Report of Geotechnical Exploration

Strothers Lane Pump Station
Winchester, Virginia

Triad Project No. 07-17-0166

Prepared for:

Mr. Glenn F. Rogers III, PE, CCCA, ENV SP
Hazen and Sawyer
4807 Hermitage Road, Suite 203
Richmond, VA 23227

Prepared by:

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February 6, 2018 (Revised March 19, 2019)

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Mr. Glenn F. Rogers III, PE, CCCA, ENV SP
Hazen and Sawyer
4807 Hermitage Road, Suite 203
Richmond, VA 23227

RE: Report of Geotechnical Exploration
Strothers Lane Water Tank
Winchester, Virginia
Triad Proposal No. 07-17-0166

Dear Mr. Rogers:

Triad Engineering, Inc. (Triad) has completed a detailed geotechnical exploration for the proposed water tank to be constructed at the terminus of Strothers Lane in Winchester, Virginia. Our scope of services was completed in substantial conformance with our proposal dated July 10, 2017 (Revised December 6, 2017) and executed by receipt of the subconsultant agreement and email acceptance of the scope of services and fee revisions. This report outlines the results of our field exploration and laboratory tests, and presents our recommendations for design and construction of the geotechnical elements of the project.

The subsurface exploration was performed to evaluate the subsurface conditions encountered at the approximate tank location for the limited purposes of preparing design and construction recommendations for geotechnical aspects of the project. It is emphasized that subsurface conditions may vary dramatically between the borings, and Triad makes no representations as to subsurface conditions other than those encountered at the specific boring locations.

This report has been prepared for the exclusive use of Hazen and Sawyer for specific application to the design of the proposed water tank in Winchester, Virginia. Triad's responsibilities and liabilities are limited to our Client and apply only to their use of our report for the purposes described above. To observe compliance with design concepts and specifications, and to facilitate design changes in the event that subsurface conditions differ from those anticipated prior to construction, it is recommended that Triad be retained to provide continuous engineering and testing services during the earthwork and foundation construction phases of the work.

We appreciate the opportunity to provide our services during the design phase of the project. If you should have any questions concerning this report, or if you require any additional information, please do not hesitate to contact us.

Sincerely,

TRIAD ENGINEERING, INC.

Raymond A. Strother II, P.E.
Project Engineer

Randy L. Moulton, P.E.
Principal Engineer

COMMONWEALTH OF VIRGINIA
RANDY L. MOULTON
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PROFESSIONAL ENGINEER

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REPORT OF GEOTECHNICAL EXPLORATION
STROTHERS LANE WATER TANK
WINCHESTER, VIRGINIA

TRIAD PROJECT NO. 07-17-0166

FOREWORD

This report has been prepared for the exclusive use of Hazen and Sawyer for specific application to the design of the planned water tank located at the terminus of Strothers Lane in Winchester, Virginia. The work has been performed in accordance with generally accepted geotechnical engineering practices. No other warranty, expressed or implied, is made.

This report should not be used for estimation of construction quantities and/or costs, and contractors should conduct their own exploration of site conditions for these purposes. Please note that Triad is not responsible for any claims, damages or liability associated with any other party's interpretation of the data or re-use of these data or engineering analyses without the express written authorization of Triad. Additionally, this report must be read in its entirety. Individual sections of this report may cause the reader to draw incorrect conclusions if considered in isolation from each other.

The conclusions and recommendations contained in this report are based, in part, upon our field observations and data obtained from the borings at the site. The nature and extent of variations may not become evident until construction. If variations then appear evident, it may be necessary to re-evaluate the recommendations presented herein. Similarly, in the event that any changes in the nature, design or location of the tank are planned, the conclusions and recommendations contained herein shall not be considered valid unless the changes are reviewed and the conclusions are modified or verified in writing by Triad. If we are not afforded the privilege of making this review, we will not assume responsibility for misinterpretation of our recommendations, as our recommendations are strictly limited to conditions represented to Triad at the time this report was issued.

SITE AND PROJECT DESCRIPTION

The site for the proposed water storage tank is located within an approximate 3.5 acre parcel situated at the terminus of Strothers Lane in Winchester, Virginia. The approximate location of the project site is illustrated on Figure No. A-1 in Appendix A. The site generally consists of gently to steeply sloping grass, gravel and densely wooded terrain. In addition, massive rock ledges were observed around the perimeter of the northern two-thirds of the existing water tank. General descriptions of the observed rock and soil conditions are shown on Figure No. A-3 in Appendix A. On-site utilities include underground water lines and overhead electric lines that extend from Strothers Lane. A small storage shed and utility pole were observed on the southern side of the existing water tank.

It is our understanding that the project will include demolition of the existing water tank and construction of a new tank. The pre-tensioned concrete tank will have a total water height of approximately 51 feet with a diameter of 100 feet. Specific structural details have not been provided at this time. However, based upon our experience with similar projects of this type, we estimate that the applied bearing pressure will be on the order of 3,200 ksf for a mat foundation. Proposed grading has not been provided at this time. However, based upon our site reconnaissance and our understanding of the project, we assume that maximum cuts and fills for the site grading will be on the order of 5 feet or less.

GEOLOGIC SETTING

General

The project is located within the Valley and Ridge Geologic Province. Based on review of the Geologic Map of Frederick County, Virginia, the project site is situated within sedimentary bedrock units belonging to the Conococheague Formation of Cambrian Age. This formation is generally described as algal limestone with interbedded aphanitic limestone and dolomite. Siliceous and dolomitic laminations are common, and minor sandy beds are present throughout the formation. A geologic contact between the Conococheague Formation and the Stonehenge Limestone of Ordovician Age is located immediately east of the project site. Residual overburden soils weathered from the parent bedrock generally consist of medium to high plasticity silty clay with varying amounts of sand and rock fragments.

Development in Karst Areas

The carbonate rock formations in the general geographic area are moderately solution-prone, highly calcareous and weather differentially to produce a pinnacled or "sawtooth" top of rock profile. The degree of weathering or solutioning within the bedrock is controlled by joint orientation and frequency. Where joints intersect or are highly fractured, subsequent solutioning is intensified creating low areas and seams that are generally filled with residual clay soils. Conversely, more competent high areas represent slightly too non-fractured rock units that are often coarse grained and only slightly solution prone.

