ATTACHMENT 3

REPORT OF SUBSURFACE EXPLORATION AND GEOTECHNICAL ENGINEERING EVALUATION

TRILITH STUDIOS ABOVE GROUND STORAGE TANK
461 SANDY CREEK ROAD
FAYETTEVILLE, GEORGIA

Oasis Project No. 224927

Prepared For:
Arcadis
2839 Paces Ferry Road SE
Suite 900
Atlanta, Georgia 30339

Prepared By:
Oasis Consulting Services
45 Woodstock Street
Roswell, Georgia 30075

October 4, 2022

October 4, 2022

Arcadis
2839 Paces Ferry Road SE
Suite 900
Atlanta, Georgia 30339

Attention: Mr. Travis Thomas

Subject: **Report of Subsurface Exploration and Geotechnical Engineering Evaluation**
Trilith Studios Above Ground Storage Tank
461 Sandy Creek Road
Fayetteville, Georgia
Oasis Project No. 224927

Dear Travis:

Oasis Consulting Services (Oasis) is pleased to provide this report of our subsurface exploration and geotechnical engineering evaluation for the above referenced project. The field study and this report were accomplished in general accordance with Oasis Proposal No. P22082 dated May 25, 2022.

The following report presents a brief summary of our pertinent findings and recommendations followed by our understanding of the proposed construction, methods of exploration employed, site and subsurface conditions encountered, and conclusions and recommendations regarding the geotechnical aspects of the project. We request that we be provided with a copy of the approved plans for review to verify that the design recommendations are incorporated into the design. We will also be able to make supplemental recommendations to address conditions that were not known at the time this report was prepared.

Should you have any questions regarding items discussed in this report, please do not hesitate to contact the undersigned.

Sincerely,
Oasis Consulting Services

Andrew W. Graff, E.I.T.
Project Engineer
GA Registration #: EIT027799

GEORGIA REGISTERED No. PE038504 PROFESSIONAL ENGINEER DARREN J. BRAY
10/4/22

Darren J. Bray, P.E.
Technical Director
GA Registration #: PE038504

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1.0 SUMMARY

A brief summary of pertinent findings, conclusions and recommendations are presented below. This information should not be utilized in design without first referring to the more detailed expansion of these ideas presented in the text of this report.

1.1. For the purpose of this report, we anticipate the construction of a 0.25-to-0.5-million-gallon elevated water storage tank for Fayette County. The water storage tank will be located on a 0.6-acre site within the Trilith Studios development in Fayetteville, Georgia. No other details of the proposed construction were available at the time this report was prepared.

1.2. General subsurface conditions encountered by the borings consist of topsoil, aggregates, residual soils, partially weathered rock (PWR), and groundwater. Boring B-3 encountered approximately 3 inches of surficial topsoil and associated root zone. Borings B-1 and B-2 encountered about 7 to 18 inches of surficial gravel or gravel/soil mix. Residual soils were encountered in all soil test borings below the surficial topsoil or gravel layer. The residuum extended to partially weathered rock or to the planned boring termination depths of up to 100 feet below existing grade. Partially weathered rock was encountered in boring B-2 and at a depth 82 feet below existing grade and extended to the boring termination depth of 100 feet below existing grade. A PWR-like material was reported by the drill operator at a depth of approximately 90 feet below existing grade in boring B-3; however, SPT sampling was not performed due to excessive water pressure. Groundwater was encountered at the time of drilling in all soil test borings at depths ranging from 38 to 43 feet below existing grade. After varying delay periods, the groundwater levels were remeasured and found to range from 29 feet to 37 feet below existing grades.

1.3. **Based on the anticipated foundation loads and the subsurface conditions encountered, shallow foundation support of the structure is not a suitable option. We anticipate supporting the proposed elevated water storage tank on a conventional deep foundation system such as auger cast in place piles (ACIPs) or driven piles. Once final loads are established, we can provide further information regarding foundation types, capacities, and expected settlements.**

1.4. Another concern is related to the presence of elastic silts (MH) encountered in the upper 5 feet of boring B-2. Elastic soils may be reused as structural fill but should not be placed immediately beneath footings, slabs, and/or paved areas due to their moisture sensitivity and potential volume change. We recommend maintaining a minimum of 24-inch-thick layer of separation between the bottom of slabs-on-grade and a 12-inch-thick layer of separation between pavements and shallow footings and the top of the elastic silt layer. After mass grading, where elastic material exists near footings, slabs, or pavement elevations, it should be undercut and replaced with approved structural fill (PI less than 30) or compacted graded aggregate base stone. Moisture conditioning will be a critical factor in achieving minimum density criteria, as such, we recommend grading take place during warmer times of the year

1.5. For slab-on-grade design, we recommend a modulus of subgrade reaction of 125 pci. An Oasis geotechnical engineer should carefully evaluate all subgrade conditions prior to fill placement or at-grade construction. In the event that soft soils or materials containing deleterious materials are encountered in other areas at the time of construction, typical recommendations would include undercutting and replacing with structural fill or stabilizing in place.

1.6. The on-site residual soils visually appear suitable for reuse as structural fill. Based on the local geology, the on-site soils are typically moisture sensitive and may be more problematic to work with should earthwork operations take place during periods of wet weather. Moisture control may be necessary, primarily depending on the weather conditions at the time of construction. In addition, a significant amount of the residual soils encountered at depths greater than about 18 feet below the existing ground surface were noted as wet. If wet soils are excavated for reuse as structural fill, drying of these soils will likely be required prior to reuse.

1.7. An Oasis geotechnical engineer should carefully evaluate all subgrade conditions prior to fill placement or construction. If soft soils or materials containing deleterious materials are encountered in other areas at the time of construction, typical recommendations would include undercutting and replacing with structural fill or stabilizing in place. New fill should be compacted to 95 percent of the standard Proctor (ASTM D 698) maximum dry density. Compaction of the subgrade immediately beneath grade slabs and pavements should be increased to 98 percent.

1.8. The site class is based on the Site Class Definitions in Section 1613.2.2, Design Spectral Response Acceleration Parameters in Section 1613.2.4, and Determination of Seismic Design Category in Section 1613.2.5 of the 2018 International Building Code. Based on the standard penetration resistance data from the borings, we recommend that **Site Class "D"** be used for the seismic design considerations.

1.9. Additional recommendations relative to earth pressures, slopes, site preparation, and foundation construction are discussed in the report.

2.0 PROPOSED CONSTRUCTION

Based on our correspondence with you, we understand that you are planning to provide design services for a 0.25-to-0.5-million-gallon elevated water storage tank for Fayette County. The water storage tank will be located on a 0.6-acre site within the Trilith Studios development in Fayetteville, Georgia. No other details of the proposed construction were available at the time this report was prepared.

3.0 METHODS OF EXPLORATION

To evaluate the subsurface conditions, the property was explored by a combination of a visual site reconnaissance and drilling a total of three (3) soil test borings to depths of 100 feet below the existing grade. The borings were located in the field using handheld GPS and by measuring distances and estimating directions from identifiable site features. Therefore, their locations as shown on the Boring Location Plan in the Appendix should be considered approximate.

The borings were advanced using a power rotary drill and twisting continuous hollow stem auger flights into the ground. At selected intervals, Standard Penetration Tests (SPT) were performed in general accordance with ASTM standard D-1586 by driving a standard 1-$^3/_8$" I.D. (2" O.D.) split spoon sampler with an automatic 140-pound hammer falling 30 inches. The number of hammer blows needed to drive the sampler 18 inches, in 6-inch increments, was recorded. The Standard Penetration Test value or "N" value is the summation of the last two 6-inch increments and is shown on the boring logs adjacent to their corresponding depths. In very dense soils or partially weathered rock, the sampler is driven a few inches instead of the 6-inch increment and the number of blows needed versus the penetration depth is recorded. The results of the penetration tests, when properly evaluated, provide an indication of the relative consistency of the soil being sampled, the potential for difficult excavation, and the soil's ability to support loads.

Soil samples recovered during the drilling process were returned to Oasis' lab where they were visually classified in general accordance with the Unified Soil Classification System (USCS). Detailed descriptions of the materials encountered at each boring location, along with a graphical representation of the Standard Penetration Test results, are shown on the Boring Logs in the Appendix.

Elevations on the Boring Logs were interpolated from available Fayette County GIS maps and should be considered approximate. If encountered, groundwater depth was measured at the time of drilling, at completion of drilling, and, if possible, after a 24-to-96-hour delay.

4.0 SITE DESCRIPTION, GEOLOGY AND SUBSURFACE CONDITIONS

4.1 SITE DESCRIPTION

The site is located within the Trilith Studios development east of the intersection of Veterans Parkway and Iver Place in Fayetteville, Georgia. At the time of our field work, the site was cleared with a mix of grass and scrub vegetation. The site is primarily flat with steep slopes along the south and east peripheries.

4.2 GEOLOGY

The site is located in the Piedmont Physiographic Province of Georgia, an area underlain by ancient igneous and metamorphic rocks. The residual soils in the Piedmont are the result of the chemical and physical weathering of the underlying parent rock. The weathering profile usually results in fine-grained clayey silts and silty clays near the surface, where weathering is more advanced. With depth, sandy silts and silty sands are found, often containing mica. Below the residual soils, partially weathered rock is often found as a transition above relatively unweathered rock. In local practice, partially weathered rock is arbitrarily defined as residual soils with Standard Penetration Resistances in excess of 100 blows per foot (50 blows per 6 inches), and which can be penetrated by a power auger. The upper surface of bedrock is generally very erratic and the depth at which bedrock is encountered can vary greatly. Typically, bedrock is encountered at shallow to moderate depths. This typical profile can be altered by the process of erosion and deposition and recent development.

4.3 SUBSURFACE CONDITIONS

The subsurface conditions encountered during this study are generally typical of those described in the previous geology section of this report. Topsoil, gravel, residual soils, partially weathered rock (PWR), and groundwater were encountered in the soil test borings. The following briefly describes the subsurface conditions encountered.

4.3.1 TOPSOIL

Topsoil is a dark-colored surficial material with a high organic content and is generally unsuitable for structural support. Boring B-3 encountered approximately 3 inches of surficial topsoil and associated root zone. Measurable amounts of surficial topsoil were not encountered at the remaining boring locations, likely due to previous grading activities. Some variation in topsoil thickness should be anticipated during site stripping operations.

4.3.2 AGGREGATES

Borings B-1 and B-2 encountered about 7 to 18 inches of surficial gravel or gravel/soil mix.

4.3.3 RESIDUUM

Residuum is a term used to define soils formed in-place by the chemical and physical weathering process of the underlying rocks. Residual soils were encountered in all soil test borings below the surficial topsoil or gravel layer. The residuum extended to partially weathered rock or to the planned boring termination depths of up to 100 feet below existing grade. The residuum was typically classified as silty sand (SM), sandy silt (ML), and/or clayey sand (SC) with varying amounts of mica. Most of the residual soils below depths of about 22 feet were noted as wet.

Standard Penetration Test results ranged from 0 to 40 bpf with typical values ranging between 10 and 18 bpf. Based on SPT results, the consistency of the residuum would be considered low to moderately low consistency.

4.3.4 PARTIALLY WEATHERED ROCK

Partially weathered rock (PWR) is defined as residual material exhibiting standard penetration resistances of 50 blows per 6 inches or less penetration that can be penetrated by a power auger. Partially weathered rock is generally a transition zone between residual soils and bedrock. Partially weathered rock was encountered in boring B-2 and at a depth 82 feet below existing grade and extended to the boring termination depth of 100 feet below existing grade. The PWR was generally sampled as very dense silty sand (SM). A PWR-like material was reported by the drill operator at a depth of approximately 90 feet below existing grade in boring B-3; however, SPT sampling could not be performed due to excessive water pressure.

4.3.5 GROUNDWATER

Groundwater was encountered at the time of drilling in all soil test borings at depths ranging from 38 to 43 feet below existing grade. After varying delay periods, the groundwater levels were remeasured and found to range from 29 feet to 37 feet below existing grades. All three boreholes were found to have caved at depths ranging from 41 feet to 53 feet below existing grade. The caved depths are generally an indication of the elevation of stabilized groundwater. Fluctuations in measured groundwater elevations of 5 feet or more are common in this geology due to seasonal fluctuations and could be encountered at higher elevations in the future.

Table 1: Soil Test Boring Summary

Boring ID	Existing Elev. (ft)	Boring Depth (ft)	PWR Depth (ft)	Refusal Depth (ft)	Groundwater Depth - initial (ft)	Groundwater Depth - final (ft)
B-1	872	100	-	-	38	36
B-2	874	100	82	-	43	37
B-3	875	100	90	-	40	32

- Not Encountered

4.4 LABORATORY TEST RESULTS

The laboratory test results indicate that most of the soils encountered at this site are classified as silty sands (SM) or sandy silts (ML/MH). The percentage of fines (% passing #200 sieve) ranged from 38.2 to 67.7 percent. The results of the Atterberg Limit testing indicate that four (4) of the five (5) samples were non-plastic soils. The remaining Atterberg Limit test indicated a Liquid

Limit (LL) of 65 and a Plasticity Index (PI) 18. As a result, the soils range from non-plastic to highly plastic.

A copy of the laboratory testing procedures is provided in Appendix B. A summary of the Laboratory test results and test reports are included in Appendix D of this report.

The conditions described in the preceding paragraphs, and those shown in the Appendix, have been based on interpolation of the results of the previously described data using generally accepted principles and practices of geotechnical engineering. However, conditions in this geology may vary intermediate of the tested locations and even more so on previously filled property.

Although individual soil test borings are representative of the subsurface conditions at the precise boring and test pit locations on the day(s) performed, they are not necessarily indicative of the subsurface conditions at other locations or other times. The nature and extent of variation between the borings and test pits may not become evident until the course of construction. If such variations are then noted, it will be necessary to reevaluate the recommendations of this report after on-site observation of the conditions.

5.0 CONCLUSIONS AND RECOMMMENDATIONS

The following conclusions and recommendations are based on the data gathered during this exploration, our understanding of the proposed construction, our experience with similar site and subsurface conditions and generally accepted principles and practices of geotechnical engineering. Should the proposed construction change significantly from that described in this report, we request that we be advised so that we may amend these recommendations accordingly. This report, and the conclusions and recommendations provided herein, are provided exclusively for the use of Arcadis and their design team and is intended solely for design of the referenced project.

5.1 GENERAL

Although structural loads have not been provided to Oasis, based on the nature of the project we expect foundation loads will be significant. As such, we anticipate the use of deep foundation elements to support the planned elevated storage tank. Based on the boring data, we do not anticipate end-bearing support of the deep foundation elements above 100 feet below existing grade.

