	G.S. Elev:
Driller: **B&C (Auto Hammer)** / GWT at 24 hrs: **5 feet**	Logged By: **LEH**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N
	0			Topsoil (Approximately 1 inch)	
				Firm dark brown silty fine sand (SM) with root fragments (RESIDUUM)	11
	5	▼ ▽		Dense red-brown silty fine to medium sand (SM)	37
				Very firm dark red-brown silty fine to medium sand (SM)	24
	10			Very stiff red-brown fine sandy silt (ML)	25
				Auger Refusal at 13 feet	

Standard Penetration Test (Blows/Foot)



Remarks:

P-15

Test Boring Record

GEO HYDRO ENGINEERS

Project: **Indian Creek Water Reclamation Facility Expansion**		Project No: **160205.20**
Location: **Locust Grove, Henry County, Georgia**		Date: **3/31/16**
Method: **HSA- ASTM D1586**	GWT at Drilling: **Not Encountered**	G.S. Elev:
Driller: **B&C (Auto Hammer)**	GWT at 24 hrs: **14 feet**	Logged By: **LEH**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N	Standard Penetration Test (Blows/Foot)
	0			Topsoil (Approximately 1 inch)		
	0–6		SM	Firm orange and brown silty fine sand (SM) (RESIDUUM)	11, 16	
	6–8.5		ML	Stiff brown slightly micaceous fine sandy silt (ML)	10	
	8.5–15	▼ 14	ML	Firm tan and white fine sandy silt (ML)	7, 8	
	15			Boring Terminated at 15 feet		

Remarks:

P-16 Test Boring Record

GEO HYDRO ENGINEERS

Project: **Indian Creek Water Reclamation Facility Expansion**		Project No: **160205.20**
Location: **Locust Grove, Henry County, Georgia**		Date: **3/31/16**
Method: **HSA- ASTM D1586**	GWT at Drilling: **Not Encountered**	G.S. Elev:
Driller: **B&C (Auto Hammer)**	GWT at 24 hrs: **N/A (Boring Backfilled)**	Logged By: **LEH**

Elev. (Ft)	Depth (Ft)	GWT	Symbol	Description	N	Standard Penetration Test (Blows/Foot)
	0			Topsoil (Approximately 1 inch)		
				Loose red silty fine sand (SM) (FILL)	5	
				Loose to firm brown slightly micaceous silty fine sand (SM) (RESIDUUM)		
	5				10	
					12	
	10				11	
	15				15	
				Boring Terminated at 15 feet		
	20					
	25					
	30					
	35					
	40					
	45					
	50					

Remarks: Boring P-16 was backfilled with bentonite chips.

Laboratory Test Results

GEO HYDRO ENGINEERS

TIMELY ENGINEERING SOIL TESTS, LLC	1874 Forge Street Tucker, GA 30084 Phone: 770-938-8233 Fax: 770-923-8973 Web: www.test-llc.com	Tested By	EB
		Date	04/08/16
		Checked By	

Client Pr. #	160205.20	Lab. PR. #	1607A-13-1
Pr. Name	Indian Creek WRF Expansion	S. Type	Bag
Sample ID	21605/Boring P-11	Depth/Elev.	-
Location	P-11	Add. Info	-

ASTM G51/AASHTO T289

Standard Test Method for Determining pH of Soil for Use in Corrosion Testing

SAMPLE PREPARATION

Air dried Material passing #10 sieve was used for testing.