Karst terrain is characterized by caves, internal drainage, lack of surface streams and topographic features such as sinkholes. These features are the result of the dissolution of soluble bedrock, such as limestone or dolomite, by groundwater and/or infiltration of surface water. As groundwater enters fractures and bedding planes in soluble carbonate bedrock, it slowly dissolves the rock and enlarges the fractures. This results in the formation of solutioning channels or underground streams or ravines that typically develop slowly over geologic time.

Based on our site reconnaissance, we did not observe any apparent naturally depressed areas within the proposed construction area. There will always be some risk that an owner must accept when developing in karst areas. These risks can include groundwater contamination, subsidence and flooding. In all these instances, water is

the primary cause of the problem. Planning and implementation of appropriate site drainage both during and after construction can help significantly reduce this risk. The level of these risks, however, cannot be clearly defined since they are partially controlled by nature. Detailed explorations, such as electrical resistivity testing of the site can help better define these risks, if desired.

It is important to note that alterations in the ground surface, particularly in cut areas, during construction can impact the natural drainage within the site, and it is common to have some solutioning features develop in these areas as a result of construction. Also, normal blasting required to remove hard rock can create micro-fractures within the bedrock that will allow greater surface water infiltration into areas that may normally not receive water and, in turn, disturb old solutioning features and/or possibly create new features. These features can develop during and/or after construction and they will result in some minor construction delays and unanticipated costs for repairs. Certain design and construction measures can and should be implemented to help reduce potential risks associated with future sinkhole development within the site. All of these suggested measures are associated with implementing proper site drainage, minimizing water infiltration, and reducing groundwater fluctuation during and after construction. These additional measures include the following:

- Positive slopes should be maintained away from the tank area.

- Do not locate any new deep utilities within the tank area, if possible. Utility trenches are common routes along which subsurface water can travel, and this can increase the risk of future subsidence. Consequently, water line trenches should be backfilled with well-graded crushed aggregate to reduce the potential for significant accumulation of water.

- Maintain positive slopes around footing excavations and the tank footprint prior to and after placement of concrete.

- All underground water lines should be pressure tested prior to backfilling to verify that no leakage is present.

FIELD EXPLORATION

Due to the presence of the existing tank and access constraints, the subsurface conditions at the site were evaluated by drilling two (2) test borings adjacent to the existing tank. The approximate boring locations are illustrated on Figure No. A-2 in Appendix A. The final exploration locations were recorded and surface elevations were determined by Triad survey personnel.

The test borings included Standard Penetration Testing (SPT) and split barrel sampling (ASTM D 1586) at standard intervals to auger refusal on hard rock at depths of 4 and 8 feet below existing grades for borings B-1 and B-2, respectively. NQ rock coring was performed in borings B-1 and B-2 to depths of 14.5 and 18 feet below existing grade, respectively. Geotechnical engineering personnel from our office were present full time

during the field exploration to direct the drill crew, log all recovered soil samples and observe groundwater and geologic conditions. The recovered soil samples were transported to our laboratory for further testing. Detailed descriptions of materials encountered in the test borings are contained on the boring logs in Appendix B. Figure Nos. 1 and 2 contains a description of the classification system and terminology utilized for soil and rock.

SUBSURFACE CONDITIONS

Subsurface Strata

Surface Materials: Approximately 1 inch of root mat/topsoil was encountered at the surface of the borings. The topsoil generally consisted of red-brown organic silt/clay with a thin surface root mat.

Possible Fill: Possible fill material was encountered below the surficial material in boring B-1. The possible fill material extended to an approximate depth 4 feet below existing grade at this location. The possible fill material consisted of red-brown fat clay with minor proportions of sand and rock fragments. Both SPT N-values obtained in the fill exhibited 45 blows per foot indicating very stiff consistencies. In addition, pocket penetrometer tests were performed for each of the samples, and the readings obtained in the fill exhibited apparent unconfined strengths of 2.25 tsf.

Residual Soils: Residual soil was encountered below the surficial material in boring B-2. The residuum was present to auger refusal on hard rock at a depth of 8 feet below existing grades. In general, the residual soils consisted of red-brown, high plasticity clay with varying amounts of sand and rock fragments. SPT N-values obtained in the residuum ranged from weight of hammer (WOH) material to 8 blows per foot which indicated very soft to medium stiff consistencies. The N-values exhibiting soft consistencies were obtained in the 0-1.5 foot surface and 5-6.5 foot samples in boring B-2. It is typical in this general geographic region to encounter softer soils with elevated moisture contents at the surface and immediately above hard limestone. In addition, pocket penetrometer tests were performed for each of the samples, and the readings obtained in the residuum exhibited apparent unconfined strengths of 0.5 and 1 tsf.

Rock: Bedrock was encountered in both test borings. The top of hard rock is defined as the depth at which auger refusal was encountered during the drilling. The top of rock level ranged from approximately 4 to 8 feet below existing grades. NQ rock coring was conducted in borings B-1 and B-2 to evaluate the general competency of the bedrock. Gray limestone was encountered in both borings to termination depths of 14.5 and 18 feet below grades in borings B-1 and B-2, respectively. The recovery of the core samples ranged from 85 to 98 percent. The Rock Quality Designation (RQD) of the recovered rock ranged from 0% to 95% which indicated very poor to excellent quality rock.

Groundwater Observations

The borings were checked for the presence of groundwater both during and upon completion of the drilling. We did not detect an apparent static groundwater level in any of the borings during or upon completion of the auger drilling. It is important to note that fluctuations in perched water and groundwater levels may occur due to variations in environmental conditions, surface drainage and other factors which may not have been evident at the time measurements were made and reported herein. Therefore, the earthwork contractor should be prepared to implement dewatering measures during the earthwork phase of the project.

LABORATORY TESTING

Laboratory tests were performed to supplement the field classifications, assess potential volume change characteristics and establish geotechnical design criteria. All laboratory tests were completed in accordance with appropriate ASTM standard test methods. Detailed results of the laboratory tests are contained in Appendix C. A summary of the test results is presented below.