Another concern is related to the presence of elastic silts (MH) encountered in the upper 5 feet of boring B-2. Elastic soils may be reused as structural fill but should not be placed immediately beneath footings, slabs, and/or paved areas due to their moisture sensitivity and potential volume change. We recommend maintaining a minimum of 24-inch-thick layer of separation between the bottom of slabs-on-grade and a 12-inch-thick layer of separation between pavements and shallow footings and the top of the elastic silt layer. After mass grading, where elastic material exists near footings, slabs, or pavement elevations, it should be undercut and replaced with approved structural fill (PI less than 30) or compacted graded aggregate base stone. Moisture conditioning will be a critical factor in achieving minimum density criteria, as such, we recommend grading take place during warmer times of the year.

The on-site residual soils visually appear suitable for reuse as structural fill. Based on the local geology, the on-site soils are typically moisture sensitive and may be more problematic to work with should earthwork operations take place during periods of wet weather. Moisture control may be necessary, primarily depending on the weather conditions at the time of construction. In addition, a significant amount of the residual soils encountered at depths greater than about 18 feet below the existing ground surface were noted as wet. If wet soils are excavated for reuse as structural fill, drying of these soils will likely be required prior to reuse.

5.2 SITE PREPARATION

As an initial step in site preparation, all trees and unwanted vegetation should be removed, stumps grubbed, and organic topsoil stripped in all areas of at-grade construction or areas to receive fill. Existing utilities should be rerouted around the proposed building location or removed so as not to negatively impact the new development. Any excavations created to demolish existing utilities should be properly backfilled according to the earthwork recommendations contained in this report.

Subgrades should be evaluated by an Oasis geotechnical engineer prior to at-grade construction or fill placement. The evaluation process should include proofrolling the subgrade with a fully loaded tandem axle dump truck (20 tons) during a period of dry weather and under the observation of the geotechnical engineer. Any areas which "pump" or "rut" excessively under the weight of the proofrolling vehicle should be further evaluated. After evaluation by Oasis, remedial options could include recompaction, undercutting and replacing with soil and/or rock, partial over-excavation with geogrid placement, or drying and recompaction. Proofrolling can occasionally detect pits where stumps or other debris may have been buried, or other areas where weak surface conditions exist. If encountered, weak soils should be evaluated by Oasis and remedial options could include replacing with structural fill or compacted crushed stone. As needed, backhoe test pits or hand augers with Dynamic Cone Penetrometer (DCP) testing can be used to delineate any unsuitable material found during proofrolling.

5.3 EARTHWORK

The residual soils on the property appear suitable for reuse as structural fill based on visual examination. Any topsoil or otherwise organic-laden soils may be reused in non-structural areas of the site such as landscape areas or slopes. Most of the residual soils encountered contain varying amounts of mica. These soils are typically more moisture sensitive and can often be problematic to work with especially during periods of wet weather. In addition, we anticipate that a significant amount of the residual soils encountered below a depth of approximately 18 feet may contain excessive moisture contents and will likely require adequate drying prior to reuse as structural fill.

Where fill is placed against slopes steeper than 5H:1V, it will be necessary to "bench" the new fill into the existing soils to insure an adequate bonding of the fill with the existing material. Inadequate benching may create a predefined plane of weakness and adversely affect slope stability.

All structural fill should be compacted to at least 95 percent of the soil's standard Proctor maximum dry density, as determined by ASTM standard D-698. The upper one foot of fill which will support structures, pavements or slabs-on-grade should be compacted to at least 98 percent of the soil's standard Proctor maximum dry density for improved support. Further, the fill material should have a maximum dry density of 90 pcf or above. In areas which are at or above the finished grade, and which will support pavements or slabs, the upper 8 inches immediately below these systems should be scarified and recompacted to the 98 percent criteria. Structural fill should be free of topsoil, organic materials, or highly plastic silts and clays, have a liquid limit (LL) less than 40 and a plasticity index (PI) less than 20 and contain rock sizes no larger than 4 inches. Unacceptable materials removed during grading operations should be either stockpiled for later use in landscaped areas or placed in approved disposal areas either on site or off site.

Fill operations should be observed on a full-time basis by an Oasis soils technician and density testing should be performed to determine the degree of compaction and to verify compliance with the project specifications. Fill materials should be placed in loose lifts not exceeding 8 inches and moisture conditioned to within 3 percent of the optimum moisture content to facilitate proper compaction. For underfloor areas, at least one field density test should be made per 2,500 square feet of fill area for each two-foot lift. Testing frequency should be increased in confined areas. Areas which do not meet the compaction specifications should be recompacted to achieve compliance. In confined areas, such as utility trenches, the use of portable compaction equipment and thin lifts of 3 to 4 inches may be required to achieve compaction.

5.4 GROUNDWATER CONTROL

Groundwater was encountered at depths ranging from approximately 29 feet to 43 feet below the existing ground surface and may be encountered at higher elevations in the future due to seasonal fluctuations.

If encountered, temporary groundwater control may be required in the lower elevations during excavations for underground utilities, foundations, and slabs. Groundwater should be properly controlled such that it is maintained 2 feet to 3 feet below the bottom of proposed excavations. Pumping from sumps in the excavations may suffice for limited depths of dewatering; however, deeper excavations may require systems such as deep wells or well points.

5.5 FOUNDATIONS

Based on the anticipated foundation loads and the subsurface conditions encountered, shallow foundation support of the structure is **not** a suitable option. **We anticipate supporting the proposed elevated water storage tank on a conventional deep foundation system such as auger cast in place piles (ACIPs) or driven piles. Once final loads are established, we can provide further information regarding foundation types, capacities, and expected settlements.**

An Oasis geotechnical engineer should observe deep foundation construction operations to verify that the foundation system is installed in accordance with the plans and specifications. Engineering inspection is considered critical to the success of the foundation system installation and performance.

We recommend the installation criteria for the piles be verified by an Oasis engineer by performing a load test in general accordance with ASTM D1143. The load test location should be selected after installing two probe piles throughout the water storage tank footprint. The probe piles would assist the pile contractor and geotechnical engineer in evaluating the equipment and pile response to the specific site conditions and in determining tentative installation criteria for the test pile. All production piles should be placed using the same procedures and equipment used for installation of the test pile. If ultimate uplift loads are to be in excess of 1/8th of the vertical capacity, a modified load test must be performed on a separate pile to verify tensile or uplift capacity.

It is recommended that the installation of the probe piles, test pile(s) and all production piles be monitored by a representative of Oasis. The installation of auger-cast piles should be sequenced such that adjacent piles within the same cap should not be constructed within the same 24-hour period. This is required to provide adequate time for curing. Minimum center-to-center spacing between driven piles should be three (3) times the maximum pile diameter.

5.6 SOIL SUPPORTED SLABS

After successful completion of the site preparation measures slabs-on-grade may be soil supported. We recommend a modulus of subgrade reaction of 125 pci for use in the slab design. Due to the underlying low-consistency residual soils, we recommend a minimum of 6 inches of crushed stone beneath the slab to address slab performance issues. The crushed stone should be compacted to a minimum of 98% Modified Proctor Test (ASTM D698). Prior to at-grade construction, the subgrade soils should be evaluated by an Oasis representative. Unstable material should either be removed and replaced or scarified and recompacted. The extent of undercutting can be determined at the time of construction. We also recommend that a vapor barrier be placed beneath the slab to prevent the infiltration of soil moisture into finished areas.

5.7 TEMPORARY AND PERMANENT SLOPES

Temporary and permanent slopes may be used to accommodate grade changes. If temporary slopes are used, they should be constructed no steeper than 1.5H:1V for slopes less than 15 feet high. Permanent slopes should be constructed no steeper than 2H:1V for slopes less than 20 feet high. These recommendations are based on our experience with similar conditions and no detailed slope stability analyses have been performed. Further, these recommendations assume that no groundwater or seepage is present in the proposed slope location. If groundwater or seepage is present, then a detailed slope stability analyses will need to be performed. Likewise, if the maximum slope heights indicated in this section are exceeded then a detailed slope stability analysis will need to be performed. Buildings should be set back at least 10 feet from the top of slopes; a minimum 5-foot setback is considered sufficient for pavement areas. All finished slopes should be suitably protected from erosion.

5.8 LATERAL EARTH PRESSURES

Lateral earth pressures imposed on a retaining wall are a function of the soil properties, the inclination of the backfill behind the retaining wall, any surcharge loads applied behind the wall, and the amount of deflection the wall system can undergo. Lateral earth pressures developed from the "active" condition are applicable for design of temporary or permanent free-standing retaining walls, if adequate wall movement can occur to fully mobilize the shear strength of the retained soil. Permanent laterally restrained walls, such as basement walls, should be designed for pressures using the full "at-rest" case. The following equivalent fluid pressures are based on our experience and correlations with our field testing. Site specific laboratory soil strength testing was not performed for this project. However, based on the conditions found, the following equivalent fluid pressures are recommended using a horizontal backfill configuration with no surcharge loads and providing "typical" Piedmont soils (silty sand and sandy silt) are used for backfill. We assume the soils have a moist unit weight of 120 pounds per cubic foot (pcf), an angle of internal friction (ϕ) of 28 degrees and a sliding coefficient of friction of .45 x N where N is the vertical force component of the foundation system per linear foot. For concrete on soil, a sliding coefficient of friction of 0.53 may be used in the *ultimate design value* of the retaining wall.

Earth Pressure Condition	Earth Pressure Value	Recommended Equivalent Fluid Pressure (psf/f) Above Groundwater	Recommended Equivalent Fluid Pressure (psf/f) Below Groundwater
Active (K_A)	0.36	45	85
At-Rest (K_O)	0.53	60	90
Passive (Kp)	2.77	160*	160*

*safety factor of at least 2 for material properties and service criterion

Heavy compaction equipment should not be used to compact backfill immediately behind any retaining wall, unless the wall is designed for the increased pressure. Retaining wall backfill should be compacted to at least 95% of the soil's standard Proctor maximum dry density; therefore hand operated compaction equipment will be necessary in these areas. Areas exposed to groundwater or surface infiltration of water should include a properly filtered footing and wall drain. The drain should include a perforated schedule 40 PVC pipe, placed in clean crushed stone, encapsulated in a 4-ounce needle-punched nonwoven filter fabric.

For structures supported on shallow foundations, lateral loads can be resisted by passive pressures against the face of the foundation or sliding resistance on the base of the footing. Because significant wall movements are required to develop the passive pressure, the recommended passive equivalent fluid pressure (160 psf/f) is one-half of the total calculated passive pressure, a safety factor of at least 2. Additional resistance to movement can be gained by developing sliding friction on the base of the footing and an allowable friction factor of 0.35 may be used. This includes a factor of safety of about 1.5. If the structural engineer is designing according to the International Building Code (IBC) 2018, the structural engineer can increase the values for passive pressure and sliding friction factor to 250 psf and 0.4, respectively. These values have a factor of safety for material properties but no service criterion factor of safety since the service criterion factor of safety is accounted for in the structural calculations per the IBC code.

5.9 SEISMIC SITE CLASSIFICATION

We have been asked to provide the Site Class as defined by the *International Building Code 2018* as adopted by the State of Georgia. The following recommendations are based on the Site Class Definitions in Section 1613.2.2, Design Spectral Response Acceleration Parameters in Section

1613.2.4, and Determination of Seismic Design Category in Section 1613.2.5 of the *2018 International Building Code and Section 11.4.7 of ASCE 7.* Based on the boring data, the site does not correspond to any of the categories for Site Class "F". Since there is not a total thickness of soft clay greater than 10 feet, the site does not meet the requirements for Site Class "E". Therefore, based on the standard penetration resistance data from the borings, we recommend that **Site Class "D"** be used for seismic design considerations.

5.10 LIQUEFACTION POTENTIAL AT THE SITE

Under cyclic loading (i.e., during an earthquake) loose non-cohesive materials (gravels, sands, silty-sands) tend to decrease in volume. This tendency to decrease in volume is much greater in loose than dense soils. When loose non-cohesive soils are saturated and rapid loading occurs under undrained conditions, the soil densification causes excess pore water pressure to increase. The increase in pore water pressure results in a loss of soil strength due to a decrease in effective stress and eventually liquefaction occurs, once the effective stress drops to zero. Liquefaction of loose sands can lead to large displacements of foundations, flow failures of slopes, ground surface settlement, sand boils, and post-earthquake stability failures.
It is our opinion that the potential for liquefaction of the native soils at the site due to earthquake activity is relatively low based on the information obtained from the soil test borings.

6.0 QUALIFICATIONS OF RECOMMENDATIONS

This evaluation of the geotechnical aspects of the proposed design and construction has been based on our understanding of the project and the data obtained during this study. The general subsurface conditions used in our evaluation were based on interpolation of the subsurface data between the borings. Regardless of the thoroughness of a subsurface exploration, there is the possibility that conditions will differ between boring locations, that conditions are not as anticipated by the designers, or that the construction process has modified the soil conditions. Therefore, experienced Oasis soil engineers and technicians should evaluate earthwork and foundation construction to verify that the conditions anticipated in design actually exist. Otherwise, we assume no responsibility for construction compliance with the design concepts, specifications or recommendations.

The recommendations contained in this report have been developed on the basis of the previously described project characteristics and subsurface conditions. If project criteria change, we should be permitted to determine if the recommendations should be modified. The findings of such a review will be presented in a supplemental report. Even after completion of a subsurface study, the nature and extent of variation between borings may not become evident until the course of construction. If such variations then become evident, it will be necessary to reevaluate the recommendations of this report after on-site observations of the conditions.

These professional services have been performed, the findings derived, and recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices. This warranty is in lieu of all warranties either expressed or implied. This company is not responsible for the conclusions, opinions or recommendation of others based on these data.

