WAUFORD
J. R. Wauford & Company, Consulting Engineers, Inc.

ADDENDUM NO. 1
WATER SYSTEM IMPROVEMENTS
CONTRACT 18-02
INDUSTRIAL PARK 500,000 GALLON ELEVATED WATER STORAGE TANK
DAYTON, TENNESSEE
WAUFORD PROJECT NO. 4630

Date of Addendum: Friday, March 8, 2019

1. Geotechnical Report:

   Add the attached Geotechnical Report to the End of Section 1 of the Specifications.

J. R. WAUFORD & COMPANY,
CONSULTING ENGINEERS, INC.

Scott B. Carroll

Scott B. Carroll, P.E.
Tennessee License # 116799

Subsurface Exploration

The City of Dayton, Tennessee
Dayton Industrial Park
500,000 Gallon Elevated Water Storage Tank
Dayton, Tennessee

02/25/2019

FSE Project No. 318194

Prepared For: The City of Dayton, TN – c/o Mr. Scott Carroll, P.E.

Foundation Systems Engineering, P.C.
Geotechnical Engineering and Consulting

February 25, 2019

The City of Dayton, Tennessee
c/o Mr. Scott Carroll, P.E.
J. R. Wauford & Company
908 West Broadway Avenue
Maryville, TN 37801

Email: scottc@jrwauford.com

RE: Subsurface Exploration
500,000 Gallon Elevated Water Storage Tank
Dayton Industrial Park
Dayton, Tennessee
FSE Project No. 318194

Dear Mr. Carroll:

We have completed a subsurface exploration for the above-referenced project. Our services were performed in general accordance with our proposal dated July 18, 2018. Following are the results of our exploration.

If you have any questions concerning the data obtained, or if we may be of further service, please feel free to call us. It has been a pleasure to be of service to you on this project

Sincerely,
Foundation Systems Engineering, P.C.

Eric M. Peterson, P.E.	Jack F. Llewellyn, Jr., P.E.
Senior Geotechnical Engineer	President/Principle Engineer
Tennessee No. 109536	Tennessee No. 19187

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1.0 EXECUTIVE SUMMARY

The City of Dayton Tennessee proposed new 500,000 gallon, elevated, water storage tank is to be located at the northwestern terminus of Greenway Boulevard, in the Dayton Industrial Park. The proposed new tank is to be located at the crest of a northeast to southwest tending ridge. There is currently no direct site access. From Greenway Boulevard to the tank site the hillside up to the proposed tank site is relatively steep and is overgrown in mature trees and some brush. The hillside is gullied, and a front-end loader was required to clear an access trail and pull the drilling rig to the tank site.

The tank site is located approximately 4,100 feet (airline) to the northwest from the intersection of Rhea County Highway (US 27/SR 29) and Manufacturers Road. *Rhea of Sunshine, Inc.* is located at the northwest end of Greenway Boulevard, adjacent to the subject hillside.

J. R. Wauford & Company, Consulting Engineers, Inc. (project engineers) is preparing plans and specifications for the proposed new 500,000-gallon multi-column, elevated water storage tank. Boring locations were selected, and the tank center stake was established in the field by the project engineers. The purpose of the geotechnical engineering report was to provide geotechnical engineering recommendations for the proposed new water storage tank foundations.

To meet this objective, the following field and laboratory testing were performed:

- Four (4) test pits were excavated at the proposed tank foundation locations using a PC 200 Komatsu Trackhoe. The test pits were excavated to refusal.
- Five (5) soil test borings with Standard Penetration testing were used to investigate the subsurface conditions at the proposed new tank foundations. The borings were placed using a CME 55 Truck Mounted Drill Rig. The borings were drilled to soil auger refusal.
- NX-sized rock coring was performed at four (4) boring locations at the tank site. The rock was core drilled to a depth of 10 feet into the bedrock at the core boring locations.

Based on the results of our site reconnaissance, review of topographic and geologic mapping, and the field and laboratory testing, we offer the following summary of findings:

- The proposed new elevated tank is to be located at the northwestern terminus of Greenway Boulevard, in the Dayton Industrial Park. The new tank is to be located at the crest of a northeast to southwest tending ridge.
- Mapping provided indicates that the area surrounding the tank slopes downward in elevation to the northeast, southeast, and northwest, and upward in elevation to the southwest. Existing ground surface elevations at the boring locations varied from approximately 933.4 feet above MSL to 928 feet above MSL. The proposed ground surface elevation at the tank is 924 feet above MSL.
- A thin veneer of topsoil and roots (approximately 6 to 8 inches) was encountered at existing grade. Residual soil and very soft rock were encountered beneath the topsoil and root veneer. The residual soils and very soft rock were encountered to depths of from approximately 1 foot to 2 feet below existing grade. The residual soils and very soft rock transitioned quickly to weathered to partially weathered, soft to hard, rock. Weathered to partially weathered rock was typically encountered above the proposed ground elevation of 924 feet above MSL.
- Published mapping indicates that the site is underlain by three geologic formations. These are the Fort Payne Chert, Chattanooga Shale and Rockwood Formation. The Fort Payne Chert is mapped to underlie the approximate northern half of the site, the Rockwood Formation the approximate southern half, with a thin band of Chattanooga Shale across the approximate center of the ridge. We did not identify the Chattanooga Shale across the middle of the site; instead, our borings encountered the Fort Payne Chert beneath the approximate center and northern portion of the tank area.

Based on the results of the subsurface exploration, and our understanding of proposed new construction, we offer the following summary of the design recommendations for the proposed new elevated water storage tank. We recommend that our firm be retained to review the recommendations contained in this report once Plans (Site Grading and Structural) and Specifications are completed. The following summary should not be considered a replacement for the detailed recommendations located within the body of this report.

RECOMMENDATION SUMMARY	
Borings Performed At Tank Site	• Five soil test borings with SPT testing. • Four test pits. • Four rock core borings (10-foot core drilling depth). • The borings are representative of subsurface condition at the borehole location only. The subsurface conditions can vary beyond/between boring locations.