TEST TYPE	TEST RESULTS
Natural Moisture Contents	6.3 % to 30.6%
Atterberg Limits: Liquid Limit Plasticity Index	27 18
Percent Passing No. 200 Sieve	35 %
USCS Soil Classification	GC
Unconfined Compressive Strength (Rock)	8,770 to 13,150 psi

DESIGN RECOMMENDATIONS

Foundations

Based on the underlying karst terrain and the results of the test borings, we believe that subsurface conditions for the tank site are generally suitable for the proposed tank construction from a geotechnical standpoint. It is our opinion that the proposed tank can be founded on a conventional mat foundation bearing on approved in-situ soils or new controlled fill. We recommend that a maximum allowable bearing pressure of 3,500 psf (3.5 ksf) be utilized for design of foundations bearing on competent limestone bedrock. However, the soft soils in boring B-2 and the erratic top of rock surface at the site present a concern in terms of overall bearing services. Due to these concerns, over-excavation and replacement of poor bearing soils will be necessary. Due to the limited access with the drill rig, the extent of over-excavation and replacement cannot be accurately determined at this time. Therefore, we recommend extensive proof-rolling with a fully loaded tandem axle dump truck within the new tank footprint once the old tank and foundation elements are removed. The proof-rolling should include several

passes in both the general north-south and east-west orientation. Additionally, all bearing testing should be extended to at least 2 feet below the final bearing surface prior to placement of steel and concrete. In an effort to protect the approved bearing surface during placement of steel reinforcement, we recommend placing a 3" thick layer of "lean mix" concrete (i.e. mud mat). This will help protect the subgrade and provide a very stable surface for placing and tying steel reinforcement.

Based on the assumed structural loads and the results of the exploration, it is estimated that the total settlement for the tank bearing on approved residual materials or new fill will be on the order of one (1) inch or less. We do not anticipate significant differential settlement due to the uniform loading of the tank, provided that the unsuitable soils are removed and replaced with new controlled fill.

Floor Slab

The tank will include a concrete slab supported on grade that will likely bear on new controlled fill. We recommend that a modulus of subgrade reaction, "k", equal to 110 pci be adopted for analysis and design of the slab-on-grade. Depending on the actual type of fill material used to achieve the final subgrade level once the over excavation and replacement efforts are complete, the modulus value may be increased. The floor slab should be underlain by a minimum six (6) inch thick layer of well-graded, crushed aggregate such as VDOT 21B aggregate or similarly graded material.

Seismic Site Classification

The project site is located in Winchester, Virginia which is considered to be a low seismic risk region. We recommend that site class "C" be utilized for seismic design of foundations. This recommendation is for the designer utilizing the International Building Code (IBC) 2015 guidelines. Liquefaction potential of the on-site soils is considered to be negligible.

CONSTRUCTION RECOMMENDATIONS

Site Preparation

Initial site preparation should include removal of all topsoil, overburden soil, old foundation elements, fractured/decomposed rock and any other deleterious materials within 5 feet of the proposed tank footprint. Upon removal of all deleterious material and prior to any fill placement or new construction, the underlying soil not removed for foundation construction should be re-densified with appropriate compaction equipment and proof-rolled with approved construction equipment. The proof-rolling should be conducted with equipment such as a fully loaded tandem-axle dump truck or approved equivalent to locate isolated soft spots or areas of excessive "pumping" which are too wet to accommodate compacted fill. Correction of unstable areas can include scarification, air-drying to a sufficient moisture content and re-compaction prior to fill placement or excavation to a level of stable soils and replacement with new fill.

Excavation Areas

As indicated in previous sections of this report, the carbonate bedrock present beneath the site generally weathers differentially to produce an irregular top of rock profile. Consequently, it is quite impossible to predict where rock will be encountered at locations between specific exploration points. In general, overburden soils present can be excavated with conventional earth moving equipment such as backhoes and tracked loaders. Decomposed rock encountered can possibly be removed to a very limited extent with a ripper. This layer, however, is typically thin and the transition from soil to hard rock is somewhat abrupt. Hard bedrock or large boulders will require blasting, hoe-ram chipping or hydraulic splitting for effective removal. Any blasting should be performed by a licensed contractor with at least 5 years of blasting experience. All cut areas should be sloped and/or supported in accordance with current Occupational Safety and Health Administration (O.S.H.A.) Guidelines.

Proper drainage of excavation areas will be very important in overall construction progress. During excavation work, dry conditions should be maintained within the cut areas at all times in order to reduce the need for additional undercutting or aeration of soils. The contractor should be prepared to implement, if necessary, temporary de-watering measures in these areas during construction. These measures can include sloping the cut areas to appropriate sump pit(s) and pumping accumulated surface runoff from precipitation events. All cut areas consisting of soil should be sealed at the end of each day, to the extent which construction practicality will permit, to help prevent infiltration of precipitation and subsequent unsuitable soil conditions.

Controlled Fill

Satisfactory Soils

On-site materials excavated from cut areas can generally be used for fill provided that compaction criteria are strictly maintained. The on-site materials may have to be conditioned to attain satisfactory moisture contents for compaction. The moisture content for any new fill should be maintained within three (3) percent of the optimum moisture content based on the Standard Proctor method (ASTM D 698). This will be very dependent upon seasonal conditions at the time of earthwork construction. Also, the fat clays are relatively sensitive to moisture fluctuations and typically can be effectively placed and compacted only during drier seasons.

Fill materials should not contain any debris, waste, or frozen materials and they should contain less than two (2) percent vegetation-organic materials by weight. Also, materials classified as OL, OH, or Pt are not suitable for use as structural fill. On-site high plasticity soils are generally suitable for re-use as structural fill provided that proper drainage, grading, and sloping away from the structure is maintained both during and after construction. Blasted or "shot" limestone rock can be utilized for fill provided that certain construction procedures are observed, if applicable. These procedures include maintaining the maximum particle size of the rock, prohibiting nesting of boulders, and mixing sufficient amounts of soil fines with the rock to fill in open voids between the rock particles.

Controlled fill for the project should be free of rock larger than four (4) inches in any dimension. All proposed fill materials should be approved by a geotechnical engineer prior to placement as controlled fill, and representative samples should be obtained one week prior to placement of that material to allow time for completion of the necessary laboratory tests.

Placement and Compaction

Prior to compaction, the moisture content of each layer should be adjusted, as necessary, to obtain the required moisture content to achieve the specified compaction level. Each layer should be compacted to the required percentage of maximum dry density. Fill should not be placed on surfaces that are muddy or frozen, or have not been approved by testing and/or proof-rolling. Free water should be prevented from appearing on the surface during or subsequent to compaction operations.

Soil material which is removed because it is too wet to permit proper compaction can be spread and allowed to dry. Drying can be facilitated by discing or harrowing until the moisture content is reduced to an acceptable level. When the soil is too dry, water should be applied uniformly to the subgrade surface or to the layer to be compacted.