APPENDIX A

SITE VICINITY MAP AND BORING LOCATION PLAN

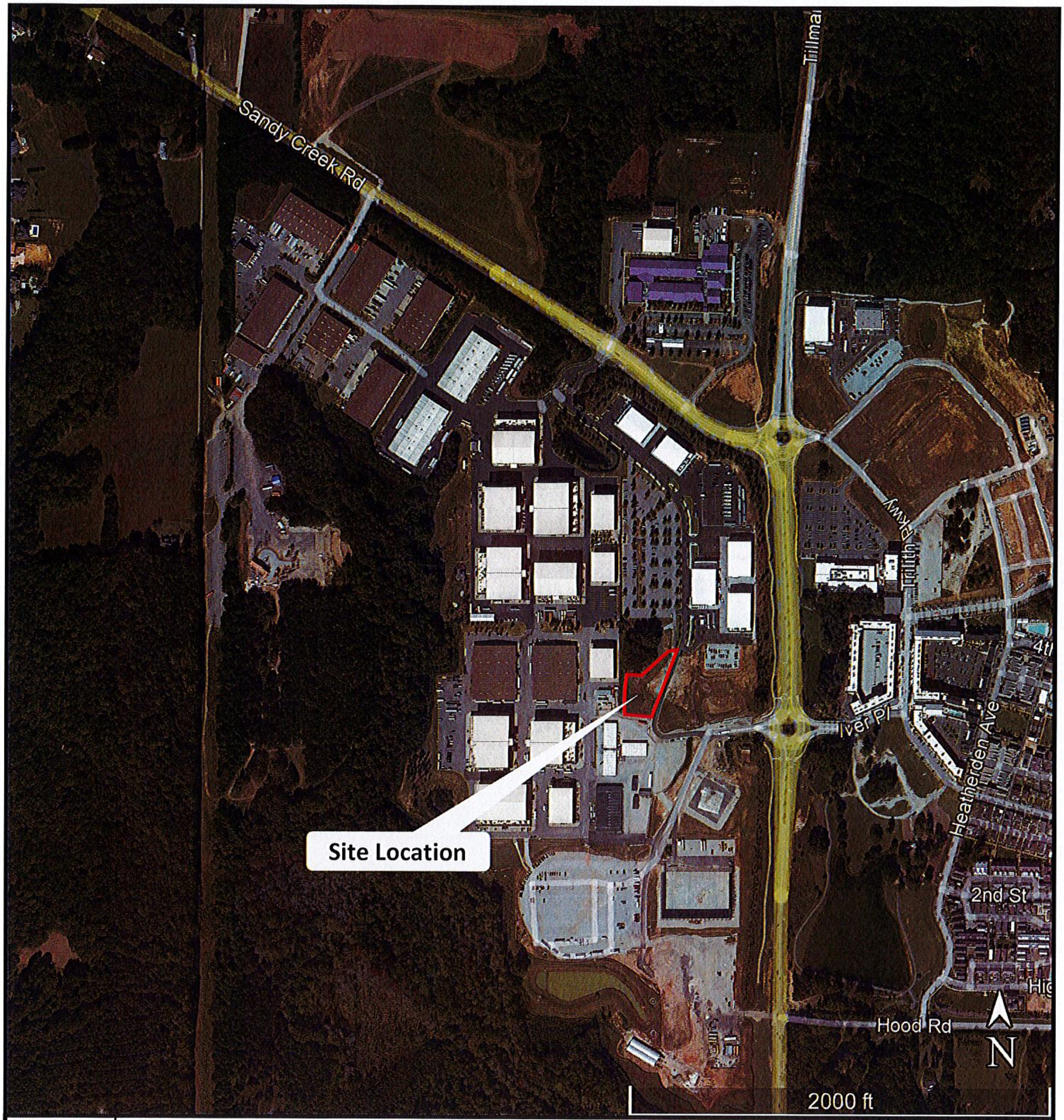

Figure No.:	SITE VICINITY MAP		
1	Source: Google Earth	FCWS Elevated Storage Tank – Trilith Studios 461 Sandy Creek Road Fayetteville, Georgia	OASIS CONSULTING SERVICES
	Oasis Project No.: 224927	Scale: Map Scale	**Oasis Consulting Services** **45 Woodstock Street** **Roswell, Georgia 30075**
	Date drawn: 08/2022	Drawn By: AG	

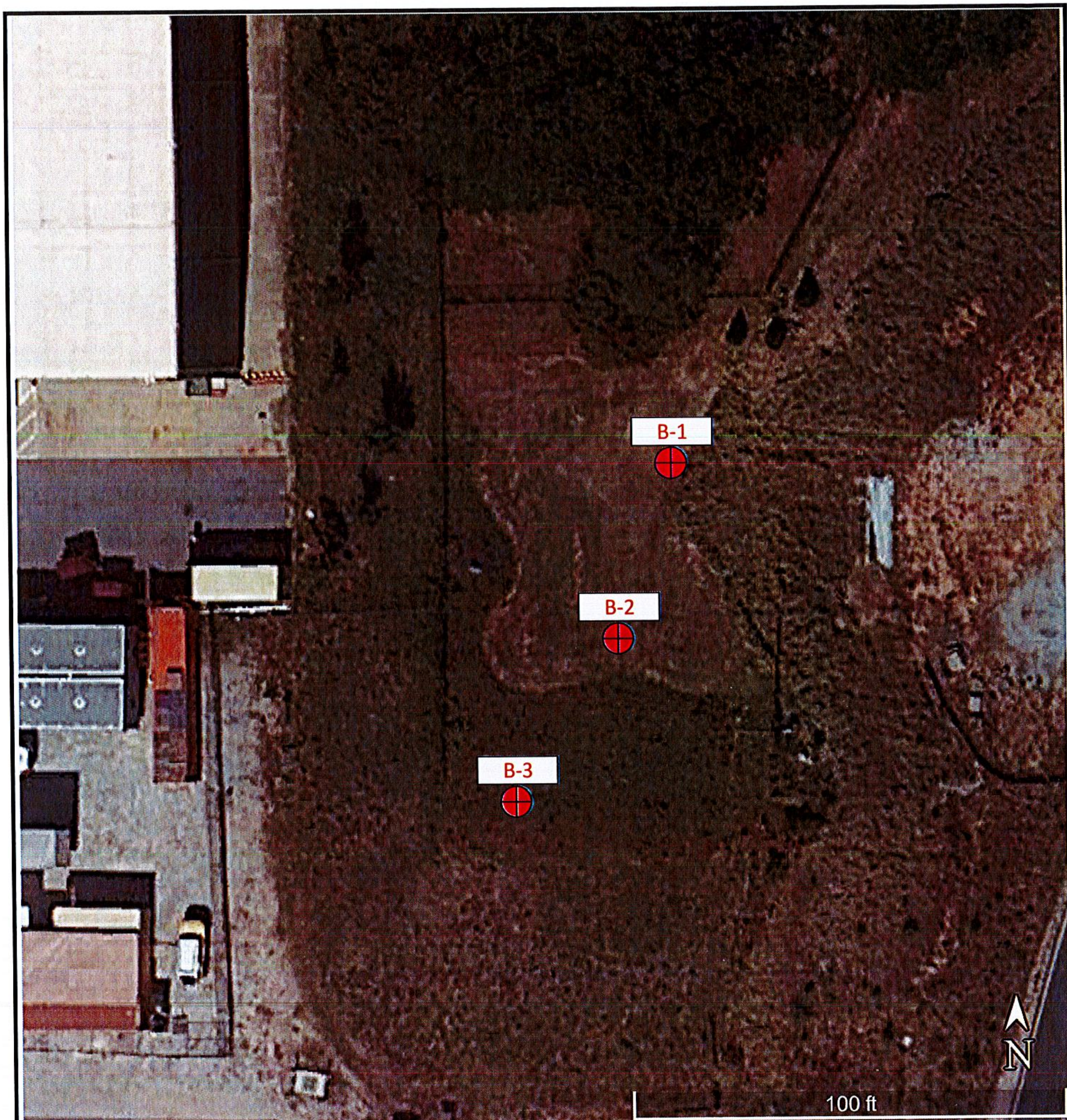

Figure No.:	BORING LOCATION PLAN			
2	Source: Google Earth = Boring Location	FCWS Elevated Storage Tank – Trilith Studios 461 Sandy Creek Road Fayetteville, Georgia		OASIS CONSULTING SERVICES
	Oasis Project No.: 224927	Scale: Map Scale	**Oasis Consulting Services** **45 Woodstock Street** **Roswell, Georgia 30075**	
	Date drawn: 08/2022	Drawn By: AG		

APPENDIX B

FIELD TEST PROCEDURES AND LABORATORY TEST PROCEDURES

TEST PROCEDURES

The general field procedures employed by Oasis Consulting Services (OCS) are summarized in the American Society for Testing and Materials (ASTM) Standard D 420 - Investigating and Sampling Soil and Rock. This practice lists recognized methods for determining soil, rock and groundwater conditions. These methods include geophysical and in-situ methods as well as borings.

Standard Drilling Techniques

To obtain subsurface samples, borings are drilled using one of several alternate techniques depending upon the subsurface conditions. Some of these techniques are:

In Soils:

a) Continuous hollow stem augers.
b) Rotary borings using roller cone bits or drag bits and water or drilling mud.
c) Hand augers.

In Rock:

a) Core drilling with diamond-faced, double or triple tube core barrels.
b) Core boring with roller cone bits.

Typical drilling methods used are presented in the following paragraphs.

Hollow Stem Augering: A hollow stem augers consists of a hollow steel tube with a continuous exterior spiral flange termed a flight. The auger is turned into the ground, returning the cuttings along the flights. The hollow center permits a variety of sampling and testing tools to be used without removing the auger.

Sampling and Testing in Boreholes

Several techniques are used to obtain samples and data in soils in the field; however the most common methods in this area are:

a) Standard Penetrating Testing
b) Undisturbed Sampling
c) Dynamic Cone Penetrometer Testing
d) Water Level Readings

The procedures utilized for this project are presented below.

Standard Penetration Testing: At regular intervals, soil samples are obtained with a standard 2-inch diameter split tube sampler connected to an A or N-size rod. The sampler is first seated 6 inches to penetrate any loose cuttings and then driven an additional 12 inches with blows of a 140-pound hammer falling 30 inches. Generally, the number of hammer blows required to drive the sampler the final 12 inches is designated the "penetration resistance" or "N" value, in blows per foot (bpf). The sampler is designed to retain the soil penetrated, so that it may be returned to the surface for observation. Representative portions of the soil samples obtained from each sample are placed in jars, sealed and transported to our laboratory.

The standard penetration test, when properly evaluated, provides an indication of the soil strength and compressibility. The tests are conducted according to ASTM Standard D1586. The depths and N-values of standard penetration tests are shown on the Boring Logs. Split tube samples are suitable for visual observation and classification tests but are not sufficiently intact for quantitative laboratory testing.

Water Level Readings: Water table readings are normally taken in the borings and are recorded on the Boring Logs. In sandy soils, these readings indicate the approximate location of the hydrostatic water table at the time of our field exploration. In clayey soils, the rate of water seepage into the borings is low and it is generally not possible to establish the location of the hydrostatic water table through short term water level readings. Also, fluctuation in the water table should be expected with variations in precipitation, surface run-off, evaporation, and other factors. For long-term monitoring of water levels, it is necessary to install piezometers.

The water levels reported on the Boring Logs are determined by field crews immediately after the drilling tools are removed, and several hours after the borings are completed, if possible. The time lag is intended to permit stabilization of the groundwater table which may have been disrupted by the drilling operation.

Occasionally the borings will cave-in, preventing water level readings from being obtained or trapping drilling water above the cave-in zone. The cave-in depth is measured and recorded on the Boring Logs.

Boring Logs

The subsurface conditions encountered during drilling are reported on a field boring log prepared by the driller or an OCS representative. The log contains information concerning the boring method, samples attempted and recovered, indications of the presence of coarse gravel, cobbles, etc., and observations of groundwater. It also contains the field representative's interpretation of the soil conditions between samples. Therefore, these boring records contain both factual and interpretive information. The field boring records are kept on file in our office.

After the drilling is completed, a geotechnical engineer or geologist classifies the soil samples and prepares the final Boring Logs, which are the basis for our evaluations and recommendations.

Soil Classification

Soil classifications provide a general guide to the engineering properties of various soil types and enable the engineer to apply his past experience to current problems. In our investigations, samples obtained during drilling operations are examined in our laboratory and visually classified by an engineer or geologist. The soils are classified according to consistency (based on number of blows from standard penetration tests), color and texture. These classification descriptions are included on our Boring Logs.

The classification system discussed above is primarily qualitative and for detailed soil classification two laboratory tests are necessary; grain size tests and plasticity tests. Using these test results the soil can be classified according to the AASHTO or Unified Classification Systems (ASTM D-2487). Each of these classification systems and the in-place physical soil properties provides an index for estimating the soil's behavior. The soil classification and physical properties are presented in this report.

The Key to Symbols and Classifications presents criteria that are typically used in the classification and description of soil and rock samples for preparation of Boring Logs.

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LABORATORY TEST PROCEDURES

Soil Compaction

Compaction tests are run on representative soil samples to determine the dry density obtained by a uniform compactive effort at varying moisture contents. The results of the test are used to determine the moisture content and unit weight desired in the field for similar soils. Proper field compaction is necessary to decrease future settlements, increase the shear strength of the soil and decrease the permeability of the soil.

The two most commonly used compaction tests are the standard Proctor test and the modified Proctor test. They are performed in accordance with ASTM Standards D698 and D1557, respectively. Generally, the standard compaction test is run on samples from building areas and areas where small compaction equipment is anticipated. The modified compaction test is generally used for analyses of highways and other areas where large compaction equipment is expected. In both tests, dry portions of each soil are mixed with varying quantities of water and representative portions are placed in a mold and compacted with a compaction hammer. Each portion is compacted with exactly the same compactive effort. Both tests have four alternate methods.

In the standard Proctor test, compaction is achieved by twenty-five blows of a 5.5 pound hammer falling 12 inches on each of three equal layers in a 4 inch diameter, 1/30 cubic foot cylinder. The moisture content and unit weight (dry density) of each compacted sample is determined and a graph of the results is plotted with the optimum moisture content occurring at the maximum dry density.

California Bearing Ratio

The results of the previously described compaction tests were used in preparing samples for California Bearing Ratio (CBR) tests. CBR tests are performed in accordance with ASTM D1883. The CBR is a punching shear test that provides data that is a semi-empirical index of the strength and deflection characteristics of a soil correlated with pavement performance to establish design curves for pavement thickness. The test is performed on a 6-inch diameter, 5-inch thick sample of compacted soil confined in a steel cylinder. Before testing, the sample is soaked in water under a confining pressure roughly equivalent to the weight of the future pavement to determine the potential swelling and to simulate subgrade saturation in the field. A 1.95-inch diameter piston is then forced into the soil at a standard rate to determine the resistance to penetration. The CBR is the ratio, expressed as a percentage, of the load required to produce a 0.1-inch deflection of the test soil to that required to produce the same deflection in a standard crushed stone.

Moisture Content

The moisture content of soil is defined as the weight of water in a given soil mass divided by the weight of dry soil solids in the same mass. Natural moisture contents are determined in accordance with ASTM Standard D2216.

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Soil Plasticity
Representative samples of the soils were selected for Atterberg limits testing to determine the soil plasticity characteristics. The soil's Plasticity Index (PI) is representative of this characteristic and is bracketed by the Liquid Limit (LL) and the Plastic Limit (PL). The LL is the moisture content at which the soil will flow as a heavy viscous fluid, and is determined in accordance with ASTM D423. The PL is the moisture content at which the soil begins to lose its plasticity and is determined in accordance with ASTM D424.

Certain soils swell and shrink with increases and decreases in soil moisture. The PI is related to this potential volume change ability. When such volume changes occur in soils confined beneath foundations, structural deformations can be produced. Past experience has shown that soils having a PI of less than 30 are only slightly susceptible to volume changes. Soils having a PI greater than 50 are generally very susceptible to these volume changes. Soils with a PI between these limits have moderate volume change potential.

Grain Size/Gradation
Grain size tests are performed to determine the soil classification and the grain size distribution. The soil samples are prepared for testing according to ASTM D421 (dry preparation) or ASTM D2217 (wet preparation). The grain size distribution of soils coarser than a number 200 sieve (0.074 mm opening) is determined by passing the samples through a standard set of nested sieves. A sample of known weight is passed through the sequence of sieves with decreasing size of openings and the portions retained on each sieve weighed. Materials passing the number 200 sieve are suspended in water and the grain size distributing calculated from the measured settlement rate (hydrometer analysis). Hydrometer analysis determines the density of a suspension of soil at various times after agitation. Using Stokes's law, the particle size remaining suspended at each particular time is calculated and the corresponding density is a measure of the quantity of soil smaller than the computed size. Test results are presented in the form of percent finer versus grain size curves.

