CONTRACT DOCUMENTS

For the Construction of the

SOUTHEAST WATER TREATMENT PLANT

GEOTECHNICAL REPORTS

VOLUME 4

OCTOBER 2014

Prepared For:

Huntsville Utilities

Prepared By:

TETRA TECH

For information regarding this project, contact:
Christian P. Dunaway, P.E.
Tetra Tech, Inc.
101 Quality Circle, Suite 140
Huntsville, AL 35806
(256) 424-4077

Tetra Tech #200-11740-10003

BID SET

OMI, Inc.

SUBSURFACE EXPLORATION AND
GEOTECHNICAL ENGINEERING STUDY
Proposed
Guntersville Dam WTP
Intake Structure and Raw Water Line
Marshall County, Alabama

OMI Job No. 5085

March 31, 2008

5151 Research Drive, NW Huntsville, AL 35805 Tel: (256) 837-7664 Fax: (256) 837-7677 E-mail: omi@omi-eng.com

OMI, Inc.

March 31, 2008

Tetra Tech
101 Church Street Suite 201
Huntsville, AL 35801

ATTN: Ms. Ashley Turnbull

SUBJECT: Report of Geotechnical Engineering Study
Proposed Guntersville Dam WTP Intake Structure and Raw Water Line
Marshall County, Alabama
OMI Job No. 5085

Gentlemen:

OMI, Inc., has completed a subsurface exploration and geotechnical engineering study for the referenced project. Enclosed is the report of the findings as well as recommendations for foundation design and construction, site preparation, and other geotechnically related site activities. This work was authorized on March 4, 2008, by Ms. Ashley Turnbull of Tetra Tech, Inc.

OMI, Inc., appreciates the opportunity to be of service to Tetra Tech and Huntsville Utilities, and looks forward to continued involvement with the construction monitoring phase of this project. Please direct any questions concerning this report to the undersigned.

Respectfully submitted,

OMI, Inc.

Jeremy C. Jess, E.I.
Staff Engineer

John M. Ozier, P.E.
Senior Engineer

Distribution: 3 Copies to Addressee

5151 Research Drive, NW Huntsville, AL 35805 Tel: (256) 837-7664 Fax: (256) 837-7677 E-mail: omi@omi-eng.com

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APPENDICES

1.0 EXECUTIVE SUMMARY

OMI has completed the subsurface study for the proposed intake facility and raw water line at Guntersville, Dam in Marshall County. Based on the conditions encountered in the six borings that were performed on the site it appears rock will be encountered on the south end of the proposed line. Soil cover increases in thickness from 0-ft at the south end to greater than 10-ft on the north end. The intake structure at the south end will bear on intact clean limestone rock. For the raw water line rock excavation will vary from 5-ft of rock excavation to no rock excavation. Most of the rock excavation will occur at the south end, but variations should be expected between borings.

Specific recommendations for the intake facility's foundation design and trench methods are given in the body of this report.

2.0 INTRODUCTION

OMI, Inc., has completed a design geotechnical engineering study for the proposed Guntersville Dam water treatment plant intake structure and raw water line. This report outlines the scope of services provided and presents comments and recommendations based on professional opinions formed during the course of this study. This work was verbally authorized on March 4, 2008, by Ms. Ashley Turnbull, project manager for Tetra Tech Inc. The work was performed in general accordance with OMI Proposal No. P-3176.

Assessment of the environmental aspects of this site, including previous land use or the determination of the presence of any chemical, industrial, or hazardous waste is beyond the scope of this study. However, OMI can provide these services if desired.

3.0 EXPLORATION METHODS

The procedures used by OMI for field and laboratory testing are in general accordance with ASTM procedures and established engineering practice. Brief descriptions of the procedures used in this exploration are contained in the Appendix of this report.

Six soil borings were performed during this study. This includes one boring at the intake location to 20-ft, and five borings to 10-ft each except for Boring B-5 which encountered auger refusal at 4-ft below the surface. Boring locations are shown on the appended Boring Location Plan. A member of the OMI professional staff directed the drilling. Subsequently, each sample was sealed and transported to the office. The material classifications are provided on the Soil Boring Records in the Appendix of this report.

4.0 SITE CONDITIONS

The site for the proposed intake and raw water line is located northeast of the existing dam. The proposed intake location is 650-ft south of the end of Guntersville Dam Road. A old rock cut area is located approximately 100-ft northwest of the proposed intake. Several abandoned structures are located in a fenced area in the rock cut. The general area is wooded, but a well defined trail leads from the rock cut area to the proposed location of the intake structure. Rock outcroppings and loose boulders are visible at the surface. The southern portion of the proposed raw water line is wooded approximately 400-ft north form the proposed intake structure. The 250-ft length from the woods to the cul-de-sac of Guntersville Dam road is covered with "surge stone" or "rip-rap."

The section of the proposed raw water line that is proposed to be along Guntersville dam road will be approximately 5-ft east of the edge of the asphalt paved road. This area is currently a grassed ditch. Topographically, the area along the line is relatively well drained, and appears to drain toward the south to Guntersville Lake on the southern half and the northern half appears to drain toward the west and then to the Tennessee river.

5.0 SUBSURFACE CONDITIONS

Raw Water Intake Structure

Subsurface conditions at the proposed intake location included Limestone rock from surface to 13.5-ft below the surface. Underlying this rock layer is 2 feet thick layer of low plastic clay. Rock again was encountered from 15-ft to the boring termination depth of 20-ft below the surface. Rock Quality Designation, RQD, of 50 percent was measured for the rock layer from 15 to 20-ft below the surface.

Raw Water Line

Along the proposed raw water line, rock was encountered at various depths from at the surface at the intake location to deeper than boring termination depth in Borings B-4 and B-6. Intermittent rock and clay layers were encountered in the 10-ft boring B-2. Boring B-3 found 7.5-ft of rip-rap or surge-stone fill before bedded limestone was encountered. Boring's B-4 and B-6 encountered stiff to very stiff low plastic clays before the boring termination depth of 10-ft below the surface. B-5 encountered similar clays until auger refusal was encountered at 4-ft below the surface.

Groundwater:

No groundwater was observed during drilling. Due to the concerns of TVA the borings were filled after drilling; therefore, extended water levels were not taken. OMI recommends estimating groundwater to be near the elevation of the Guntersville Lake in the area of the intake structure. Lower water tables may be expected near borings B-4 through B-6. Because of the geology of this region, the groundwater levels are generally a function of seasonal precipitation and locally heavy rainfall events. Consequently, the groundwater levels can and do fluctuate with time.

6.0 SITE GEOLOGY

Review of the published geologic literature and field verification by OMI personnel, indicated that the site is underlain by the Bangor Limestone. Geologically, the Bangor Limestone in Marshall County generally lies above the Hartselle Sandstone and below the Pennington Formation.

In Marshall County, the Bangor Limestone is composed of about 350 to 420-ft of bioclastic and oolitic limestone, dolomite, and shale. Chert contained in the Bangor is generally small black nodules found in the upper part of the formation. The upper part of the Bangor grades northeastward into the Pennington Formation and is generally composed of green to gray, calcareous shales and thin beds of dolomite and limestone. The middle part of the Bangor is generally medium to massive-bedded limestone and dolomite. The basal part of the formation is generally medium to massive-bedded, argillaceous limestone with occasional partings of yellow calcareous shale.

7.0 PROJECT INFORMATION

Raw Water Intake Structure:

The proposed intake line for the Guntersville Dam water treatment Plant is located on the North side of the Tennessee River in Marshall County. The intake for the line is expected to be located approximately 1000-ft east of the dam. Near the intake point, rock is evident at the surface. The intake structure is expected to be a cast in-place concrete building housing the pumps. Typically, intake structures have plan dimensions of 30 feet square and extend 15 to 20 feet below the water surface. Loads on the intake structure are expected to be less than 10 kips per linear foot wall loads. The foundation of the intake structure should bear on rock and the raw water line will be in rock excavation.

Raw Water Line

The line is currently expected to have an invert elevation approximately 5-ft below the existing ground surface elevations. The line will follow along Guntersville Dam Road on the north east side of the pavement. The line length for this study included 2000 linear feet.

8.0 DESIGN RECOMMENDATIONS

8.1 Intake Structure Foundation Design

Due to the shallow nature and high quality of the bedrock, a rock bearing spread footing appears to be more suitable at this site. To prepare the site, the area of the foundation should be excavated to remove all soil from the top of the rock. The foundation can be designed using a net bearing pressure of 10,000 psf. This bearing pressure assumes that the footings are bearing on sound, competent rock. OMI recommends that this strip footing have a minimum width of 18-in and bear directly on sound, clean rock. It appears based on Boring B-1 that an adequate bearing surface will be 17-ft below the surface which is estimated to be 10-ft below the lake elevation.

Dowels should be used to anchor the foundation to the rock and provide resistance to lateral movement. OMI recommends that No. 8 bars be grouted into 2-in diameter holes which are drilled at least 3-ft into the rock. Each dowel hole should be checked and probed by OMI to make sure that no major joints, caves or voids exist in the upper surface of the rock that could result in the entire mass moving under the lateral loads.

Uplift resistance can be developed by grouting bars into dowel holes drilled into the rock. The depth of the dowel hole will be determined by the quality of the rock, the bond between the grout, the inside of the dowel hole and the development length of the rebar. For design purposes, OMI recommends that a 2-in diameter hole with a No.8 bar grouted into a 48-in deep probe hole will have an allowable uplift capacity of 15,000-lb.

Any overlying structures should be designed to bear on the walls and foundation system.

8.2 Below-Grade Walls

The walls must be designed to withstand lateral earth pressures induced on them. OMI recommends the use of cast-in-place, reinforced concrete for these walls. The following recommendations for design may only be used if the backfill conditions are also followed.

The walls are fixed and are not allowed to deflect under lateral loads. These walls should be designed using at-rest lateral earth pressures (K_o of 0.67) plus hydrostatic pressures and may be approximated by using an equivalent fluid weight of 101 pcf if backfilled with soil, but use 87 pcf (K_o of 0.43) if backfilled with No. 57 stone as recommended by OMI. These values assume the water table to be at the surface. Any loads that will be placed near the top of the wall should also be considered. Surcharge loads must also be considered. Appropriate factors of safety must be applied.

8.3 Below-Grade Walls Backfill

OMI recommends the use of an open graded stone such as No. 57 for backfill around the intake structure. Using stone will allow for drainage during construction and will speed up the process. The use of soil backfill is also suitable, but will require more compaction effort. Compaction of the soil will be difficult at deeper depths due to groundwater infiltration.

Stone should extend from the base of the wall up to about 2-ft below final grade. The backfill should be placed in lifts not exceeding 24-in and densified. Heavy compaction equipment should not be operated near the wall. All unsupported walls should be adequately braced during backfilling operations to prevent damage to the wall. A cap of compacted clay should be placed over the stone to limit migration of surface water into the backfill. Please be aware that the use of "drainboards" and impervious backfill may significantly increase the actual load on the wall.

Soil Backfill

If on site soil is to be used for backfilling, the fill should be placed in 8-in loose lifts and compacted to 95 to 97 percent of the soils Standard Proctor maximum dry density. Higher compaction should be avoided. This method should result in 6-in compacted layers up to the surface.

8.4 Seismic Classification

OMI has reviewed the soils at the site with respect to the criteria for seismic classification. In accordance with Section 1615.1, Table 1615.1.1 of the 2000 International Building Code, OMI judges the soil to be Site Class B.

9.0 CONSTRUCTION CONSIDERATIONS

9.1 Raw Water Line Excavation

Based on the borings it appears rock excavation methods should be expected from the intake structure continuously to approximately 200-ft along the raw water line from the intake point. From that point intermittent rock excavation should be expected, but rock excavation in this portion will likely only consist of 10 percent or less of the excavation. It appears the remaining 90 percent of the trench may be excavated by typical earth excavation, or track hoe methods.

9.2 Rock Bearing Foundation Construction For The Intake Structure

Construction of the rock bearing foundations is relatively straightforward. However, there are several items that are important. The excavation must extend to suitable rock and all mud and loose soil must be cleaned from the rock surface.

Weathered joints will likely cross the footing area. These joints may be less than one inch across and may be several feet wide. The depth can also be several feet deep. Any joints must be cleaned of soil. The soil should be dug out to a depth equal to the width of the joint.

Two-inch diameter probe holes should be drilled into the rock to check for the quality of the bedrock at the footing location. The probe holes should be 6-ft deep. OMI recommends that four probe holes be drilled for each footing. Additional probe holes may be required depending on the quality of the rock. An engineer from OMI should check the footing, the probe holes and the dowel holes before any dowels are grouted or the rebar is placed.

Dowels into the rock are planned to resist lateral loads. OMI recommends that No. 8 rebar be used as dowels. Each dowel hole should be drilled and cleaned out before the footing is checked. The dowel holes should be 2-in in diameter and drilled a sufficient depth into the rock. For this job, dowel holes should be drilled at least 3-ft into the rock. Just prior to grouting the following procedure should be followed.

1) Each dowel hole should be filled with clean water and blown out with compressed air to clean the inside of the dowel hole to ensure an adequate bond between the grout and the rock. Repeat as necessary.
2) All water should be blown out of the hole.
3) The hole should be filled with a high strength non-shrink grout.
4) The No. 8 dowel should be inserted into the grout filled hole and rodded up and down several times to ensure the dowel, grout and dowel hole bond to each other and to remove air pockets.
5) The dowel should extend up into the footing the proper distance, and be allowed to set properly before any other work is performed.

9.3 Estimated Topsoil Removal Along Raw Water Line

The depth of topsoil varies greatly across the site. OMI believes that the stripping depth to remove the topsoil will average about 3-in.

9.4 Groundwater Control

Due to the proximity of the lake, dewatering will be required at the intake and along the southern end of the water line where the trench depth will be lower than the lake elevation. Once the trench elevation is above the lake, less infiltration can be expected and dewatering can be accomplished with typical methods. Cofferdam construction will be difficult. The bottom surface of the lake is likely composed of rock and driving sheet piling will not be an option. Other cofferdam methods must be planned for.

9.5 Fill Placement

After piping is complete, placement of structural fill may begin, as necessary. Specific requirements of the soil properties are discussed previously. The soil should be placed in loose lifts, not exceeding 8-in in thickness, and systematically compacted to at least 95 percent of the soil's standard Proctor maximum dry density (ASTM D698) except the top 1-ft should be compacted to 100 percent SPMDD in areas to be paved.

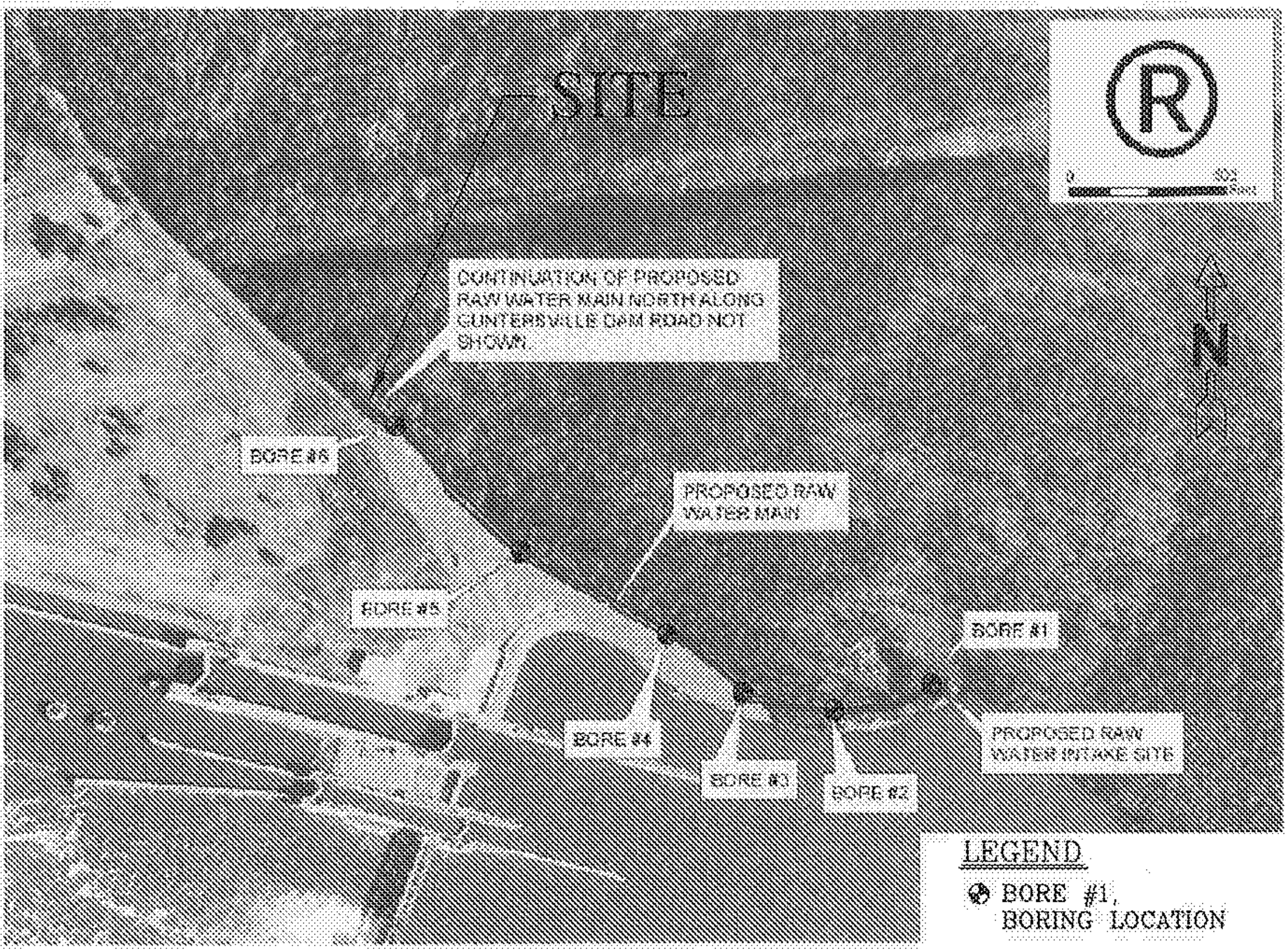
9.6 Construction Monitoring

The foundation and site preparation recommendations contained in this report are based on the conditions encountered during the subsurface exploration and past experience in this geologic setting. Because subsurface conditions may vary from the anticipated, it is important to have a well-rounded quality control program. Construction monitoring on a project of this nature can serve as an economical means to achieve the best possible foundation system and reduce the potential for future problems. The involvement in the subsurface exploration portion of this project uniquely qualifies OMI, Inc., to provide these services as a party responsible to the Owner. OMI, Inc., strongly recommends that all construction monitoring be performed under contract with the Owner or the Owner's representative.

APPENDICES

OMI, Inc.

PH: (256) 837 - 7664
5151 Research Dr. NW
Huntsville, AL 35805
FAX: (256) 837 - 7677



LEGEND
BORE #1, BORING LOCATION

JOB NAME: GUNTERSVILLE DAM WTP INTAKE AND RAW WATER LINE MARSHALL COUNTY, AL	TEST BORE LOCATIONS DRAWING NO: 5085-1	JOB NO: 5085 DATE: 03-31-2008 SCALE: AS SHOWN DRAWN BY: MCS

OMI, Inc.

DRILL LOG	PROJECT: Guntersville Dam WTP Intake		PROJECT NO.	BORING NO.: B-1
SITE	BEGUN: 3/11/08	COMPLETED: 3/11/08	HOLE SIZE: NQ	PAGE NO.: 1 of 1
COORDINATES	DEPTH GROUND WATER	AT FIRST CHECK: 5-ft		GROUND ELEVATION
DRILLER: RS	CORE RECOVERY (%)	# SAMPLES: 4	# CORE BOXES: 2	DEPTH TOP OF ROCK: 0-ft
DRILL MAKE AND MODEL	LOGGED BY: JJ			DEPTH BOTTOM OF HOLE: 20

SAMPLE DATA					REMARKS: NOTES ON WATER LOSSES AND LEVELS, CAVING, AND DRILLING CONDITIONS CASING DEPTH	FRACTURES PER FOOT	STRATA ELEVATION/ DEPTH	DEPTH	GRAPHIC LOG	DESCRIPTION
SAMPLE TYPE AND DIAMETER	SAMPLE NO.	SAMPLE LENGTH	SAMPLE RECOVERY	N-VALUE/ RQD (%)						
NQ 0.0-ft	Run 1	60"	48" 80%	50%		0, 0, 2, X, X	0 0.0	0		LIMESTONE, slightly weathered, gray, hard, medium to fine grained, medium bedded.
NQ 5.0-ft	Run 2	60"	60" 100%	100%		0, 0, 0, 0, 0	-5 5.0	5		LIMESTONE, weathered, gray, hard, medium to fine grained medium bedded.
NQ 10.0-ft	Run 3	60"	46" 77%	60%		0, 0, 0, 1, X	-13.5 13.5	10		SANDY SILTY CLAY layer.
NQ 15.0-ft	Run 4	60"	56" 93%	50%		X, 5, 0, 0, 5	-15.5 15.5	15		LIMESTONE, weathered, gray, hard, medium to fine grained, mediuim bedded.
								20		Rock Core terminated at 20-ft.
								25		
								30		
								35		

OMI, Inc.