All fill material compacted by heavy compaction equipment should be placed in maximum 9-inch loose lifts. All fill material compacted by hand-operated tampers or light compaction equipment should be placed in maximum 4-inch loose lifts.

Any fill material placed within the tank footprint and extending five (5) feet beyond its perimeter should be compacted to at least 100 percent of the laboratory maximum dry density as determined by the Standard Proctor method (ASTM D 698). The moisture content of the soils should be at or within three (3) percentage points of the optimum moisture content. In areas where mixtures of shot rock and soil fill are placed, minimum passes with compaction equipment should be established during construction, and all areas should be proof-rolled with approved equipment for acceptance of compaction where in-place moisture-density tests are not feasible.

Foundation Construction

It is anticipated that conventional earth excavation equipment such as a backhoe or trackhoe can be utilized to excavate the overburden to firm, approved soils and to some extent the fractured/decomposed limestone rock for mat foundation construction. After all the overburden soil and fractured/decomposed rock has been removed, the over-excavated materials should be backfilled with approved soil or dense-graded aggregate such as VDOT 21B. We recommend that a 3” thick layer of “lean mix” concrete (i.e. mud mat) be placed to provide a uniform and level bearing surface upon which to place and tie reinforcing steel for the mat foundation. All excavations adjacent to the mat foundation should be backfilled with soil and properly compacted after the foundation has been constructed to the tank bottom level. This measure will help prevent potential ponding of water during precipitation events and subsequent strength reduction of any soils located adjacent to the foundation.

Any underground utilities which are located below or adjacent to the new tank foundation should be backfilled with lean mix concrete or flowable fill grout to grades which are at or above the design bearing levels immediately after foundation construction is complete. In addition, minimal thicknesses of bedding stone should be utilized beneath the utility lines in order to help prevent significant accumulation of water from precipitation developing within the utility trench area.

Utility Construction

Locations and invert elevations for proposed utilities have not been provided to us. In general, we anticipate that conventional excavation equipment such as a backhoe or trackhoe can be used for utility excavations in the residual soils and any new controlled fill. Any excavations which encounter hard bedrock will require blasting or hoe-ram chipping to attain scheduled invert elevations. Blasting should be performed by a licensed contractor with at least 5 years of experience. All utility trenches should be sloped and/or supported in accordance with current O.S.H.A. requirements.

In areas where “shot” rock and soil fill has been placed during mass earthwork construction, an acceptable substitute backfill material should be used for new utility trench backfill. This is recommended because of the inherent difficulty in re-compacting “shot” rock materials in trenches using small, hand-operated equipment. The substitute material should comply with the maximum particle size restrictions specified for the particular utility. Trenches below the tank footprint should be backfilled in accordance with the Controlled Fill section of this report.

Construction Observations

We recommend that the geotechnical engineering firm of record, Triad, be retained to observe the construction activities to verify that the field conditions are consistent with the findings of our exploration. If significant variations are encountered, or if the design is altered, we should be notified. We should provide personnel as required to:

- observe final surface material removal and witness and document proof-rolling of the original subgrade prior to any fill placement.

- examine, test and approve fill construction. Field density tests should be performed in accordance with ASTM D 6938 (nuclear method). At least three field density tests should be conducted for each lift, or at a frequency determined to be sufficient by the geotechnical engineer based on the amount of fill being placed, to confirm the required soil compaction.

- examine, test and verify all foundation bearing surfaces, foundation depths, and reinforcing steel size, amount and placement for the structure. All foundation bearing levels should be examined immediately prior to placing reinforcing steel and concrete to confirm that the required bearing support is available.

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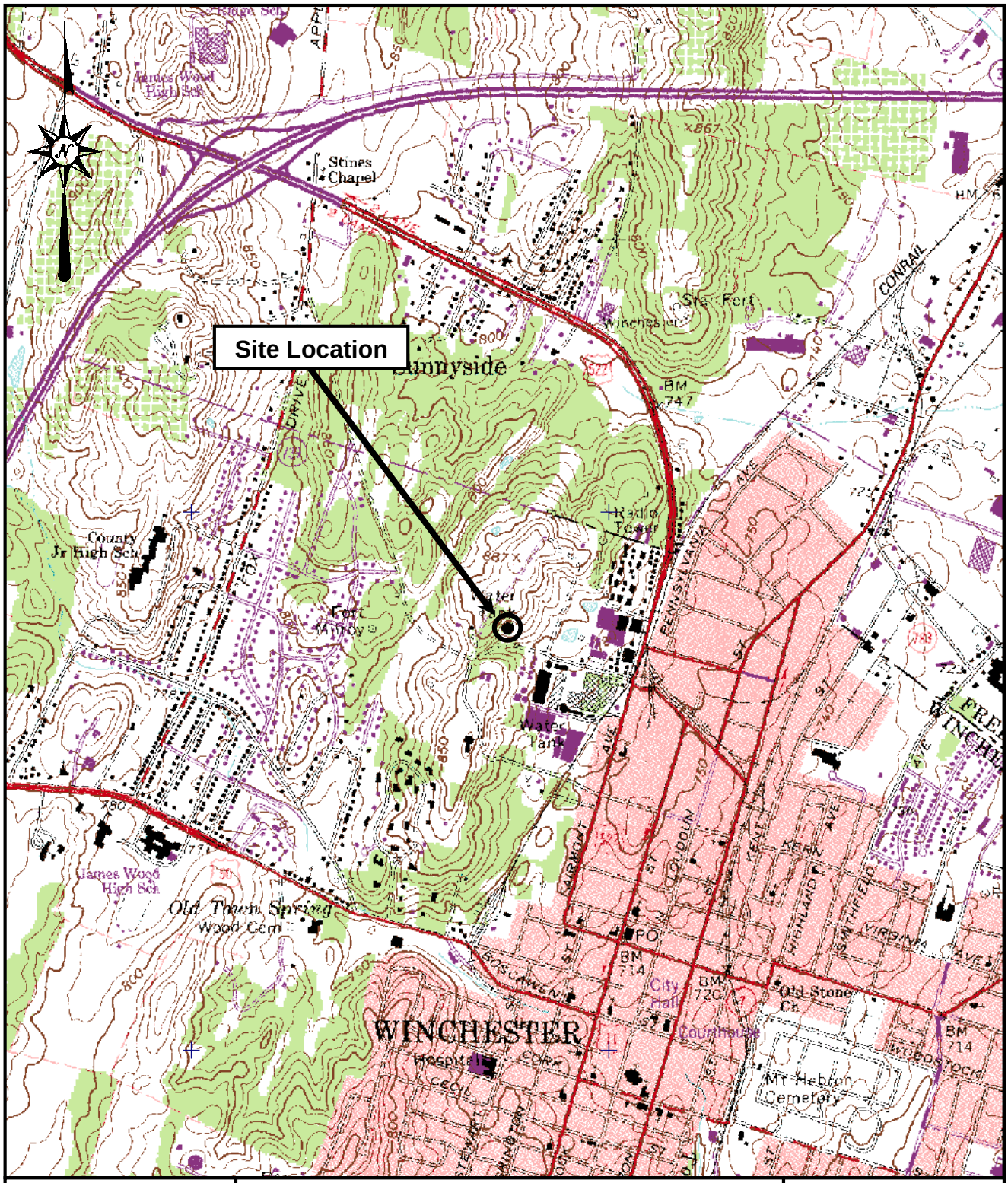
APPENDIX A