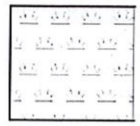
APPENDIX C

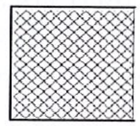
KEYS TO SYMBOLS AND CLASSIFICATIONS AND BORING LOGS

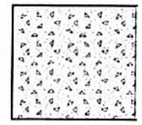
KEY TO SYMBOLS AND CLASSIFICATIONS

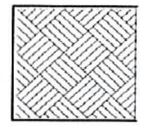
SYMBOL	TYPE OF SAMPLE
	Split Tube Sample (SPT)
	Shelby Tube Sample
	Bulk Sample
	Core Run

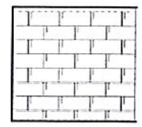

Asphalt


Topsoil


Fill


Partially Weathered Rock


Bedrock


Concrete

PARTICLE SIZE DEFINITIONS

COMPONENT	SIZE RANGE
Boulders	Larger than 12 inches
Cobbles	3 to 12 inches
Gravel	3 inches to 4.5 mm (Sieve No.4)
Coarse Gravel	3 inches to 3/4 of an inch
Fine Gravel	3/4 of an inch 4.5
Sand	4.5 mm to 0.074 mm (Sieves No.4 to No.200)
Coarse Sand	4.5 mm to 2.0 mm (Sieves No.4 to No.10)
Medium Sand	2.0 mm to 0.42 mm (Sieves No.10 to No.40)
Fine Sand	0.42 mm to 0.074 mm (Sieves No.40 to No.200)
Silt and Clay	Smaller than 0.074 mm (passing sieve No. 200)

MOISTURE CONTENT

Dry	Absence of moisture, dusty, dry to the touch
Damp	Some perceptible moisture, below optimum
Moist	No visible water, near optimum moisture content
Wet	Visible free water, usually soil is below water table

RELATIVE HARDNESS OF ROCK

Very Soft	Desintegrates or easily compresses to touch
Soft	May be broken with fingers
Moderately Soft	May be scratched with nail, edges may be broken with fingers
Moderately Hard	Light blow of hammer required to break sample
Hard	Hard blow of hammer required to break sample

ROCK CONTINUITY

DESCRIPTION	RQD*
Incompetent	Less than 40%
Competent	40% to 70%
Fairly Continuous	71% to 90%
Continuous	91% to 100%

*RQD=Rock Quality Designation

RELATIVE DENSITY OR CONSISTENCY VERSUS SPT N-VALUE

COHESIONLESS SOIL			COHESIVE SOILS		
Density	N (blows/ foot)	Approximate Relative Density (%)	Consistency	N (blows/foot)	Approximate Undrained Shear Strength (psf)
Very Loose	0 to 4	0 to 15	Very Soft	0 to 1	Less than 250
Loose	5 to 10	15 to 35	Soft	2 to 4	250 to 500
Medium Dense	11 to 30	35 to 65	Firm	5 to 8	500 to 1000
Dense	31 to 50	65 to 85	Stiff	9 to 15	1000 to2000
Very Dense	over 50	85 to 100	Very Stiff	16 to 30	2000 to 4000
			Hard	31 to 50	Greater than 4000
			Very Hard	over 50	

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISION			GRAPHIC SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	Clean Gravels (Little or no fines)		GW	Well graded gravels, gravel-sand mixtures, little or no fines
				GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO.4 SIEVE	Gravels with fines (Appreciable amount of fines)		GM	Silty gravels, gravel-sand-silt mixtures
				GC	Clayey gravels, gravel-sand-clay mixtures
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SANDS AND SANDY SOILS	Clean sands (Little or no fines)		SW	Well graded sands, gravelly sands, little or no fines
				SP	Poorly graded sands, gravelly sands, little or no fines
	MORE THAN 50% OF COARSE FRACTIONPASSING NO.4 SIEVE	Sands with fines (Appreciable amount of fines)		SM	Silty sands, sand-silt mixtures
				SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS	SILTS AND CLAYS	Liquid Limit less than 50		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
				CL	Inorganic clays of low to medium plasticity, gravely clays, sandy clays, silty clays, lean clays
				OL	Organic silts and organic silty clays of low plasticity
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	Liquid Limit greater than 50		MH	Inorganic silts, micaceous or diatomaceous fine sand or silty soils
				CH	Inorganic clays of high plasticity, fat clays
				OH	Organic clays of medium high plasticity, organic silts
HIGHLY ORGANIC SOILS				PT	Peat, humus, swamp soils with high organic contents

Note: Dual symbols are used to indicate borderline soil classifications

Oasis Consulting Services
45 Woodstock Street
Roswell, Georgia 30075
Telephone: (678) 739-2400

BORING NUMBER B-1

CLIENT Arcadis
PROJECT NAME FCWS Elevated Storage Tank - Trilith Studios
PROJECT NUMBER 224927
PROJECT LOCATION 461 Sandy Creek Road, Fayetteville, GA
DATE STARTED 8/5/22 **COMPLETED** 8/5/22
GROUND ELEVATION 872 ft **HOLE SIZE** 6
DRILLING CONTRACTOR Nicholson Exploration
GROUND WATER LEVELS:
DRILLING METHOD HSA-Auto Hammer
AT TIME OF DRILLING 38.00 ft / Elev 834.00 ft
LOGGED BY NE **CHECKED BY** BT
AT END OF DRILLING 33.00 ft / Elev 839.00 ft
NOTES Borehole caved at 41'
96hrs AFTER DRILLING 36.00 ft / Elev 836.00 ft

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ELEVATION (ft)	BLOW COUNTS (N VALUE)
0	AGGREGATE: Approximately 7 inches of gravel			
	(SC) RESIDUUM: Medium dense red brown clayey medium to fine SAND	SS 1	870	8-10-15 (25)
5		SS 2		4-8-10 (18)
	(SM) Medium dense red orange clayey silty medium to fine SAND, slightly micaceous	SS 3	865	11-5-7 (12)
10	(SM) Loose orange tan silty fine SAND, slightly micaceous	SS 4		4-3-7 (10)
	(ML) Stiff red tan fine sandy SILT, micaceous		860	
15		SS 5		4-3-6 (9)
			855	
20		SS 6		4-5-9 (14)
	(ML) Stiff tan orange fine sandy SILT, slightly micaceous, wet		850	
25		SS 7		6-5-6 (11)
			845	
30		SS 8		4-4-7 (11)
	(ML) Firm to stiff brown orange black fine sandy SILT, micaceous, very wet		840	
35		SS 9		3-3-4 (7)

SPT N VALUE: 20 40 60 80
PL MC LL: 20 40 60 80
FINES CONTENT (%): 20 40 60 80

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BORING NUMBER B-1

CLIENT Arcadis
PROJECT NAME FCWS Elevated Storage Tank - Trilith Studios
PROJECT NUMBER 224927
PROJECT LOCATION 461 Sandy Creek Road, Fayetteville, GA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ELEVATION (ft)	BLOW COUNTS (N VALUE)
35		(ML) Firm to stiff brown orange black fine sandy SILT, micaceous, very wet *(continued)*		835	
40			SS 10		4-4-7 (11)
		(SM) Medium dense brown tan silty medium to fine SAND, micaceous, very wet		830	
45			SS 11		5-5-6 (11)
				825	
50			SS 12		4-5-6 (11)
				820	
55			SS 13		6-7-10 (17)
				815	
60			SS 14		3-6-8 (14)
				810	
65			SS 15		6-8-12 (20)
				805	
70			SS 16		10-14-16 (30)
		Auger only		800	
75					

▲ SPT N VALUE ▲
20 40 60 80
PL MC LL
20 40 60 80
☐ FINES CONTENT (%) ☐
20 40 60 80

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BORING NUMBER B-1

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PROJECT NUMBER 224927
PROJECT LOCATION 461 Sandy Creek Road, Fayetteville, GA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ELEVATION (ft)	BLOW COUNTS (N VALUE)	▲ SPT N VALUE ▲ 20 40 60 80 / PL MC LL 20 40 60 80 / ☐ FINES CONTENT (%) ☐ 20 40 60 80
75		Auger only *(continued)*				
				795		
80						
				790		
85						
				785		
90						
				780		
95						
				775		
100						

Borehole terminated at 100.0 feet.

BORING NUMBER B-2

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CLIENT Arcadis	PROJECT NAME FCWS Elevated Storage Tank - Trilith Studios
PROJECT NUMBER 224927	PROJECT LOCATION 461 Sandy Creek Road, Fayetteville, GA
DATE STARTED 8/9/22 COMPLETED 8/9/22	GROUND ELEVATION 874 ft HOLE SIZE 6
DRILLING CONTRACTOR Nicholson Exploration	GROUND WATER LEVELS:
DRILLING METHOD HSA-Auto Hammer	AT TIME OF DRILLING 43.00 ft / Elev 831.00 ft
LOGGED BY NE CHECKED BY BT	AT END OF DRILLING 29.00 ft / Elev 845.00 ft
NOTES Borehole caved at 47'	24hrs AFTER DRILLING 37.00 ft / Elev 837.00 ft

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ELEVATION (ft)	BLOW COUNTS (N VALUE)
0	AGGREGATE: Approximatelt 18" of gravel/soil mix			
	(MH) RESIDUUM: Very stiff red brown clayey medium to fine sandy SILT	SS 1		8-8-9 (17)
5		SS 2	870	9-7-10 (17)
	(SM) Medium dense red orange clayey silty medium to fine SAND	SS 3		7-7-6 (13)
	(SM) Medium dense orange tan silty fine SAND, slightly micaceous	SS 4	865	5-3-8 (11)
10				
	(SM) Medium dense to loose tan orange silty fine SAND, slightly micaceous			
15		SS 5	860	6-3-8 (11)
20		SS 6	855	4-2-6 (8)
	(ML) Stiff purple tan file sandy SILT, slightly micaceous, wet			
25		SS 7	850	3-3-7 (10)
	(ML) Loose to medium dense purple tan silty fine SAND, slightly micaceous, very wet			
30		SS 8	845	2-3-9 (12)
35		SS 9	840	3-6-9 (15)

▲ SPT N VALUE ▲ 20 40 60 80
PL MC LL 20 40 60 80
☐ FINES CONTENT (%) ☐ 20 40 60 80

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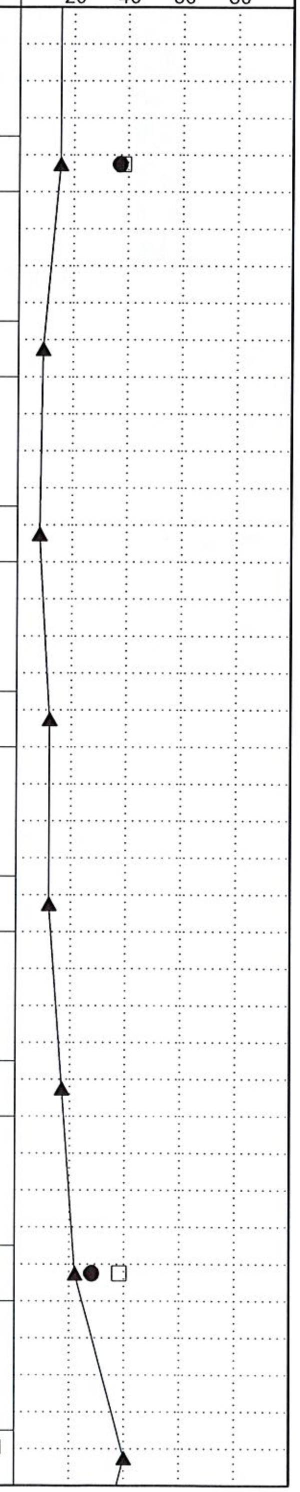
BORING NUMBER B-2

CLIENT Arcadis
PROJECT NAME FCWS Elevated Storage Tank - Trilith Studios
PROJECT NUMBER 224927
PROJECT LOCATION 461 Sandy Creek Road, Fayetteville, GA

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ELEVATION (ft)	BLOW COUNTS (N VALUE)
35	(ML) Loose to medium dense purple tan silty fine SAND, slightly micaceous, very wet *(continued)*			
40		SS 10	835	3-5-10 (15)
	(SM) Loose to medium dense tan brown silty medium to fine SAND, slightly micaceous, very wet			
45		SS 11	830	4-3-6 (9)
50		SS 12	825	3-4-4 (8)
55		SS 13	820	4-4-8 (12)
60		SS 14	815	3-5-7 (12)
65		SS 15	810	6-6-11 (17)
70		SS 16	805	6-9-13 (22)
	(SM) Dense to medium dense tan brown silty medium to fine SAND, slightly micaceous, very wet			
75		SS 17	800	13-19-21 (40)

▲ SPT N VALUE ▲ 20 40 60 80
PL MC LL 20 40 60 80
☐ FINES CONTENT (%) ☐ 20 40 60 80



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BORING NUMBER B-2

CLIENT Arcadis

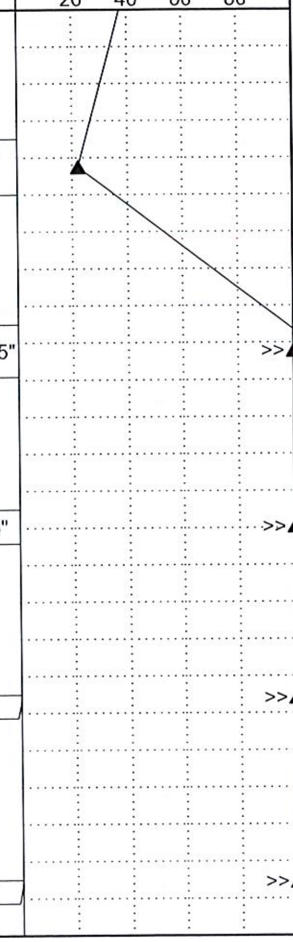
PROJECT NAME FCWS Elevated Storage Tank - Trilith Studios

PROJECT NUMBER 224927

PROJECT LOCATION 461 Sandy Creek Road, Fayetteville, GA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ELEVATION (ft)	BLOW COUNTS (N VALUE)
75		(SM) Dense to medium dense tan brown silty medium to fine SAND, slightly micaceous, very wet *(continued)*			
			SS 18	795	6-9-13 (22)
80					
		(SM) PARTIALLY WEATHERED ROCK: Sampled as very dense tan brown silty medium to fine SAND, slightly micaceous, wet			
			SS 19	790	6-12-50/5"
85					
			SS 20	785	47-50/5"
90					
			SS 21	780	50/2"
95					
		No sample recovered			
			SS 22	775	50/1"
100					

▲ SPT N VALUE ▲ (20 40 60 80); PL MC LL (20 40 60 80); □ FINES CONTENT (%) □ (20 40 60 80)



Borehole terminated at 100.0 feet.