DRILL LOG

PROJECT: Guntersville Dam WTP Intake			PROJECT NO.	BORING NO. B-2
SITE	BEGUN 3/13/08	COMPLETED 3	HOLE SIZE	PAGE NO. 1 of 1
COORDINATES	DEPTH GROUND WATER	AT FIRST CHECK		GROUND ELEVATION
DRILLER RS	CORE RECOVERY (%)	# SAMPLES 2	# CORE BOXES 1	DEPTH TOP OF ROCK 8.5-ft
DRILL MAKE AND MODEL	LOGGED BY: JJ			DEPTH BOTTOM OF HOLE 10

SAMPLE DATA: SAMPLE TYPE AND DIAMETER	SAMPLE NO.	SAMPLE LENGTH	SAMPLE RECOVERY	N-VALUE/ RQD (%)	REMARKS: NOTES ON WATER LOSSES AND LEVELS, CAVING, AND DRILLING CONDITIONS CASING DEPTH	FRACTURES PER FOOT	STRATA ELEVATION/ DEPTH	DEPTH	GRAPHIC LOG	DESCRIPTION
NQ 0.0-ft.	Run 1	60"	13" 22%	15%		0	0	0		LIMESTONE.
						X	0.0 -1 1.0			SANDY SILTY CLAY with 15% limestone boulders, 10% medium to coarse sand, 75% fines, medium plasticity, tan, stiff, moist, fill, CL.
						X				
						X				
						X	-4 4.0	5		LIMESTONE.
NQ 5.0-ft	Run-2	60"	20" 33%	30%		0	-5 5.0			SANDY SILTY CLAY with 15% limestone boulders, 10% medium to coarse sand, 75% fines, medium plasticity, tan, stiff, moist, fill, CL.
						X				
						X				
						X	-8.5 8.5			
						0		10		LIMESTONE, slightly weathered light gray, hard, fine grained, medium bedded.
										Rock Core terminated at 10-ft.
								15		
								20		
								25		
								30		
								35		

OMI, Inc.

DRILL LOG

PROJECT: Guntersville Dam WTP Intake			PROJECT NO.	BORING NO. B-3
SITE	BEGUN 3/13/08	COMPLETED 3/13/08	HOLE SIZE NQ	PAGE NO. 1 of 1
COORDINATES	DEPTH GROUND WATER	AT FIRST CHECK		GROUND ELEVATION
DRILLER RS	CORE RECOVERY (%)	# SAMPLES 2	# CORE BOXES 1	DEPTH TOP OF ROCK 7.5-ft
DRILL MAKE AND MODEL	LOGGED BY: JJ			DEPTH BOTTOM OF HOLE 10

SAMPLE DATA					REMARKS: NOTES ON WATER LOSSES AND LEVELS, CAVING, AND DRILLING CONDITIONS CASING DEPTH	FRACTURES PER FOOT	STRATA ELEVATION/ DEPTH	DEPTH	GRAPHIC LOG	DESCRIPTION
SAMPLE TYPE AND DIAMETER	SAMPLE NO.	SAMPLE LENGTH	SAMPLE RECOVERY	N-VALUE/ RQD (%)						
NQ 0.0-ft	Run 1	60"	13" 22%	0%		X X X X X	0 0.0	0		Surge stone and/or rip rap fill.
NQ 5.0-ft	Run 2	60"	35" 58%	44%		X X 0 0 0	-7.5 7.5	5		LIMESTONE, slightly weathered, fine to medium grained, hard, gray, medium bedded.
								10		Rock Core terminated at 10-ft.
								15		
								20		
								25		
								30		
								35		

OMI, Inc.

5151 Research Drive, N.W. Huntsville, AL 35805

JOB NO.: 5085 JOB: Guntersville Dam WTP Intake LOG OF BORING: B-4

JOB LOCATION: BORING LOCATION:

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (Elevation=)	BLOWS PER FT	NATURAL MOISTURE	POCKET PENT TSF	Pp ■ (tsf) 1 2 3 4 / "N" values blows/ft ▲ / WATER CONTENT, % - ● / PL ✕- - - -✕ LL 20 40 60 80
0			5-IN Topsoil.				
			SANDY SILTY CLAY, 15% coarse to fine sand, 85% fines, medium plasticity, brownish tan, stiff to very stiff, moist, residuum, CL.			---	
2.5						3.5	
						3.5	
5							
						3.5	
7.5							
						3.25	
10			Boring terminated at 10-ft. Note: Split spoon samples pushed and not hammered as required by TVA to avoid disturbing eagles.				
12.5							
15							
17.5							
20							

COMPLETION DEPTH: 10 DEPTH TO WATER INITIAL: Dry OMI,Inc.

DATE: 3/14/08 DEPTH TO WATER FINAL:

OMI, Inc.

5151 Research Drive, N.W. Huntsville, AL 35805

JOB NO.: 5085 JOB: Guntersville Dam WTP Intake LOG OF BORING: B-5

JOB LOCATION: BORING LOCATION:

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL Elevation=	BLOWS PER FT	NATURAL MOISTURE	POCKET PENT TSF	Pp ■ (tsf) 1 2 3 4 "N" values blows/ft ▲ WATER CONTENT, % - ● PL ✕— — — ✕ LL 20 40 60 80
0			11-IN Topsoil.			---	
2.5			SANDY SILTY CLAY, 15% coarse to fine sand, 85% fines, medium plasticity, brownish tan, stiff to very stiff, moist, residuum, CL.			2.5 ---	■
5			Auger refusal at 4-ft. Note: Split spoon samples pushed and not hammered as required by TVA to avoid disturbing eagles.				
7.5							
10							
12.5							
15							
17.5							
20							

COMPLETION DEPTH: 4 DEPTH TO WATER INITIAL: Dry OMI,Inc.

DATE: 3/14/08 DEPTH TO WATER FINAL:

OMI, Inc.

5151 Research Drive, N.W. Huntsville, AL 35805

JOB NO.: 5085 JOB: Guntersville Dam WTP Intake LOG OF BORING: B-6

JOB LOCATION: BORING LOCATION:

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (Elevation=)	BLOWS PER FT	NATURAL MOISTURE	POCKET PENT TSF	Pp ■ (tsf) 1 2 3 4; "N" values blows/ft ▲; WATER CONTENT, % - ●; PL × – – – × LL 20 40 60 80
0			3-IN Topsoil.				
			SANDY SILTY CLAY, with traces of weathered chert, 20% medium to fine sand, 80% fines, low plasticity, orange, very stiff, moist, residuum, CL.			3.0	
2.5						3.0	
						4.0	
5							
						3.5	
7.5							
						3.5	
10			Boring terminated at 10-ft. Note: Split spoon samples pushed and not hammered as required by TVA to avoid disturbing eagles.				
12.5							
15							
17.5							
20							

COMPLETION DEPTH: 10 DEPTH TO WATER INITIAL: Dry OMI,Inc.

DATE: 3/14/08 DEPTH TO WATER FINAL:

BORING LEGEND

SOIL SYMBOLS

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAIN SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVELS 50% OR MORE OF COARSE FRACTION RETAINED ON #4 SIEVE	CLEAN GRAVELS	GW	WELL-GRADED GRAVELS AND GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
			GP	POORLY GRADED GRAVELS AND GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN 50% OF COARSE FRACTION PASSES #4 SIEVE	CLEAN SANDS	SW	WELL-GRADED SANDS AND GRAVELLY SANDS, LITTLE OR NO FINES
			SP	POORLY GRADED SANDS AND GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES	SM	SILTY SANDS, SAND-SILT MIXTURES
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAIN SOILS 50% OR MORE PASSES NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT 50% OR LESS	ML	INORGANIC SILTS, VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50%	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDS OR SILTS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS			PT	PEAT, MUCK AND OTHER HIGHLY ORGANIC SOILS

ABBREVIATIONS:

SS- SPLIT SPOON SAMPLE

UD- UNDISTURBED SAMPLE

REC-SAMPLE RECOVERY

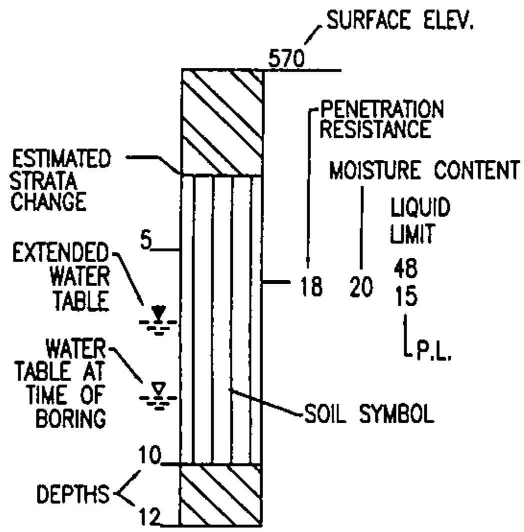
USC-VISUAL UNIFIED SOIL CLASSIFICATION

POCKET PENET- POCKET PENETROMETER READING, TSF

RQD-ROCK QUALITY DESIGNATION

FF- FRACTURE FREQUENCY PER FOOT OF CORE

KEY TO BORING RECORDS OR PROFILES



ROCK SYMBOLS

SANDSTONE

SHALE

GNEISS OR SCHIST

CONGLOMERATE

LIMESTONE OR DOLOMITE

OMI,INC.

5151 Research Drive
Huntsville, AL 35805

FIELD TEST PROCEDURES

OMI, Inc., generally follows field and laboratory testing procedures as outlined by the American Society for Testing and Materials (ASTM) and the U. S. Army Corps of Engineers. Field procedures are outlined and an overview description is provided in ASTM Standard D-420, "Standard Guide to Site Characterization for Engineering, Design, and Construction Purposes." This document is a guide to the selection of various standards for investigating soil, rock, and ground water for earth related construction. Applicable procedures include geophysical, in-situ, and boring methods. A summary of each procedure used during this study is presented below.

SOIL DRILLING PROCEDURES

Several techniques are used to advance borings for collection of soil, rock, or ground water samples. Different techniques are used, depending on the samples desired and the soil and water conditions. Depths for sample intervals, strata changes, and boring termination or refusal are recorded to the nearest 1/10 of a foot. The techniques include the following.

Soil Borings

A) Solid stem continuous flight augers (ASTM D-1452)

B) Hollow stem continuous flight augers (ASTM D-1452)

C) Rotary drilling techniques using roller cone bits or drag bits and water with or without drilling mud or other additives to flush the hole

D) Hand augers

E) Backhoes or other excavating equipment.

Rock Borings

A) Core borings with diamond bits with double or triple core barrels (ASTM D-2113)

B) Rock borings with roller cone bit

C) Rotary hammer drilling.

Hollow and Solid Stem Auger: An auger is a center post with a continuous spiral flange wrapped around it. The post is called the stem. Augers are usually constructed in 5-foot long sections that can be coupled together. As the auger is turned and advanced into the ground; the soil "cuttings" are brought to the surface. Solid stem augers have a solid core and have to be removed from the boring to allow access for sampling tools. Hollow stem augers have the spiral flange connected to a hollow tube (stem). Sampling tools can access the bottom of the boring without removing the augers from the hole.

Rotary Borings: Rotary drilling involves the use of roller cone or drag type drill bits attached to the end of hollow drill rods. A flushing medium, normally water or bentonite slurry, is pumped through the rods to clear the cuttings from the bit face and flush them to the surface. Casing is sometimes set behind the advancing bit to prevent the hole from collapsing and to restrict the penetration of the drilling fluid into the surrounding soils. Cuttings returned to the surface by the drilling fluid are usually collected in a settling tank to allow the fluid to be re-circulated.

Hand Auger Borings: Hand auger borings are advanced by manually twisting a 4-inch diameter steel bucket auger into the ground and withdrawing it when filled to observe the sample collected. Other equipment such as post-hole diggers is sometimes used in lieu of augers to obtain shallow soil samples. Occasionally, these hand auger borings are used for driving 3-inch diameter steel tubes to obtain intact soil samples.

Test Pits: A backhoe or other construction equipment is sometimes used to excavate into soils to observe the soil and collect samples.

Core Drilling: Soil drilling methods are not normally capable of penetrating through hard cemented soil, weathered rock, coarse gravel or boulders, thin rock seams, or sound continuous rock. Material which cannot be penetrated by auger or rotary soil drilling methods at a reasonable rate is designated as "refusal material." Core drilling procedures are required to penetrate and sample refusal materials.

Prior to coring, casing may be set in the drilled hole through the overburden soils to keep the hole from caving and to prevent excessive water loss. The refusal materials are then cored according to ASTM D-2113 using a diamond bit fastened to the end of a hollow, double, or triple tube core barrel. This device is rotated at high speeds and the cuttings are brought to the surface by circulating water. Core samples of the material penetrated are protected and retained in the swivel-mounted inner tube. Upon completion of each drill run, the core is brought to the surface, recovery is measured, and the core is sequentially placed in boxes and transported to our laboratory for review and storage.

SAMPLING AND TESTING IN BOREHOLES

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

A) Standard Penetration Testing
B) Undisturbed Sampling
C) Dynamic Cone Penetration Testing
D) Hand-Held Static Cone Penetrometer
E) Water Level Readings.

These procedures are presented below. Any additional testing techniques employed during this exploration are contained in other sections of the Appendix.

Standard Penetration Testing: At regular intervals, the drilling tools are removed and soil samples are obtained with a standard 2-inch diameter split tube or "split spoon" sampler connected to a drill rod. The sampler is first seated 6 inches to penetrate any loose cuttings then driven an additional 12 inches with blows of a 140 pound safety 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, defined in blows per foot (bpf). The split spoon sampler is designed to retain the soil penetrated so it may be returned to the surface for observation. Representative portions of the soil samples obtained from each split spoon sample are placed in jars, sealed, and transported to the 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 D-1586. The depths and N-values of standard penetration tests are shown on the Boring Records. Split spoon samples are suitable for visual observation and classification tests, but generally are not sufficiently intact for quantitative laboratory testing.

Undisturbed Sampling: Relatively undisturbed samples are obtained by pushing 3 inch outside diameter (OD), 30 inch long steel tubes with hydraulic pressure supplied by the drill rig into the soil at the desired sampling levels (ASTM Standard D-1587). These tubes are also known as Shelby tubes. Each tube, together with the encased soil, is removed from the ground, sealed, and transported to the laboratory. Locations and depths of undisturbed samples are shown on the Boring Records.

Dynamic Cone Penetrometer: The dynamic cone is a hand-operated penetrometer used in hand auger borings and observation pits. This test is intended to provide data that can be correlated to the standard penetration test. A 1.5-inch OD cone is seated to penetrate any loose cuttings, and then driven for 3 intervals of 1.75 inch with blows from a 15-pound weight falling 20 inches. The average number of blows required to drive the cone over 1 increment is an index to soil strength and compressibility.

Water Level Readings: Water table readings are normally taken in the borings and are recorded on the Boring Records. In sandy soils, these readings indicate the approximate location of the hydrostatic water table at the time of the 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 level reported on the Boring Records is determined by field crews immediately after the drilling tools are removed, and again several hours after the borings are completed, if possible. The time lag is intended to permit stabilization of the ground water 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 Records.

BORING RECORDS

The subsurface conditions encountered during drilling are reported on a Boring Record. The record contains information concerning the boring method, samples attempted and recovered, indications of the presence of coarse gravel, cobbles, etc., and observations of ground water. It also contains the driller's and the geotechnical engineer's interpretation of soil conditions between samples. Therefore, these boring records contain both factual and interpretative information. A geotechnical engineer visually classifies the soil samples and prepares the Boring Records which are the basis for all evaluations and recommendations.

LABORATORY TEST PROCEDURES

OMI, Inc., generally follows laboratory testing procedures as outlined by the American Society for Testing and Materials (ASTM), the U. S. Army Corps of Engineers, and other applicable procedures. All work is initiated and supervised by qualified engineers. Laboratory tests are performed by technicians trained to perform the work according to the appropriate procedures. The equipment is well maintained and inspected and calibrated annually or as specified by ASTM.

A description of the procedures used during this exploration or study are included in this Appendix.

SOIL CLASSIFICATION

Classification of soils provides a record and general guide to the engineering properties of the soils encountered during this study. Samples obtained during the field testing (drilling) operations are visually examined and classified by the geotechnical engineer. OMI, Inc., generally follows ASTM procedure No. D-2488 "Visual-Manual Procedure for Classifying Soils." Soil consistency and relative density is based on the number of blows from the standard penetration test. Representative or special samples are then selected for laboratory testing. Soil Boring Records are developed which present the data from the field testing as well as the soil description, water level information, and other data.

MOISTURE CONTENT

Moisture content values, when used in conjunction with other data, can be a useful and inexpensive tool to the engineer as an indicator of the engineering characteristics and parameters of the soil when compared to other data. Moisture content is performed by weighing a moist sample, drying, then re-weighing the dry sample. The moisture content is expressed as a percent of the dry weight of the soil. ASTM Method D-2216 is used to determine the moisture content of soil.

ATTERBERG LIMITS

Atterberg limits include the liquid limit (LL), plastic limit (PL), and shrinkage limit (SL) tests. These tests are performed to aid in the classification of soils and to determine the plasticity and volume change characteristics of the soil. The liquid limit is the minimum moisture content at which the soil will flow as a heavy viscous fluid. The plastic limit is the minimum moisture content at which the soil behaves as a plastic material. The shrinkage limit is the moisture content below which no further volume change will occur with continued drying. The plasticity index (PI) is the difference between the liquid limit and the plastic limit. The PI is the range of moisture at which the soil remains plastic. Many engineering characteristics have been correlated to the Atterberg limits. These are ASTM procedures D-4318, D-4943, and D-427.

STANDARD PROCTOR COMPACTION TEST

This test is used to establish a curve that predicts the effect of moisture and compactive effort on the dry density of the soil sample. It is useful as a comparative value in monitoring contractors' efforts during fill placement and compaction during construction. Also, correlations of engineering parameters such as strength, compressibility, and permeability are related to the percent compaction and soil type.

A representative sample of the proposed fill material (soil or stone) is collected. The sample is divided into four or more samples. Each sample is then brought to a different moisture content about 2% apart. Each sample is then placed in a standard 4-inch diameter mold in 3 equal layers with each layer being compacted with 25 blows from a 5.5-pound hammer falling 12 inches. The sample is trimmed to a known volume of 1/30 cubic foot then weighed. The moisture content of the sample is determined and the dry density is calculated. A graph of dry density (pcf) versus moisture content is developed. The maximum density and its corresponding moisture content known as the optimum moisture content are derived from the curve. A graph of the moisture-density relationship is given in the Appendix. ASTM D-698 describes the procedure.

UNCONFINED COMPRESSION TESTS - ROCK CORES

The strength of rock is important in many engineering applications. This strength is usually desired and reported as the unconfined or simple shear strength. Selected samples of rock cores are cut using a diamond saw. The cores are usually cut to a length equal to about twice the core diameter. The capped length and diameter of each core is measured and recorded. The cores are then loaded to failure in a compression machine. The unconfined compressive strength is calculated by dividing the cross-sectional area of the core into the maximum load required to crush the sample. If the length to diameter ratio is less than 2.0, then the maximum strength is adjusted mathematically. The results are reported in psi. This procedure is similar to ASTM D-2938.

CONSOLIDATION TESTING

The consolidation test provides data for estimating the settlement and time rate of settlement of the soil in response to the applied loads. Representative soil samples are collected from undisturbed samples, trimmed into a disk about 2.5 inches in diameter and 1 inch thick, then placed in the consolidometer. The disk is confined in a brass ring and sandwiched by porous stones on the top and bottom. The sample ring and stones are placed in a testing device, inundated, then loaded in increments. The sample height is measured as each load caused it to compress. The resulting loads and deformations are reduced to a graph which is presented in the Appendix. These results may be presented in load versus percent strain or load versus void ration. This procedure is described in ASTM D-2435.

Report of
Subsurface Soil/Rock Exploration and
Geotechnical Engineering Evaluation for
an Intake Structure Near Guntersville Dam
Guntersville, Marshall County, Alabama

File No. 113-11-40-1019A
May 18, 2011

Ardaman & Associates, Inc.

OFFICES

Orlando, 8008 S. Orange Avenue, Orlando, Florida 32809, Phone (407) 855-3860
Bartow, 1525 Centennial Drive, Bartow, Florida 33830, Phone (863) 533-0858
Cocoa, 1300 N. Cocoa Blvd., Cocoa, Florida 32922, Phone (321) 632-2503
Fort Myers, 9970 Bavaria Road, Fort Myers, Florida 33913, Phone (239) 768-6600
Miami, 2608 W. 84th Street, Hialeah, Florida 33016, Phone (305) 825-2683
Port Charlotte, 740 Tamiami Trail, Unit 3, Port Charlotte, Florida 33954, Phone (941) 624-3393
Port St. Lucie, 460 Concourse Place NW, Unit 1, Port St. Lucie, Florida 34986, Phone (772) 878-0072
Sarasota, 2500 Bee Ridge Road, Sarasota, Florida 34239, Phone (941) 922-3526
Tallahassee, 3175 West Tharpe Street, Tallahassee, Florida 32303, Phone (850) 576-6131
Tampa, 3925 Coconut Palm Drive, Suite 115, Tampa, Florida 33619, Phone (813) 620-3389
West Palm Beach, 2200 North Florida Mango Road, Suite 101, West Palm Beach, Florida 33409, Phone (561) 687-8200

Ardaman & Associates, Inc.