TEST DATA

T.E.S.T. Sample ID	Client Sample ID	pH meter Reading	Reported pH
21605	Boring P-11	4.94	**4.9**

Standard buffer solutions used to standardize pH meter:

4.0 pH

7.0 pH

10.0 pH

pH Meter ID: 375

REMARKS

TIMELY ENGINEERING SOIL TESTS, LLC

1874 Forge Street Tucker, GA 30084
Phone: 770-938-8233
Fax: 770-923-8973
Web: www.test-llc.com

Tested By	RI		
Date	04/08/16		
Checked By	[signature]		
Client Pr. #	160205.20	Lab. PR. #	1607A-13-1
Pr. Name	Indian Creek WRF Expansion	S. Type	Bag
Sample ID	21605/Boring P-11	Depth/Elev.	-
Location	P-11	Add. Info	-

ASTM G 57/G187/AASHTO T 288
Standard Test Method for Determining Minimum Laboratory Soil Resistivity

Determination of Resistivity at as-received moisture content

As-received Moisture Content

Mass of Wet Sample & Tare, g	
Mass of Dry Sample & Tare, g	
Mass of Tare, g	
Moisture Content, %	NA

Remarks

TEST DATA

Mass of Soil Box, g	-	Meter Dial Reading, ohms	-
Mass of Soil Box + Soil, g	-	Reading of Meter Range Multiplier	-
Mass of Soil, g	-	Measured Resistance, ohms	-
Calibrated Volume of Soil Box, ft^3	0.0027	Calibrated Soil Box Multiplier, cm	1.0
Wet Density of as-placed Soil, pcf	-		
Dry Density of as-placed Soil, pcf	-	**Reported Soil Resistivity, ohms-cm**	**NA**

Determination of Minimum Soil Resistivity

TEST DATA

	Trials at Various Moisture Content								
TRIAL #	1	2	3	4	5	6	7	8	9
Meter Dial Reading, ohms	173	75	64	55.5	49.3	49.3			
Reading of Meter Range Multiplier	K	K	K	K	K	K			
Measured Resistance, ohms	173000	75000	64000	55500	49300	49300			
Calibrated Soil Box Multiplier, cm	1.0	1.0	1.0	1.0	1.0	1.0			
Measured Resistivity, ohms-cm	173000	75000	64000	55500	49300	49300			

Reported Soil Minimum Resistivity, ohms-cm **49300**

Note: Material passed # 10 sieve used for testing

Oven ID #	496/610
Balance ID #	563/700
Soil Box ID #	612/613/707
Resistivity Meter ID #	706

Description

NA

USCS (D2487; D2488)	NA
AASHTO (M145)	NA

TIMELY ENGINEERING SOIL TESTS, LLC

1874 Forge Street Tucker, GA 30084
Phone: 770-938-8233
Fax: 770-923-8973
Web: www.test-llc.com

Tested By	EB
Date	04/08/16
Checked By	[illegible]

Client Pr. #	160205.20	Lab. PR. #	1607A-13-1
Pr. Name	Indian Creek WRF Expansion	S. Type	Bag
Sample ID	21605/Boring P-11	Depth/Elev.	-
Location	P-11	Add. Info	-

ASTM G200

Standard Test Method for Measurement of Oxidation Reduction Potential (ORP) of Soil

SAMPLE PREPARATION

Roots, Stones, Gravel and other deleterious material was removed prior to testing

Measurements performed ar room temperature condition: 20.1 °C

TEST DATA

T.E.S.T. Sample ID	Client Sample ID	ORP meter Reading #1, mV	ORP meter Reading #2, mV	ORP meter Reading #3, mV	Reported ORP value, mV
21605	Boring P-11	348.0	346.0	361.0	**352**

REMARKS

Standard ORP calibration solution (420+/-mV) used to standardize ORP meter: P.D.1/21/15 Exp.10/16

ORP Meter ID 375

ORP Probe ID 417

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Tested By	EB
Date	04/08/16
Checked By	[signature]

Client Pr. #	160205.20	Lab. PR. #	1607A-13-1
Pr. Name	Indian Creek WRF Expansion	S. Type	Bag
Sample ID	21606/Boring P-15	Depth/Elev.	-
Location	P-15	Add. Info	-

ASTM G51/AASHTO T289

Standard Test Method for Determining pH of Soil for Use in Corrosion Testing

SAMPLE PREPARATION

Air dried Material passing #10 sieve was used for testing.