RECOMMENDATION SUMMARY	
Clearing/ Grubbing	• The site should be stripped of all topsoil, roots, and organics. • Care should be taken to remove all stumps from proposed foundation areas. • We expect that excavation of rock will be required to reach proposed ground elevation and bottom of foundation elevation. • Rock should be removed using dozers with rippers or similar, and/or trackhoe and hoe ram or similar. Blasting to remove rock should not be permitted. Care should be taken to remove all overbreak material from beneath the tank foundations, down to undisturbed material. • It is our experience that only very large sized dozers (Cat D-8 or D-9 for example) with rock rippers (not soil scarifiers) can remove material below our auger refusal depth.
Undercutting (Foundation Preparation)	• Tank spread footings should bear on soft to hard rock. • The footings should be extended down into the rock a minimum of 36 inches. • Foundation bearing material is expected to consist of Fort Payne Chert (limestone/dolomite/chert) at borings B-1, B-3, B-4, and B-5. The foundation bearing material at boring B-2 is expected to consist of Shale. • The rock across the bottom of the footing should be leveled. • The rock across the bottom of the footings and footing rock sidewalls should be hand cleaned to remove all loose material. Care should be taken to remove all overbreak material. Any surface water/runoff that accumulates (as well as any silt/clay) should be completely removed prior to placing concrete. Footing reinforcing should be removed to facilitate cleaning if required, and then be replaced. • Footing concrete should be cast directly against the clean bedrock. • Rock dowels/anchors may be used, embedded into the Fort Payne Chert or Rockwood/Chattanooga shale formations. Design values for the formations are not the same. Specific design data can be provided if dowels/anchors are utilized. • Footings should be poured with concrete as soon after excavation as possible.
Fill Compaction	• Soil Fill – Compact soil fill to 100% Standard Proctor maximum dry density, ASTM D 698 unless noted otherwise. • Stone Fill (No. 4 or No. 57) – Compact stone fill to 95% Modified Proctor maximum dry density, ASTM D 1557, unless noted otherwise. • Mineral Aggregate Base (TDOT 303D) over top of spread footing – Compact to 100% Standard Proctor maximum dry density. • Rework and recompact soil at undercut level to 98% Standard Proctor maximum dry density, ASTM D 698.
Soil Fill & Stone Fill	• Lean Silty Clay: PI < 30, LL < 50, Standard Proctor maximum dry density > 92 PCF. No organic material, or rock > 4” in greatest dimension. • ASTM C 33 Size No. 4 and Size No. 57 clean, washed, crushed limestone gravel. • Mineral Aggregate Base – TDOT 303 Grading D.

RECOMMENDATION SUMMARY	
Cut and Fill Slopes	• The excavation sidewalls should be sloped or shored for stability. For short term slopes less than 12 feet in height, we recommend that excavation sidewalls in chert and shale rock be sloped no steeper than 3/4H:1V. Temporary construction excavations should be sloped or shored in accordance with local, state and federal regulations including OSHA (29 CFR Part 1926) excavation and trench safety. Excavations should be observed and classified by an OSHA competent person.
Groundwater Management	• No groundwater was encountered in the borings or test pits. We do not anticipate that groundwater will present construction related problems. Surface runoff into the footings should be removed.
Foundation Bearing Material	• Fort Payne Chert (limestone, dolomite, chert ledges) bedrock and Chattanooga Shale/Rockwood Formation shale bedrock.
Foundation Bearing Capacity	• Use spread foundations for design, bearing on weathered bedrock (limestone, dolomite, chert, shale). Allowable bearing capacity 10,000 psf. Estimated total settlement less than 1/2-inch. Estimated differential settlement 1/4-inch or less.
Seismic Considerations	• ASCE/SEI 7-10; Site Soil Class B; Risk Category IV; Seismic Design Category C. • S_S = 0.339 g; S_1 = 0.121 g; S_{MS} = 0.339 g; S_{M1} = 0.121; S_{DS} = 0.226 g; S_{D1} = 0.081 g; Fa = 1; Fv = 1

The above summary provides an overview only and should not be used as a separate document or in place of reading the entire report including the appendices. The summary is not a substitute for the following detailed sections of this report. A complete discussion of finding and recommendations are included in the following sections of this report.

2.0 OBJECTIVE OF SUBSURFACE EXPLORATION

The purpose of our exploration was to provide geotechnical engineering recommendations for the design of tank foundations and to provide the geotechnical data as further outlined in the July 2, 2018, *Tentative Work Scope* at the Dayton Industrial Park – 500,000 Gallon Elevated Water Storage Tank site located in Dayton, Tennessee. To meet our objective, we placed five soil test borings and four test pits at the tank site. The borings were drilled to auger refusal. NX sized rock core drilling was performed at four locations to determine the quality of the underlying bedrock unit. Area topographic and geologic mapping and USGS Seismic Design Maps were reviewed.

3.0 SCOPE OF SERVICES

Drilling was performed using a CME 55 drilling rig. The borings were advanced using 8-inch, nominal diameter, Hollow-Stem augers. Standard Penetration testing was performed through the Hollow-Stem augers. Test pits were excavated at the proposed tank foundation locations using a PC 200 Komatsu Trackhoe. NX-sized rock coring was performed at four boring locations. A tank center stake was established in the field by your firm before drilling. FSE notified Tennessee-One-Call for utility location. An FSE geotechnical engineer classified all soil and rock samples collected at the site.

Our soils lab, Construction Materials Laboratory (CML), has demonstrated proficiency for the testing of construction materials and has met the requirements of AASHTO R18 set forth by the AASHTO Highway Subcommittee on Materials. CML received a Certificate of Accreditation from the American Association of State Highway and Transportation Officials AASHTO Accreditation Program.

3.1 Subsurface Exploration

- Five (5) soil test borings with Standard Penetration testing (B-1 through B-5) were used to investigate subsurface conditions at the tank site. The borings were placed at locations as selected by your firm, with our concurrence.

  The borings were drilled to auger refusal. Standard Penetration Testing (SPT) was performed at 2.5-foot intervals to a depth of 10 feet below the existing grade. Thereafter penetration testing was performed at 5-foot intervals. The SPT testing was performed through the Hollow-Stem augers.

NX-sized rock core drilling was performed at boring locations B-1, B-3, B-4, and B-5. The bedrock was core drilled depth of 10 feet below auger refusal. The results of the rock core drilling may be seen in Table II (page 13) of this report.