Geotechnical, Environmental and Materials Consultants

May 18, 2011
File No. 113-11-40-1019A

Tetra Tech, Inc. (IER)
101 Church Street, Suite 201
Huntsville, Alabama 35801

Attention: Ms. Shannon Bailey-Partlow, P.E., Project Manager

Subject: **Report** of Subsurface Soil/Rock Exploration and Geotechnical Engineering Evaluation for a Proposed Intake Structure Near Guntersville Dam

Dear Ms. Bailey-Partlow:

As authorized by Tetra Tech (TT), Ardaman and Associates, Inc. (Ardaman) has completed the subsurface exploration and geotechnical engineering evaluation for the proposed intake structure east of the Guntersville Dam. The purposes were to evaluate subsurface conditions encountered in test borings performed at the site, and to provide geotechnical recommendations regarding: foundation support; rock anchor pullout capacity; soil parameters for below grade walls; and earthwork preparation.

This report has been prepared for the exclusive use of Tetra Tech for specific application to the subject project.

We are pleased to be of assistance to you on this phase of your project. When we may be of further service to you or should you have any questions, please do not hesitate to contact us.

Sincerely,

ARDAMAN & ASSOCIATES, INC.
Alabama License No. 2687

Jeremy M. Clark, E.I.
Staff Engineer

William S. Jordan, P.E.
Senior Project Manager

Michael S. Wilson, P.E.
Branch Manager/Senior Engineer
Alabama License No. 24013

JMC/MSW/WSJ/mea

3175 W Tharpe Street, Tallahassee, Florida 32303 Phone (850) 576-6131 FAX (850) 576-0790
Offices in: Bartow, Cocoa, Fort Myers, Miami, Orlando, Port Charlotte, Port St. Lucie, Sarasota, Tallahassee, Tampa, W. Palm Beach

1.0 PROJECT DESCRIPTION AND SCOPE OF SERVICES

It is our understanding that the intake structure will be constructed on the north side of Guntersville Lake, approximately 1,000-feet east of the dam. The inlet invert of the Intake Structure will be at approximately elevation +583-feet, about 14-feet below the grade at our test hole locations, and approximately 12-feet below Guntersville Lake "normal" water elevation. Adjacent to the intake structure, at grade elevation, will be paved areas for vehicular traffic.

Based on the drawings provided to Ardaman from Tetra Tech (drawings D-100B through D-100G), we interpret that the structure consists of two reinforced concrete platforms, one of which will bear on the rock formation and consist of the intake "cells" of the structure, and the other platform housing the pumps, will sit atop the stem walls of the intake cells, effectively directing the loads to the rock bearing platform. The lower platform will likely consist of a structural mat foundation, with average loading on the order of 2 to 3 ksf.

Granular backfill is planned along three sides of the perimeter of the intake cells, which will allow groundwater to accumulate around the exterior walls. When water is contained within the cells a balance of hydrostatic pressure will occur. However, unbalanced uplift and lateral hydrostatic forces will be applied to the structure when/if de-watering occurs within the intake cells. Rock anchors are planned to resist these loads.

Geotechnical services were based on the authorized proposal for the project, as follows:

1. Ardaman mobilized a drill rig and crew to the site, and performed two (2) test borings in locations designated by Tetra Tech. The borings were initiated by performing Standard Penetration Test (SPT) sampling in general accordance with ASTM D 1586. Once "hard rock" was encountered, the borings were then advanced by coring techniques in general accordance with ASTM D 2113.

2. Ardaman's Drill Crew Chief prepared a field log for each boring, which documented specific information including: SPT "N"-values (for soil); Rock Quality Designation (RQD) and rock recovery values (%); visual classification of soil and rock; depth to stratum changes; depth of drilled fluid losses, and depth to encountered groundwater, if apparent. Our drill crew chief packaged portions of the soil samples and boxed all rock cores for transportation to our office.

3. Ardaman's engineers visually/manually classified recovered soil samples and examined, classified, and photographed the rock cores. Based on the soil and rock examination, our engineers developed a boring profile for each test hole. Laboratory tests of selected soil and rock samples were directed to further assess engineering and index properties of the encountered materials.

4. Our engineers analyzed and evaluated subsurface conditions encountered and developed recommendations regarding: foundation support; rock anchor pullout resistance; earthwork preparation; and soil stress parameters for backfill against subgrade walls. Recommendations are presented in this report, along with a test boring location plan and the soil/rock profiles.

2.0 GEOLOGY

Much of the Tennessee Valley is located within the physiographic province of the Interior Low Plateaus. The Tennessee Valley is a rolling upland having an average altitude of approximately 600-feet above sea level and a maximum relief of about 400-feet. The rock underlying the Tennessee Valley mostly consists of carbonates ranging in age from the Late Ordovician to Early Mississippian. Limestone bluffs commonly border the Tennessee River.

The intake structure is near the south edge of the demarcation between the Highland Rim to the north, and the Cumberland Plateau to the south. The Plateau was formerly a vast tableland that sloped southward. Subsequent erosion dissected and roughened the original surface resulting in only a few flat-topped remnants. The upper rocks of this region were mainly shales and sandstones of Pennsylvanian age underlain by older carbonate rocks, but due to erosion, the sandstone was weathered away, exposing the limestone and dolomite underneath. In these less resistant rocks, long and narrow valleys developed along the axes of the folds.

Colluvium deposits tend to decrease in thickness traveling from north to south along Guntersville Dam Road. This decrease in the colluviums deposits are likely due to the strongly sloping rock formations.

The limestone encountered during the subsurface exploration, presented below, matches the geologic description of the Bangor Limestone (Upper Mississippian), which consists mainly of bluish-gray to pale greenish-gray, thick-bedded, coarsely crystalline or finely granular bioclastic and oolitic limestone. The Bangor Limestone formation ranges in thickness from 100 to 700-feet, with a few interbeds of shale and secondary rock formations of maroon mudstone. The base of the formation includes interbeds of cherty limestone and grayish-yellow dolomicrite.

The Hartselle sandstone lies below the Bangor Limestone. It consists of thick and thin-bedded sandstone, but it is covered with soil material that has been washed or has rolled from the higher lying formations.

3.0 FIELD SUBSURFACE EXPLORATION-LOCATIONS AND METHODS

The approximate locations of the test borings are shown on the attached **Figure 1** under the *Test Boring Location Plan*. The borings were located on site by our staff using a wheel tape measuring from existing site features. The boring locations indicated shall be considered accurate only to the degree implied by the methods of measurement used. We understand that TT retained surveyors to document the two test boring locations at a later date.

The portions of the test holes performed by SPT testing were advanced by rotary drilling with 4-inch diameter flight augers, using a Model CME-55 drill rig mounted on a flat-bed truck. The SPT depths were sampled at 18-inch intervals continuously until "hard rock" was encountered. The holes were then advanced in 5-foot intervals using a diamond impregnated drill bit that cuts a core approximately 2-3/8 inch in diameter. The two test holes were grouted with tremie-placed "neat" Portland cement grout upon completion.

4.0 LABORATORY TESTING OF SOILS

Laboratory testing was directed by our engineers on selected soil and rock samples from the test borings, to aid classification and to further define the engineering properties of the soils and rock. The laboratory tests on the soils included Nature Moisture Content (ASTM D 2216), and Percent Finer than the U.S. No. 200 Sieve (ASTM D 1140, percent silt and clay).

The laboratory tests on the rock cores included Unconfined Compressive Strength (ASTM D 2938), and Splitting Tensile Strength (ASTM D 3967). The results of the laboratory tests are presented adjacent to the *Soil/Rock Boring Profiles* on the attached **Figure 1**, at the respective depths from which the tested samples were recovered. In addition to the results indicated on Figure 1, the rock strength data is presented in tabular form in **Appendix A**.

5.0 SUBSURFACE SOIL/ROCK AND GROUNDWATER CONDITIONS

5.1 General

Ardaman's interpretations of subsurface conditions encountered are depicted on the *Soil/Rock Boring Profiles* on the attached Figure 1. The soil and rock descriptions shown in the *Soil/Rock Legend* are based upon visual/manual and laboratory test-based classification procedures in general accordance with ASTM D 2488; ASTM D 2487; and AASHTO M145.

Photographs of the rock cores are presented on the attached **Figure 2.** The approximate depth of each core is labeled adjacent to the core. Note that soil deposits within the rock joints are often washed away during the coring process, and the recovery percentages, in part, imply those combined thicknesses, although "honeycombs" and void spaces also reduce recovery percentages. Notably, our drillers monitored down-pressure during coring and other than the less resistant zone from 8' to 10'-4" in TH-IP1, no further soft zones or voids were apparent.

The stratification lines on the *Profiles* represent the approximate boundaries between the soil and rock types, but the actual transitions may be more gradual than implied. This report does not address variations which occur between or away from the borings. The nature and extent of such variations may not become evident until during the course of construction. If any variations become evident, Ardaman must be contacted and authorized to provide additional testing and evaluations concerning the projects geotechnical evaluations and recommendations.

5.2 Soil/Rock Conditions

Initially, we encountered a surficial layer of approximately 1.5-feet of a dark brown silty fine sand with traces of grass, surficial roots, and organics-topsoil (Stratum 1) with limestone rubble. Next, we encountered brown and light brown silty lean clay with sand and with inclusions of fat clay, and occasional sandstone and limestone fragments to ½-inch maximum (Stratum 3).

Underlying Stratum 3, at approximately 5 to 5.5-feet below grade, was a bluish-gray to pale greenish-gray very hard limestone. In test-hole TH-IP1 there was no recovery from 8' to 10'-4", so a split-spoon sample was taken next, and a 0.5-foot thick layer of Stratum 3 (saturated) was encountered to almost 11-feet below grade.

Stratum 3, atop the limestone formation, was stiff to very stiff, in accordance with the *Engineering Classification* system displayed on Figure 1. Based on the rock recovery percentages and the RQD values, the rock appears to be high quality, thick-bedded limestone. The only potential location of concern encountered during the exploration of the intake structures is from 5 to 11-feet below grade in TH-IP1, where relatively low recovery and RQD values were obtained, and an interbedded layer of clay was encountered near 11-feet depth.

5.3 Groundwater Conditions

Due to the drilling technique used, groundwater elevations could not be estimated. Groundwater elevations at the intake structure should reflect, or be somewhat above, the elevation of Guntersville Lake. Groundwater and Lake elevations will be affected by seasonal variations in regional precipitation, and the lake control elevation(s) of the Dam spillway.

6.0 DESIGN RECOMMENDATIONS

6.1 General Soil/Rock Evaluation

In our opinion, subsurface conditions encountered at the intake structure appear adequate to support the structure on a mat foundation, bearing directly upon sound limestone, at about 14 to 15-feet below the surface at our borings. The exposed bearing surface must consist of clean, solid rock, absent of soil layers. The encountered rock conditions also appear adequate to allow for the construction of anchors to resist lateral, and uplift conditions.

After soil stripping and rock excavation to proposed mat bearing depth, cleaning and leveling of the rock surface is required to provide a smooth uniform bearing surface. Additional boreholes within the rock mass shall be performed beneath the proposed mat foundation. These additional boreholes will serve the purposes of both further exploration of the quality of the rock in more locations, and to serve as holes for the installation of foundation anchors.

Ardaman shall be requested to inspect the exposed rock surfaces and probe the additional boreholes (and any added anchor holes) to delineate additional excavation, if required. We recommend minimum 8 additional boreholes, or one (1) hole every 200 square feet of bearing surface, whichever results in a greater number of boreholes. The boreholes and any added anchor holes shall be probed by a professional engineer and/or his/her qualified representative, under his/her direct supervision, to "feel" along the sidewalls of the drilled boreholes to assess the void spaces and intermittent layers of soil, which would affect bearing and uplift resistance.

Dewatering of the excavation will be required. The limits of the excavation must be safely sloped, or shored in accordance with 29 CFR 1926.65.

6.2 Foundation and Anchor Design

Provided anomalies are not encountered within the additional boreholes, the Intake Structure may be designed for a maximum allowable bearing pressure of up to 10,000 pounds per square foot. We estimate little, if any, measurable foundation settlement, and estimate maximum foundation settlement to be on the order of 1/8th inch or less.

The additional boreholes may be used for rock anchor installation. The total number of anchors shall be as necessary to resist lateral and uplift loading conditions. The required number of anchors can be calculated based on anchor design length and ultimate bond strength estimated below, along with an appropriate factor of safety. However, no two anchors shall be placed closer than 3-feet from one another.

Once the additional boreholes and any additional anchor holes have been drilled and cleared of anomalies, we recommend blowing groundwater out of the holes with compressed air and then filling the holes with high strength, non-shrink grout, or the grout shall be tremmied. At anchor holes, the grouting must immediately be followed by placement of a reinforcing bar (the reinforcement bar diameter shall be specified by the anchor designer) into the grouted borehole, to be tied into the foundation system.

Anchorage resistance is provided by friction between the grout and rock. Theoretical values of ultimate bond strength are determined from shear strength data, and must be verified via field pullout tests.

Anchor pullout capacity is dependent upon rock strength and continuity, grout strength, and the intimacy and roughness of the contact between the grout and rock. Provided the anchors are less than 6-feet deep, and are installed as recommended above, and provided the boreholes encounter hard rock absent of what are judged to be significant soil seams and voids, we estimate that an **ultimate bond strength on the order of 100 psi, or more, is available. Therefore, 9 kips per foot embedment length for 2.5-inch diameter boreholes, or 7 kips per foot for 2.0-inch diameter boreholes** is recommended for design of anchors 6 feet deep or less. We recommend discounting the top 1 foot of embedment in calculating anchor capacity. If more than 6 feet of anchor embedment is needed to achieve required capacity, a higher bond strength is likely available. We will provide additional analysis and recommendations, if needed.

We recommend verification of anchor pullout bond capacities by testing at least 25% of the production anchors, or at least five (5) tests, whichever is greater, to 1.5 times the design load. If any of the anchors pull out at or below 1.5 times the design load, we recommend testing all of the anchors installed to that date and adjusting the anchor lengths of subsequent anchors based on the pull-out test results. If the lengths are adjusted due to insufficient capacity, we recommend testing at least five (5) of the modified anchors, to the above specification.

6.3 Backfill Types and Subsurface Walls

We recommend the use of well tamped No. 57 Stone as backfill around the intake structure's subsurface walls. The stone shall be placed in lifts not exceeding 6-inches in uncompacted thickness, and then tamped using a hand operated vibratory plate compactor. Granular soils meeting AASHTO A-1 or A-3 soil and soil-aggregate mixture classifications may also be used.

For granular soil backfill, we recommend placement in maximum 6-inch loose lifts, and each lift shall be compacted to between 95 and 97 percent of the Standard Proctor maximum dry density (ASTM D 698). Adjust the moisture content of the soils as necessary to achieve compaction. Do not over-compact the backfill, and heavy, self propelled compactors and construction equipment shall be kept at least 10-feet away from walls, to avoid over-stressing them.

Areas where pavement will not be placed shall be covered with a 2-foot thick compacted clayey sand cap, consisting of A-2-4 soils, but also with at least 20% passing the US No. 200 sieve, and that exhibit a degree of cohesion consistent with clayey soil fines.

The subsurface cast-in-place, reinforced concrete walls, shall be designed to resist "at rest" lateral soil pressures plus differential hydrostatic stresses. The unit weight of backfill (provided below) shall be multiplied by the coefficient of lateral earth pressure to determine the stresses acting on the walls, and unbalanced hydrostatic stress shall be added when buoyant soil unit weight is used. Anticipated surcharge loads adjacent to the wall shall be multiplied by the coefficient of lateral pressure to determine the resulting additional uniform horizontal load. The following parameters shall be utilized in the design of subsurface walls:

- <u>No. 57 Stone Backfill*</u>
  - Total moist unit weight (γ_t) = 115 pcf
  - Buoyant Unit Weight (γ_b) = 53 pcf
  - Angle of internal friction (Φ) = 34 degrees
  - Undrained Cohesion "Cu"="0" (zero)
  - Lateral "at rest" earth pressure coefficient (K_0) = 0.44 (fixed walls)
- <u>A-1 or A-3 Granular Soil Backfill*</u>
  - Total Soil moist unit weight (γ_t) = 115 pcf
  - Buoyant Soil Unit Weight (γ_b) = 53 pcf
  - Angle of internal friction (Φ) = 30 degrees
  - Undrained Cohesion "Cu"-"0" (zero)
  - Lateral "at rest" earth pressure coefficient (K_0)=0.5 (fixed walls)

* No Factor of Safety is built into these values. If there is no potential for net uplift of the Intake Structure, then an ultimate concrete/rock interface frictional value of 1.0 times the normal *effective* stress may be used in the sliding resistance evaluation (with an appropriate factor of safety).

7.0 CLOSURE

The recommendations submitted in this report are based upon the data obtained from the soil and rock borings presented on the attached Figure 1. This report does not reflect any variations which may occur between or away from the borings. The nature and extent of site variations may not become evident until during the course of construction. If site variations appear evident, it will be necessary to reevaluate the recommendations of this report after performing further on-site observations during the construction period and noting the characteristics of such variations.

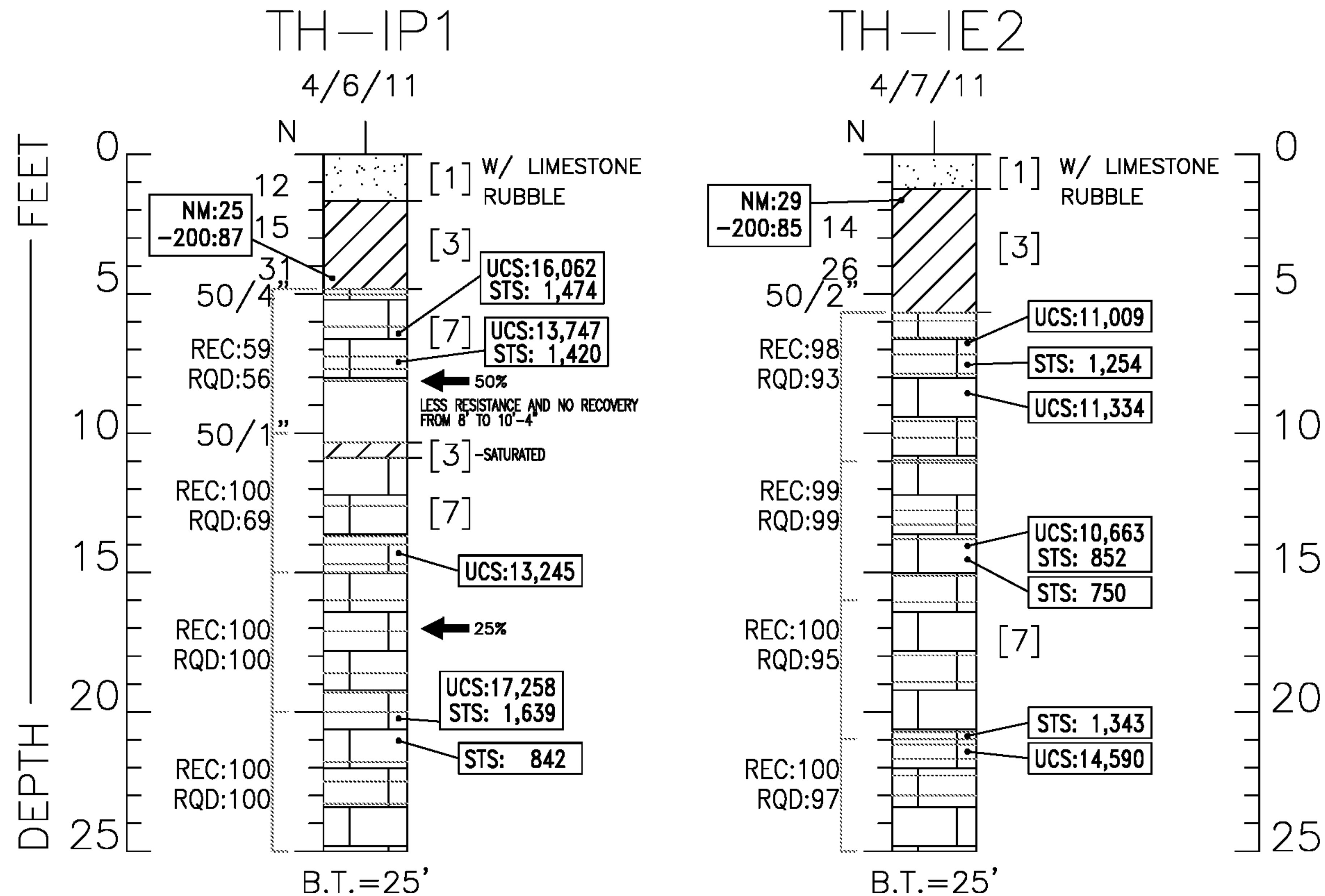
In the event any changes occur in the design, nature, location of the facility, or assumed structural loads, Ardaman and Associates, Inc. must be contacted to review the applicability of the conclusions and recommendations in this report. Ardaman and Associates, Inc. must also perform a general review of final design drawings and specifications to determine if earthwork and foundation recommendations have been properly interpreted and implemented in the design specifications.

This study does not deal with the possibility of eventual sinkhole development at the site. This exploration and analysis covers only the near surface materials. It is not intended to include deep soil or rock strata where cavities and caverns may exist.

This report has been prepared in accordance with generally accepted geotechnical engineering practices. No other warranty, expressed or implied, is made.