Illustrations



SOURCE: USGS 7.5 Winchester (VA) 1987; Topographic Maps		STROTHERS LANE WATER TANK WINCHESTER, VIRGINIA
DRAWN BY: MWB	CHECKED BY: RLM	**SITE VICINITY PLAN**
DATE: 01-05-2018	SCALE: 1"= 2000'	**TRIAD PROJECT NO. 07-17-0166**

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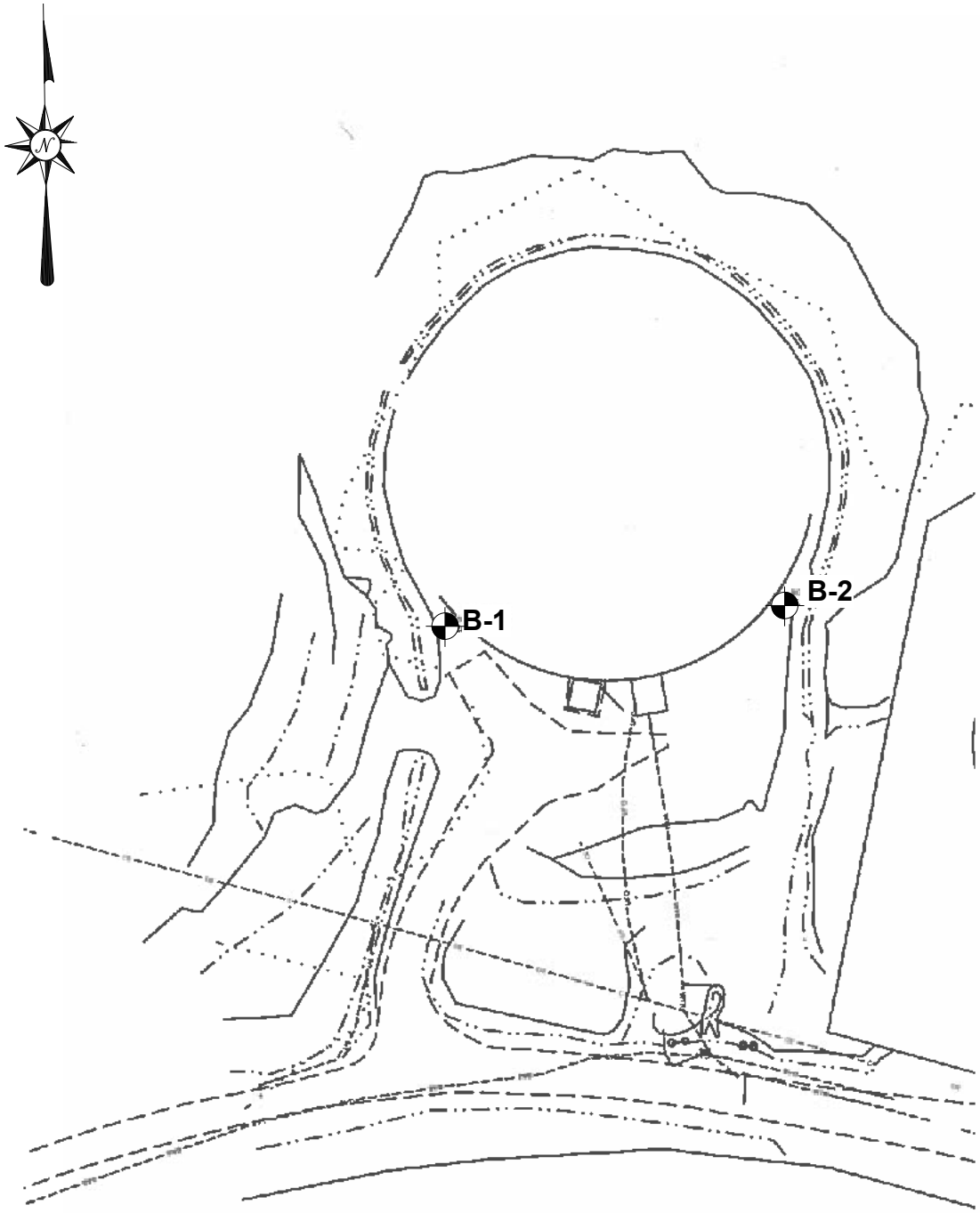
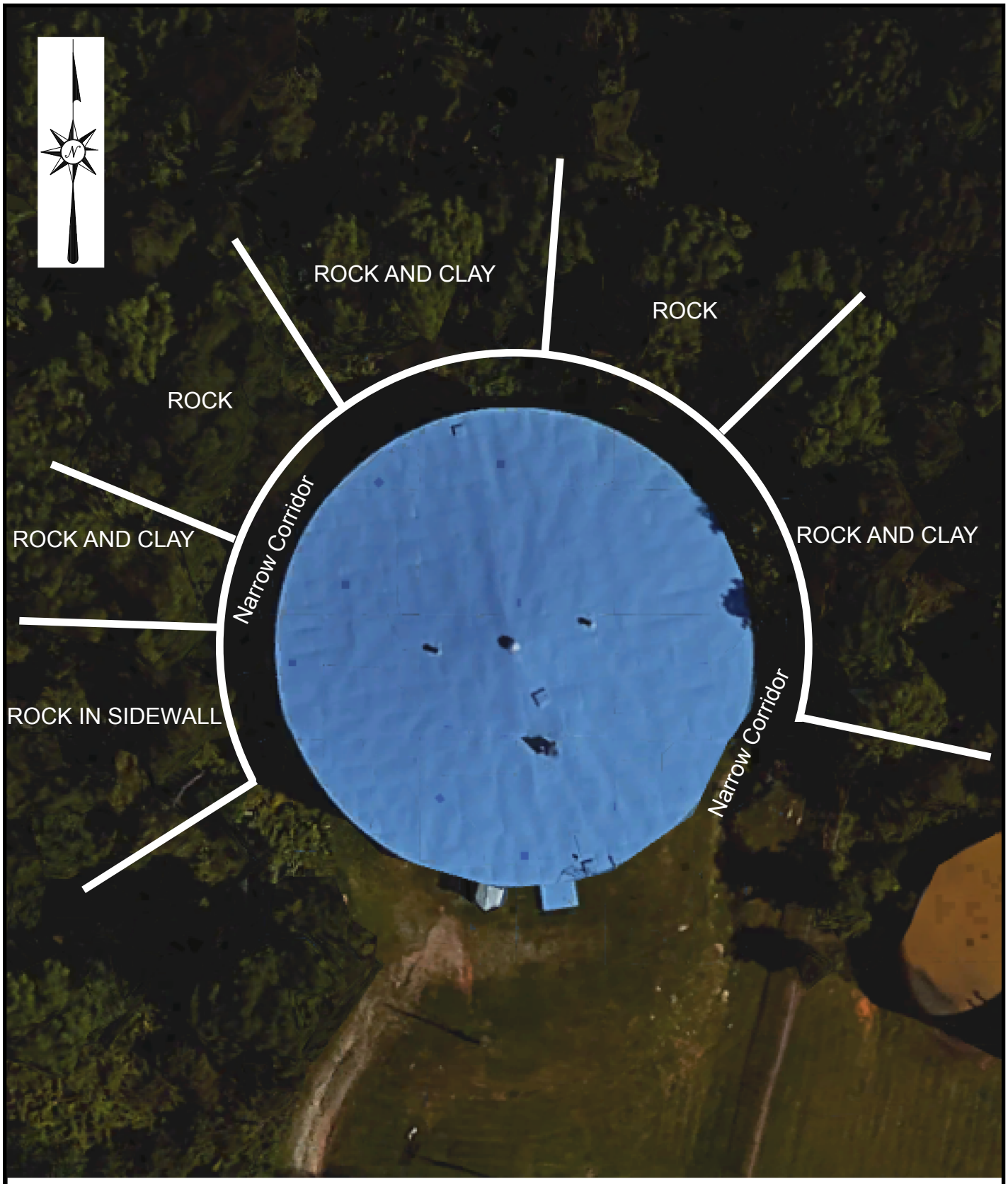
FIGURE NO.: A-1