The approximate locations of the borings may be seen on the attached *Boring Location Plan*. The subsurface stratification encountered at the boring locations may be seen on the attached *Test Boring Records* located in the Appendix. A summary of the soils encountered may be seen in Table I (page 11) of this report.

- Four (4) test pits (TP-1, TP-3, TP-4, and TP-5) were used to investigate subsurface conditions at the corresponding boring locations B-1, B-3, B-4, and B-5. The test pits were excavated using a PC 200 Komatsu Trackhoe. The test pits were excavated to refusal.

- A brief description of the testing performed on this project may be found in the Appendix of this report.

4.0 SITE LOCATION AND CONDITIONS

The City of Dayton Tennessee proposed new 500,000 gallon, elevated, water storage tank is to be located at the northwestern terminus of Greenway Boulevard, in the Dayton Industrial Park. The proposed new tank is to be located at the crest of a northeast to southwest tending ridge. There is currently no direct site access. From Greenway Boulevard to the tank site the hillside up to the proposed tank site is relatively steep and is overgrown in mature trees and some brush. The hillside is gullied, and a front-end loader was required to clear an access trail and pull the drilling rig to the tank site.

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Mapping provided indicates that the area surrounding the tank slopes downward in elevation to the northeast, southeast, and northwest, and upward in elevation to the southwest. Existing ground surface elevations at the boring locations varied from approximately 933.4 feet above MSL to 928 feet above MSL. The proposed ground surface elevation at the tank is 924 feet above MSL.

A thin veneer of topsoil and roots (approximately 6 to 8 inches) was encountered at existing grade. Residual soil and very soft to soft rock were encountered beneath the topsoil and root veneer. The residual soils and very soft rock were encountered to depths of from approximately 1 foot to 2 feet below existing grade. The residual soils and very soft rock transitioned quickly to weathered to partially weathered, soft to hard, rock. Weathered to partially weathered rock was typically encountered above the proposed ground elevation of 924 feet above MSL.

Published mapping indicates that the site is underlain by three geologic formations. These are the Fort Payne Chert, Chattanooga Shale and Rockwood Formation. The Fort Payne Chert is mapped to underlie the approximate northern half of the site, the Rockwood Formation the approximate southern half, with a thin band of Chattanooga Shale across the approximate center of the ridge. We did not identify the Chattanooga Shale across the middle of the site. Instead, our borings encountered the Fort Payne Chert beneath the approximate center and northern portion of the tank area.

5.0 SUBSURFACE STRATIFICATION

Five soil test borings with Standard Penetration testing (B-1 through B-5) and four test pits were used to investigate subsurface conditions at the tank site. The borings were placed utilizing a CME 55 drilling rig. The borings were advanced using 8-inch, nominal diameter, Hollow-Stem augers. Standard Penetration testing was performed through the Hollow-Stem augers. Test pits were excavated at the proposed tank foundation locations using a PC 200 Komatsu Trackhoe. NX-sized rock coring was performed at four boring locations.

Following is a summary of the soils encountered at the boring locations. Additional subsurface details may be seen on the attached *Test Boring Records and Test Pit Records*. Subsurface stratification indicated on the boring log is approximate.

5.1 Soil Overburden

The following table summarizes soil subsurface stratification encountered at the boring and test pit locations. Refer to the boring logs located in the appendix for a soil description and the approximate subsurface stratification encountered. The subsurface stratification indicated on boring, and test pit logs are approximate and were developed by a geotechnical engineer based on his interpretation of the driller's field log, split spoon sampling, and rock cores.

5.2 Table I – Soil Overburden Data Summary

Boring No.	Stratum*	Description*	Origin
B-1 / TP-1	0"- 6"	Topsoil & Roots.	
	6"– 4'	Very soft to hard, dry, weathered to partially weathered, gray and black, chert (Auger/Test Pit Refusal).	Residual
	4'–14' NX Core Drill	Hard, dry, light gray to light olive gray and dark gray, broken, weathered to partially weathered, chert. (Recovery = 10% & RQD = 0%)	Residual
	14'	Core Drilling Terminated.	
B-2	0" - 6"	Topsoil & Roots.	
	6" - 6.5'	Very soft to soft, slightly moist to dry, dark gray, black, brown and gray, partially weathered shale.	Residual
	6.5' - 28.5'	Very soft to soft, slightly moist to dry, black, partially weathered shale.	Residual
	28.5'-33.5'	Very soft, slightly moist, light gray, black and tan, partially weathered shale and siltstone.	Residual
	33.5'-35'	Soft, slightly moist to dry, medium to dark gray and maroon, partially weathered, shale.	Residual
	35'	Auger Refusal.	
B-3 / TP-3	0"-4"	Topsoil & Roots.	
	4"-1.5'	Soft, very moist to wet, reddish tan, sandy, silty clay mixed with chert.	Residual
	1.5'-2'	Very soft to hard, dry, weathered to partially weathered, gray and black, chert (Auger/Test Pit Refusal).	Residual
	2'-7'	Hard, dry, weathered to partially weathered, light gray to light olive gray and dark gray, chert (CME 55 Rotary Cone).	Residual
	7'-12' NX Core Drill	Hard, dry, weathered to partially weathered, gray and black, chert. (Recovery = 13% & RQD = 0%)	Residual
	12' – 15.5' NX Core Drill	Very soft to moderately hard, olive gray, partially weathered siltstone. (Recovery = 96% & RQD = 0%)	Residual

Boring No.	Stratum*	Description*	Origin
	15.5’ – 17’ NX Core Drill	Hard, dry, black chert. (Recovery = 72% & RQD = 58%)	Residual
	17’	Core Drilling Terminated.	
B-4 / TP-4	0”-4”	Topsoil & Roots.	Residual
	4”- 4’	Soft to hard, slightly moist, light gray and black, chert (Auger/Test Pit Refusal).	Residual
	4’-7.5’ NX Core Drill	Hard, dry, weathered to partially weathered, light gray to light olive gray and dark gray, chert. (Recovery = 12% & RQD = 0%)	Residual
	7.5’-12’	Hard, dry, weathered to partially weathered, light gray to dark brown, chert. (Recovery = 13% & RQD = 0%)	Residual
	12’-15.5’	Hard, dry, weathered to partially weathered, light to dark brown, chert. (Recovery = 77% & RQD = 0%)	Residual
	15.5	Core Drilling Terminated.	
B-5 / TP-5	0”-6”	Topsoil & Roots.	
	6”-2’	Very soft to hard, dry, weathered to partially weathered, gray and black, chert (Auger/Test Pit Refusal).	Residual
	2’-4’	Very soft to soft, dry, weathered to partially weathered, light gray to light olive gray and dark gray, chert (CME 55 Rotary Cone).	Residual
	4’-14’ NX Core Drill	Hard, dry, weathered to partially weathered, light gray to light olive gray and dark gray, chert. (Recovery = 13% & RQD = 8%)	Residual
	14’	Core Drilling Terminated	

***Stratum Depths and Descriptions Are Approximate.**

5.3 Bedrock

Borings B-1, B-3, B-4, and B-5, were selected for rock core drilling. An NX sized diamond-studded core bit was used to advance the core drill hole.