End of Report

SOIL/ROCK BORING PROFILES



INDICATES LOCATION OF FRACTURES, JOINTS, OR FISSURES

NOTES:
1. BOREHOLES WERE TREMIE FILLED WITH PORTLAND CEMENT GROUT UPON COMPLETION
2. DUE TO DRILLING TECHNIQUE USED, GROUNDWATER ELEVATIONS COULD NOT BE ESTIMATED OR MEASURED
3. SURVEY DATA REGARDING LAND ELEVATIONS AT THE TEST BORINGS ARE NOT YET AVAILABLE. BASED UPON THE TOPOGRAPHIC MAP, THE ELEVATIONS APPEAR TO BE BETWEEN +596 AND +598 FEET

SOIL/ROCK LEGEND

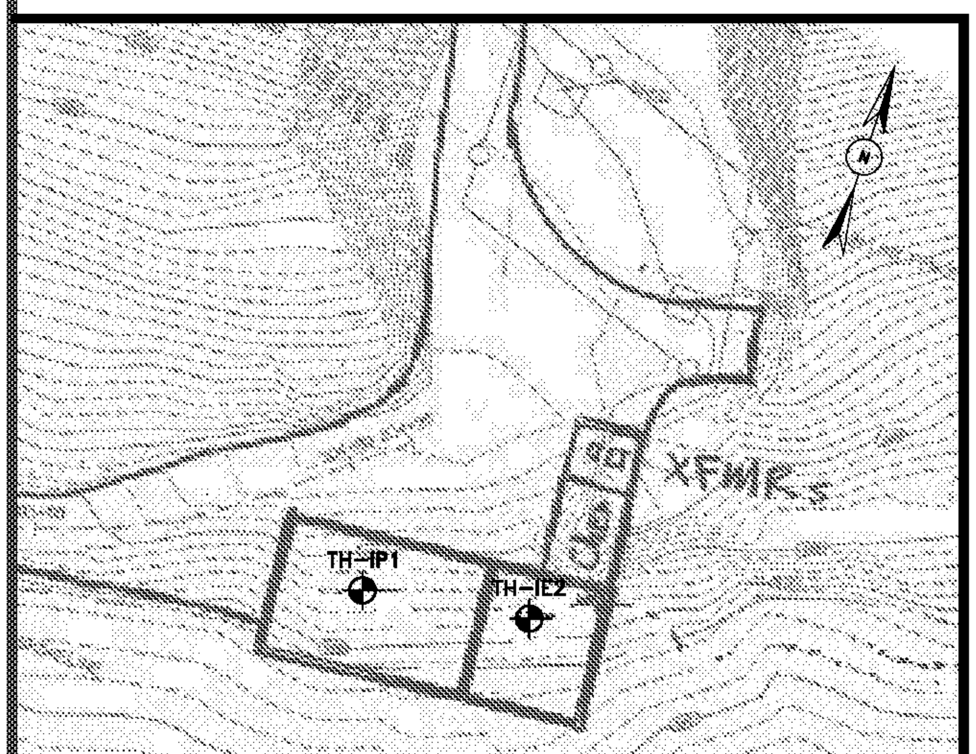
(1)	DARK BROWN SILTY FINE SAND W/ TRACES OF GRASS AND SURFICIAL ROOTS AND TRACE ORGANICS; TOPSOIL (SM; A-2-4)
(3)	BROWN AND LIGHT BROWN SILTY LEAN CLAY W/ SAND AND W/ INCLUSIONS OF FAT CLAY AND OCC. SANDSTONE AND LIMESTONE FRAGMENTS TO 1/2-INCH MAX (CL W/ CH; A-6 W/ A-7)
(7)	BLUISH-GRAY TO PALE GREENISH-GRAY VERY HARD LIMESTONE

NOTE: THE ABOVE STRATA WERE ENCOUNTERED IN THE TWO INTAKE STRUCTURE BORINGS. THE REMAINING STRATA (NOT LISTED) WERE ENCOUNTERED IN THE RAW WATER PIPELINE ALIGNMENT BORINGS.

LEGEND

TH	STANDARD PENETRATION TEST (SPT) BORING LOCATION
N	STANDARD PENETRATION RESISTANCE IN BLOWS PER FOOT (ASTM D-1586)
50%	PROPORTIONAL LOSS OF DRILLING FLUID CIRCULATION
50/3"	50 BLOWS FOR 3-INCHES PENETRATION INTO SOIL
NM	NATURAL MOISTURE CONTENT IN PERCENT (ASTM D-2216)
-200	PERCENT PASSING NO. 200 SIEVE SIZE (PERCENT FINES)(ASTM D-1140)
REC	PERCENT ROCK RECOVERY
RQD	ROCK QUALITY DESIGNATION (SUMMATION OF PIECES>4"/ LENGTH OF CORE) (ASTM D 6032)
UCS	UNCONFINED COMPRESSIVE STRENGTH IN POUNDS PER SQUARE INCH (ASTM D 2938)
STS	SPLITTING TENSILE STRENGTH IN POUNDS PER SQUARE INCH (ASTM D 3967)
SP-SM,SM,SC	UNIFIED SOIL CLASSIFICATION SYSTEM
A-3,A-2-4	AASHTO SOIL CLASSIFICATION SYSTEM
DRILLERS:	KM, JDA,
DRILL RIG:	CME 55, MANUAL HAMMER ABOVE 10.5'; AUTO-HAMMER BELOW

TEST BORING LOCATION PLAN



ENGINEERING CLASSIFICATION

I COHESIONLESS SOILS

DESCRIPTION	BLOW COUNT "N"
VERY LOOSE	0 TO 4
LOOSE	4 TO 10
MEDIUM DENSE	10 TO 30
DENSE	30 TO 50
VERY DENSE	>50

II COHESIVE SOILS

DESCRIPTION	UNCONFINED COMPRESSIVE STRENGTH, QU, TSF	BLOW COUNT "N"
VERY SOFT	<1/4	0 TO 2
SOFT	1/4 TO 1/2	2 TO 4
MEDIUM STIFF	1/2 TO 1	4 TO 8
STIFF	1 TO 2	8 TO 15
VERY STIFF	2 TO 4	15 TO 30
HARD	>4	>30

WHILE THE BORINGS ARE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT THEIR RESPECTIVE LOCATIONS & FOR THEIR RESPECTIVE VERTICAL REACHES, LOCAL VARIATIONS CHARACTERISTIC OF THE SUBSURFACE MATERIALS OF THE REGION ARE ANTICIPATED & MAY BE ENCOUNTERED. THE BORING LOGS & RELATED INFORMATION ARE BASED ON THE DRILLER'S LOGS & VISUAL EXAMINATION OF SELECTED SAMPLES IN THE LABORATORY. THE DELINEATION BETWEEN SOIL TYPES SHOWN ON THE LOGS IS APPROXIMATE & THE DESCRIPTIONS REPRESENT OUR INTERPRETATION OF SUBSURFACE CONDITIONS AT THE DESIGNATED BORING LOCATIONS ON THE PARTICULAR DATE DRILLED.

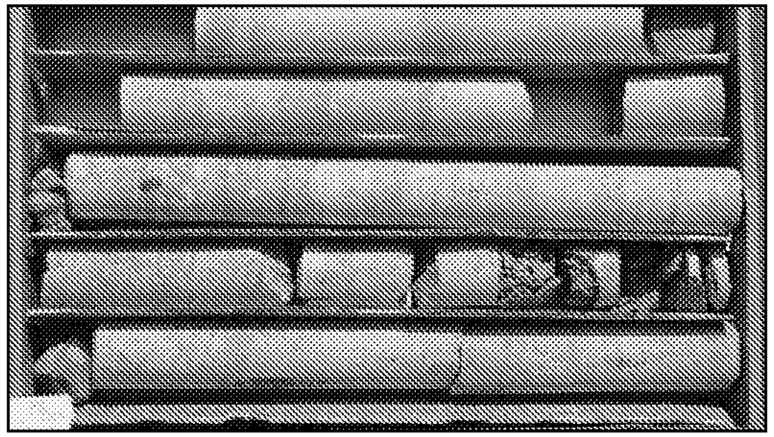
GROUNDWATER ELEVATIONS SHOWN ON THE BORING LOGS REPRESENT GROUNDWATER SURFACES ENCOUNTERED ON THE DATES SHOWN. FLUCTUATIONS IN WATER TABLE LEVELS SHOULD BE ANTICIPATED THROUGHOUT THE YEAR. ABSENCE OF GROUNDWATER DATA ON CERTAIN BORINGS IMPLIES THAT NO DATA IS AVAILABLE, BUT DOES NOT NECESSARILY MEAN THAT GROUNDWATER WILL NOT BE ENCOUNTERED AT THESE LOCATIONS OR WITHIN THE VERTICAL REACHES OF THESE BORINGS IN THE FUTURE.

Ardaman & Associates, Inc.

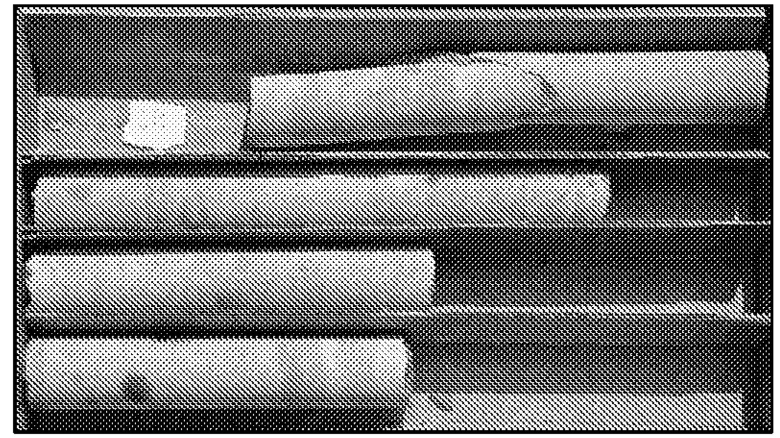
SUBSURFACE EXPLORATION FOR
A PROPOSED INTAKE STRUCTURE
EAST OF GUNTERSVILLE DAM
GUNTERSVILLE, MARSHALL COUNTY, ALABAMA

		MAY 18, 2011
113-11-40-1019A	M.S. WILSON, P.E.	1

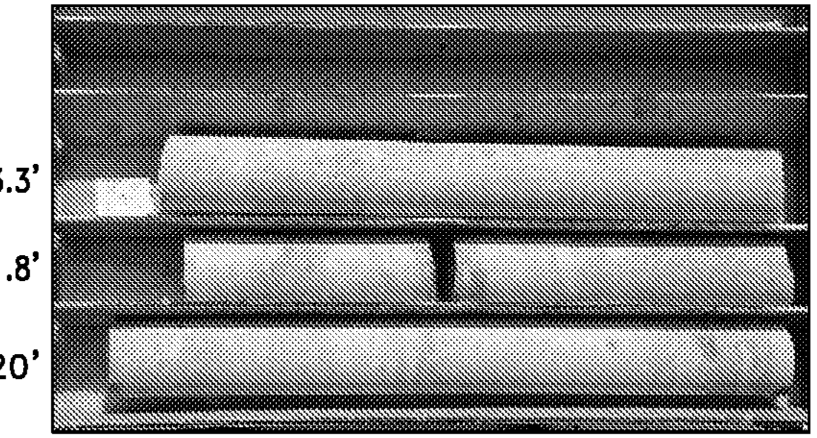
ROCK CORE PHOTOGRAPHS



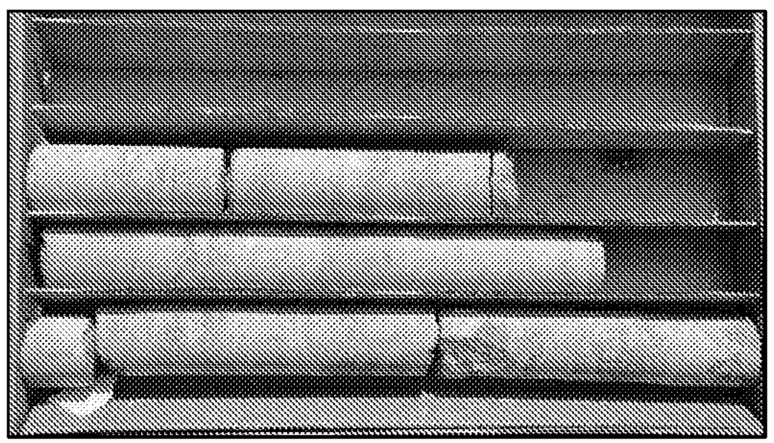
TH-IP1
FROM 4.8 TO 15.2-FEET BELOW GRADE



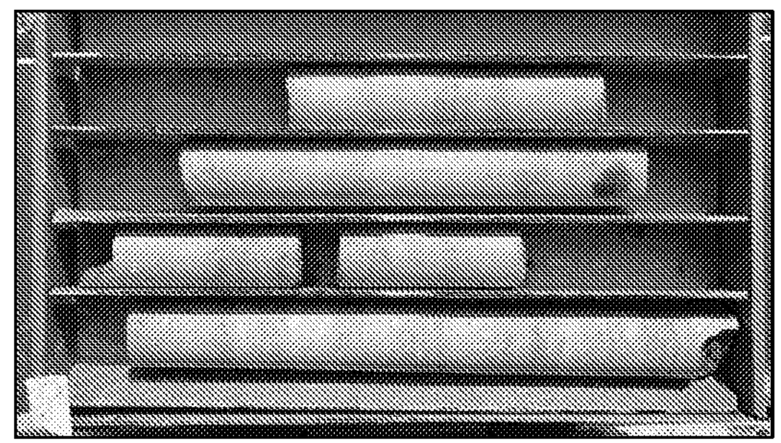
TH-IP1
FROM 15.2 TO 20-FEET BELOW GRADE



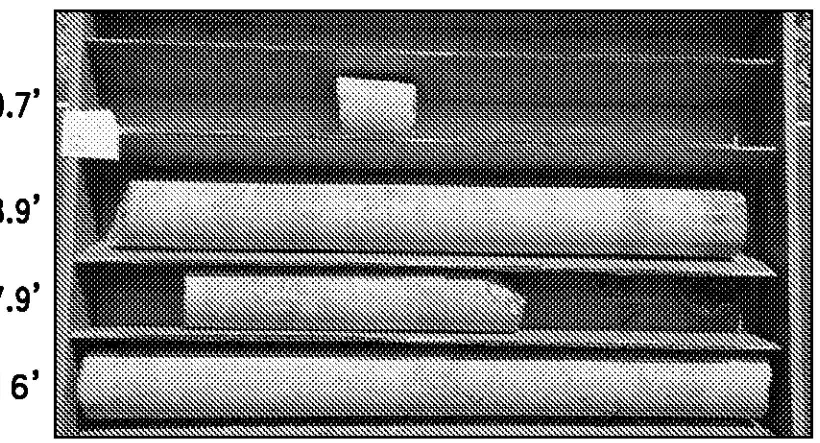
TH-IP1
FROM 20 TO 25-FEET BELOW GRADE



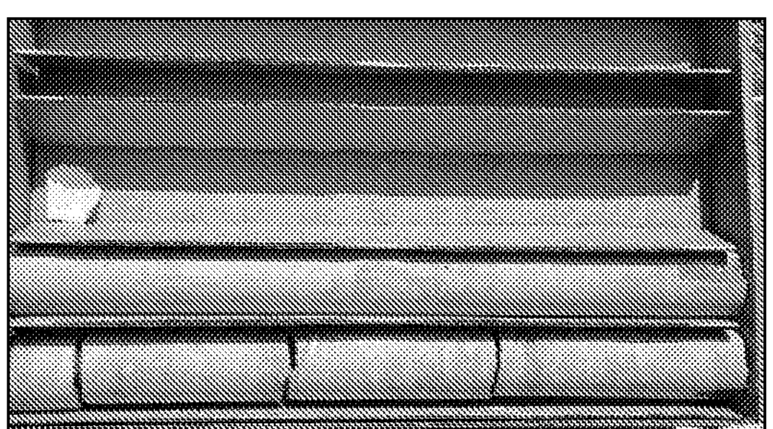
TH-IE2
FROM 6 TO 11-FEET BELOW GRADE



TH-IE2
FROM 11 TO 16-FEET BELOW GRADE



TH-IE2
FROM 16 TO 21-FEET BELOW GRADE



TH-IE2
FROM 21 TO 25-FEET BELOW GRADE

DIMENSIONS SHOWN ARE APPROXIMATE AND ROUNDED TO THE NEAREST 0.2-FT.

Ardaman & Associates, Inc.

ROCK CORE PHOTOGRAPHS FROM
INTAKE STRUCTURE TEST BORINGS
EAST OF GUNTERSVILLE DAM
GUNTERSVILLE, MARSHALL COUNTY, ALABAMA

MAY 18, 2011

113-11-40-1019A | M.S. WILSON, P.E. | 2

Appendix A

Results of Unconfined Compressive Strength and Splitting Tensile Strength on Selected Rock Specimens

Intake Structure Cores - Rock Strength Data

Summary by Ardaman & Associates, Inc. W.S.Jordan, P.E. & Jeremy M. Clark, E.I. 5/6/11

Test Hole	Approximate Depth (ft)	Unconfined Compressive Strength (PSI)*	Splitting Tensile Strength (PSI)**
1	6.2	16,062	1,474
1	7.4	13,747	1,420
1	14.3	13,245	NA***
1	20.3	17,258	1,640
1	21	NA	842
2	6.7	11,009	NA
2	7.5	NA	1,255
2	8.4	11,334	NA
2	14.1	10,663	852
2	14.6	NA	750
2	20.7	NA	1,343
2	21.4	14,590	NA

* Unconfined Compressive Strength Tests and calculations in accordance with ASTM D 2938

**Splitting Tensile Strength Tests and calculations in accordance with ASTM D 3967

***NA indicates that a test was not designated at the corresponding depth

Report of
Subsurface Soil/Rock Exploration and
Geotechnical Engineering Evaluation for
an Intake Structure Near Guntersville Dam
Guntersville, Marshall County, Alabama

File No. 113-11-40-1019A
May 18, 2011
Revised September 19, 2011
Final Revisions January 3, 2012

Final Revisions January 3, 2012
Revised September 19, 2011
May 18, 2011
File No. 113-11-40-1019A

Tetra Tech, Inc. (IER)
101 Church Street, Suite 201
Huntsville, Alabama 35801

Attention: Ms. Shannon Bailey-Partlow, P.E., Project Manager

Subject: **Report** of Subsurface Soil/Rock Exploration and Geotechnical Engineering Evaluation for a Proposed Intake Structure Near Guntersville Dam

Dear Ms. Bailey-Partlow:

As authorized by Tetra Tech (TT), Ardaman and Associates, Inc. (Ardaman) has completed the subsurface exploration and geotechnical engineering evaluation for the proposed intake structure east of the Guntersville Dam. The purposes were to evaluate subsurface conditions encountered in test borings performed at the site, and to provide geotechnical recommendations regarding: foundation support; rock anchor pullout capacity; soil parameters for below grade walls; and earthwork preparation.

This revision includes updated details regarding rock anchors, soil/rock properties, and seismic criteria. This report has been prepared for the exclusive use of Tetra Tech for specific application to the subject project.

We are pleased to be of assistance to you on this phase of your project. When we may be of further service to you or should you have any questions, please do not hesitate to contact us.

Sincerely,

ARDAMAN & ASSOCIATES, INC.
Alabama License No. 2687

Jeremy M. Clark, E.I.
Staff Engineer

William S. Jordan, P.E.
Senior Project Manager

Michael S. Wilson, P.E.
Branch Manager/Senior Engineer
Alabama License No. 24013

JMC/MSW/WSJ/mss

1.0 PROJECT DESCRIPTION AND SCOPE OF SERVICES

It is our understanding that the intake structure will be constructed on the north side of Guntersville Lake, approximately 1,000-feet east of the dam. The inlet invert of the Intake Structure will be at approximately elevation +583-feet, about 14-feet below the grade at our test hole locations, and approximately 12-feet below Guntersville Lake "normal" water elevation. Adjacent to the intake structure, at grade elevation, will be paved areas for vehicular traffic.

Based on the drawings provided to Ardaman from Tetra Tech (drawings D-100B through D-100G), we interpret that the structure consists of two reinforced concrete platforms, one of which will bear on the rock formation and consist of the intake "cells" of the structure, and the other platform housing the pumps, will sit atop the stem walls of the intake cells, effectively directing the loads to the rock bearing platform. The lower platform will likely consist of a structural mat foundation, with average loading on the order of 2 to 3 ksf.

Granular backfill is planned along three sides of the perimeter of the intake cells, which will allow groundwater to accumulate around the exterior walls. When water is contained within the cells a balance of hydrostatic pressure will occur. However, unbalanced uplift and lateral hydrostatic forces will be applied to the structure when/if de-watering occurs within the intake cells. Rock anchors are planned to resist these loads.

Geotechnical services were based on the authorized proposal for the project, as follows:

1. Ardaman mobilized a drill rig and crew to the site, and performed two (2) test borings in locations designated by Tetra Tech. The borings were initiated by performing Standard Penetration Test (SPT) sampling in general accordance with ASTM D 1586. Once "hard rock" was encountered, the borings were then advanced by coring techniques in general accordance with ASTM D 2113.

2. Ardaman's Drill Crew Chief prepared a field log for each boring, which documented specific information including: SPT "N"-values (for soil); Rock Quality Designation (RQD) and rock recovery values (%); visual classification of soil and rock; depth to stratum changes; depth of drilled fluid losses, and depth to encountered groundwater, if apparent. Our drill crew chief packaged portions of the soil samples and boxed all rock cores for transportation to our office.