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Roswell, Georgia 30075
Telephone: (678) 739-2400

BORING NUMBER B-3

CLIENT Arcadis
PROJECT NAME FCWS Elevated Storage Tank - Trilith Studios
PROJECT NUMBER 224927
PROJECT LOCATION 461 Sandy Creek Road, Fayetteville, GA
DATE STARTED 8/10/22 **COMPLETED** 8/10/22
GROUND ELEVATION 875 ft **HOLE SIZE** 6
DRILLING CONTRACTOR Nicholson Exploration
GROUND WATER LEVELS:
DRILLING METHOD HSA-Auto Hammer
∇ **AT TIME OF DRILLING** 40.00 ft / Elev 835.00 ft
LOGGED BY NE **CHECKED BY** BT
▼ **AT END OF DRILLING** 32.00 ft / Elev 843.00 ft
NOTES Borehole caved at 53'
AFTER DRILLING ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ELEVATION (ft)	BLOW COUNTS (N VALUE)
0	TOPSOIL: Approximately 3" of topsoil and associated root zone		875	
	(SC) RESIDUUM: Medium dense red brown clayey medium to fine SAND			
		SS 1		8-9-11 (20)
5		SS 2	870	5-9-11 (20)
	(SM) Medium dense red orange clayey silty medium to fine SAND, slightly micaceous			
		SS 3		4-5-8 (13)
	(SM) Loose red tan silty fine SAND, slightly micaceous			
10		SS 4	865	4-4-6 (10)
	(SM) Loose purple tan silty fine SAND, slightly micaceous			
15		SS 5	860	3-4-5 (9)
20		SS 6	855	3-3-5 (8)
	(SM) Medium dense purple tan silty fine SAND, slightly mciaceous, wet			
25		SS 7	850	4-6-8 (14)
30		SS 8	845	5-8-10 (18)
	▼ (SM) Loose to medium dense purple tan silty fine SAND, slightly micaceous, very wet			
35		SS 9	840	3-4-5 (9)

▲ SPT N VALUE ▲ 20 40 60 80
PL MC LL 20 40 60 80
□ FINES CONTENT (%) □ 20 40 60 80

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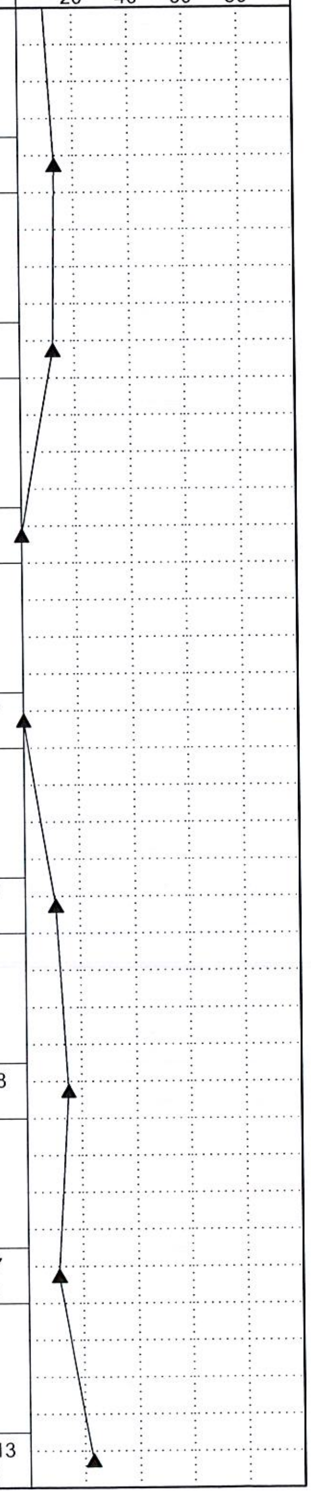
BORING NUMBER B-3

CLIENT Arcadis
PROJECT NAME FCWS Elevated Storage Tank - Trilith Studios
PROJECT NUMBER 224927
PROJECT LOCATION 461 Sandy Creek Road, Fayetteville, GA

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ELEVATION (ft)	BLOW COUNTS (N VALUE)
35	(SM) Loose to medium dense purple tan silty fine SAND, slightly micaceous, very wet *(continued)*		840	
40	∇	SS 10	835	5-5-8 (13)
45		SS 11	830	4-6-6 (12)
	(SM) Very loose tan black silty medium to fine SAND, slightly micaceous, very wet			
50		SS 12	825	0-0-0 (0)
55		SS 13	820	0-0-0 (0)
	(SM) Medium dense tan white silty coarse to fine SAND, slightly micaceous, wet			
60		SS 14	815	2-4-7 (11)
65		SS 15	810	12-7-8 (15)
70		SS 16	805	8-4-7 (11)
75		SS 17	800	7-10-13 (23)

▲ SPT N VALUE ▲ 20 40 60 80
PL MC LL 20 40 60 80
☐ FINES CONTENT (%) ☐ 20 40 60 80



(Continued Next Page)

Oasis Consulting Services
45 Woodstock Street
Roswell, Georgia 30075
Telephone: (678) 739-2400

BORING NUMBER B-3

CLIENT Arcadis **PROJECT NAME** FCWS Elevated Storage Tank - Trilith Studios

PROJECT NUMBER 224927 **PROJECT LOCATION** 461 Sandy Creek Road, Fayetteville, GA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ELEVATION (ft)	BLOW COUNTS (N VALUE)	▲ SPT N VALUE ▲ 20 40 60 80 / PL MC LL 20 40 60 80 / □ FINES CONTENT (%) □ 20 40 60 80
75		Auger only. Driller described PWR-like material at 90 feet.		800		
80				795		
85				790		
90				785		
95				780		
100				775		

Borehole terminated at 100.0 feet.

APPENDIX D

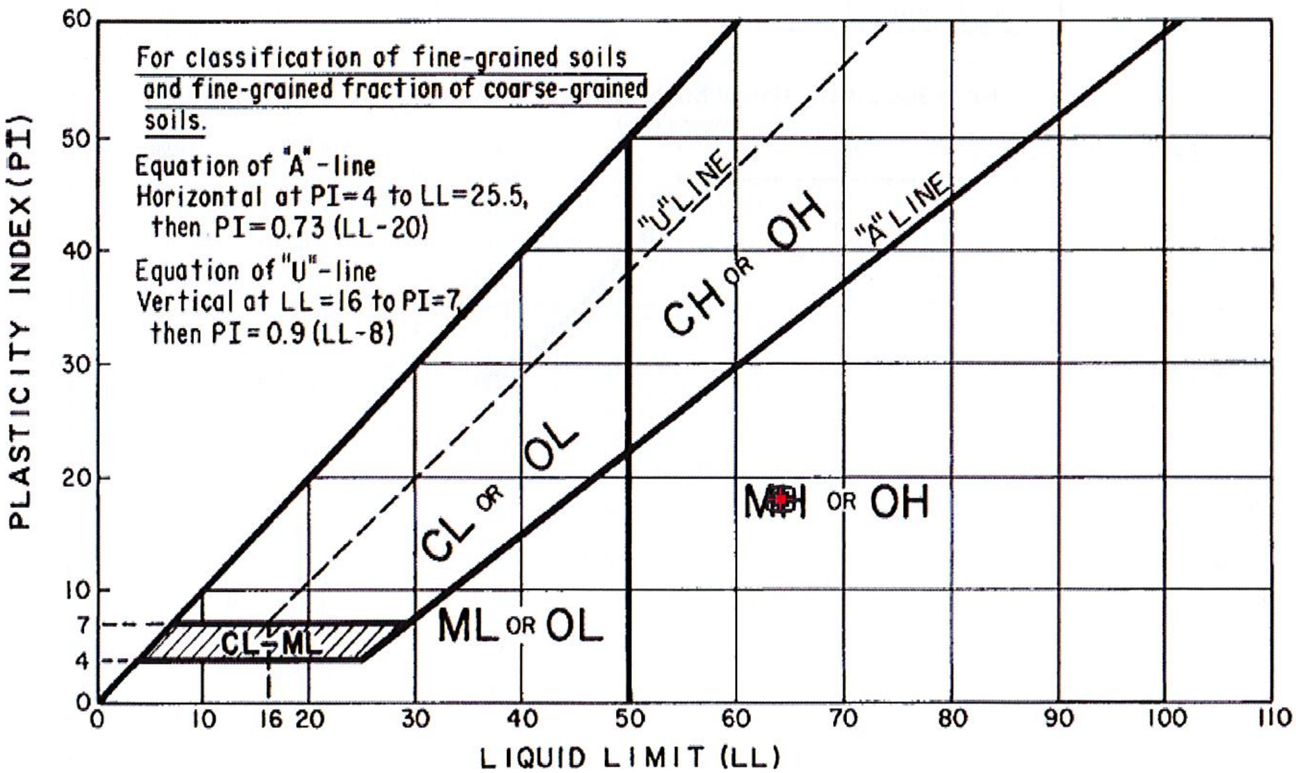
LABORATORY TEST RESULTS

REPORT OF ATTERBERG LIMITS TEST RESULTS (ASTM D4318)

Project Name:	Trilith Studios Above Ground Storage Tank	Lab#:	5068	Project Number:	224927
Location	B-2 3.5'-5'	Technician	JS	Test Date:	8/26/2022
Type of Test:	Atterberg Limits	Checked by	DW	USCS Classification:	MH
Sample Description:	Red sandy SILT (MH)			Boring B-2	Depth 3.5'-5'

*USCS Classification is based on the Atterberg Limit test and the Grain Size Analysis results.

Liquid Limit	**65**
Plastic Limit	**47**
Plasticity Index	**18**



OASIS CONSULTING SERVICES

45 Woodstock Street
Roswell, Georgia 30075
678-739-2400

Report of Grain Size Analysis of Material Larger Than #200 Sieve

(ASTM D422), (ASTM D1140)

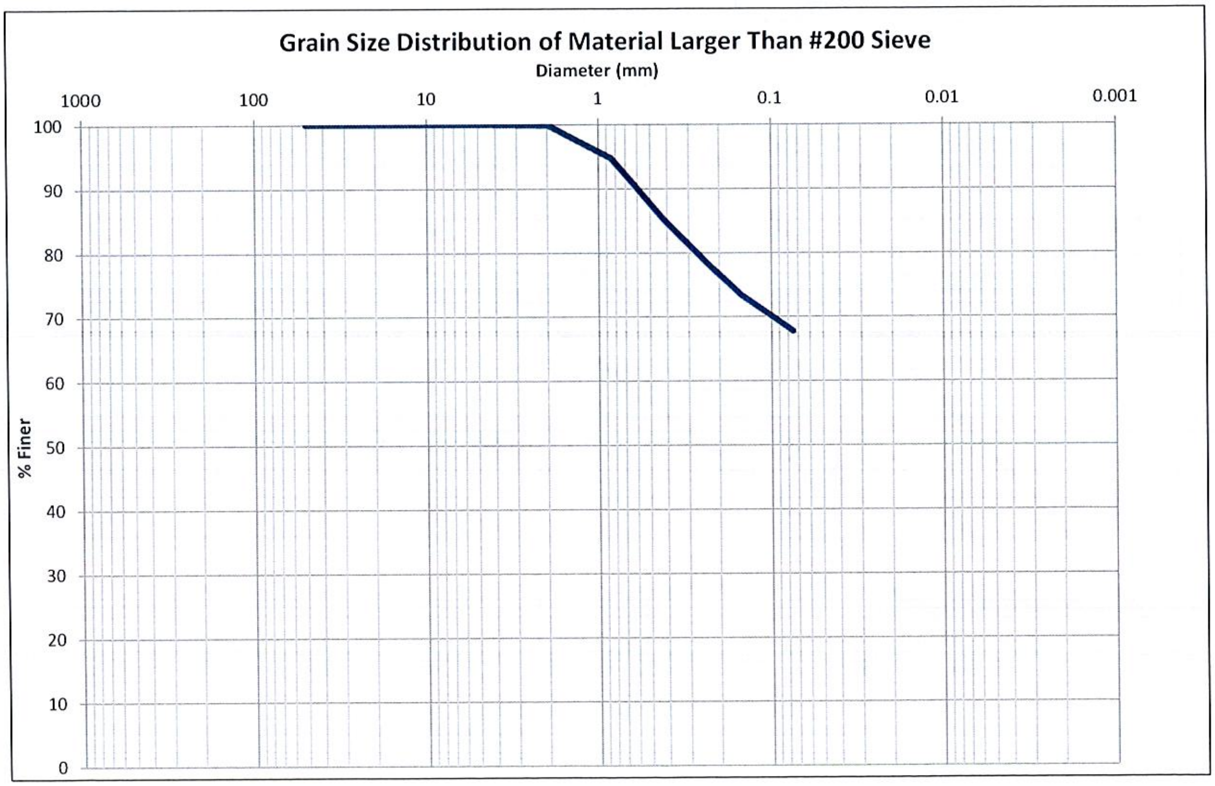
Client	Arcadis			
Project Name	Trilith Studios Above Ground Storage Tank			
Project Number	224927			
Date	8/26/2022	Technician	JS	
Sample #	B-2 3.5'-5'	Lab #	5068	
Classification	Red sandy SILT (MH)			

Sieve Analysis		
Sieve #	Diameter mm	Passing %
2	50.8	100.0
1.5	38.09	100.0
1	25.4	100.0
3/4	19.04	100.0
1/2	12.7	100.0
3/8	9.5	100.0
#4	4.75	100.0
#10	2	99.9
#20	0.85	94.7
#40	0.425	85.2
#60	0.25	78.9
#100	0.15	73.3
#200	0.075	67.7