Core recovery and rock quality designation were determined for the rock core drilling. Core recovery is the ratio, expressed as a percentage, of the total length of core recovered to the total length of core advanced. The RQD (Rock Quality Designation) is the ratio, expressed as a percentage, of the summed length of core retrieved in segments longer than 4 inches, to the total length of core advance. These values are related to the quality of the drilling and soundness of the rock. In homogenous sound rock a recovery of 100 percent may be expected; in rocks with seams a recovery of about 50 percent is typical; however, in decomposed or seamy rock the recovery may be little or nothing. Rock with an RQD value of 90 percent or more denotes excellent rock; 75 percent to 90 percent, good rock; 50 percent to 75 percent, fair rock; 25 percent to 50 percent, poor rock; below 25 percent very poor rock.

The following table summarizes bedrock stratification encountered at the core borings. Refer to the boring logs located in the appendix for a rock description and the approximate subsurface stratification encountered. The subsurface stratification indicated on boring logs is approximate and was developed by a geotechnical engineer based on his interpretation of the driller’s field log and the recovered rock core. Core recovery and RQD for the rock core may be seen in the following table (Table II).

5.4 Table II – Rock Core-Drilling Subsurface Stratification Summary

Boring Number	Stratum Depth, Ft.*	Description*	Core Recovery - %	Rock Quality Designation - %
B-1	0” – 6”	Topsoil.	N/A	N/A
	6” – 4’	Weathered rock – Auger Drill	N/A	N/A
	4’ – 14’	Hard, gray, weathered chert.	10	0
	14’	Core Drilling Terminated.		
B-3	0” – 4”	Topsoil.	N/A	N/A
	4” – 7’	Very soft to hard, dry, weathered to partially weathered, light gray to light olive gray and dark gray, chert.	N/A	N/A
	7’ – 12’	Hard, dry, weathered to partially weathered, gray and black, chert.	13	0

Boring Number	Stratum Depth, Ft.*	Description*	Core Recovery - %	Rock Quality Designation - %
	12' – 15.5'	Very soft to moderately hard, olive gray, partially weathered siltstone.	96	0
	15.5' – 17'	Hard, dry, black chert.	72	58
	17'	Core Drilling Terminated		
B- 4	0" – 4"	Topsoil & Roots.	N/A	N/A
	4" – 4'	Soft to hard, slightly moist, light gray and black, chert.	N/A	N/A
	4' – 7.5'	Hard, dry, weathered to partially weathered, light gray to light olive gray and dark gray, chert.	12	0
	7.5' – 12'	Hard, dry, weathered to partially weathered, light gray to dark brown, chert.	13	0
	12' – 15.5'	Hard, dry, weathered to partially weathered, light to dark brown, chert.	77	0
	15.5'	Core Drilling Terminated		
B-5	0" – 6"	Topsoil & Roots.	N/A	N/A
	6" – 2'	Very soft to hard, dry, weathered to partially weathered, gray and black, chert.	N/A	N/A
	2' – 4'	Very soft to soft, dry, weathered to partially weathered, light gray to light olive gray and dark gray, chert.	N/A	N/A
	4' – 14'	Hard, dry, weathered to partially weathered, light gray to light olive gray and dark gray, chert.	13	8
	14'	Core Drilling Terminated		

***Stratum depths and Descriptions are approximate. N/A = 0% Recovery & 0% RQD for soil overburden (this material was not core drilled).**

The underlying Fort Payne Chert bedrock was encountered at the core locations. The formation is composed of limestone and dolomite, with chert ledges. Chert was encountered at the core boring locations. The chert is weathered and broken, with recovery and rock quality described as fair to very poor.

Very soft rock disintegrates or easily compresses to touch and can have a high percentage of soil. The unconfined compressive strength of very soft rock ranges from 1 to 2,500 psi. Soft rock is coherent but breaks very easily to thumb pressure at sharp edges and crumbles with firm hand pressure. The unconfined compressive strength of soft rock ranges from 2,500 psi to 8,000 psi. Moderately hard rock can be broken off along sharp edges by considerable hand pressure and can be broken with light hammer blows. Hard rock cannot be broken by thumb pressure but can be broken by moderate hammer blows. The unconfined compressive strength of hard rock ranges from 8,000 to 20,000 psi. The unconfined compressive strength of moderately hard rock is near the lower end of the hard rock unconfined compressive strength scale. Very hard rock can be broken only by heavy hammer blows. The unconfined compressive strength of very hard rock is greater than 20,000 psi.

Upon completion, the borings were backfilled with soil auger cuttings.

5.5 Groundwater

By definition, groundwater is the continuous body of subsurface water that fills the soil, rock voids and fissures and is free to move under the influence of gravity. The water table or phreatic surface is the level of zero (atmospheric) pressure in a continuous body of groundwater. The groundwater level is not a static level surface as the term *water table* implies. Instead, it is the sloping surface of a moving stream of water in the voids and fissures.

No groundwater was encountered in the borings or test pits at the time of drilling/excavation or after a period of 24 hours. We do not anticipate that groundwater will present construction-related problems for this project

A deeper bedrock aquifer exists at some depth into the bedrock below the ground surface. This depth is well below the level of our borings. A study of the deeper bedrock aquifer was beyond the scope of our exploration.