3. Ardaman's engineers visually/manually classified recovered soil samples and examined, classified, and photographed the rock cores. Based on the soil and rock examination, our engineers developed a boring profile for each test hole. Laboratory tests of selected soil and rock samples were directed to further assess engineering and index properties of the encountered materials.

4. Our engineers analyzed and evaluated subsurface conditions encountered and developed recommendations regarding: foundation support; rock anchor pullout resistance; earthwork preparation; and soil stress parameters for backfill against subgrade walls. Recommendations are presented in this report, along with a test boring location plan and the soil/rock profiles.

2.0 GEOLOGY

Much of the Tennessee Valley is located within the physiographic province of the Interior Low Plateaus. The Tennessee Valley is a rolling upland having an average altitude of approximately 600-feet above sea level and a maximum relief of about 400-feet. The rock underlying the Tennessee Valley mostly consists of carbonates ranging in age from the Late Ordovician to Early Mississippian. Limestone bluffs commonly border the Tennessee River.

The intake structure is near the south edge of the demarcation between the Highland Rim to the north, and the Cumberland Plateau to the south. The Plateau was formerly a vast tableland that sloped southward. Subsequent erosion dissected and roughened the original surface resulting in only a few flat-topped remnants. The upper rocks of this region were mainly shales and sandstones of Pennsylvanian age underlain by older carbonate rocks, but due to erosion, the sandstone was weathered away, exposing the limestone and dolomite underneath. In these less resistant rocks, long and narrow valleys developed along the axes of the folds.

Colluvium deposits tend to decrease in thickness traveling from north to south along Guntersville Dam Road. This decrease in the colluviums deposits are likely due to the strongly sloping rock formations.

The limestone encountered during the subsurface exploration, presented below, matches the geologic description of the Bangor Limestone (Upper Mississippian), which consists mainly of bluish-gray to pale greenish-gray, thick-bedded, coarsely crystalline or finely granular bioclastic and oolitic limestone. The Bangor Limestone formation ranges in thickness from 100 to 700-feet, with a few interbeds of shale and secondary rock formations of maroon mudstone. The base of the formation includes interbeds of cherty limestone and grayish-yellow dolomicrite.

The Hartselle sandstone lies below the Bangor Limestone. It consists of thick and thin-bedded sandstone, but it is covered with soil material that has been washed or has rolled from the higher lying formations.

3.0 FIELD SUBSURFACE EXPLORATION-LOCATIONS AND METHODS

The approximate locations of the test borings are shown on the attached **Figure 1** under the *Test Boring Location Plan*. The borings were located on site by our staff using a wheel tape measuring from existing site features. The boring locations indicated shall be considered accurate only to the degree implied by the methods of measurement used. We understand that TT retained surveyors to document the two test boring locations at a later date.

The portions of the test holes performed by SPT testing were advanced by rotary drilling with 4-inch diameter flight augers, using a Model CME-55 drill rig mounted on a flat-bed truck. The SPT depths were sampled at 18-inch intervals continuously until “hard rock” was encountered. The holes were then advanced in 5-foot intervals using a diamond impregnated drill bit that cuts a core approximately 2-3/8 inch in diameter. The two test holes were grouted with tremie-placed “neat” Portland cement grout upon completion.

4.0 LABORATORY TESTING OF SOILS

Laboratory testing was directed by our engineers on selected soil and rock samples from the test borings, to aid classification and to further define the engineering properties of the soils and rock. The laboratory tests on the soils included Nature Moisture Content (ASTM D 2216), and Percent Finer than the U.S. No. 200 Sieve (ASTM D 1140, percent silt and clay).

The laboratory tests on the rock cores included Unconfined Compressive Strength (ASTM D 2938), and Splitting Tensile Strength (ASTM D 3967). The results of these tests are presented adjacent to the *Soil/Rock Boring Profiles* on the attached **Figure 1**, at the respective depths from which the tested samples were recovered. The rock strength data are also presented in tabular form in **Appendix A**.

5.0 SUBSURFACE SOIL/ROCK AND GROUNDWATER CONDITIONS

5.1 General

Ardaman's interpretations of subsurface conditions encountered are depicted on the *Soil/Rock Boring Profiles* on the attached Figure 1. The soil and rock descriptions shown in the *Soil/Rock Legend* are based upon visual/manual and laboratory test-based classification procedures in general accordance with ASTM D 2488; ASTM D 2487; and AASHTO M145.

Photographs of the rock cores are presented on the attached **Figure 2**. The approximate depth of each core is labeled adjacent to the core. Note that soil deposits within the rock joints are often washed away during the coring process, and the recovery percentages, in part, imply those combined thicknesses, although "honeycombs" and void spaces also reduce recovery percentages. Notably, our drillers monitored down-pressure during coring and other than the less resistant zone from 8' to 10'-4" in TH-IP1, no further soft zones or voids were apparent.

The stratification lines on the *Profiles* represent the approximate boundaries between the soil and rock types, but the actual transitions may be more gradual than implied. This report does not address variations which occur between or away from the borings. The nature and extent of such variations may not become evident until during the course of construction. If any variations become evident, Ardaman must be contacted and authorized to provide additional testing and evaluations concerning the projects geotechnical evaluations and recommendations.

5.2 Soil/Rock Conditions

Initially, we encountered a surficial layer of approximately 1.5-feet of a dark brown silty fine sand with traces of grass, surficial roots, and organics-topsoil (Stratum 1) with limestone rubble. Next, we encountered brown and light brown silty lean clay with sand and with inclusions of fat clay, and occasional sandstone and limestone fragments to ½-inch maximum (Stratum 3).

Underlying Stratum 3, at approximately 5 to 5.5-feet below grade, was a bluish-gray to pale greenish-gray very hard limestone. In test-hole TH-IP1 there was no recovery from 8' to 10'-4", so a split-spoon sample was taken next, and a 0.5-foot thick layer of Stratum 3 (saturated) was encountered to almost 11-feet below grade.

Stratum 3, atop the limestone formation, was stiff to very stiff, in accordance with the *Engineering Classification* system displayed on Figure 1. Based on the rock recovery percentages and the RQD values, the rock appears to be high quality, thick-bedded limestone.

The only potential location of concern encountered during the exploration of the intake structures is from 5 to 11-feet below grade in TH-IP1, where relatively low recovery and RQD values were obtained, and an interbedded layer of clay was encountered near 11-feet depth.

5.3 Groundwater Conditions

Due to the drilling technique used, groundwater elevations could not be estimated. Groundwater elevations at the intake structure should reflect, or be somewhat above, the elevation of Guntersville Lake. Groundwater and Lake elevations will be affected by seasonal variations in regional precipitation, and the lake control elevation(s) of the Dam spillway.

6.0 DESIGN RECOMMENDATIONS

6.1 General Soil/Rock Evaluation

In our opinion, subsurface conditions encountered at the intake structure appear adequate to support the structure on a mat foundation, bearing directly upon sound limestone, at about 14 to 15-feet below the surface at our borings. The exposed bearing surface must consist of clean, solid rock, absent of soil layers. The encountered rock conditions also appear adequate to allow for the construction of anchors to resist lateral, and uplift conditions.

After soil stripping and rock excavation to proposed mat bearing depth, cleaning and leveling of the rock surface is required to provide a smooth uniform bearing surface. Additional boreholes within the rock mass shall be performed beneath the proposed mat foundation. These additional boreholes will serve the purposes of both further exploration of the quality of the rock in more locations, and to serve as holes for the installation of foundation anchors.

Ardaman shall be requested to inspect the exposed rock surfaces and probe the additional boreholes (and any added anchor holes) to delineate additional excavation, if required. We recommend minimum 8 additional boreholes, or one (1) hole every 200 square feet of bearing surface, whichever results in a greater number of boreholes. The boreholes and any added anchor holes shall be probed by a professional engineer and/or his/her qualified representative, under his/her direct supervision, to “feel” along the sidewalls of the drilled boreholes to assess the void spaces and intermittent layers of soil, which would affect bearing and uplift resistance.

Dewatering of the excavation will be required. The limits of the excavation must be safely sloped, or shored in accordance with 29 CFR 1926.65.

Shallow foundations may be supported upon “structural” backfill, placed in lifts and properly compacted in accordance with Section 6.3, below.

6.2 Foundation and Anchor Design

Provided anomalies are not encountered within the additional boreholes, the Intake Structure may be designed for a maximum allowable bearing pressure of up to 10,000 pounds per square foot. We estimate little, if any, measurable foundation settlement, on the order of 1/8th inch or less. We estimate a modulus of subgrade for sound limestone (Recovery ≥90%; RQD≥60%) to be at least 700 pci. This is a theoretical value based upon literature review. No plate load tests have been performed.

<u>The additional boreholes recommended in Section 6.1, above, may be used for rock anchor installation. The total number of anchors shall be as necessary to resist lateral and uplift loading conditions</u>. The required number of anchors can be calculated based on anchor design length and ultimate bond strength estimated below, along with an appropriate factor of safety. However, no two anchors shall be placed closer than 3-feet from one another.

Once the additional boreholes and any additional anchor holes have been drilled and checked for anomalies, we recommend blowing groundwater out of the holes with compressed air and then filling the holes with high strength, non-shrink grout, or the grout shall be tremmied. At anchor holes, the grouting must immediately be followed by placement of a reinforcing bar (the reinforcement bar diameter shall be specified by the anchor designer) into the grouted borehole, to be tied into the foundation system.

Anchor resistance is provided by friction between the grout and rock. Theoretical values of ultimate bond strength are estimated from rock unconfined compressive strength and splitting tensile strength data, and must be verified via field pullout tests.

Anchor pullout capacity is dependent upon rock strength and continuity, grout strength, and the intimacy and roughness of the contact between the grout and rock. Provided the anchors are installed as recommended above, and provided the boreholes encounter hard rock absent of what are judged to be significant soil seams and voids, we estimate that <u>**ultimate bond strength at the interface of the grout and hard limestone of 100 psi (or more) is available. Therefore, the ultimate bond strength for 2.5 inch diameter core-holes is 9.4 kips per foot embedment length, and 7.5 kips per foot for 2.0-inch diameter core-holes**</u>. We recommend discounting the top 1 foot of embedment in calculating anchor capacity.

We also recommend verification of anchor pullout bond capacities by testing at least 25% of the production anchors, or at least five (5) tests, whichever is greater, to 1.5 times the design load. If any of the anchors pull out at or below 1.5 times the design load, we recommend testing all of the anchors installed to that date and adjusting the anchor lengths of subsequent anchors based on the pull-out test results. If the lengths are adjusted due to insufficient capacity, we recommend testing at least five (5) of the modified anchors, to the above specification.

Shallow foundations may be supported upon “structural” backfill meeting the requirements of Section 6.3 below, placed and compacted in accordance with section 6.3. An allowable bearing value of 2.5 ksf is recommended for foundations 5 feet wide or less. This bearing value is based upon allowable settlement of 1 inch or less for a post-construction saturated condition. Minimum allowable foundation width is 18 inches for strip footings and 24 inches for spread footings. Minimum required embedment is 18 inches from bottom of footing to surrounding finish grade.

6.3 Backfill Types and Subsurface Walls

We recommend use of well tamped No. 57 Stone, or granular AASHTO A-1 or A-3 soil and soil-aggregate mixtures, around the intake structure’s subsurface walls. Stone shall be placed in maximum 6-inch loose lifts, each lift thoroughly tamped using a hand operated vibratory plate compactor. For granular soil backfill, we recommend placement in maximum 6-inch loose lifts, each lift compacted to between 95 and 97 percent of the Standard Proctor maximum dry density (ASTM D 698). Adjust the moisture content of the soils as necessary to achieve compaction. Do not over-compact the backfill. Heavy, self propelled compactors and construction equipment shall be kept at least 10-feet away from walls, to avoid over-stressing them.

Areas where pavement will not be placed shall be covered with a 2-foot thick compacted clayey sand cap, consisting of A-2-4 soils, but also with at least 20% passing the US No. 200 sieve, and that exhibit a degree of cohesion consistent with clayey soil fines.

The subsurface cast-in-place, reinforced concrete walls, shall be designed to resist "at rest" lateral soil pressures plus differential hydrostatic stresses. The unit weight of backfill (provided below) shall be multiplied by the coefficient of lateral earth pressure to determine the stresses acting on the walls, and unbalanced hydrostatic stress shall be added when buoyant soil unit weight is used. Anticipated surcharge loads adjacent to the wall shall be multiplied by the coefficient of lateral pressure to determine the resulting additional uniform horizontal load. The following parameters shall be utilized in the design of subsurface walls:

- **<u>No. 57 Stone Backfill*</u>**
  - Total moist unit weight (γ_t) = 115 pcf
  - Saturated unit weight (γ_s) = 125 pcf
  - Buoyant Unit Weight (γ_b) = 63 pcf
  - Angle of internal friction (Φ) = 34 degrees
  - Undrained Cohesion "Cu"="0" (zero)
  - Subgrade Modulus (k) = 200 pci**
  - Lateral "at rest" earth pressure coefficient (K_0) = 0.44 (fixed walls)
  - Friction Coefficient (δ)=0.45 (=2/3*tan(Φ))(No. 57 Stone/structure interface)***

- **<u>A-1 or A-3 Granular Soil Backfill*</u>**
  - Total Soil moist unit weight (γ_t) = 115 pcf
  - Saturated unit weight (γ_s) = 125 pcf
  - Buoyant Soil Unit Weight (γ_b) = 63 pcf
  - Angle of internal friction (Φ) = 30 degrees
  - Undrained Cohesion "Cu"-"0" (zero)
  - Subgrade Modulus (k) = 175 pci**
  - Lateral "at rest" earth pressure coefficient (K_0)=0.5 (fixed walls)
  - Friction Coefficient (δ)=0.38 (=2/3*tan(Φ))(Granular soil backfill/structure interface)***

* No Factor of Safety is built into these values. If there is no potential for net uplift of the Intake Structure, then an ultimate concrete/rock interface frictional value of 1.0 times the normal *<u>effective</u>* stress may be used in the sliding resistance evaluation (with an appropriate factor of safety).

** These are theoretical values based upon literature review. No plate load tests have been performed.

***Unit Shearing Resistance $(F)=(K_0)(\delta)[\gamma_t h_w+\gamma_b(h-h_w)]$

where: h_w is the vertical depth (feet) below grade to groundwater table; and h is the vertical depth (feet) below grade at which shearing resistance is determined. If the groundwater table was not encountered, h_w will equal h.

6.4 Seismic Criteria

Based upon review of the International Building Code, the Guntersville Intake Structure area is site class "B" (rock), with S_s=0.31 and S_1=0.105. The USGS web site indicates a Peak Horizontal Acceleration (PHA) of 0.049g for the site.

7.0 CLOSURE

The recommendations submitted in this report are based upon the data obtained from the soil and rock borings presented on the attached Figure 1. This report does not reflect any variations which may occur between or away from the borings. The nature and extent of site variations may not become evident until during the course of construction. If site variations appear evident, it will be necessary to reevaluate the recommendations of this report after performing further on-site observations during the construction period and noting the characteristics of such variations.

In the event any changes occur in the design, nature, location of the facility, or assumed structural loads, Ardaman and Associates, Inc. must be contacted to review the applicability of the conclusions and recommendations in this report. Ardaman and Associates, Inc. must also perform a general review of final design drawings and specifications to determine if earthwork and foundation recommendations have been properly interpreted and implemented in the design specifications.

This study does not deal with the possibility of eventual sinkhole development at the site. This exploration and analysis covers only the near surface materials. It is not intended to include deep soil or rock strata where cavities and caverns may exist.

This report has been prepared in accordance with generally accepted geotechnical engineering practices. No other warranty, expressed or implied, is made.

End of Report

Appendix A

Results of Unconfined Compressive Strength and Splitting Tensile Strength on Selected Rock Specimens

Intake Structure Cores - Rock Strength Data

Summary by Ardaman & Associates, Inc.:W.S.Jordan, P.E. & Jeremy M. Clark, E.I. 5/6/11

Test Hole	Approximate Depth (ft)	Unconfined Compressive Strength (PSI)*	Splitting Tensile Strength (PSI)**
1	6.2	16,062	1,474
1	7.4	13,747	1,420
1	14.3	13,245	NA***
1	20.3	17,258	1,640
1	21	NA	842
2	6.7	11,009	NA
2	7.5	NA	1,255
2	8.4	11,334	NA
2	14.1	10,663	852
2	14.6	NA	750
2	20.7	NA	1,343
2	21.4	14,590	NA

* Unconfined Compressive Strength Tests and calculations in accordance with ASTM D 2938

**Splitting Tensile Strength Tests and calculations in accordance with ASTM D 3967

***NA indicates that a test was not designated at the corresponding depth

OMI, Inc.

SUBSURFACE EXPLORATION AND
GEOTECHNICAL ENGINEERING STUDY
Proposed
Water Intake Road
Guntersville Dam
Marshall County, Alabama

OMI Job No. 6590-A

April 3, 2013

5151 Research Drive, N.W., Suite A Huntsville, AL 35805 Tel: (256) 837-7664 Fax: (256) 837-7677 E-mail: omi@omi-eng.com

OMI, Inc.

April 3, 2013

Tetra Tech
2110 Powers Ferry Road, Suite 202
Atlanta, GA 30339

ATTN: Mr. Michael Schmidt

SUBJECT: Report of Geotechnical Engineering Study
Proposed Water Intake Road
Guntersville Dam
Marshall County, Alabama
OMI Job No. 6590-A

Gentlemen:

OMI, Inc., has completed a subsurface exploration and geotechnical engineering study for the referenced project. Enclosed is the report of the findings as well as recommendations for pavement design and construction, site preparation, and other geotechnically related site activities. This work was authorized on March 20, 2013 by Mr. Michael Schmidt of Tetra Tech.

OMI, Inc., appreciates the opportunity to be of service to Tetra Tech and looks forward to continued involvement with the construction monitoring phase of this project. Please direct any questions concerning this report to the undersigned.

Respectfully submitted,

OMI, Inc.

Christopher S. Jones, E.I.
Staff Engineer

John M. Ozier, P.E.
Senior Engineer

Distribution: 3 Copies to Addressee

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APPENDICES

1.0 EXECUTIVE SUMMARY

Visual observation of the site and review of soil boring records from this study as well as from previous studies conducted by OMI and others, indicates the area proposed for the new access road is underlain by layers of soil and limestone rock boulders. OMI anticipates both soil and rock fill will be required to construct the access road. Specific recommendations for pavement design and site earthwork are given in the body of this report.

2.0 INTRODUCTION

OMI, Inc., has completed a geotechnical engineering study for the proposed water intake access road near Guntersville Dam in Marshall County, AL. This report outlines the scope of services provided and presents comments and recommendations based on professional opinions formed during the course of this study. This work was authorized on March 20, 2013, by Mr. Michael Schmidt of Tetra Tech. The work was performed in general accordance with correspondence between Mr. John M. Ozier and Mr. Michael Schmidt and Mr. Chris Coleman of Tetra Tech.

Assessment of the environmental aspects of this site, including previous land use or the determination of the presence of any chemical, industrial, or hazardous waste is beyond the scope of this study. However, OMI can provide these services if desired.

3.0 EXPLORATION METHODS

The procedures used by OMI for field testing are in general accordance with ASTM procedures and established engineering practice. Brief descriptions of the procedures used in this exploration are contained in the Appendix of this report.

OMI performed two shallow hand auger borings during this study near Station 301+75 and Station 299+50 as requested by Tetra Tech. Soil boring records for these borings as well as borings B-2 and B-3 from previous studies, OMI Job No. 5085 are included in the appendix of this report. Borings B-2 and B-3 from previous studies were located near the previously mentioned stations.

4.0 SITE CONDITIONS

The site for the proposed water intake road is located near an existing gravel road that extends east/southeast along the Guntersville Lake shoreline from a paved access road and parking area. The site is undeveloped and wooded with numerous visible rock outcrops. Topographically, the site slopes downward toward Guntersville Lake. Visual observations of the area indicate excavation of rock was required to construct the existing paved road.

5.0 SUBSURFACE CONDITIONS

Based on two shallow hand auger soil borings performed during this study, review of soil boring records from previous studies conducted within the area by OMI and others, and visual observations, the subsurface conditions beneath the area proposed for road consists of various layers of soil and limestone rock boulders. Soil layers encountered by the shallow borings performed during this study indicate soil layers are relatively thin.

6.0 SITE GEOLOGY

Bangor Limestone

The Bangor Limestone is composed of about 350 to 420-ft of bioclastic and oolitic limestone, dolomite, and shale. Chert contained in the Bangor is generally small black nodules found in the upper part of the formation. The upper part of the Bangor grades northeastward into the Pennington Formation and is generally composed of green to gray, calcareous shales and thin beds of dolomite and limestone. The middle part of the Bangor is generally medium to massive-bedded limestone and dolomite. The basal part of the formation is generally medium to massive-bedded, argillaceous limestone with occasional partings of yellow calcareous shale. Only the basal portion of the Bangor is present at this site.

Sinkhole Activity

Sinkholes have occurred in this formation within the vicinity of this site. However, surface observations and the subsurface exploration did not disclose evidence of sinkhole activity on this site. This exploration does not, nor was it intended to, address the possibility of future sinkhole development.

7.0 PROJECT INFORMATION

OMI understands Huntsville Utilities plans to construct a water treatment plant with an intake structure located east of the dam. Further, current plans include the construction of a paved access road to the proposed intake structure. The proposed raw water line and access road will extend from the proposed pump intake building to the west/northwest along the Guntersville Lake shoreline and tie into the existing paved road that was previously discussed. Review of plans provided by Tetra Tech indicate the majority of proposed access road will be in cut sections; however, up to 4-ft of fill will be required in some areas, primarily between Station 299+00 and Station 302+00.