Natural Moisture Content
%
19.6

Wash #200 Soak Time
(At least 120 min to the nearest 10 min)
960

Test Method		
A		B
		x



Checked By: ____________________

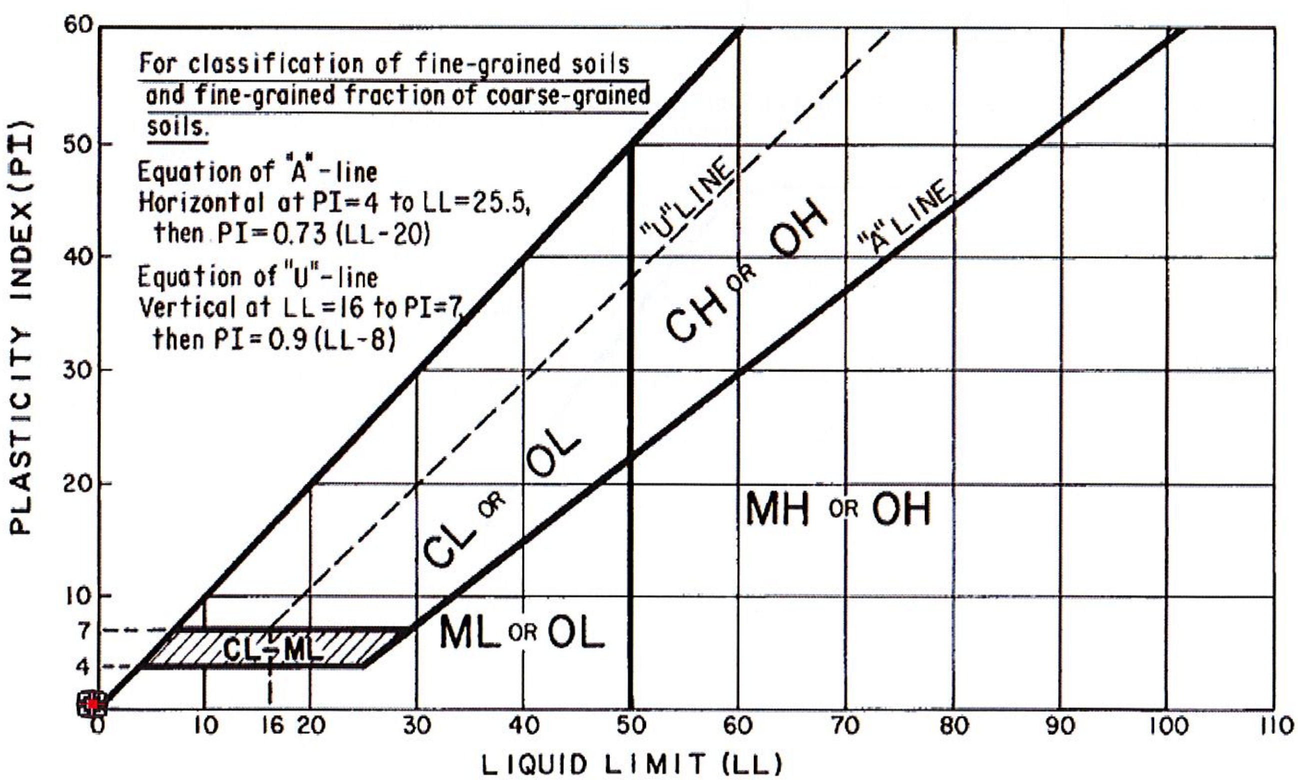
OASIS CONSULTING SERVICES

REPORT OF ATTERBERG LIMITS TEST RESULTS (ASTM D4318)

Project Name:	Trilith Studios Above Ground Storage Tank	Lab#:	5068	Project Number:	224927
Location	B-2, 18.5'-20'	Technician	JS	Test Date:	8/26/2022
Type of Test:	Atterberg Limits	Checked by	DW	USCS Classification:	SM
Sample Description:	Reddish brown silty SAND (SM)			Boring B-2	Depth 18.5'-20'

*USCS Classification is based on the Atterberg Limit test and the Grain Size Analysis results.

Liquid Limit	**N/A**
Plastic Limit	**N/A**
Plasticity Index	**NP**



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Roswell, Georgia 30075
678-739-2400

Report of Grain Size Analysis of Material Larger Than #200 Sieve

(ASTM D422), (ASTM D1140)

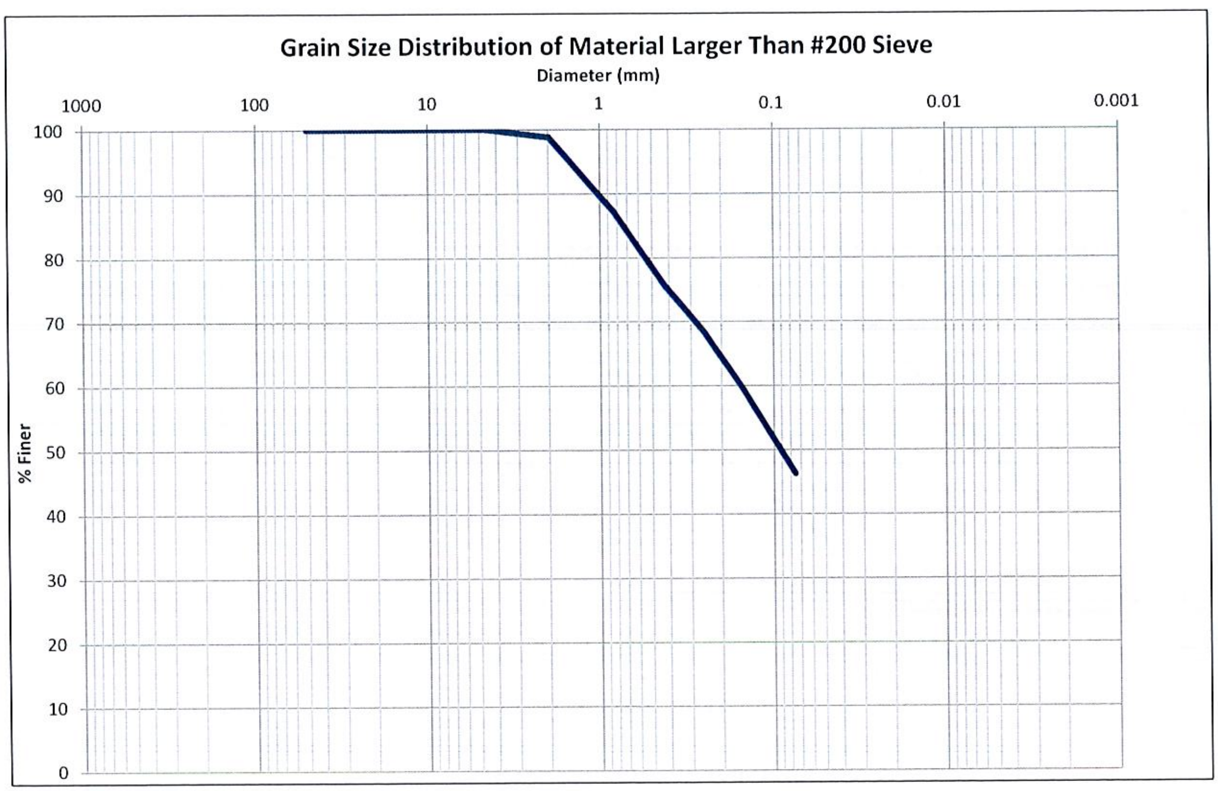
Client	Arcadis			
Project Name	Trilith Studios Above Ground Storage Tank			
Project Number	224927			
Date	8/26/2022	**Technician**	JS	
Sample #	B-2, 18.5'-20'	**Lab #**	5068	
Classification	Reddish brown silty SAND (SM)			

Sieve Analysis		
Sieve #	Diameter mm	Passing %
2	50.8	100.0
1.5	38.09	100.0
1	25.4	100.0
3/4	19.04	100.0
1/2	12.7	100.0
3/8	9.5	100.0
#4	4.75	100.0
#10	2	98.7
#20	0.85	87.3
#40	0.425	75.6
#60	0.25	68.3
#100	0.15	59.4
#200	0.075	46.3

Natural Moisture Content
%
27.8

Wash #200 Soak Time
(At least 120 min to the nearest 10 min)
960

Test Method		
A		B
		x



Jun-19

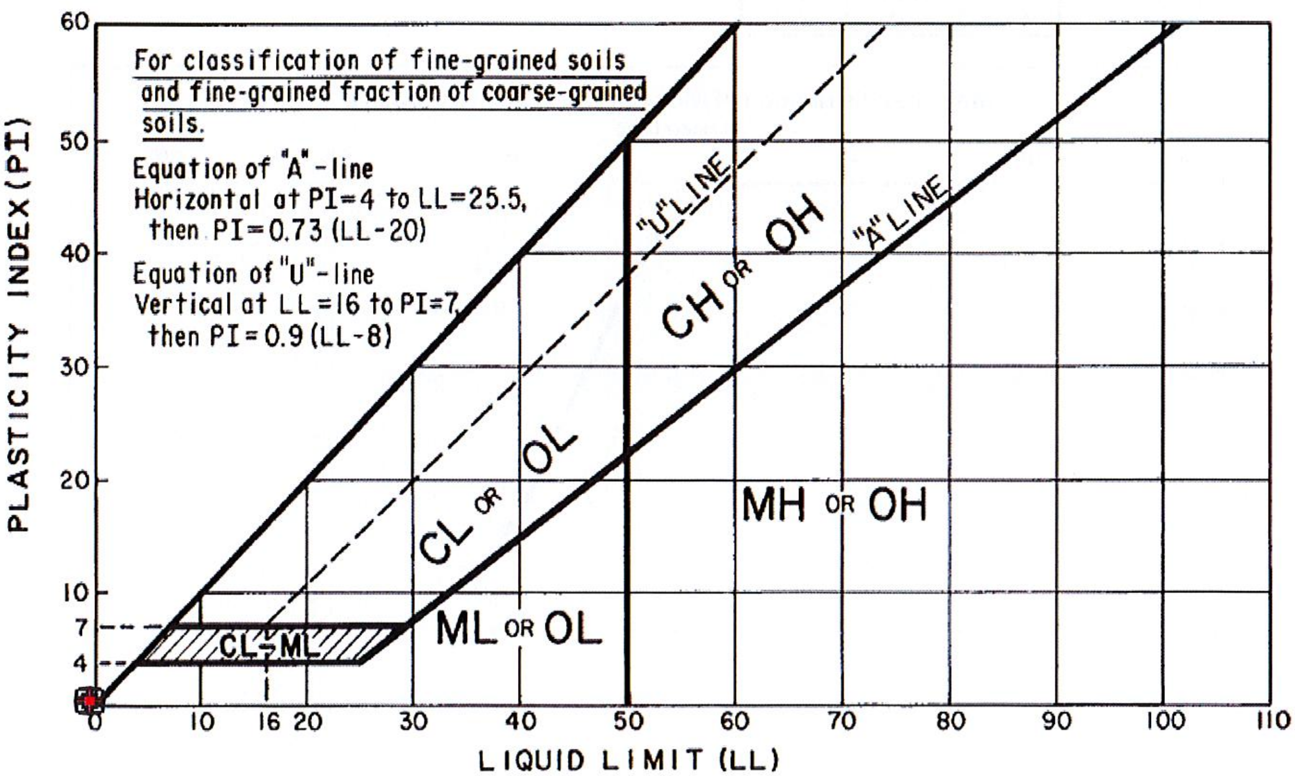
Checked By: ____________________

REPORT OF ATTERBERG LIMITS TEST RESULTS (ASTM D4318)

Project Name:	Trilith Studios Above Ground Storage Tank	Lab#: 5068		Project Number: 224927	
Location	B-2 38.5'-40'	Technician	JS	Test Date: 8/26/2022	
Type of Test:	Atterberg Limits	Checked by	DW	USCS Classification: SM	
Sample Description:	Light red silty SAND (SM)			Boring B-2	Depth 38.5'-40'

*USCS Classification is based on the Atterberg Limit test and the Grain Size Analysis results.

Liquid Limit	**N/A**
Plastic Limit	**N/A**
Plasticity Index	**NP**



OASIS CONSULTING SERVICES

45 Woodstock Street
Roswell, Georgia 30075
678-739-2400

Report of Grain Size Analysis of Material Larger Than #200 Sieve

(ASTM D422), (ASTM D1140)

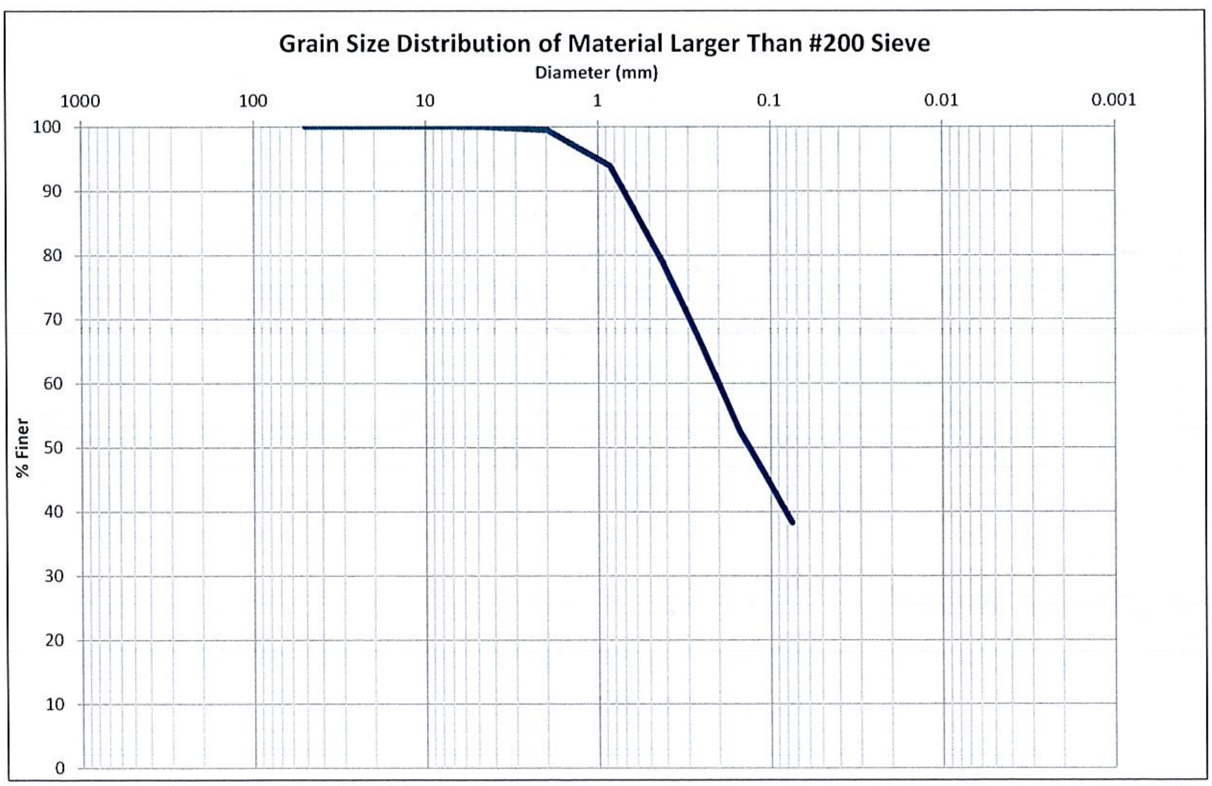
Client	Arcadis			
Project Name	Trilith Studios Above Ground Storage Tank			
Project Number	224927			
Date	8/26/2022	**Technician**	JS	
Sample #	B-2, 38.5'-40'	**Lab #**	5068	
Classification	Light red silty SAND (SM)			

Sieve Analysis		
Sieve #	Diameter mm	Passing %
2	50.8	100.0
1.5	38.09	100.0
1	25.4	100.0
3/4	19.04	100.0
1/2	12.7	100.0
3/8	9.5	100.0
#4	4.75	100.0
#10	2	99.4
#20	0.85	93.8
#40	0.425	79.0
#60	0.25	65.8
#100	0.15	52.4
#200	0.075	38.2