6.0 GEOLOGY

Physiographically the site is located in the Appalachian Ridge and Valley Province. The site is located in a region of folded and faulted rock formations of Cambrian to Pennsylvanian age. The subject site is located on the *Geologic Map of Evensville*

Quadrangle, Tennessee (GM 118-SW). Mapping was performed by George D. Swingle, assisted by Pratt Finlayson, Herbert A. Tiedemann, John W. Jewell, Edward T. Luther, G. A. Boyd, and Ernest E. Russell. The mapping was published in 1964 by George D. Swingle of the *Tennessee Division of Geology* at a map scale of 1:24,000.

Published mapping indicates that the site is underlain by three geologic formations. These are the Fort Payne Chert, Chattanooga Shale and Rockwood Formation. The Fort Payne and Chattanooga Shale formations are of Missippian geologic age, and the Rockwood Formation is of Silurian geologic age. The Fort Payne Chert is mapped to underlie the approximate northern half of the site, the Rockwood Formation the approximate southern half, with a thin band of Chattanooga Shale across the approximate center of the ridge. We did not identify the Chattanooga Shale across the middle of the site; instead, our borings encountered the Fort Payne Chert beneath the approximate center and northern portion of the tank area.

The formations strike from northeast to southwest. Published mapping indicates that the Fort Payne Chert and Chattanooga Shale dip steeply to the northwest and that the Rockwood formation is near vertically bedded.

Rock strength testing performed by the Tennessee Valley Authority indicates compressive strength values of 5,000 to 20,000 PSI for the underlying rock formations, depending upon the degree of weathering. Limestones, dolomites and chert rock is classified as moderately hard to very hard and typically requires blasting, ram hoe or other similar hard rock excavation technique for removal.

7.0 SEISMIC SITE CLASS

The following Seismic Site Class data is based on data published by the USGS Earthquake Hazards Program. The system allows input of desired Building Code; soil classification; and site latitude and longitude, and provides site-specific seismic coefficients based on input values. Latitude and longitude were obtained from *Google Mapping*. Seismic Site Class is based on Table 20.3-1 *Site Classification*.

Site Soil Classification varies from A to F. Site Class A consists of Hard Rock. Site Class B consists of Rock; Site Class C is Very Dense Soil and Hard Rock; Site Class D is Stiff Soil; Site Class E is Soft Clay Soil, and Site Class F are soils requiring seismic response analysis.

Soil site classification is based on the upper 100 feet of material beneath the site. At the Tank site, soil overburden (topsoil and residual soil) was encountered to a maximum depth of approximately 1 to 2 feet below existing grade. Below the soil overburden, very soft to hard chert and shale bedrock was encountered.

In determining Site Class, the engineer may consider seismic shear wave, Standard Penetration Test N values, and soil undrained shear strength. With regard to the subject site and assigned seismic site class, Standard Penetration Test N values, and rock core drilling were considered.

Based on the above discussion, we recommend the following Site Modified Design Spectral Acceleration values be used in determining earthquake loading.

In our professional opinion, the Tank site is classified as soil "Class B" ("Rock"); and seismic design category C. Using the ASCE/SEI 7-10 Table 1.5-2) information, the short and 1.0-second spectral accelerations were determined for ground motion with a 2 percent probability of exceedance in 50 years. The following spectral acceleration requirements indicate the following values.

7.1 Table III – Seismic Values

ASCE/SEI 7-10	0.2 SEC Period Spectral Response	1.0 SEC Period Spectral Response
Horizontal Spectral Accelerations, (g) for Class B Sites	S_s = 0.339 g	S_1 = 0.121 g
Site Coefficients	F_a = 1	F_v = 1
Site Modified Spectral Accelerations, (g) for Class B Sites	S_{MS} = 0.339 g	S_{M1} = 0.121 g
Site Modified Design Spectral Accelerations, (g) for Class B Sites	S_{DS} = 0.226 g	S_{D1} = 0.081 g

USGS Seismic Design Maps based on ASCE/SEI 7-10, incorporating Supplement 1 and errata of March 31, 2013, and ASCE/SEI 7-10 Table 1.5-2. 1. ASCE Hazards Report included in the Appendix.

The acceleration values were determined using IBC spectral acceleration data provided by the United States Geological Survey (USGS) Earthquake Hazards Program for Site Soil Class B. Site coefficients were then used to modify the results per the ASCE/SEI 7-10. The design spectral acceleration values are listed in the lower row of Table III above. These values were not determined by a site-specific seismic study but were derived from interpolation of values provided by ASCE/SEI 7-10.

8.0 RECOMMENDATIONS

We offer the following engineering recommendations based on the geotechnical data obtained during our subsurface exploration, and the construction data provided in the "*Tentative Scope of Work For Use by Geotechnical Engineering Firms...*" prepared by J. R. Wauford & Company, Consulting Engineers.

The proposed new tank is to consist of a 500,000-gallon, multi-column, elevated water storage tank. The proposed base elevation is approximately 924 feet above MSL, while proposed overflow elevation is 1077.5 feet above MSL.

The site is underlain by shallow soil overburden consisting of a 4 to 6-inch-thick veneer of topsoil, and 12 inches to 18 inches of soil overburden and very soft, weathered rock. The topsoil, soil and very soft rock are underlain by soft to hard, chert and shale bedrock. Both soil augers and trackhoe refused at relatively shallow depths of 2 to 5 feet below existing grade on chert bedrock at boring B-1, B-3, B-4, and B-5. Boring B-2 encountered shale bedrock and refused at a depth of approximately 35 feet below existing grade on moderately hard shale. No groundwater was encountered at the boring locations at the time of drilling or after a period of 24 hours.

8.1 Elevated Water Storage Tank - Foundation Preparation And Design

We recommend that the proposed new elevated water storage tank be supported on shallow spread foundations. The footings should bear on Fort Payne Chert or Rockwood/Chattanooga Shale bedrock. The spread footings should be excavated down to and bear on the underlying Chert and/or Shale bedrock unit. The bottom of the spread footings should bear a minimum depth of 36 inches (3 feet) below the proposed ground elevation of 924 feet (bottom of spread footing elevation 921 feet).

As noted, based on the results of the borings some rock excavation will be required to reach the required ground and bottom of footing elevation. The top of rock across the Tank footprint may be undulating with higher ledges/seams than adjacent areas. Rock excavation should be performed as needed to level the rock surface in the footings. The foundation excavations (bottom and sides) should be hand cleaned of all loose material (soil, loose rock, mud/muck, water, etc.).