- Approximate Boring Location

Location Plan is Approximate.
For Reference Purposes Only.

SOURCE: Base Map Triad		STROTHERS LANE WATER TANK WINCHESTER, VIRGINIA	TRIAD ENGINEERING, INC. www.triadeng.com
DRAWN BY: RAS	CHECKED BY: RAS	BORING LOCATION PLAN	
DATE: 1-23-2018	SCALE: 1"=50'	TRIAD PROJECT NO: 07-17-0166	FIGURE NO.: A-2



Location Plan is Approximate.
For Reference Purposes Only.

Base Map Provided By: GOOGLE EARTH		STROTHERS LANE WATER TANK WINCHESTER, VIRGINIA	TRIAD ENGINEERING, INC. www.triadeng.com
DRAWN BY: MWB	CHECKED BY: RLM	ROCK OUTCROP LOCATION PLAN	
DATE: 01-05-2018	SCALE: NTS	TRIAD PROJECT NO: 07-17-0166	FIGURE NO.: A-3

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APPENDIX B

Field Exploration

FIELD EXPLORATION

The subsurface conditions at the site were explored by drilling two (2) test borings with Standard Penetration Tests (SPT) and sampling. NQ rock coring was performed at both locations to further evaluate the rock at the site. The borings were drilled by Triad utilizing a track-mounted rotary auger drill rig and hollow stem augers to advance the hole. The field exploration was supervised by geotechnical engineering personnel from our office.

SPT and sampling was performed in accordance with ASTM D 1586. The SPTs were performed to depths indicated on the attached boring log using a split barrel sampler with an outside diameter of two (2) inches and an inside diameter of one and three-eighths (1-3/8) inches. The split barrel sampler was driven eighteen (18) inches with a hammer weighing approximately 140 pounds and falling thirty (30) inches. The number of blows required to drive the split barrel sampler at six (6) inch increments was recorded on the boring logs. The method utilized to classify the soils is defined in Figure No. 1, Key To Identification Of Soils And Weathered Rock Samples. The method utilized to classify the rock is identified in Figure No. 2, Key To Identification of Hard Rock.

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KEY TO IDENTIFICATION OF SOIL AND WEATHERED ROCK SAMPLES

The material descriptions on the logs indicate the vis ual identification of the soil and rock recovered from the exploration and are based on the following criteria. Major soil components are designated by capital letters and minor components are described by terms indicating the percentage by weight of each component. Standard Penetration Testing (SPT) and sampling was conducted in accordance with ASTM D1586. N-values in blows per foot are used to describe the *relative density* of coarse-grained soils or the *consistency* of fine-grained soils.

The MAJOR components constitute more than 50% of the sample and have the following size designation.		The MINOR components have the following percentage designation.	
COMPONENT	**PARTICLE SIZE**	**ADJECTIVE**	**PERCENTAGE**
Boulders	12 inches plus		
Cobbles	3 to 12 inches	and	35 - 50
Gravel -coarse	¾ to 3 inches		
-fine	#4 to ¾ inches	some	20 - 35
Sand -coarse	#10 to #4		
-medium	#40 to #10	little	10 - 20
-fine	#200 to #40		
Silt or Clay	Minus #200 (fine-grained soil)	trace	0 - 10

Relative Density – Coarse-grained Soils		**Consistency – Fine-grained Soils**	
Term	***N-Value***	***Term***	***N-Value***
Very Loose	≤4	ery Soft	≤ 2
Loose	5 to 10	Soft	3 to 4
Medium Dense	11 to 30	Medium Stiff	5 to 8
Dense	31 to 50	Stiff	9 to 16
Very Dense	>50	Very Stiff	>16

Soil Plasticity	**Plasticity Index (PI)**	**Rock Hardness**	
None	Nonplastic	***Term***	***N-Value***
Low	1 to 5	Very Weathered	≤ 50/.5
Medium	5 to 20	Weathered	50/.4
High	20 to 40	Soft	50/.3
Very High	over 40	Medium hard	50/.2 to 50/.1
Moisture Description		Hard	Auger Refusal
Dry - Dusty, dry to touch			
Slightly Moist - damp			
Moist - no visible free water			
Wet - visible free water, saturated			

FIGURE NO. 1

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KEY TO IDENTIFICATION OF HARD ROCK SAMPLES

The material descriptions on the logs indicate the visual identification of the rock recovered from the NQ/NX coring operations and are based on the following criteria. Core recovery is the ratio of the length of core recovered in each run to the total length of the core run in percent. Rock Quality Designation (RQD) is the ratio of the sum of the lengths of rock core pieces 4 inches or longer divided by the length of the core run in percent.