Natural Moisture Content
%
36.9

Wash #200 Soak Time
(At least 120 min to the nearest 10 min)
960

Test Method		
A		B
		x



Jun-19

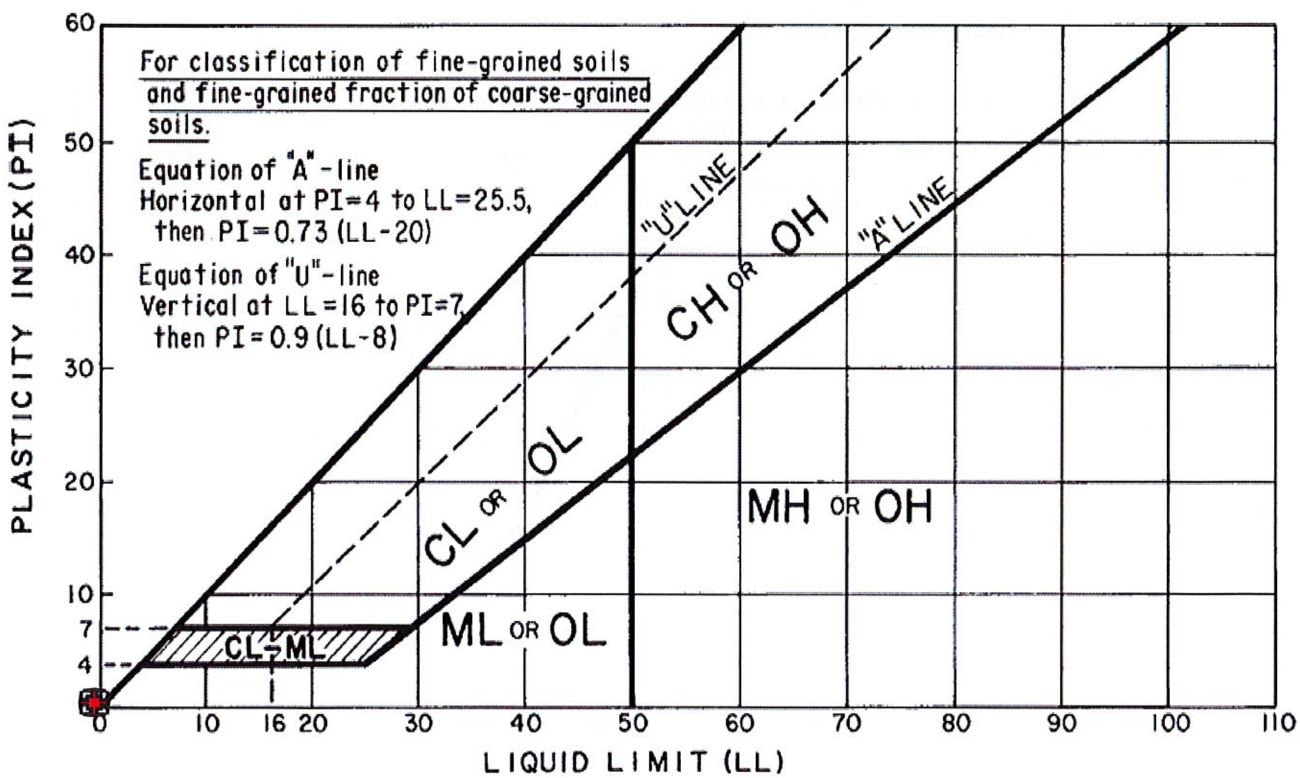
Checked By: ____________________

REPORT OF ATTERBERG LIMITS TEST RESULTS (ASTM D4318)

Project Name:	Trilith Studios Above Ground Storage Tank	Lab#:	5068	Project Number:	224927
Location	B-2, 68.5'-70'	Technician	JS	Test Date:	8/26/2022
Type of Test:	Atterberg Limits	Checked by	DW	USCS Classification:	SM
Sample Description:	Gray silty SAND (SM)			Boring B-2	Depth 68.5'-70'

*USCS Classification is based on the Atterberg Limit test and the Grain Size Analysis results.

Liquid Limit	**N/A**
Plastic Limit	**N/A**
Plasticity Index	**NP**



OASIS CONSULTING SERVICES

45 Woodstock Street
Roswell, Georgia 30075
678-739-2400

Report of Grain Size Analysis of Material Larger Than #200 Sieve

(ASTM D422), (ASTM D1140)

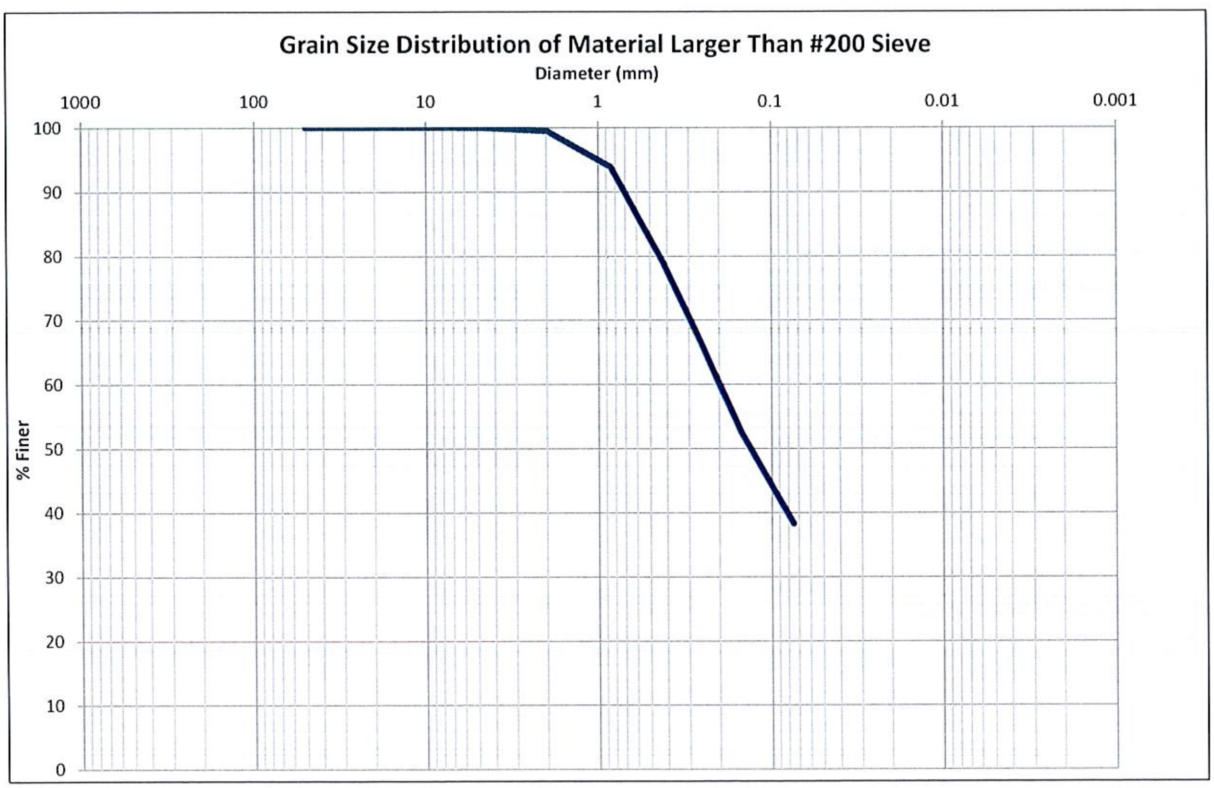
Client	Arcadis			
Project Name	Trilith Studios Above Ground Storage Tank			
Project Number	224927			
Date	8/26/2022	**Technician**	JS	
Sample #	B-2, 68.5'-70.0'	**Lab #**	5068	
Classification	Gray silty SAND (SM)			

Sieve Analysis		
Sieve #	Diameter mm	Passing %
2	50.8	100.0
1.5	38.09	100.0
1	25.4	100.0
3/4	19.04	100.0
1/2	12.7	100.0
3/8	9.5	100.0
#4	4.75	100.0
#10	2	99.4
#20	0.85	93.8
#40	0.425	79.0
#60	0.25	65.8
#100	0.15	52.4
#200	0.075	38.2

Natural Moisture Content
%
28.1

Wash #200 Soak Time
(At least 120 min to the nearest 10 min)
960

Test Method		
A		B
		x



Jun-19

Checked By: ____________________

November 22, 2022

Arcadis
2839 Paces Ferry Road SE, Suite 900
Atlanta. Georgia 30339

Attention: Mr. Travis Thomas

Subject: **Addendum to Report of Subsurface Exploration and Geotechnical Engineering Evaluation**
FCWS Elevated Storage Tank
461 Sandy Creek Road
Fayetteville, Georgia
Oasis Project No. 224927

Dear Travis:

As you are aware, Oasis Consulting Services (Oasis) previously submitted a Report of Subsurface Exploration and Geotechnical Engineering Evaluation dated October 4, 2022 (fka - Trilith Studios Above Ground Storage Tank, Oasis Project No 224927). In that report we provided the recommendation of deep foundations for foundation support of the Above Ground Storage Tank. Recently, you inquired as to what type of deep foundations would be appropriate for the anticipated loads of the structure and asked that we review the foundation support options in light of the newly provided anticipated loads.

This addendum report should be used in conjunction with our previous report and not as a separate report. The recommendations contained in our original report remain in effect unless otherwise modified in this addendum report.

PROJECT INFORMATION

We understand the project consists of the construction of a 400,000-gallon elevated storage tank in an area of the Trilith Development. Based on our review of the provided load calculations, we understand the elevated storage tank will have maximum outside column loads of 593.5 kips and a maximum center riser load 1134.9 kips.

FOUNDATION OPTIONS

As noted above, we were asked to review the foundation support options in light of the anticipated loads and provide an axial pile and lateral pile analysis along with pile uplift resistance recommendations. For our analysis, we used a combination of soil conditions from borings B-2 and B-3 as a conservative approach. Boring B-2 soil consistency from the depth of 47 feet to 57 feet below existing grade was interpolated from boring B-3.

Our analysis considered a deep foundation system as the most appropriate foundation solution for the design as they will provide the necessary axial and lateral support for the anticipated loads. We considered caissons and auger cast-in-place (ACIP) piles for this project. After review, caissons do not appear feasible due to the depth needed to support the provided loads. In our opinion, auger cast piles will be the most cost-effective and feasible deep foundation system at this site due to the availability to achieve the depth needed to support the provided loads and therefore, detailed recommendations are provided for this system only.

For this project we analyzed 16-inch and 18-inch diameter ACIP piles. Estimated allowable compressive capacity was calculated using the SPT N-values and American Association of State Highway Transportation Officials (AASHTO) methods. Axial pile analysis was performed using the computer program RSPile by Rocscience with a deflection of 0.5-inches. Lateral pile analysis was performed using the computer program LPile. Results are attached.

Based on the boring data, ACIP piles will develop their capacity from a combination of skin friction and end bearing but mainly skin friction for this site. ACIP piles consisting of 16 or 18 inches in diameter, depending on the anticipated loads, appear to be a feasible foundation option. Auger refusal depths in the borings varied significantly in the parking deck area, and we estimate pile lengths on the order of 85 to 95 feet based on the existing grade elevation. If desired, we recommend additional air track borings be performed at each column location to better quantify the depth to rock and estimated ACIP pile depths, which should help to provide a more accurate pile construction costs estimate. If partially weathered rock is encountered (PWR) refusal should be defined as a penetration rate of one foot or less per minute using a drive box with a minimum dead weight of 5,000 pounds and a torque of at least 20,000 foot-pounds. It is recommended that a center-to-center pile spacing of at least three (3) pile diameters be maintained to minimize settlement and pile capacity reductions caused by group effects. Where piles are spaced no closer than three pile diameters, a group reduction factor will not be required.

The allowable load design capacity generated is based on pile skin resistance since end bearing support varied significantly. The allowable axial compression design capacities include a factor of safety of 2.0 for skin friction in soil and 3.0 for the end bearing resistance in weak rock. The

28-day compressive strength of the grout should be at least 4,000 psi. To provide tension reinforcement, a full-length steel-reinforcing cage should be installed into the center of each pile immediately following grouting. The cage should be designed by the structural engineer based on design allowable capacities. Spacing devices (Centralizers) should be attached to the cage at one-third points but not in the cage area. Piles subject to uplift forces must be provided with adequate reinforcement steel through the entire length.

We recommend the design loads and pile lengths for the piles be verified by performing at least one (1) static load test and monitored in general accordance with ASTM D1143. We recommend that the pile(s) be tested to a minimum of two (2) times their allowable compression design capacity. After completing the pile load test and failure does not occur first, we recommend loading the test pile to three (3) times the design load. The primary purpose of the testing program would be to evaluate the axial/compression capacity of the proposed piles at the recommended minimum depth. The load tests are used to provide evidence that the contractor can produce an ACIP pile, which can safely support the design loads at the project site, and to satisfy project requirements. The load test location should be selected after installing 2 to 4 probe piles throughout the water tower foundations. The probe piles would assist the pile contractor and geotechnical engineer in evaluating the equipment and pile response to the specific site conditions and in determining tentative installation criteria for the test pile. All production piles should be placed using the same procedures and equipment used for installation of the test pile. If ultimate uplift loads are to be in excess of 1/8th of the vertical capacity, a modified load test must be performed on a separate pile to verify tensile or uplift capacity.

It is recommended that the installation of the probe piles, test pile(s) and all production piles be monitored by a representative of Oasis. The installation of auger-cast piles should be sequenced such that adjacent piles with a center-to-center pile spacing of at least three (3) pile diameters within the same cap should not be constructed within the same 24-hour period. This is required to provide adequate time for curing.

All piles must be installed with a grout ratio in excess of 1.15. The grout ratio is the actual volume of pumped grout divided by the theoretical volume of the pile. During the forming of the pile, the minimum required pump strokes per linear foot of pile, as determined by pump calibration, should be achieved. Should less than the required pump strokes occur in any one-foot increment, the auger should be immediately advanced three (3) feet below the point in question and forming of the pile resumed. Pressure of the grout during pumping should be maintained between 75 and 300 pounds per square inch (psi). If the pressure falls below 75 psi, the auger should be advanced to a point three (3) feet lower than the elevation at which the pressure loss occurred. If the auger jumps upward during withdrawal or if the grouting process is interrupted, the auger should be inserted at least three (3) feet below the point in question and the pumping process continued.

Qualified personnel should be present to cast grout compressive test specimens. At a minimum, at least two sets of specimens, six specimens per set, should be cast per day of pile installation, or at least one set per every 50 cubic yards of grout. A flow cone should be used to check the fluidity of the grout mix.

UPLIFT RESISTANCE

Uplift resistance will rely on side friction developed between the various soils in contact with the ACIP piles. Table 1 below presents our recommendations.

Table 1

Soil Type	Uplift-Allowable Side Friction (ksf)
Fill Soils	0.25
Residual Soils	0.5
PWR	1.5

The upper five (5) feet of ACIP piles should be neglected for uplift calculations due to disturbance and other factors. The recommended friction values include a factor of safety of at least 2 and assume the pile has full depth reinforcement. The actual uplift resistance will depend on the thicknesses of the different strata at each pile location.