The tank spread footings may be sized, bearing on the Chert and Shale bedrock, utilizing an allowable bearing capacity of 10,000 psf (10 ksf). The footings should bear on bedrock and be cast against bedrock in excavation sidewalls (as noted, sidewalls should be cleaned of soil, loose rock, mud/muck, water, etc.).

Spread footings should be sized no smaller than 5' X 5' or equivalent. Exterior footings should bear a minimum of 36 inches below grade for frost protection, and to provide minimum foundation embedment.

The Tank foundation excavations should be observed by a geotechnical engineer for the presence of seams, slots, etc., and to ensure that all loose, unsuitable, material has been completely removed from foundation bottom and sidewalls. A test/probe hole should be drilled into the bedrock to allow the geotechnical engineer to probe for the presence of weathered seams beneath the structure. The geotechnical engineer should select the locations of the test holes. Based on his visual observations, more than one location may be recommended. The test/probe holes should be a **minimum** depth of 5-feet into the chert bedrock. Test/probe holes are not required in Shale (such as encountered at boring B-2) unless specifically requested by the engineer. If thick soil seams, soft areas, cavities, etc. are encountered, then additional test/probe holes should be placed as directed by the geotechnical engineer.

Dynamic Cone Penetrometer testing performed in the footings to confirm the assigned allowable bearing capacity. Footings should be poured with concrete as soon after excavation as possible. Foundation bearing soils exposed to inclement weather should be cleaned out down to the undisturbed bedrock. Footings requiring undercutting may be backfilled with concrete.

Positive drainage should be maintained away from the Tank area during and after construction such that no ponding of water around Tank foundations is allowed.

8.2 Elevated Water Storage Tank – Site And Foundation Preparation

We anticipate that a modest amount of site grading (primarily cut) will be required for the Tank site preparation. Backfill material may be required over the top of the Tank spread footings (the footings may also be poured with concrete up to finished grade).

We recommend that the Tank pad, cut, and fill areas be stripped of all topsoil, roots, and organics. The Tank area should then be excavated down to finished subgrade elevation.

Once site stripping/undercutting, etc., as outlined above, is completed, the Tank area should be proofrolled. Proofrolling should be performed utilizing a heavily loaded vehicle, such as a loaded dump truck, and a criss-cross proof rolling pattern. A minimum of two passes of the proof rolling equipment should be made. Proofrolling should be performed under the direction of a geotechnical engineer.

All fill should be compacted to 100 percent Standard Proctor maximum dry density, ASTM D 698. The moisture content of the fill should be controlled to within the range of +/- 2 percent of Optimum moisture content during compaction.

All fill should be placed and compacted in horizontal lifts. Benching should be performed in sloping areas where fill soil placement is required, to tie existing soil and new fill together. In no instance should new fill be placed on a sloping surface. Backfill lift thickness should be limited to a maximum of 8-inches, loose. In-place density testing should be performed concurrently with the placement of all fill to ensure the desired density is achieved. The recommended minimum rate of testing is 1 test per 2,500 square feet or less of fill area for each soil fill lift. A qualified soil technician, under the direction of a geotechnical engineer, should perform the soil density testing.

If fill is placed over the top of the tank spread footings, the new fill should consist of TDOT 303 Grading D Mineral Aggregate Base or excavatable flowable fill.

Soil fill may be placed as needed in all other areas of the site. The fill should consist of lean, silty clay, free of organics, and rock fragments larger than 4 inches in greatest dimension. The soil fill should have a plasticity index of less than 30, a minimum Standard Proctor maximum dry density of 92 PCF, and Optimum moisture content below the soil plastic limit. The geotechnical engineer should evaluate and approve proposed fill soils prior to use.

The TDOT 303 D aggregate base backfill should be placed in loose horizontal lifts not to exceed 8 inches in thickness. Compaction of the stone should be performed until at least 100% of its Standard Proctor Density is achieved. The flowable fill should have a minimum 28-day compressive strength of at least 100 psi.

In general, site finished grading should be performed around the Tank to ensure that positive drainage is maintained away from the Tank foundations.

9.0 ADDITIONAL RECOMMENDED WORK

We recommend that our firm be retained to review the above recommendations once project plans and specifications (foundation plans and site grading plans) have been completed. We have outlined our construction assumptions in the above paragraphs. Any proposed structure location changes or change in planned construction should be made available to our firm for review. It is likely that some changes/modifications/clarifications to our recommendations will be needed.

We recommend that our firm is selected to provide all field and laboratory, quality control engineering and testing services during construction. Quality control testing and observation services are recommended to prepare the subgrade for foundation construction, confirm the bearing capacity of the foundations, perform proofrolling, subgrade observation, compaction testing, and to test concrete for foundations.

10.0 GENERAL QUALIFICATIONS

This geotechnical report has been prepared for the exclusive use of J. R. Wauford & Company and the City of Dayton Tennessee. This report has been prepared in accordance with generally accepted geotechnical engineering practice for specific application to the geotechnical design elements of this project. The conclusions and recommendations contained in this report are based on applicable standards of our practice in this geographic area at the time this report was prepared. No other warranty expressed or implied is made. Foundation Systems Engineering, P.C is not responsible for any claims, damages, or liability associated with any other party's interpretation of this report's subsurface data or reuse of this report's subsurface data or engineering analysis without our express written authorization. An environmental site assessment (ESA) was not performed by our firm for this project and was beyond the scope of work for this preliminary subsurface exploration report.