TEST DATA

T.E.S.T. Sample ID	Client Sample ID	pH meter Reading	Reported pH
21606	Boring P-15	5.30	**5.3**

Standard buffer solutions used to standardize pH meter:

4.0 pH

7.0 pH

10.0 pH

pH Meter ID: 375

REMARKS

TIMELY ENGINEERING SOIL TESTS, LLC

1874 Forge Street Tucker, GA 30084
Phone: 770-938-8233
Fax: 770-923-8973
Web: www.test-llc.com

Tested By	RI
Date	04/08/16
Checked By	[signature]

Client Pr. #	160205.20	Lab. PR. #	1607A-13-1
Pr. Name	Indian Creek WRF Expansion	S. Type	Bag
Sample ID	21606/Boring P-15	Depth/Elev.	-
Location	P-15	Add. Info	-

ASTM G 57/G187/AASHTO T 288
Standard Test Method for Determining Minimum Laboratory Soil Resistivity

Determination of Resistivity at as-received moisture content

As-received Moisture Content	
Mass of Wet Sample & Tare, g	
Mass of Dry Sample & Tare, g	
Mass of Tare, g	
Moisture Content, %	NA

Remarks

TEST DATA

Mass of Soil Box, g	-	Meter Dial Reading, ohms	-
Mass of Soil Box + Soil, g	-	Reading of Meter Range Multiplier	-
Mass of Soil, g	-	Measured Resistance, ohms	-
Calibrated Volume of Soil Box, ft^3	0.0027	Calibrated Soil Box Multiplier, cm	1.0
Wet Density of as-placed Soil, pcf	-		
Dry Density of as-placed Soil, pcf	-	**Reported Soil Resistivity, ohms-cm**	**NA**

Determination of Minimum Soil Resistivity

TEST DATA

	Trials at Various Moisture Content								
TRIAL #	1	2	3	4	5	6	7	8	9
Meter Dial Reading, ohms	73.1	57.1	49	45.8	44.7	40.2	40.2		
Reading of Meter Range Multiplier	K	K	K	K	K	K	K		
Measured Resistance, ohms	73100	57100	49000	45800	44700	40200	40200		
Calibrated Soil Box Multiplier, cm	1.0	1.0	1.0	1.0	1.0	1.0	1.0		
Measured Resistivity, ohms-cm	73100	57100	49000	45800	44700	40200	40200		

Reported Soil Minimum Resistivity, ohms-cm **40200**

Note: Material passed # 10 sieve used for testing

Oven ID #	496/610
Balance ID #	563/700
Soil Box ID #	612/613/707
Resistivity Meter ID #	706

Description

NA

USCS (D2487; D2488)	NA
AASHTO (M145)	NA

TIMELY ENGINEERING SOIL TESTS, LLC
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Phone: 770-938-8233
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Web: www.test-llc.com

Tested By	EB
Date	04/08/16
Checked By	[signature]

Client Pr. #	160205.20	Lab. PR. #	1607A-13-1
Pr. Name	Indian Creek WRF Expansion	S. Type	Bag
Sample ID	21606/Boring P-15	Depth/Elev.	-
Location	P-15	Add. Info	-

ASTM G200

Standard Test Method for Measurement of Oxidation Reduction Potential (ORP) of Soil

SAMPLE PREPARATION

Roots, Stones, Gravel and other deleterious material was removed prior to testing

Measurements performed ar room temperature condition: 20.1 °C

TEST DATA

T.E.S.T. Sample ID	Client Sample ID	ORP meter Reading #1, mV	ORP meter Reading #2, mV	ORP meter Reading #3, mV	Reported ORP value, mV
21606	Boring P-15	388.0	414.0	403.0	**402**

Standard ORP calibration solution (420+/-mV) used to standardize ORP meter: P.D.1/21/15, Exp.10/16

REMARKS

ORP Meter ID	375
ORP Probe ID	417

EXHIBIT "A"

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.

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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.

GEO HYDRO ENGINEERS