CLOSURE

This addendum report of professional services has been performed, the findings derived, and recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices in the local area. This warranty is in lieu of all warranties either expressed or implied. Unless otherwise stated herein, our original recommendations remain unchanged.

This addendum report and the conclusions and recommendations provided herein, are provided exclusively for the use of Arcadis and their design team and is intended solely for design of the referenced project. Oasis is not responsible for the conclusions, opinions or recommendations of others based on these data.

We sincerely appreciate the opportunity to provide you with these geotechnical services for the project. We remain available to assist you with the project if additional information is needed. Should you have any questions concerning this report, please do not hesitate to contact us.

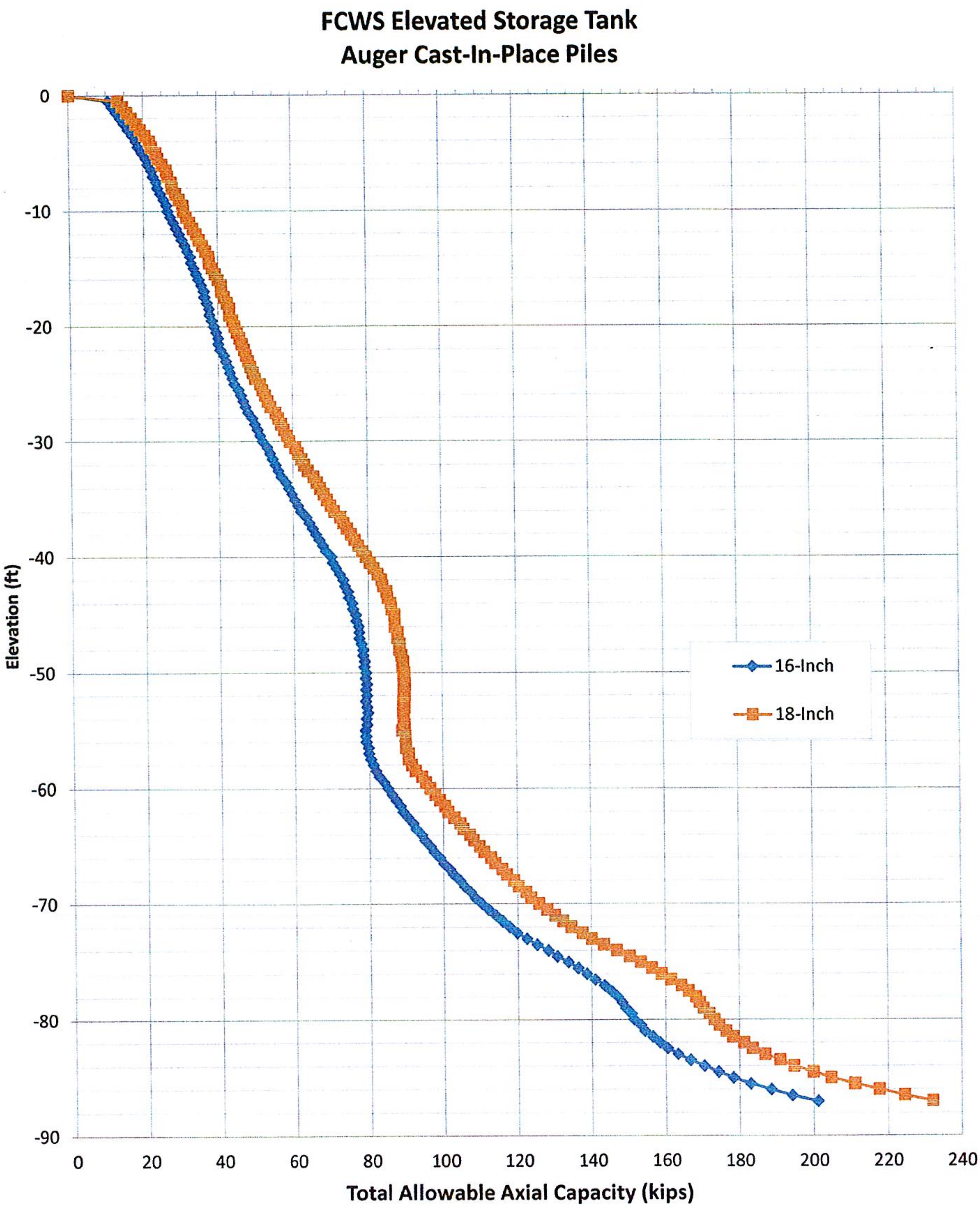
Sincerely,
Oasis Consulting Services

Benjamin D. Thomason, E.I.T.
Project Engineer
GA Registration #: EIT022461

Darren J. Bray, P.E.
Technical Director
GA Registration #: PE038504

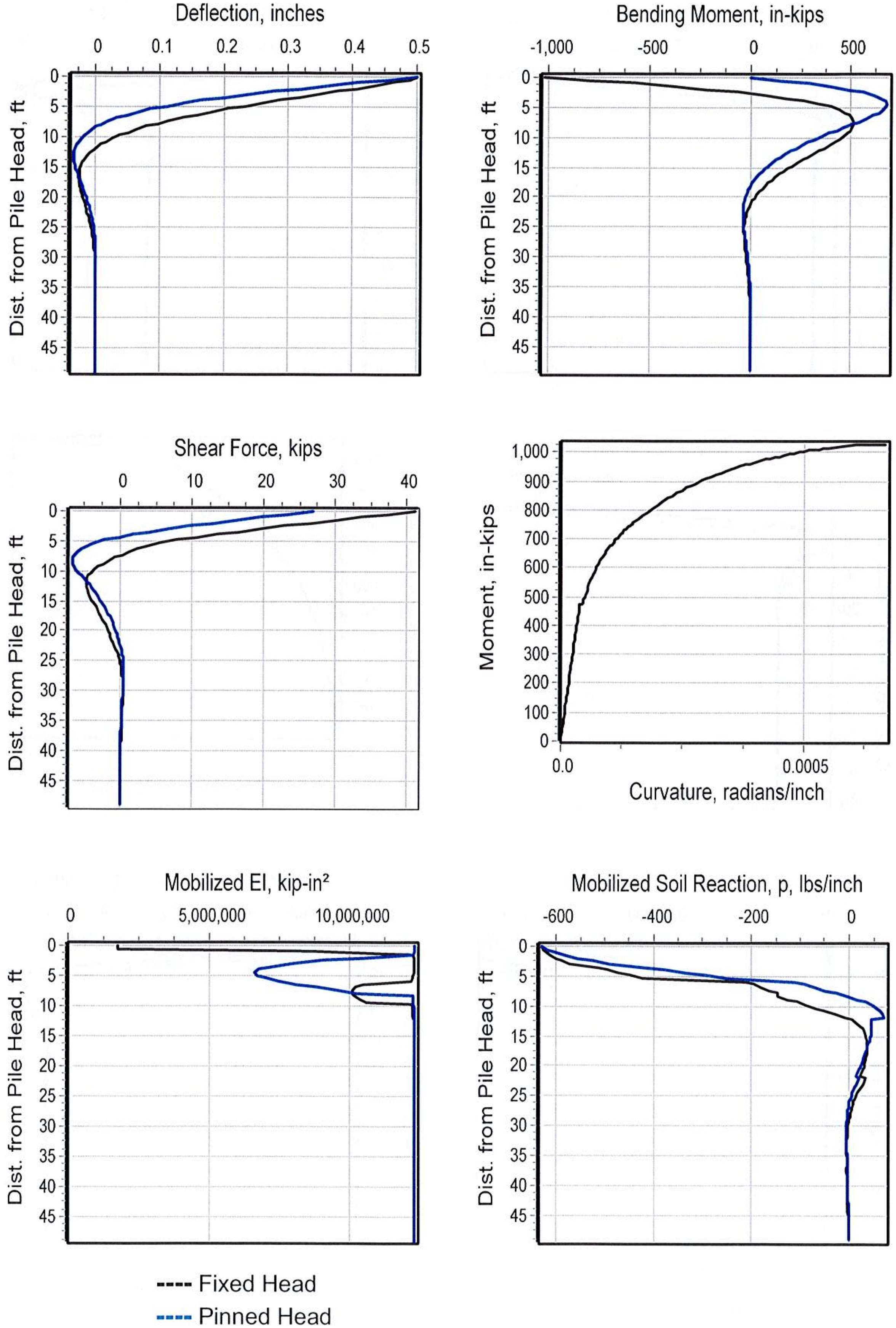
GEORGIA REGISTERED No. PE038504 PROFESSIONAL ENGINEER DARREN J. BRAY
11/22/22

Attachments: Total Allowable Axial Pile Analysis
Lateral Pile Analysis, 16-Inch ACIP
Lateral Pile Analysis, 18-Inch ACIP


FCWS Elevated Storage Tank
Auger Cast-In-Place Piles
Elevation (ft)
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
0
20
40
60
80
100
120
140
160
180
200
220
240
Total Allowable Axial Capacity (kips)
16-Inch
18-Inch

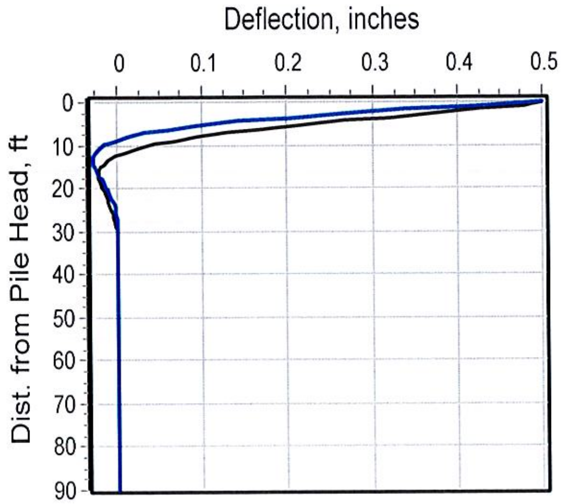
FCWS Elevated Storage Tank
Fayette County
Oasis Project No. 224927

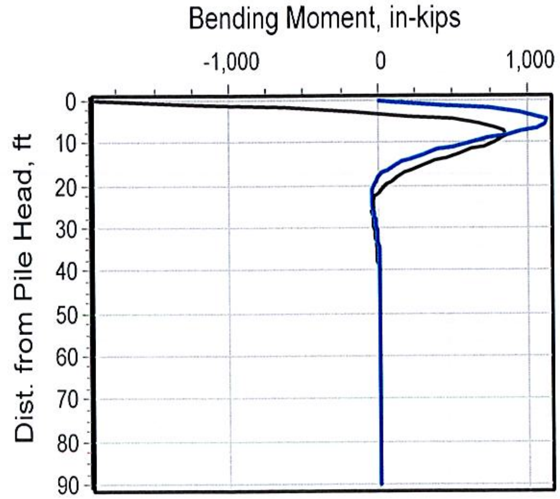
Lateral Pile Analysis
16-Inch ACIP

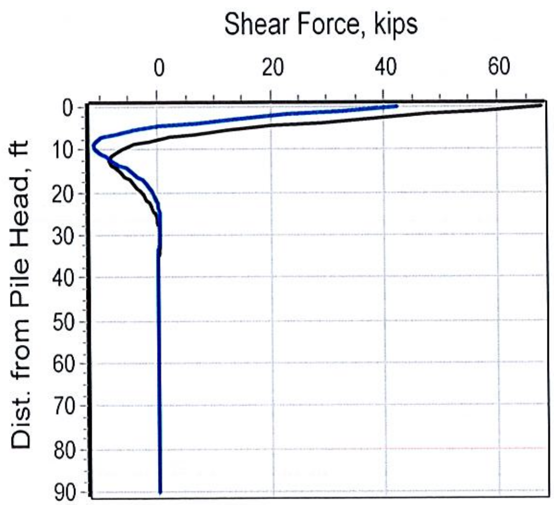


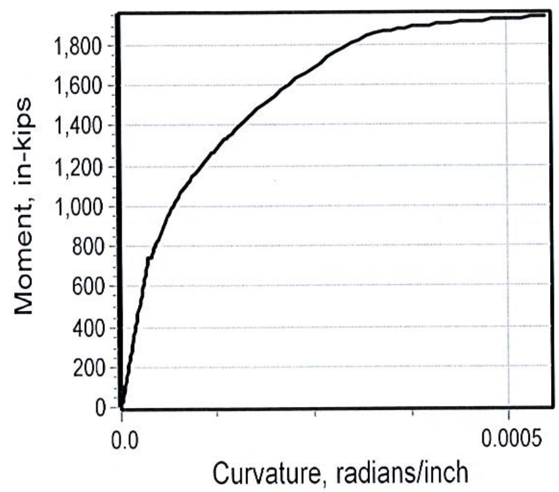
FCWS Elevated Storage Tank
Fayette County
Oasis Project No. 224927

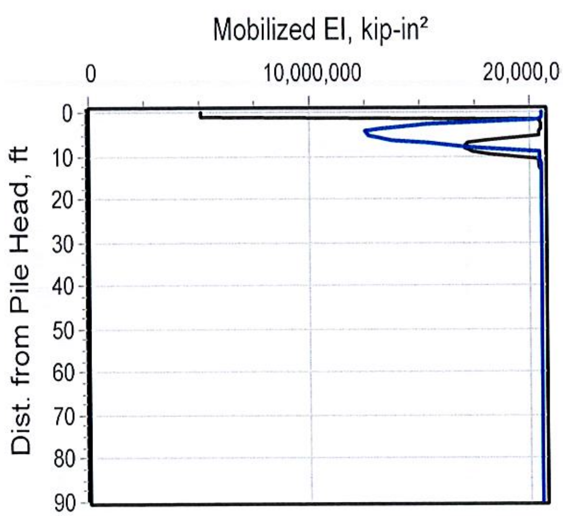
Lateral Pile Analysis
18-Inch ACIP

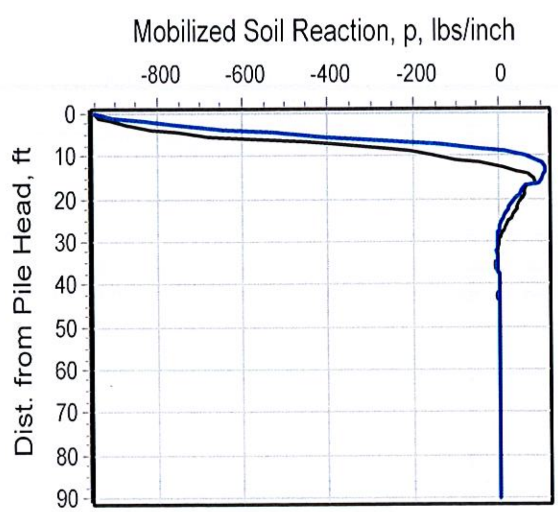












---- Fixed Head
---- Pinned Head