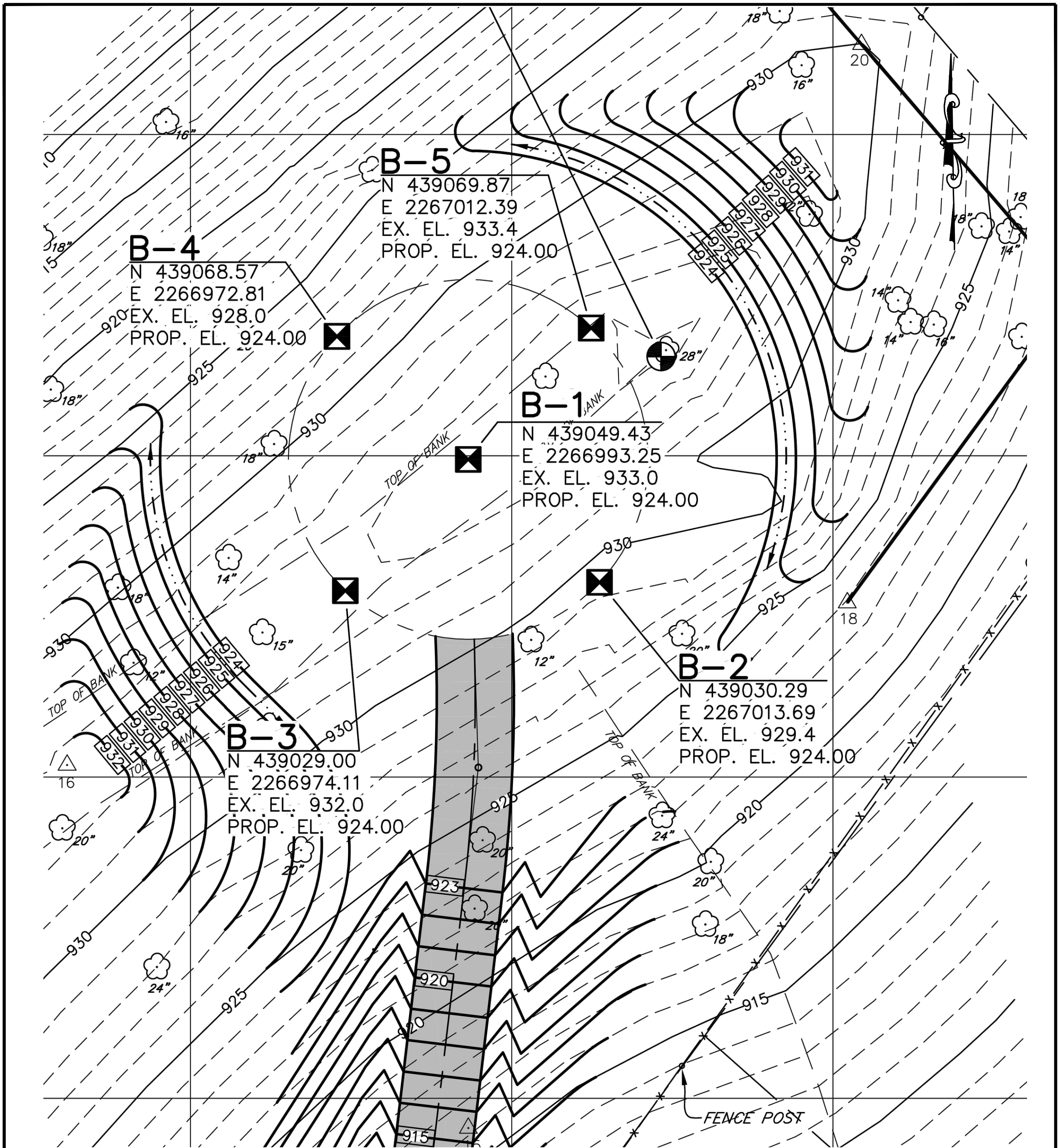
The analyses and recommendations submitted herein are based, in part, upon the data obtained from the borings. The nature and extent of variations between the boring and test pit locations may not become evident until construction. If variations appear evident, then we will re-evaluate the recommendations of this report. Plans and technical specifications for construction were not available for our use at the time this report was prepared. This report should not be made a part of the project plans and specifications but may be included with bidding documents for the convenience of the bidders.

APPENDICIES

I. Boring Location Plan & Test Boring Records
II. ASFE

APPENDIX I

Boring Location Plan & Test Boring Records



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Phone: 865.577.3361 | www.fsepc.com

DAYTON INDUSTRIAL PARK
WATER STORAGE TANK
DAYTON, TENNESSEE

FOR: MR. SCOTT CARROLL, J.R. WAUFORD	
DRAWN BY: JCH	NOTES: LOCATIONS ARE APPROXIMATE. NOT TO SCALE.
SCALE: NONE	DATE: 2/15/19
FSE #: 318194	

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TEST PIT RECORD

ELEV.	DEPTH	DESCRIPTION	DEPTH	N	CR --- RQD	S	POCKET PENETROMETER (TSF)	REMARKS
933	0.0	TOPSOIL AND ROOTS. (6")						
932.5	0.5	VERY SOFT TO HARD, DRY, WEATHERED TO PARTIALLY WEATHERED, GRAY AND BLACK, CHERT.						
929	4.0	HARD, DRY, LIGHT GRAY TO LIGHT OLIVE GRAY AND DARK GRAY.			10% / 0%			BEGIN NQ SIZED ROCK CORING.
919	14.0	BORING TERMINTAED AT 14.0 FEET. NO GROUNDWATER ENCOUNTERED AT TIME OF DRILLING.						BOREHOLE BACKFILLED WITH AUGER CUTTINGS AT TIME OF COMPLETION.

N-VALUE FROM DYNAMIC CONE PENETROMETER TESTING (ASTM STP 399)
PP - POCKET PENETROMETER
CR – 15% CORE RECOVERY, NX OR BX DESIGNATES BIT SIZE (ASTM D-2113)
RQD – ROCK QUALITY
SYMBOLES DESCRIBED BELOW:
UNDISTURBED SAMPLE (ASTM D-1587)
WATER TABLE, TIME OF BORING
WATER TABLE, 24 HR READING
LOSS OF DRILLING FLUID

5
13 70
20 100

BORING NUMBER: B-1
DATE DRILLED: 2/4/19
LAB. NO.: 318194

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TEST PIT RECORD

ELEV.	DEPTH	DESCRIPTION	DEPTH	N	CR --- RQD	S	POCKET PENETROMETER (TSF)	REMARKS
929.4	0.0	TOPSOIL AND ROOTS. (6")						
928.9	0.5	VERY SOFT TO SOFT, SLIGHTLY MOIST TO DRY, DARK GRAY, BLACK, BROWN AND GRAY, PARTIALLY WEATHERED SHALE.	2.5	50= 5"				
			5.0	50= 6"				
922.9	6.5	VERY SOFT TO SOFT, SLIGHTLY MOIST TO DRY, BLACK, PARTIALLY WEATHERED SHALE.	7.5	50= 2"				
			10.0	50= 6"				
			14.5	50= 1"				
			19.5	50= 3"				