8.0 BASIS FOR RECOMMENDATIONS

The following recommendations are based in part on the preceding project information. This study has utilized the subsurface data, historical information regarding the structural performance of similar structures, and past experience with similar geologic environments to develop professional opinions on which the recommendations are based. Because the structural elements of the design greatly influence the design recommendations, OMI must be provided the opportunity to review the following comments and recommendations in light of changes in road location, elevation, or structural loading.

9.0 DESIGN and CONSTRUCTION RECOMMENDATIONS

9.1 Fill Material

Areas Requiring More Than 2-ft of Fill

Shot rock or surge stone with a diameter of 12-in or less may be used in areas where more than 2-ft of fill is required. The shot rock or surge stone should not contain soil particles and should be placed in 18-in lifts that are walked in with a dozer or similar piece of heavy equipment. OMI should be present to observe stripped areas prior to placement of the fill. Further, OMI should monitor and direct the placement and compaction of the shot rock or surge stone.

A thin layer of No. 2 stone should be used to cap the shot rock/surge stone fill. The No. 2 stone should be placed and walked in with heavy machinery to ensure surface voids in the shot rock/surge stone are properly filled. A 3-in lift of dense grade base should be placed on the No. 2 stone and compacted to 100 percent of the materials standard Proctor maximum dry density, SPMDD. The pavement section, including the dense grade base, may be placed on the compacted dense grade base.

After stripping and/or cutting is complete, OMI should observe the subgrade or areas to receive fill. Due to the nature of the sites subsurface conditions, OMI anticipates voids between rock boulders will be exposed during stripping and cutting. OMI recommends a thin layer of No. 2 stone be used to fill the voids and as fill to raise the grade to 3-in below the pavement section. A 3-in lift of dense grade base should be placed on the No. 2 stone and compacted to 100 percent of the materials standard Proctor maximum dry density, SPMDD. The pavement section including the dense graded base may be placed on the compacted dense grade base.

9.2 Pavement Areas

The access road should be prepared in accordance with the general recommendations for stripping and fill placement stated elsewhere in this text, except the upper 1-ft must be compacted to at least 100 percent of the standard Proctor maximum dry density. Specific traffic frequency and loading information has not been provided; however, based on previous experience, the following pavement sections may be used based on the assumption that normal traffic will be 1 to 10 pickup trucks a day and an occasional heavy truck.

FLEXIBLE PAVEMENT DESIGN

PAVEMENT MATERIAL	AUTOMOBILE
ASPHALT SURFACE COURSE (Hot Mix) ALDOT No. 429A, 1/2-in ESAL Range A/B	1.0 inch
ASPHALT BINDER COURSE ALDOT No. 429B, ¾-in, ESAL Range A/B	2.0 inches
STONE BASE COURSE ALDOT No. 825 B (Compacted to 100% Standard Proctor as per AASHTO T-99)	5.0 inches
TOTAL THICKNESS	8.0 inches

All pavement materials and construction methods should conform to the guidelines and requirements of the Alabama Department of Transportation. During placement of the aggregate base and asphalt courses, density tests and thickness measurements should be performed to compare the design section to the constructed section. The soil subgrade should be graded to provide a smooth transition from one pavement section to another.

Immediately prior to placement of the aggregate base, the subgrade must be proofrolled to judge the stability of stone. The stone may require re-compaction. The stone base course should only be applied to a stable, compact subgrade. Asphalt paving should proceed closely after stone placement. If lengthy delays between stone and asphalt paving occur, the stability of the stone and soil subgrade should be checked prior to paving.

9.3 Density Testing

Field density testing should be performed on each lift prior to placement of additional lifts. Test locations should be evenly distributed throughout the fill area and should be performed at the frequencies shown on the following table.

AREA	METHOD OF PLACEMENT AND COMPACTION	INITIAL TEST FREQUENCY	RETEST FREQUENCY
General Site	Large self-propelled equipment	1 test per 100-ft of road, minimum 3 tests per lift	1 test per failed test
Isolated Areas	Hand-guided equipment	1 test per lift	1 test per failed test
Trench backfill and behind retaining walls	Hand-guided equipment	1 test per 50 linear feet per 6-in of fill	1 test per failed test

Test frequencies may be increased during the early stages of earthwork construction. Compaction requirements should apply to all excavation/backfill operations conducted on the proposed development property.

9.4 Construction Monitoring

The pavement design and site preparation recommendations contained in this report are based on the conditions encountered during the subsurface exploration and past experience in this geologic setting. Because subsurface conditions may vary from the anticipated, it is important to have a well-rounded quality control program. Construction monitoring on a project of this nature can serve as an economical means to achieve the best possible foundation system and reduce the potential for future problems. The involvement in the subsurface exploration portion of this project uniquely qualifies OMI, Inc., to provide these services as a party responsible to the Owner. OMI, Inc., strongly recommends that all construction monitoring be performed under contract with the Owner or the Owner's representative.

APPENDIX

OMI, Inc.

5151 Research Drive, N.W. Huntsville, AL 35805

JOB NO.: 6590 JOB: WTP Intake Road LOG OF TEST PIT: B-1

JOB LOCATION: Guntersville Dam TEST PIT LOCATION: Sta 299+50

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL Elevation=	MOISTURE CONTENT	STATIC CONE PENETROMETER READINGS Kg/cm2	POCKET PENT (tsf)	DYNAMIC CONE PENETROMETER	DYNAMIC CONE VALUE ○ 20 40 60 80 Pp ■ (tsf) 1 2 3 4 MOISTURE CONTENT- ▲ STATIC CONE VALUE- ● PL ——— LL 20 40 60 80
0			AUGER REFUSAL AT SURFACE					
2.5								
5								
7.5								
10								
12.5								
15								
17.5								
20								

COMPLETION DEPTH: 0 DEPTH TO WATER INITIAL: N/A OMI,Inc.

DATE: 3/28/13 DEPTH TO WATER FINAL: N/A

OMI, Inc.

5151 Research Drive, N.W. Huntsville, AL 35805

JOB NO.: 6590 JOB: WTP Intake Road LOG OF TEST PIT: B-2

JOB LOCATION: Guntersville Dam TEST PIT LOCATION: 301+75

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL Elevation=	MOISTURE CONTENT	STATIC CONE PENETROMETER READINGS Kg/cm2	POCKET PENT (tsf)	DYNAMIC CONE PENETROMETER	DYNAMIC CONE VALUE ○ 20 40 60 80 Pp ■ (tsf) 1 2 3 4 MOISTURE CONTENT- ▲ STATIC CONE VALUE- ● PL ─┼─ ─ ─ ─┼─ LL 20 40 60 80
0			SANDY SILTY CLAYwith 20% medium to coarse sand, 80% fines, low plasticity, yellowish brown, very stiff, moist, residuum, CL		15 16	2.0 2.5	15 21	
2.5			AUGER REFUSAL AT 2-FT					
5								
7.5								
10								
12.5								
15								
17.5								
20								

COMPLETION DEPTH: 2 DEPTH TO WATER INITIAL: Dry OMI,Inc.

DATE: 3-28-13 DEPTH TO WATER FINAL: N/A

OMI, Inc.

DRILL LOG

PROJECT	PROJECT NO.	BORING NO.
Guntersville Dam WTP Intake		B-2

SITE	BEGUN	COMPLETED	HOLE SIZE	PAGE NO.
	3/13/08	3		1 of 1

COORDINATES	DEPTH GROUND WATER	AT FIRST CHECK		GROUND ELEVATION

DRILLER	CORE RECOVERY (%)	# SAMPLES	# CORE BOXES	DEPTH TOP OF ROCK
RS		2	1	8.5-ft

DRILL MAKE AND MODEL	LOGGED BY:	DEPTH BOTTOM OF HOLE
	JJ	10

SAMPLE TYPE AND DIAMETER	SAMPLE NO.	SAMPLE LENGTH	SAMPLE RECOVERY	N-VALUE/ RQD (%)	REMARKS: NOTES ON WATER LOSSES AND LEVELS, CAVING, AND DRILLING CONDITIONS CASING DEPTH	FRACTURES PER FOOT	STRATA ELEVATION/ DEPTH	DEPTH	GRAPHIC LOG	DESCRIPTION
NQ 0.0-ft.	Run 1	60"	13" 22%	15%		0	0	0		LIMESTONE.
						X	0.0 / -1 / 1.0			SANDY SILTY CLAY with 15% limestone boulders, 10% medium to coarse sand, 75% fines, medium plasticity, tan, stiff, moist, fill, CL.
						X				
						X				
						X	-4 / 4.0	5		LIMESTONE.
NQ 5.0-ft	Run-2	60"	20" 33%	30%		0	-5 / 5.0			SANDY SILTY CLAY with 15% limestone boulders, 10% medium to coarse sand, 75% fines, medium plasticity, tan, stiff, moist, fill, CL.
						X				
						X				
						X	-8.5 / 8.5			LIMESTONE, slightly weathered light gray, hard, fine grained, medium bedded.
						0		10		Rock Core terminated at 10-ft.
								15		
								20		
								25		
								30		
								35		

OMI, Inc.

DRILL LOG

PROJECT	PROJECT NO.	BORING NO.
Guntersville Dam WTP Intake		B-3

SITE	BEGUN	COMPLETED	HOLE SIZE	PAGE NO.
	3/13/08	3/13/08	NQ	1 of 1

COORDINATES	DEPTH GROUND WATER	AT FIRST CHECK		GROUND ELEVATION

DRILLER	CORE RECOVERY (%)	# SAMPLES	# CORE BOXES	DEPTH TOP OF ROCK
RS		2	1	7.5-ft

DRILL MAKE AND MODEL	LOGGED BY:	DEPTH BOTTOM OF HOLE
	JJ	10

SAMPLE DATA					REMARKS: NOTES ON WATER LOSSES AND LEVELS, CAVING, AND DRILLING CONDITIONS CASING DEPTH	FRACTURES PER FOOT	STRATA ELEVATION/ DEPTH	DEPTH	GRAPHIC LOG	DESCRIPTION
SAMPLE TYPE AND DIAMETER	SAMPLE NO.	SAMPLE LENGTH	SAMPLE RECOVERY	N-VALUE/ RQD (%)						
NQ 0.0-ft	Run 1	60"	13" 22%	0%		X X X X X	0 0.0	0		Surge stone and/or rip rap fill.
NQ 5.0-ft	Run 2	60"	35" 58%	44%		X X X 0 0 0	-7.5 7.5	5		LIMESTONE, slightly weathered, fine to medium grained, hard, gray, medium bedded.
								10		Rock Core terminated at 10-ft.
								15		
								20		
								25		
								30		
								35		

Report of Subsurface Soil Exploration
And Geotechnical Engineering Evaluation
For the Raw Water Main, Intake to WTP Site
Guntersville, Marshall County, Alabama

May 23, 2011
File No.113-11-40-1019A

Ardaman & Associates, Inc.

OFFICES

Orlando, 8008 S. Orange Avenue, Orlando, Florida 32809, Phone (407) 855-3860
Bartow, 1525 Centennial Drive, Bartow, Florida 33830, Phone (863) 533-0858
Cocoa, 1300 N. Cocoa Blvd., Cocoa, Florida 32922, Phone (321) 632-2503
Fort Myers, 9970 Bavaria Road, Fort Myers, Florida 33913, Phone (239) 768-6600
Miami, 2608 W. 84th Street, Hialeah, Florida 33016, Phone (305) 825-2683
Port Charlotte, 740 Tamiami Trail, Unit 3, Port Charlotte, Florida 33954, Phone (941) 624-3393
Port St. Lucie, 460 Concourse Place NW, Unit 1, Port St. Lucie, Florida 34986, Phone (772) 878-0072
Sarasota, 2500 Bee Ridge Road, Sarasota, Florida 34239, Phone (941) 922-3526
Tallahassee, 3175 West Tharpe Street, Tallahassee, Florida 32303, Phone (850) 576-6131
Tampa, 3925 Coconut Palm Drive, Suite 115, Tampa, Florida 33619, Phone (813) 620-3389
West Palm Beach, 2200 North Florida Mango Road, Suite 101, West Palm Beach, Florida 33409, Phone (561) 687-8200

Ardaman & Associates, Inc.

Geotechnical, Environmental and Materials Consultants

May 23, 2011
File No. 113-11-40-1019A

Tetra Tech, Inc. (IER)
101 Church Street, Suite 201
Huntsville, Alabama 35801

Attention: Ms. Shannon Bailey-Partlow, P.E., Project Manager

Subject: Report of Subsurface Soil Exploration and Geotechnical Engineering Evaluation for the Raw Water Main, Intake to WTP Site, Guntersville, Marshall County, Alabama

Dear Ms. Bailey-Partlow:

As authorized, Ardaman & Associates, Inc. (Ardaman) has completed the subsurface soil exploration and geotechnical evaluations for the proposed raw water line. The purposes of these services were to portray and evaluate subsurface conditions encountered in test borings performed along the proposed pipeline alignment.

This report has been prepared for the exclusive use of Tetra Tech, Inc. for specific application to the subject project.

We are pleased to be of assistance to you on this phase of your project. When we may be of further service to you or should you have any questions, please do not hesitate to contact us.

Sincerely,

ARDAMAN & ASSOCIATES, INC.
Alabama License No.: 2687

Jeremy M. Clark, E.I.
Staff Engineer

Michael S. Wilson, P.E.
Branch Manager/Senior Engineer
Alabama License No. 24013

William S. Jordan, P.E.
Senior Project Engineer

JMC/MSW/MSWms

1.0 PROJECT BACKGROUND AND SCOPE OF SERVICES

The raw water (force) main is proposed to be installed alongside an approximate 3.2 mile length of Guntersville Dam Road, approximately 20-feet east of the east edge of pavement, running parallel to the road. Tetra Tech has estimated that the raw water main will be 42-inch diameter Ductile Iron, to be installed with its invert approximately 7 to 8-feet below grade. It will transmit raw water from the proposed intake structure at Guntersville Lake to the proposed water treatment plant (WTP) over 3-miles to the north. See **Figure 1** for location maps.

Geotechnical services performed were based on Ardaman's authorized proposal for the project, and generally accepted geotechnical practices, as follows:

1. Ardaman mobilized a drill rig and crew to the site in April of 2011, and performed a total of seventeen (17) auger borings, at approximate 1,000-foot intervals along the proposed pipeline alignment, to a depth of 10-feet each below grade.

2. During the performance of the borings, our drill crew prepared a field log for each boring, visually classified the soils, and transported portions of the samples to our office for further classification by our engineers. The depths to groundwater in the borings, when encountered, were estimated in the field by our drill crew, and visually confirmed in the laboratory by our engineers.

3. Recovered soil samples were visually/manually classified by our engineers, whom developed a soil profile for each boring. Laboratory tests were designated on selected soil samples to further define their engineering and index properties.

4. Our engineers researched site area geology. We analyzed the soil conditions encountered in the test borings and provide excavation evaluations. Our findings and recommendations are documented in this report, which includes a test boring location plan and soil boring profiles.

2.0 GEOLOGY

Much of the Tennessee Valley is located within the physiographic province of the Interior Low Plateaus. The Tennessee Valley is a rolling upland having an average altitude of approximately 600-feet above sea level and a maximum relief of about 400-feet. The rock underlying the Tennessee Valley mostly consists of carbonates ranging in age from the Late Ordovician to Early Mississippian. Limestone bluffs commonly border the Tennessee River.

The pipeline site is near the south edge of the demarcation between the Highland Rim to the north, and the Cumberland Plateau to the south. The Plateau was formerly a vast tableland that sloped southward. Subsequent erosion dissected and roughened the original surface resulting in only a few flat-topped remnants. The upper rocks of this region were mainly shales and sandstones of Pennsylvanian age underlain by older carbonate rocks, but due to erosion, the sandstone was weathered away, exposing the limestone and dolomite underneath. In these less resistant rocks, long and narrow valleys developed along the axes of the folds.

Colluvium deposits tend to decrease in thickness traveling from north to south along Guntersville Dam Road. This decrease in the colluviums deposits are likely due to the strongly sloping rock formations.

The occasional limestone encountered during the subsurface exploration, presented below, matches the geologic description of the Bangor Limestone (Upper Mississippian), which consists mainly of bluish-gray to pale greenish-gray, thick-bedded, coarsely crystalline or finely granular bioclastic and oolitic limestone. The Bangor Limestone formation ranges in thickness from 100 to 700-feet, with a few interbeds of shale and secondary rock formations of maroon mudstone. The base of the formation includes interbeds of cherty limestone and grayish-yellow dolomicrite.

The Hartselle sandstone lies below the Bangor Limestone. It consists of thick and thin-bedded sandstone, but it is covered with soil material that has been washed or has rolled from the higher lying formations.

3.0 FIELD SUBSURFACE EXPLORATION-LOCATIONS AND METHODS

The approximate locations of the seventeen (17) auger borings performed are shown on the attached **Figures 1 and 2.** The borings were located on site by our staff using a wheel tape, measuring from existing landmarks, and were later surveyed (coordinated by Tetra Tech). The boring locations indicated shall be considered accurate only to the degree implied by the methods of measurement used. Note that some of the boring locations were adjusted from the original planned location due to accessibility issues or borehole advancement limitations. The approximate actual stations of the borings are indicated on Figure 2.

The borings were performed in general accordance with ASTM D 1452, advanced with 4-inch diameter continuous flight augers and in some instances with manually operated "bucket" augers where drill rig access was not possible. Soil sampling was performed at 2-foot intervals. The borings were backfilled with auger cuttings and site soils upon completion.

4.0 LABORATORY TESTING

Laboratory testing was performed on selected soil samples from the test borings to aid classification and further define the engineering properties of the soils. The laboratory tests included Natural Moisture Content (NM)(AASHTO T-265); Percent Finer than the U.S. No. 200 Sieve (-200)(AASHTO T-11, percent silt and clay), and Atterberg Limits determinations (LL & PI) (AASHTO T-89 & 90, plasticity). The test results are presented on the attached Figure 3 adjacent to the *Soil Boring Profiles*, at their respective depths from which the tested samples were recovered.

5.0 SUBSURFACE SOIL CONDITIONS

5.1 General

Our interpretations of subsurface conditions encountered in the test borings are depicted on the *Soil Boring Profiles* presented on the attached **Figure 3.** The soil descriptions in the *Soil Legend* are based upon visual/manual classification procedures in accordance with ASTM D 2488, the laboratory index tests in accordance with ASTM D 2487, and AASHTO M-145.

The stratification lines on the *Profiles* represent the approximate boundaries between soil types, and the actual transitions may be more gradual than implied. This report does not reflect or address variations which occur between or away from the borings.

5.2 Soil Conditions

The following soil types encountered are described as follows:

- dark brown silty fine sand with traces of grass and surficial roots and trace organics; topsoil (Stratum 1);
- brown medium to fine sand with silt and clay and occasional limestone gravel pieces to 1-inch maximum (Stratum 1A);
- brown and light brown silty, clayey sand (Stratum 2);
- yellowish-brown very sandy, clayey silt to very sandy, silty lean clay (Stratum 2A);
- brown and light brown silty lean clay with sand and with inclusions of fat clay and occasional sandstone and limestone fragments to ½-inch maximum (Stratum 3);
- reddish-brown to brown silty, very clayey fine sand to sandy lean clay with occasional inclusions of sandy fat clay (Stratum 4);
- reddish-brown silty, very clayey sand to very sandy, silty lean clay (Stratum 4A);
- marbled light yellowish-brown and red silty sandy lean clay with inclusions of sandy fat clay (Stratum 5);
- marbled light gray, tan, and red clayey elastic silt to fat clay with occasional sandstone gravel to ¼-inch maximum (Stratum 6);
- marbled light brown and gray silty lean clay to fat clay with sand (Stratum 6A).

In general, the soils encountered consisted mostly of moderately plastic to highly plastic clays and silts. Crushed limestone (gravel) and standstone was observed within Strata 1A, 3, and 6. A limestone boulder was encountered at the Station 130+00 planned boring location, at 2-feet below grade, which is why AB-130 was moved an additional 11-feet from the edge of pavement. At this location, our drill crew chief indicated that the roadway appeared to be built-up using limestone boulders. Therefore, limestone boulders may be prevalent in this area, and covered "outcrops" are certainly possible in other areas of the alignment.

5.3 Groundwater Conditions

Groundwater, or wet soils, were encountered in three of the auger borings (AB-130, 140, and 168.5), as shallow as approximately 6-feet below grade.

Shallow perched groundwater may be encountered at the site during construction following rainy periods, specifically within sandy layers (Strata 1, 1A, and 2), atop or within the less permeable clayey Strata. While such a perched groundwater condition is not permanent, it could negatively impact the earthwork operations and schedule, if present during construction.

5.4 Soil Survey Information for Marshall County (Feb. 15, 2007)

According to the USDA Soil Survey, the majority of the soil along Guntersville Dam Road consists mainly of Allen-Waynesboro sandy and clayey loams, characterized as well draining with a very low frequency to flooding or ponding. The Allen-Waynesboro deposit is gently sloping with 2 to 6 percent slopes and a depth to a restrictive feature of greater than 80-inches.