Relative Degree of Rock Hardness

Term	***Defining Characteristics***
Very Soft	Can be indented by thumb or crushed under pressure of finger and/or thumb
Soft	Can be scratched by fingernail, peeled by pocket knife or crushed with pressed hammer
Medium Hard	Cannot be scraped or peeled with knife but can be scratched, breaks easily with hammer blow
Hard	Breaks under one or two strong hammer blows or scratched with knife with difficulty
Very Hard	Breaks under several strong hammer blows with very resistant sharp edges

Rock Adjectives

Seam	Thin layer (12 inches or less)
Interbedded	Thin or very thin alternating seams of bedrock occurring in equal amounts
Some	Significant amount of accessory material (15 to 40 percent)
Few	Insignificant amount of accessary material (0 to 15 percent)

Rock Quality Designation (RQD)		**Recovery**	
Term	***Percent***	***Term***	***Percent***
Very Poor	≤ 25	Poor	≤ 25
Poor	26 to 50	Low	26 to 50
Fair	51 to 75	Moderate	51 to 75
Good	76 to 90	High	76 to 90
Excellent	>90	Very High	>90

Rock Structure

Degree of Fracturing		**Thickness of Bedding**	
Term	***Spacing***	***Term***	***Spacing***
Intensely fractured or very broken	2 in.	Thinly bedded	<4 in.
Highly fractured or broken	2 in. to 8 in.	Medium bedded	4 in. to 1 ft.
Moderately fractured or blocky	8 in. to 2 ft.	Thickly bedded	1 ft. to 3 ft.
Slightly Fractured	2 ft. to 6 ft.	Massive	>3 ft.

Dip of Bed or Fracturing	
Flat	0º to 20º
Dipping	20º to 45º
Steeply Dipping	45º to 90º

FIGURE NO. 2

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TEST BORING LOG

Project Number: 07-17-0166 | Project Name: Strothers Lane Pump Station | Boring No.: B-1

Logger: KBA | Boring Location: See Figure No. A-2

Date Started: 12/28/17 | Drill/Method:

Date Completed: 12/28/17 | Driller: TRIAD | Ground Elev.: 824.2

Shelby Tube | Standard Split Spoon | Core Sample | Auger Probe

Depth (feet)	Sample No.	Sample Type	Blow Counts	Recovery (%)	RQD (RUN)	Strata Depth (ft)	MATERIAL DESCRIPTION	RQD (Strata)	Water Level	Graphic Log	Strata Elevation
	S-1		22-28-17	56%			Red-brown fat **CLAY**, little to some sand and rock fragments, very stiff, medium to high plasticity, moist PP=2.25 tsf				
	S-2		12-28-17	67%			-very stiff PP=2.25 tsf -POSSIBLE FILL TO RESIDUUM- -AUGER REFUSAL AT 4 FEET-				
						4.0					820.2
5.0	RC-1			93%	87%		Gray hard **LIMESTONE**, very high recovery, good quality, broken, few calcite stringers. Bedding planes dip approximately 45 degrees from the horizontal axis. -clay filled cavity 5.4'-5.6'				
						5.5					818.7
	RC-2			96%	84%		Gray hard **LIMESTONE**, very high recovery, good quality, blocky, few calcite stringers. Bedding planes dip approximately 45 degrees from the horizontal axis.				
10.0											
						10.5					813.7
	RC-3			98%	95%		Gray hard **LIMESTONE**, very high recovery, excellent quality, broken to blocky, few calcite stringers. Bedding planes dip approximately 45 degrees from the horizontal axis.				
						14.5					809.7
15.0							-BORING TERMINATED AT 14.5 FEET-				
20.0											

Remarks: No groundwater encountered during or upon completion of drilling.

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TRIAD_C 07-17-0166 BORINGS.GPJ TRIAD 3.GDT 2/6/18

TEST BORING LOG

Project Number: 07-17-0166 | Project Name: Strothers Lane Pump Station | Boring No.: B-2

Logger: KBA | Boring Location: See Figure No. A-2

Date Started: 12/28/17 | Drill/Method:

Date Completed: 12/28/17 | Driller: TRIAD | Ground Elev.: 823.9

Shelby Tube | Standard Split Spoon | Core Sample | Auger Probe

Depth (feet)	Sample No.	Sample Type	Blow Counts	Recovery (%)	RQD (RUN)	Strata Depth (ft)	MATERIAL DESCRIPTION	RQD (Strata)	Water Level	Graphic Log	Strata Elevation
	S-1		2-2-2	44%			Red-brown fat **CLAY**, little sand, trace rock fragments, soft, medium to high plasticity, moist PP=1.0 tsf				
	S-2		4-5-3	33%			-medium stiff PP=1.0 tsf				
5.0	S-3		WOH-WOH-2	33%			-very soft PP=0.5 tsf				
	S-4		16-16-21	78%			-very stiff, some sand and rock fragments				
						8.0	-RESIDUUM- -AUGER REFUSAL AT 8 FEET-				815.9
	RC-1			85%	0%	10.0	Gray hard **LIMESTONE**, high recovery, very poor quality, very broken, few calcite stringers. Bedding planes dip approximately 30 to 45 degrees from the horizontal axis.				813.9
10.0	RC-2			94%	72%	15.0	Gray hard **LIMESTONE**, very high recovery, fair quality, blocky, few calcite stringers. Bedding planes dip approximately 30 to 45 degrees from the horizontal axis.				808.9
15.0	RC-3			97%	23%	18.0	Gray hard **LIMESTONE**, very high recovery, poor quality, broken, few calcite stringers. Bedding planes dip approximately 30 to 45 degrees from the horizontal axis.				805.9
20.0							-BORING TERMINATED AT 18.5 FEET-				

Remarks: No groundwater encountered during or upon completion of drilling.

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APPENDIX C

Laboratory Testing

LABORATORY TESTING

The soil samples obtained during the field exploration were visually classified in the field by geotechnical engineering personnel from Triad. The recovered soils were further evaluated by laboratory testing. Laboratory soil tests were conducted in accordance with applicable ASTM Standards as listed below:

1) Moisture content tests were performed in accordance with ASTM D 2216.

2) Atterberg Limits tests, consisting of the liquid limit, plastic limit, and plasticity index, were performed in accordance with ASTM D 4318.

3) Sieve analysis tests with washed No. 200 sieve test were performed in accordance with ASTM D 422.

4) Unconfined Compressive Strength Tests (Rock) were performed in accordance with ASTM D 7012.

A summary and details of the laboratory tests are included on the following pages of this appendix.

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SOIL DATA SUMMARY

SAMPLE NO.	SAMPLE DEPTH (ft)	SAMPLE TYPE	NATURAL MOISTURE (%)	ATTERBERG LIMITS			GRADATION			USCS SOIL CLASS.	PROCTOR		UNCONFINED COMPRESSIVE STRENGTH
				LL	PL	PI	% GRAVEL	% SAND	% FINES		MAX. DD (pcf)	OPT. M (%)	
B-1	0-1.5	SS	16.2										
B-1	2.5-4	SS	26.6										
B-2	2.5-4	SS	30.6	37	19	18	46	19	35	GC			
B-2	5-6.5	SS	27.3										
B-2	7.5-9	SS	6.3										
RC-1/B-1	5.0-5.3	CORE	-										13,150 psi
RC-2/B-2	11.2-11.5	CORE	-										8,770 psi

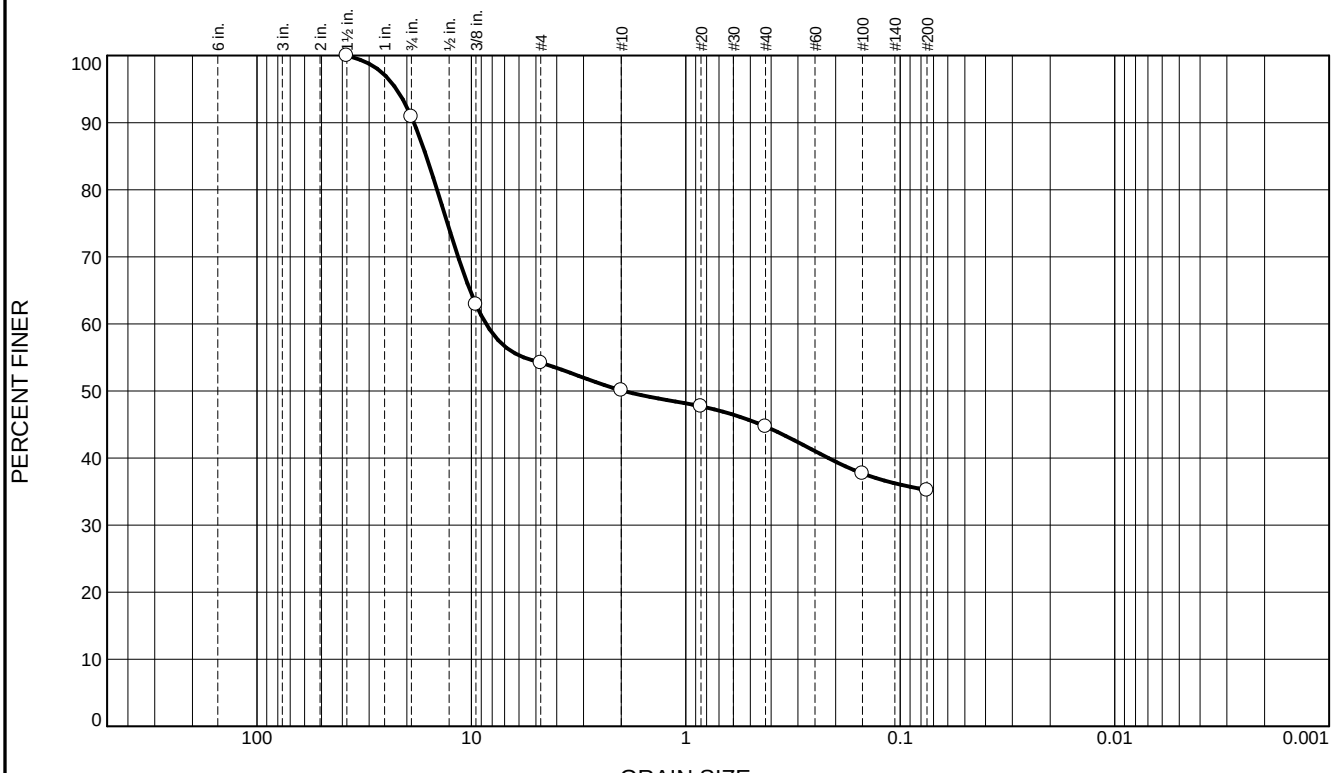
TRIAD ENGINEERING, INC.

Notes:
1) Soil tests performed in accordance with recognized ASTM testing standards.
2) SS = Split Spoon; UD = Undisturbed

PROJECT NUMBER: 07-17-0166
PROJECT NAME: Strothers Lane Water Tank
LOCATION: Winchester, VA

FIGURE C-1

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	9.1	36.7	4.1	5.4	9.5	35.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/2"	100.0		
3/4"	90.9		
3/8"	62.9		
#4	54.2		
#10	50.1		
#20	47.7		
#40	44.7		
#100	37.7		
#200	35.2		

* (no specification provided)

Soil Description

Orange-brown clayey GRAVEL, little sand

Atterberg Limits

PL= 19 LL= 37 PI= 18

Coefficients

D_{90}= 18.5300 D_{85}= 16.2292 D_{60}= 8.5418
D_{50}= 1.9481 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= GC AASHTO= A-2-6(2)

Remarks

Source of Sample: B-2 **Depth:** 2.5' - 6.5'

Date: 1-28-2018

Triad Engineering, Inc.

Client: Hazen and Sawyer
Project: Strothers Lane Water Tank
Winchester, VA
Project No: 07-17-0166 **Figure** C-2

Tested By: KBA **Checked By:** RAS