6.0 GEOTECHNICAL ENGINEERING EVALUATION

6.1 Excavation Cautions

As previously noted, auger boring AB-130 (station 130+00) had to be moved 11-feet east of the planned location due to limestone rubble. Our drill crew chief made multiple attempts to penetrate the limestone, and was unsuccessful until eventually moving 31-feet off the edge of pavement. Locations with strata containing limestone pieces include: Station 120+00; 130+00; 160+00; 180+00; 250+00; 270+00; and 280+00. At these locations, excavation may be difficult. Difficult rock excavation may be experienced in the southern portion of the proposed line, nearer the intake structure.

Difficulties in excavation may also occur around Stations 130+00, 140+00, and 168+50 due to groundwater. Provide adequate drainage in these areas, and be prepared to dewater.

6.2 Backfilling Criteria

Bedding for the proposed pipeline and backfilling requirements should be in accordance with the pipe manufacturer's recommendations. Select backfill may be required for the pipe envelope. The on-site soils have a low to high potential for corrosion, with the majority of the site rated as "moderate". Further site testing will be required to measure specific soil corrosive properties.

In generally accepted practice, for green areas (landscaped areas not supporting roadways or structures), backfill may consist of the excavated soil types, except for excessively organic soils, (not encountered during our exploration), and in some cases boulders which may be encountered in the excavation.

Compaction criteria, if any, are generally regulated by local municipal or governmental jurisdictions. In order to avoid substantial subsidence of the backfilled pipeline area, the backfill soils should generally be compacted to either 95% of the Standard Proctor maximum dry density (AASHTO T99), or if soil conditions are highly variable, 105% of the density of similar soils in the undisturbed sides of the pipeline trench at those elevations.

The soils encountered occasionally contained layers or inclusions of fat clay, but were generally of the clayey sand and sandy lean clay types, which in our opinion are slightly to moderately expansive. We would not expect backfill using these soil types to result in expansion to the degree that "egging" of the pipeline would be a concern.

7.0 CLOSURE

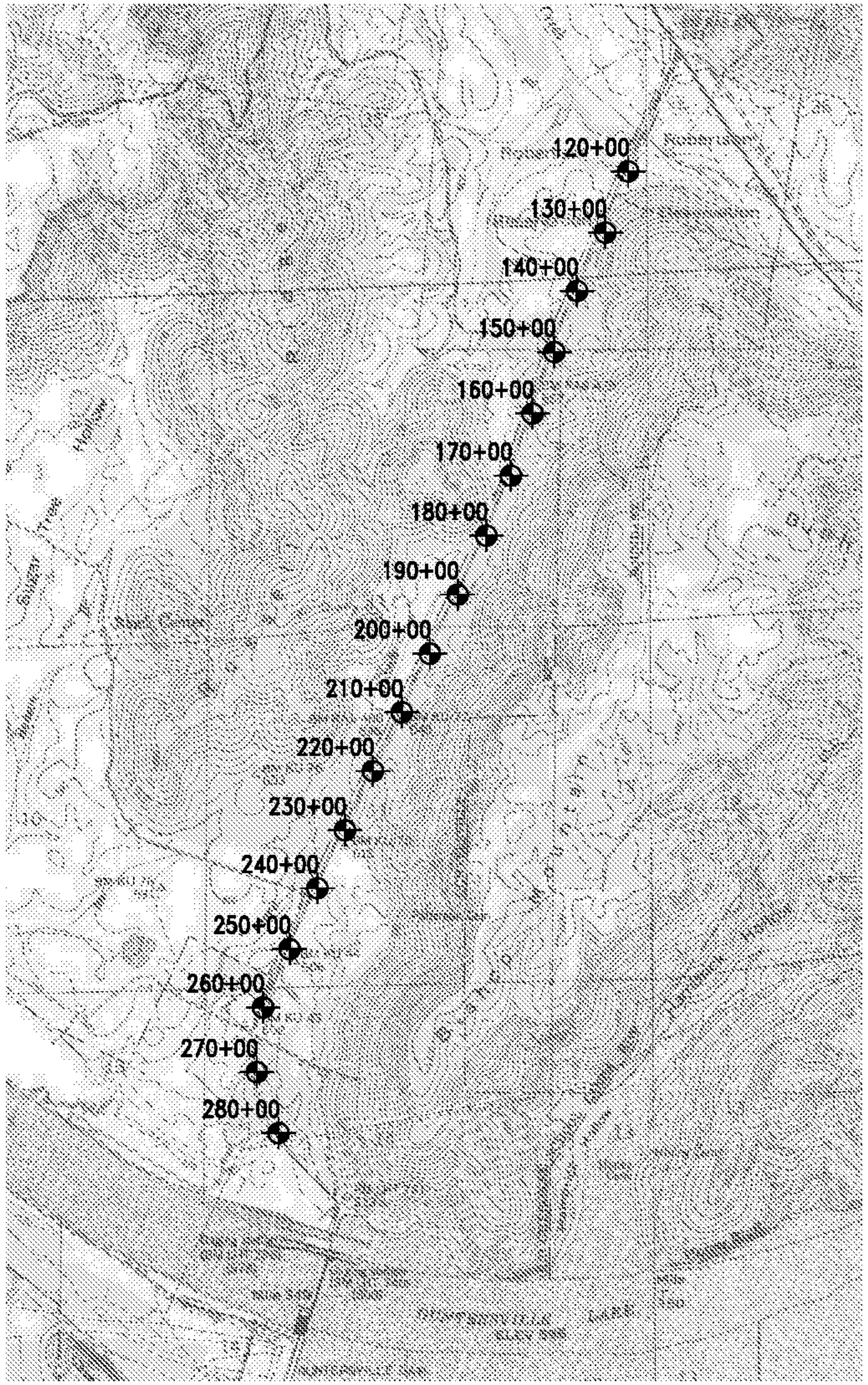
The recommendations submitted in this report are based upon the data obtained from the soil borings presented on Figure 3. This report does not reflect any variations which may occur between the borings, or over the course of time. The nature and extent of variations may not become evident until construction. If site or soil or groundwater variations appear evident, it will be necessary to reevaluate the recommendations of this report after performing further on-site observations during the construction period, and noting the characteristics of such variations.

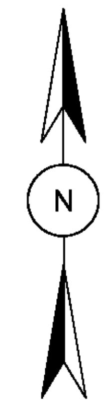
This study does not deal with the possibility of *eventual* sinkhole development at the site. This exploration and analysis covers only the near surface soil and limestone deposits explored at specific locations and to specific depths. It is not intended to include deeper soil or rock strata where cavities and caverns may exist.

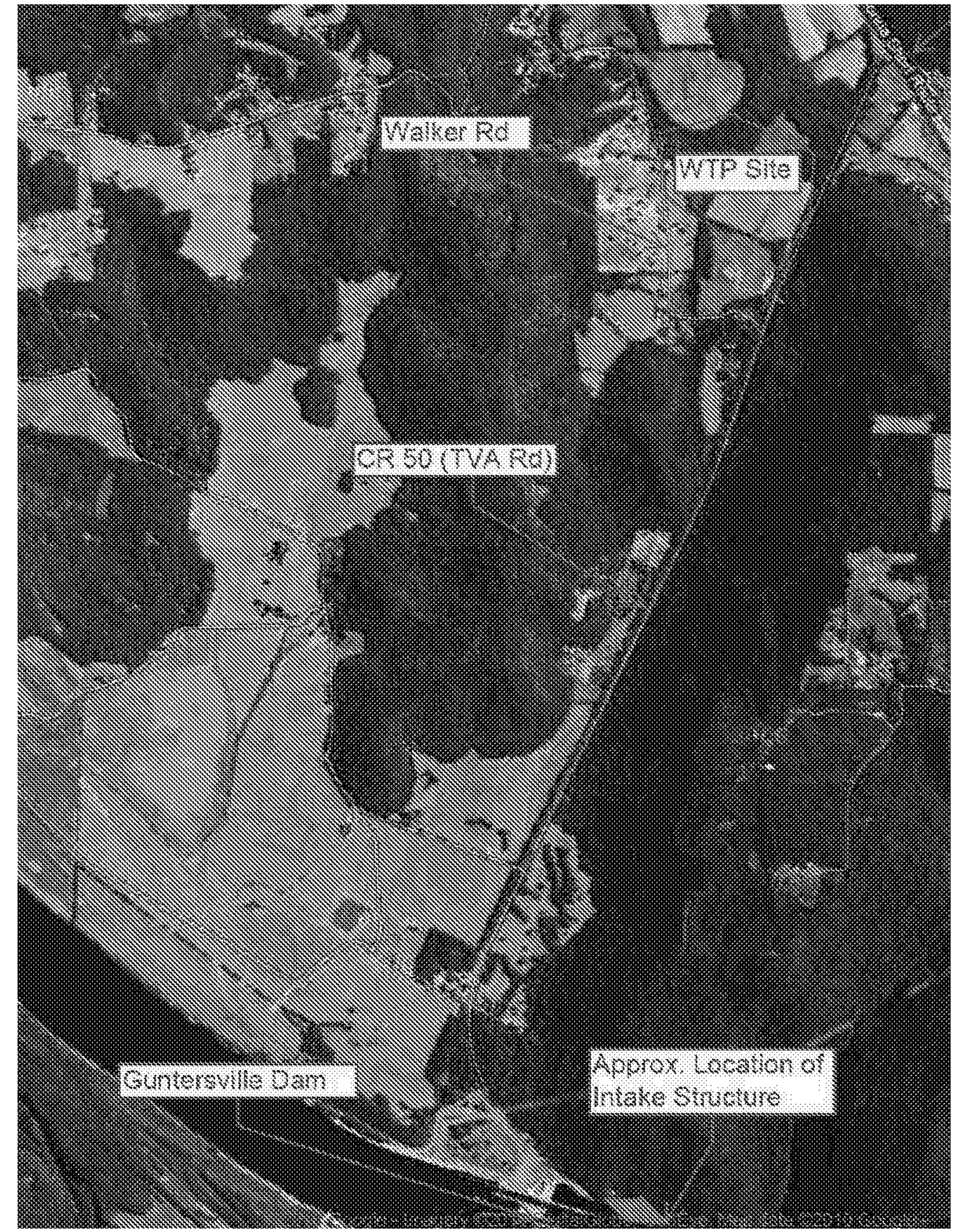
This report has been prepared in accordance with generally accepted geotechnical engineering practices. No other warranty, expressed or implied, is made.

End of Report

LOCATION MAPS





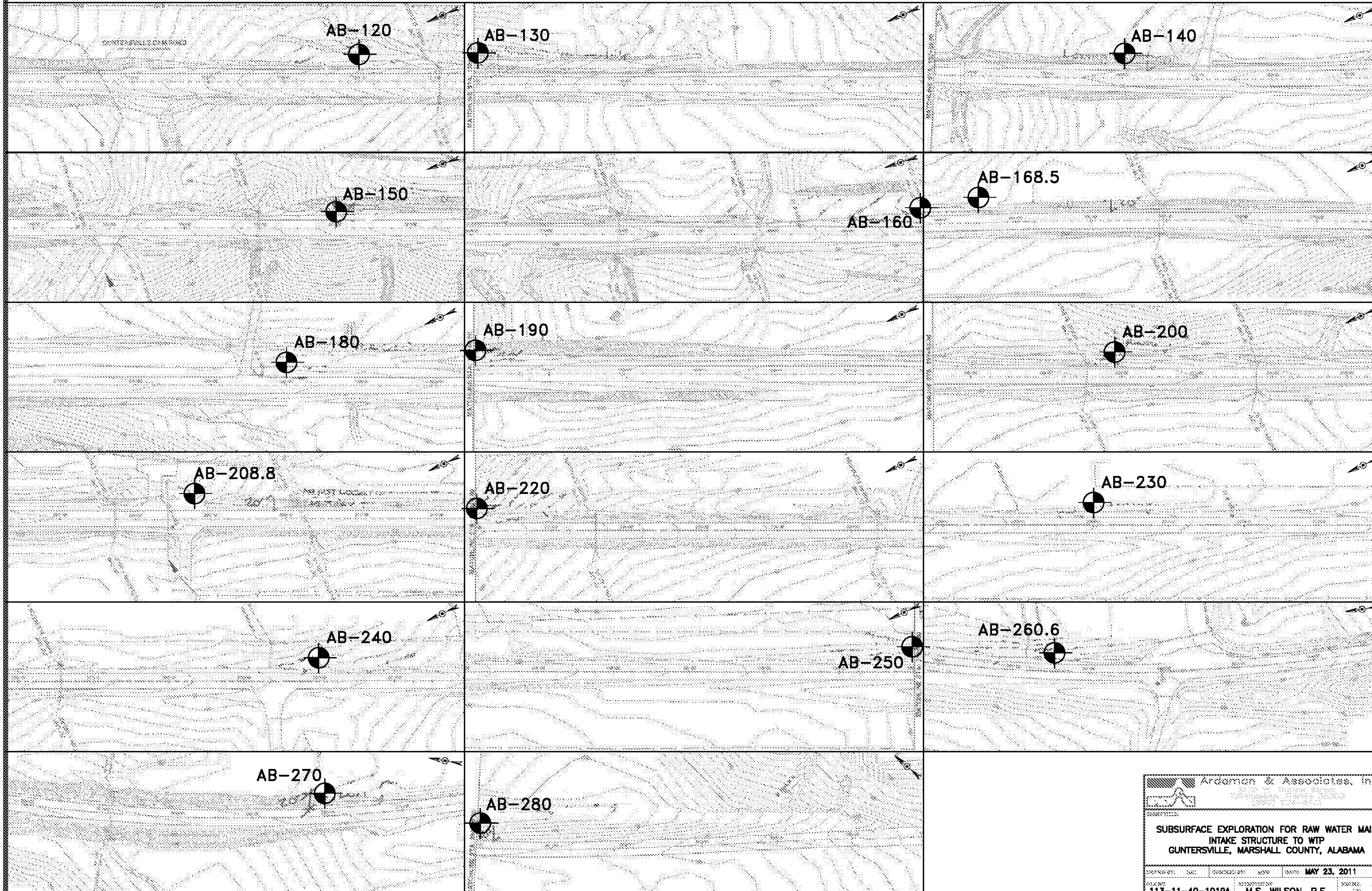


Ardaman & Associates, Inc.

SUBSURFACE EXPLORATION LOCATION MAPS
INTAKE STRUCTURE TO WTP
GUNTERSVILLE, MARSHALL COUNTY, ALABAMA

		MAY 23, 2011
113-11-40-1019A	M.S. WILSON, P.E.	1

TEST BORING LOCATION MAPS



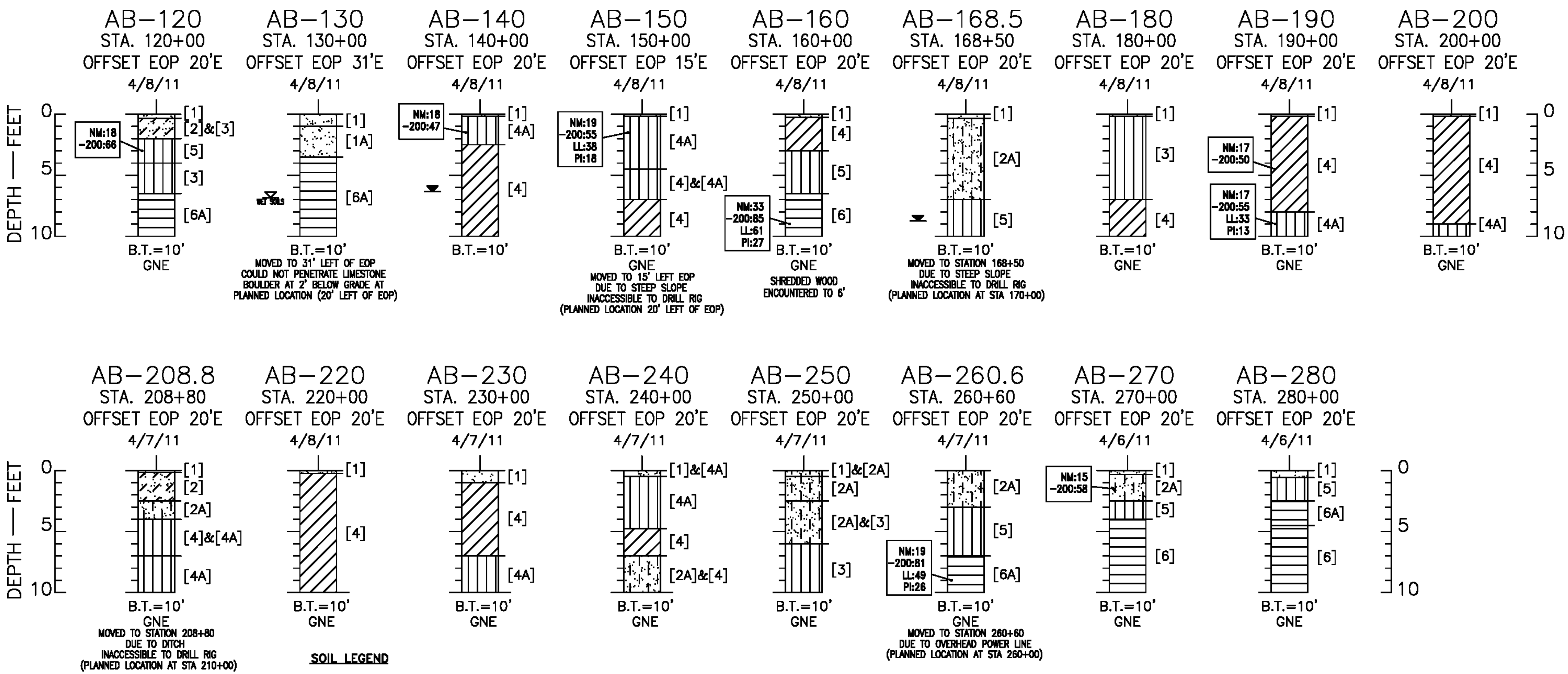
Ardaman & Associates, Inc.

SUBSURFACE EXPLORATION FOR RAW WATER MAIN
INTAKE STRUCTURE TO WTP
GUNTERSVILLE, MARSHALL COUNTY, ALABAMA

MAY 23, 2011

113-11-40-1019A | M.S. WILSON, P.E. | 2

SOIL BORING PROFILES



SOIL LEGEND

- (1) DARK BROWN SILTY FINE SAND W/ TRACES OF GRASS AND SURFICIAL ROOTS AND TRACE ORGANICS; TOPSOIL (SM; A–2–4)
- (1A) BROWN MEDIUM TO FINE SAND W/ SILT AND CLAY AND OCC. LIMESTONE GRAVEL PIECES TO 1-INCH MAX (SP–SC; A–2–4)
- (2) BROWN AND LIGHT BROWN SILTY, CLAYEY SAND (SM–SC; A–2–4)
- (2A) YELLOWISH–BROWN VERY SANDY, CLAYEY SILT TO VERY SANDY, SILTY LEAN CLAY (ML TO CL; A–4 TO A–6)
- (3) BROWN AND LIGHT BROWN SILTY LEAN CLAY W/ SAND AND W/ INCLUSIONS OF FAT CLAY AND OCC. SANDSTONE AND LIMESTONE FRAGMENTS TO 1/2–INCH MAX (CL W/ CH; A–6 W/ A–7)
- (4) REDDISH–BROWN TO BROWN SILTY, VERY CLAYEY FINE SAND TO SANDY LEAN CLAY W/ OCC. INCLUSIONS OF SANDY FAT CLAY (SC TO CL W/ CH; A–6 W/ A–7)
- (4A) REDDISH–BROWN SILTY, VERY CLAYEY SAND TO VERY SANDY, SILTY LEAN CLAY (SC TO CL; A–6)
- (5) MARBLED LIGHT YELLOWISH–BROWN AND RED SILTY SANDY LEAN CLAY W/ INCLUSIONS OF SANDY FAT CLAY (CL W/ CH; A–6 W/ A–7)
- (6) MARBLED LIGHT GRAY, TAN, AND RED CLAYEY ELASTIC SILT TO FAT CLAY W/ OCC. SANDSTONE GRAVEL TO 1/4–INCH MAX (MH TO CH; A–7)
- (6A) MARBLED LIGHT BROWN AND GRAY SILTY LEAN CLAY TO FAT CLAY W/ SAND (CL TO CH; A–7)

LEGEND

AB	APPROXIMATE LOCATION OF AUGER BORING
GNE	GROUNDWATER NOT ENCOUNTERED ON DATE DRILLED
▼	GROUNDWATER DEPTH ESTIMATED ON DATE DRILLED
▽	POSSIBLE PERCHED WATER (WET SOILS)
NM	NATURAL MOISTURE CONTENT IN PERCENT (AASHTO T–265)
–200	PERCENT PASSING NO. 200 SIEVE SIZE (PERCENT FINES)(ASHTO T–11)
LL	LIQUID LIMIT (AASHTO T–89,90)
PI	PLASTICITY INDEX (AASHTO T–89, 90)
SP–SM,SM,SC	UNIFIED SOIL CLASSIFICATION SYSTEM
A–3,A–2–4	AASHTO SOIL CLASSIFICATION SYSTEM
DRILLERS:	KM, JDA

DRILLING METHODS: CME–55 DRILL RIG WITH 4–INCH DIAMETER FLIGHT AUGERS OR 3–INCH DIAMETER MANUAL "BUCKET" AUGERS

WHILE THE BORINGS ARE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT THEIR RESPECTIVE LOCATIONS & FOR THEIR RESPECTIVE VERTICAL REACHES, LOCAL VARIATIONS CHARACTERISTIC OF THE SUBSURFACE MATERIALS OF THE REGION ARE ANTICIPATED & MAY BE ENCOUNTERED. THE BORING LOGS & RELATED INFORMATION ARE BASED ON THE DRILLER'S LOGS & VISUAL EXAMINATION OF SELECTED SAMPLES IN THE LABORATORY. THE DELINEATION BETWEEN SOIL TYPES SHOWN ON THE LOGS IS APPROXIMATE & THE DESCRIPTIONS REPRESENT OUR INTERPRETATION OF SUBSURFACE CONDITIONS AT THE DESIGNATED BORING LOCATIONS ON THE PARTICULAR DATE DRILLED.

GROUNDWATER ELEVATIONS SHOWN ON THE BORING LOGS REPRESENT GROUNDWATER SURFACES ENCOUNTERED ON THE DATES SHOWN. FLUCTUATIONS IN WATER TABLE LEVELS SHOULD BE ANTICIPATED THROUGHOUT THE YEAR. ABSENCE OF GROUNDWATER DATA ON CERTAIN BORINGS IMPLIES THAT NO DATA IS AVAILABLE, BUT DOES NOT NECESSARILY MEAN THAT GROUNDWATER WILL NOT BE ENCOUNTERED AT THESE LOCATIONS OR WITHIN THE VERTICAL REACHES OF THESE BORINGS IN THE FUTURE.

Ardaman & Associates, Inc.

SUBSURFACE EXPLORATION FOR RAW WATER MAIN
INTAKE STRUCTURE TO WTP
GUNTERSVILLE, MARSHALL COUNTY, ALABAMA

MAY 23, 2011

113–11–40–1019A | M.S. WILSON, P.E. | 3

OMI, Inc.

SUBSURFACE EXPLORATION AND
GEOTECHNICAL ENGINEERING STUDY
Proposed
Southeast Water Treatment Plant
Highway 431
Marshall County, Alabama

OMI Job No. 6644

September 18, 2013

5151 Research Drive, N.W., Suite A   Huntsville, AL 35805   Tel: (256) 837-7664   Fax: (256) 837-7677   E-mail: omi@omi-eng.com

OMI, Inc.

September 18, 2013

Tetra Tech, Inc.
2110 Powers Ferry Road, Suite 202
Atlanta, GA 30339

ATTN: Mr. Michael Schmidt

SUBJECT: Report of Geotechnical Engineering Study
Proposed Southeast Water Treatment Plant
Highway 431
Marshall County, Alabama
OMI Job No. 6644

Gentlemen:

OMI, Inc., has completed a subsurface exploration and geotechnical engineering study for the referenced project. Enclosed is the report of the findings as well as recommendations for foundation design and construction, site preparation, and other geotechnically related site activities. This work was authorized on April 24, 2013 by Mr. Christian Dunway of Tetra Tech, Inc..

OMI, Inc., appreciates the opportunity to be of service to Tetra Tech, Inc. and looks forward to continued involvement with the construction monitoring phase of this project. Please direct any questions concerning this report to the undersigned.

Respectfully submitted,
OMI, Inc.

Christopher S. Jones, E.I.
Staff Engineer

John M. Ozier, P.E.
Senior Engineer

Distribution: 3 Copies to Addressee

3151 Research Drive N.W., Suite A Huntsville, AL 35805 Tel: (256) 837-7664 Fax: (256) 837-7677 E-mail: omi@omi-eng.com

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APPENDICES

1.0 EXECUTIVE SUMMARY

OMI has completed subsurface studies for the proposed Southeast Water Treatment Plant. The elevated washwater tank is estimated to settle about 2 to 2.5-in if shallow foundations are utilized. Settlement of the remaining structures is expected to be less than 1-in. The onsite soils are suitable for support of the proposed structures on shallow foundations provided the specific recommendations and comments within this report are followed and understood. Based on information provided by Tetra Tech, varying amounts of cut and fill are planned at different locations across the site. Based on the soil borings performed during this study, the onsite soils may be used as fill; however, the deeper soils at the site are highly plastic and should only be used beneath the proposed sludge drying beds. Soft, unsuitable soils were encountered in the upper 3-ft at the proposed carbon contactors, sludge thickener and recycle pump station, and elevated water tank locations. These soils should be undercut prior to placement of structural fill. The proposed FFE of the finished water pump station and washwater recovery basin are near auger refusal depths. Based on the borings drilled during this study, it does not appear rock excavation will be required; however, should it be required, OMI anticipates the rock will consist of argillaceous limestone or shale that may be excavated with typical excavation equipment such as a trackhoe.

The excavation and construction of the proposed structures should be relatively straight forward; however, groundwater control will be required. Groundwater should be kept from the excavations as described within this report. In addition, underdrain systems or other methods of uplift resistance should be installed beneath the washwater recovery basin, the finished water storage basin/pump station, and the flocculation and sedimentation basin to mitigate bouyant forces on these structures for future maintenance. Specific recommendations for foundation design and site earthwork are given in the body of this report.

2.0 INTRODUCTION

OMI, Inc., has completed a design geotechnical engineering study for the proposed Southeast Water Treatment Plant. This report outlines the scope of services provided and presents comments and recommendations based on professional opinions formed during the course of this study. This work was authorized on April 24, 2013, by Mr. Christian Dunway of Tetra Tech, Inc.. The work was performed in general accordance with OMI Proposal No. P-4035A. Additional studies were authorized on May 31, 2013.

Assessment of the environmental aspects of this site, including previous land use or the determination of the presence of any chemical, industrial, or hazardous waste is beyond the scope of this study. However, OMI can provide these services if desired.

3.0 EXPLORATION METHODS

The procedures used by OMI for field and laboratory testing are in general accordance with ASTM procedures and established engineering practice. Brief descriptions of the procedures used in this exploration are contained in the Appendix of this report.

Thirty-two soil test borings to varying depths were performed during this study. In addition, rock coring was performed at two locations. Boring locations are shown on the appended Boring Location Plan. A member of the OMI professional staff directed the drilling and logged the soils in the field during excavation. Subsequently, each sample was sealed and transported to the office. Selected samples were tested to determine the natural moisture content and Atterberg limits of the soil. These tests assist in confirming the visual classifications as well as provide an index of certain engineering properties. The soil classifications, field testing data, and the results of the laboratory tests are provided on the Soil Boring Records in the Appendix of this report. A consolidation test was performed on an undisturbed sample from boring B-13 beneath the planned elevated washwater tank. This test allows OMI to calculate the settlement of the tank based on the soil properties.

4.0 SITE CONDITIONS

The site for the proposed Southeast Water Treatment Plant is northwest of County Road 50 between Walker Road and Highway 431 in Marshall County, AL. Vegetation across the site consists of wooded areas and open fields. Topographically, portions of the site proposed for development vary from steeply sloping to rolling topography and range in elevation from approximately 650-ft MSL near the northwestern corner of the site to 585-ft MSL near the northeastern corner of the site. A large tree line extends in a northwestern direction across the northwestern portion of the site and elevations within this area generally drop from about 645-ft MSL along the western edge to 605-ft MSL along the eastern edge. Surface drainage is generally directed to the northeast across the site; however, a small drainage channel located within a thin tree line at the base of the steeply sloping portion of the site collects surface water and directs flow to the north/northwest. This drainage channel is generally located between borings B-5 and B-13 shown on the attached boring location plan. A second larger drainage feature shown as a blue line tributary on the USGS Guntersville Dam Quadrangle Alabama, 7.5 Minute Topographical Map, photo-revised 1970, crosses the southwestern portion of the site proposed for construction.

5.0 SUBSURFACE CONDITIONS

5.1 Flocculation/Sedimentation Basin and Control/Chemical Buildings

Borings B-1A, B-1B, B-2, B-9A, B-9B, and B-15 were drilled on the western portion of the site and encountered similar conditions. Low plastic, stiff, sandy silty clays were encountered below the topsoil to about 1.5-ft to 3-ft. Standard Penetration Tests (SPTs) within this layer ranged from 7 to 14 blows per foot (bpf) with an average of about 12 bpf. Pocket penetrometer values within this layer ranged from 1.5 to 2 tons per square foot (tsf) with an average of about 2 tsf. Low plastic, stiff to very stiff, sandy silty clays were encountered below this layer to auger refusal or boring termination in borings B-2, B-9A, B-9B, and B-15 and to about 30-ft in borings B-1A and B-1B. SPT values within this layer ranged from 14 to 34 bpf with an average of about 23 bpf. Pocket penetrometer values ranged from 1.5 to 4 tsf with an average of about 3 tsf. A sample was collected from B-1B within this layer at 25-ft for an Atterberg Limits test. The results of the test showed a

Liquid Limit (LL) of 43 and Plastic Limit (PL) of 18. Below this layer in borings B-1A and B-1B, high plastic, very stiff to hard, sandy silty clays were encountered. SPT values within this layer ranged from 18 to 32 bpf with an average of about 28 bpf and pocket penetrometer values ranged from 1.5 to 4.5 tsf with an average of about 3 tsf.

Extended groundwater measurements at the time of the study indicated the groundwater table ranged in elevation from 606-ft MSL to 608-ft MSL in borings B-1A, B-1B, and B-2. Borings B-9A, B-9B, and B-15 were collapsed and dry to elevations of 623-ft, 618-ft, and 613-ft, respectively. It is noted that review of the soil samples collected from the borings B-1A and B-1B indicated the seasonal high water table may be near elevation 620-ft.

5.2 Sludge Thickener, Thickened Sludge Recycle Pump Station, Carbon Contactors, and Elevated Water Tank

Borings, B-3A, B-3B, B-4, B-5, B-13A, and B-13 were drilled on the western central portion of the site proposed for development, adjacent to or near the previously mentioned surface water drainage channel directing flow to the north through a thin tree line. The borings encountered low plastic, soft to firm, sandy silty clays below the topsoil to depths ranging from 1.5 to 3-ft below the ground surface. SPT values within this layer ranged from 3 to 6 bpf with an average of about 5 bpf and pocket penetrometer values ranged from 0 to 1.5 tsf with an average of about 0.5 tsf. Low plastic, stiff to very stiff, sandy silty clays were encountered below this layer to depths ranging from 1.5 to 5-ft. SPT values within this layer ranged from 9 to 15 bpf with an average of about 12 bpf and pocket penetrometer values ranged from 1 to 2.5 tsf with an average of about 2 tsf. An undisturbed sample was collected from this layer from boring B-13 for consolidation analysis. Low plastic, very stiff, sandy silty clays were encountered below this layer to depths ranging from 8.5 to 13.5-ft in borings B-4, B-5, B-13 and B-13A and 18.5-ft in borings B-3A and B-3B. SPT values within this layer ranged from 16 to 33 bpf with an average of about 23 bpf and pocket penetrometer values ranged from 2.5 to 4.5 tsf with an average of about 3.5 tsf. An undisturbed sample was collected from B-13A within this layer for a consolidation test and Atterberg Limits test. The results of the Atterberg Limits test showed a LL of 27 and PL of 13. High plastic, stiff to very stiff, sandy silty clays were encountered below this layer to boring termination or auger refusal in borings B-4, B-5, B-13A, and B-3A and to 13.5 ft in Boring B-13 and to 28.5-ft in Boring B-3B. SPT values within this layer ranged from 9 to 22 bpf with an average of 15 bpf and pocket penetrometer values ranged from 1 to 3 tsf with an average of about 2 tsf. Soft, high plastic, sandy silty clays were encountered below this layer to auger refusal in borings B-13 and B-3B. SPT values within this layer were 4 and 1 bpf. Pocket penetrometer values were 0.5 and 0 tsf.

OMI recommended and performed coring at boring B-13A due to marginal soils encountered in boring B-13 and the anticipated loads associated with the elevated water tank. Argillaceous limestone with layered clay seams was encountered in the upper 4-ft to 4.5-ft of the core. Calcareous shale layered with clay seams was encountered below the argillaceous limestone to boring termination. Sample recovery ranged from 72 to 83 percent. Rock Quality Design ranged from 30 to 33 percent in the upper 8-ft to 8.5-ft and was 0 percent in the lower 3-ft.

Extended groundwater measurements indicated the groundwater table ranged in elevation form 595-ft MSL to 599-ft MSL within this area of the site at the time of the study.

5.3 Washwater Recovery Basin, Finished Water Storage Basin, and Generator Building

Borings B-6A, B-6B, B-7, B-7A, B-8, B-8A, B-12, and B-14 were drilled on the eastern portion of the site proposed for development. Below the topsoil, low plastic, firm to stiff, sandy silty clays were encountered to depths ranging from 1.5 to 3-ft. SPT values within this layer ranged from 6 to 14 bpf with an average of about 10 bpf and pocket penetrometer values ranged from 0.5 to 3 tsf with an average of about 1.5 tsf. Low plastic, very stiff, sandy silty clays were encountered below this layer to about 13-ft. SPT values within this layer ranged from 16 to 24 bpf with an average of about 19 bpf and pocket penetrometer values ranged from 1.5 to 4.5 tsf with an average of about 3 tsf. High plastic, stiff to very stiff, sandy silty clays were encountered below 13-ft and extended to auger refusal in borings B-6A, B-6B, B-8, and B-14. SPT values within this layer ranged from 13 to 24 bpf with an average of about 17 bpf and pocket penetrometer values ranged from 1.5 to 4.5 tsf with an average of about 3.5 tsf. Samples were collected from B-6A and boring B-7 within this layer for Atterberg Limits tests. The results of the test showed LL of 86 and 101 and PL of 29 and 32, respectively. High plastic, soft to firm, sandy silty clays were encountered below this layer to auger refusal in borings B-7, B-7A, and B-8A. SPT values within this layer ranged from 4 to 9 bpf with an average of about 6 bpf and pocket penetrometer values ranged from 0 to 1.5 tsf with an average of about 0.5 tsf.

Due to the depth of the proposed finished water recovery basin and relatively shallow auger refusal, OMI recommended and performed 10-ft of coring at boring B-8A. Approximately 2-ft of Argillaceous limestone was encountered above about 8-ft of calcareous shale with clay seams. Sample recovery ranged from 50 to 78 percent and Rock Quality Design was 71 percent in the upper 4.5-ft and 0 percent in the lower 5.5-ft.

Extended groundwater measurements indicated the groundwater table elevation was approximately 597-ft MSL near borings B-6A, B-6B, and B-8A. The groundwater table in borings B-7, B-7A, and B-8 ranged in elevation from approximately 586-ft MSL to 592-ft MSL.

5.4 Sludge Drying Beds and Pump Station

Borings B-10A through B-10F and boring B-11 were drilled on the southern portion of the site proposed for development. The borings encountered low plastic, firm to stiff, sandy silty clays to depths ranging from 1.5 to 3-ft. SPT values within this layer ranged from 6 to 17 bpf with an average of about 10 bpf and pocket penetrometer values ranged from 1 to 2 tsf with an average of about 1.5 tsf. Low plastic, stiff to very stiff sandy silty clays were encountered below this layer to boring termination at 10-ft in borings B-10A through B-10F and to 18.5 ft in boring B-11. SPT values within this layer ranged from 14 to 32 bpf with an average of about 23 bpf and pocket penetrometer values ranged from 2 to 4.5 tsf with an average of about 3.5 tsf. High plastic, very stiff, sandy silty clays were encountered below this layer in boring B-11 to 28-ft. SPT values within this layer were 24 and 29 bpf and pocket penetrometer values were 3.5 tsf. High plastic, firm, sandy silty clays with an SPT value of 8 and a pocket penetrometer value of 0.5 were encountered below this layer to auger refusal at 32-ft.

At the time of the study, groundwater was encountered at 3-ft in borings B-10A and B-10B; however, borings B-10C through B-10F were dry to depths ranging from 3 to 7-ft. Boring B-11 was dry to 13-ft.

5.5 Access Drive

Borings B-17A and B-17B, drilled in the proposed access drive, encountered varying conditions below the topsoil. B-17A encountered low plastic, very stiff to hard, sandy silty clays with chert to boring termination at 10-ft. SPT values ranged from 20 to 100 plus bpf with an average of about 25 bpf and pocket penetrometer values ranged from 2.5 to 4 tsf with an average of about 3.5 tsf. Boring B-17B encountered low plastic, soft to stiff, sandy silty clays to about 3-ft. SPT values were 4 and 12 bpf and pocket penetrometer values were 1 and 2.5 tsf. Low plastic, very stiff, sandy silty clays were encountered below this layer to boring termination at 10-ft. SPT values ranged from 17 to 25 bpf with an average of about 21 bpf and pocket penetrometer values were 3 tsf. No groundwater was encountered within these borings.

Because of the geology of this region, the groundwater levels are generally a function of seasonal precipitation and locally heavy rainfall events. Consequently, the groundwater levels can and do fluctuate with time. Review of published information indicates variations from 5-ft to 10-ft in the ground water table elevation would not be uncommon for Marshall County, AL. Further, data collected in Marshall County shows much greater fluctuations in the seasonal groundwater table elevation can and do occur. In addition, OMI expects the proposed mass site grading may have unknown impacts to the groundwater table elevation at the site.

6.0 SITE GEOLOGY

Published geologic information indicates the proposed site is underlain by the Monteagle Limestone.

In Alabama, the name Monteagle replaces the names St. Genevieve and Gasper Limestone. The Monteagle Limestone ranges in thickness from 200 to 250-ft and is characterized by light gray oolitic limestone in crossbedded, massive beds. Near the top of the formation, the limestone is thinly bedded and separated by beds of shale ranging from 2-in to 5-ft thick.

Sinkhole Activity

Sinkholes have occurred in this formation within the vicinity of this site. However, surface observations and the subsurface exploration did not disclose evidence of sinkhole activity on this site. This exploration does not, nor was it intended to, address the possibility of future sinkhole development.

7.0 PROJECT INFORMATION

OMI understands that the planned construction will consist of a water treatment plant including several structures discussed below. Rough structure dimensions, including slab thicknesses and bearing elevations were provided for some structures by Tetra Tech. OMI understands these dimensions are preliminary and may change during the design process. No changes are anticipated in OMI's recommendations, should the thicknesses or bearing elevations vary a couple of feet during the design process.

7.1 Flocculation/Sedimentation Basin and Filters

Approximate Elevations (ft MSL)

Structure	Existing Ground Surface	Planned Ground Surface	Top of Wall	FFE Bottom of Walls and Bottom of Basin
Flocculation and Sedimentation Basin	639 to 648	623 to 634	639	621
Filters	639 to 642	623 to 625	639	624 Filter Gallery =623

The proposed flocculation and sedimentation basin will be located on the western portion of the site. The proposed structure roughly measures 212-ft by 113-ft and is 19-ft deep. OMI anticipates the structure will be constructed with cast in place concrete. Rough estimates provided by Tetra Tech indicate the structure will consist of 2 to 2.5-ft thick walls supported by 2 to 3-ft monolithic footings and a 8 to 12-in thick mat slab. No structural loading information was provided; however, OMI anticipates wall loads will range from 5 to 8 kips per linear foot and the soil pressures beneath the slab will be less than 1800 psf. The water will be about 18-ft deep.

Filters will be located on the south side of the basin and will measure approximately 88-ft by 140-ft and will be about 15.5-ft in depth. OMI understands the proposed FFE of the structure will be 623-ft MSL and the FFE of the filter gallery will be 624-ft MSL. OMI anticipates wall loads will range from 4 to 6 kips per linear foot and soil pressures beneath the slab will be less than 1500 psf. The water will be about 15-ft deep.

7.2 Control/Chemical Building and Blower Room

Approximate Elevations (ft MSL)

Structure	Existing Ground Surface	Planned Ground Surface	FFE of Structure
Control Room	623 to 630	622 to 623	623
Chemical Building and Blower Room	623 to 630	622 to 623	623

The control/chemical building and blower room will be located east of and adjacent to the flocculation/sedimentation basin and filters. OMI understands the structure will consist of two sections including the operations building and chemical feed building. The structure will roughly measure 236-ft by 62-ft. The operations portion of the structure will be two-stories and consist of offices, laboratory, control room, and other non process areas. The chemical feed and blower room portion of the structure will be one story and will contain chemical storage tanks and related feed equipment. Both structures will be slab on grade with exterior walls consisting of reinforced concrete masonry units (CMU) with spray foam insulation and brick veneer. OMI anticipates wall loads will range from 5 to 7 kips per linear foot for the operations building and 3 to 4 kips per linear foot for the chemical feed portion. Anticipated floor loads will range from 100 to 200 psf. Foundations for the Control Chemical Building will tie into the walls of the Flocculation/Sedimentation Basin and Filters.

7.3 Sludge Thickener

Approximate Elevations (ft MSL)

Existing Ground Surface	Planned Ground Surface	Top of Wall	FFE Bottom of Walls	FFE Center of Structure
598 to 610	611 to 620	623.50	609.5	601

The proposed sludge thickener will be approximately 74-ft in diameter and constructed of cast in place concrete. OMI anticipates wall loads will range from 2 to 4 kips per linear foot and soil pressures beneath the slab will be less than 1800 psf. The water will be about 22-ft deep.

7.4 Carbon Contactors Building

Approximate Elevations (ft MSL)

Existing Ground Surface	Planned Ground Surface	FFE
600	608	609 Pipe Gallery=608

The carbon contactors will be housed in a structure measuring approximately 140-ft by 80-ft and will be 21-ft tall. OMI anticipates the structure will be of CMU construction and a slab on grade. OMI anticipates wall loads will range in from 2 to 4 kips per linear foot and floor loads will be less than 300 psf.

7.5 Thickened Sludge/Recycle Pump Station

Approximate Elevations

Existing Ground Surface	Planned Ground Surface	FFE of Structure
596 to 598	608	608