Continued on page 2

N-VALUE FROM DYNAMIC CONE PENETROMETER TESTING (ASTM STP 399)
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RQD – ROCK QUALITY
SYMBOLES DESCRIBED BELOW:
UNDISTURBED SAMPLE (ASTM D-1587)
WATER TABLE, TIME OF BORING
WATER TABLE, 24 HR READING
LOSS OF DRILLING FLUID

5		
13	70	
20	100	

BORING NUMBER: B-2
DATE DRILLED: 2/4/19
LAB. NO.: 318194

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TEST PIT RECORD

ELEV.	DEPTH	DESCRIPTION	DEPTH	N	CR --- RQD	S	POCKET PENETROMETER (TSF)	REMARKS
			24.5	50= 3"				
900.9	28.5	VERY SOFT, SLIGHTLY MOIST, LIGHT GRAY, BLACK AND TAN, PARTIALLY WEATHERED SHALE AND SILTSTONE.	29.5	42				
895.9	33.5	SOFT, SLIGHTLY MOIST TO DRY, MEDIUM TO DARK GRAY AND MAROON, PARTIALLY WEATHERED SHALE.	34.5	50= 5"				BOREHOLE BACKFILLED WITH AUGER CUTTINGS AT TIME OF COMPLETION.
894.4	35.0	AUGER REFUSAL AT 35.0 FEET. NO GROUNDWATER ENCOUNTERED AT TIME OF DRILLING.						

N-VALUE FROM DYNAMIC CONE PENETROMETER TESTING (ASTM STP 399)
PP - POCKET PENETROMETER
CR – 15% CORE RECOVERY, NX OR BX DESIGNATES BIT SIZE (ASTM D-2113)
RQD – ROCK QUALITY
SYMBOLES DESCRIBED BELOW:
5
13 70
20 100
UNDISTURBED SAMPLE (ASTM D-1587)
WATER TABLE, TIME OF BORING
WATER TABLE, 24 HR READING
LOSS OF DRILLING FLUID

BORING NUMBER: B-2
DATE DRILLED: 2/4/19
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TEST PIT RECORD

ELEV.	DEPTH	DESCRIPTION	DEPTH	N	CR --- RQD	S	POCKET PENETROMETER (TSF)	REMARKS
932	0.0	TOPSOIL AND ROOTS. (4")						
931.7	0.3	SOFT, VERY MOIST TO WET, REDDISH TAN, SANDY, SILTY CLAY MIXED WITH CHERT.						
930.5	1.5	VERY SOFT TO HARD, DRY, WEATHERED TO PARTIALLY WEATHERED, GRAY AND BLACK, CHERT.						ROTARY-CONE BORING ADVANCEMENT.
930	2.0	HARD, DRY, WEATHERED TO PARTIALLY WEATHERED, LIGHT GRAY TO LIGHT OLIVE GRAY AND DARK GRAY, CHERT.						
925	7.0	HARD, LIGHT TO LIGHT OLIVE GRAY, DARK GRAY, CHERT.			13% --- 0%			BEGIN NQ SIZED ROCK CORING.
920	12.0	VERY SOFT TO MODERATELY HARD, OLIVE GRAY, PARTIALLY WEATHERED SILTSTONE.			96% --- 0%			
916.5	15.5	HARD, BLACK, CHERT.			72% --- 58%			
915	17.0	BORING TERMINTAED AT 17.0 FEET. NO GROUNDWATER ENCOUNTERED AT TIME OF DRILLING.						BOREHOLE BACKFILLED WITH AUGER CUTTINGS AT TIME OF COMPLETION.

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RQD – ROCK QUALITY
SYMBOLES DESCRIBED BELOW:
UNDISTURBED SAMPLE (ASTM D-1587)
WATER TABLE, TIME OF BORING
WATER TABLE, 24 HR READING
LOSS OF DRILLING FLUID

5
13 70
20 100

BORING NUMBER: B-3
DATE DRILLED: 2/4/19
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TEST PIT RECORD

ELEV.	DEPTH	DESCRIPTION	DEPTH	N	CR --- RQD	S	POCKET PENETROMETER (TSF)	REMARKS
928	0.0	TOPSOIL AND ROOTS. (4")						
927.7	0.3	SOFT TO HARD, SLIGHTLY MOIST, LIGHT GRAY AND BLACK, CHERT.						
924	4.0	HARD, DRY, WEATHERED TO PARTIALLY WEATHERED, LIGHT GRAY TO LIGHT OLIVE GRAY AND DARK GRAY, CHERT.			12% --- 0%			BEGIN NQ SIZED ROCK CORING.
920.5	7.5	HARD, DRY, WEATHERED TO PARTIALLY WEATHERED, LIGHT GRAY TO DARK BROWN, CHERT.			13% --- 0%			
916	12.0	HARD, DRY, WEATHERED TO PARTIALLY WEATHERED, LIGHT TO DARK BROWN, CHERT.			77% --- 0%			
912.5	15.5	BORING TERMINTAED AT 15.5 FEET. NO GROUNDWATER ENCOUNTERED AT THE TIME OF DRILLING.						BOREHOLE BACKFILLED WITH AUGER CUTTINGS AT TIME OF COMPLETION.

N-VALUE FROM DYNAMIC CONE PENETROMETER TESTING (ASTM STP 399)
PP - POCKET PENETROMETER
CR – 15% CORE RECOVERY, NX OR BX DESIGNATES BIT SIZE (ASTM D-2113)
RQD – ROCK QUALITY
SYMBOLES DESCRIBED BELOW:
5
13 70
20 100
UNDISTURBED SAMPLE (ASTM D-1587)
WATER TABLE, TIME OF BORING
WATER TABLE, 24 HR READING
LOSS OF DRILLING FLUID

BORING NUMBER: B-4
DATE DRILLED: 2/4/19
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TEST PIT RECORD

ELEV.	DEPTH	DESCRIPTION	DEPTH	N	CR --- RQD	S	POCKET PENETROMETER (TSF)	REMARKS
933.4	0.0	TOPSOIL AND ROOTS. (6")						
932.9	0.5	VERY SOFT TO HARD, DRY, WEATHERED TO PARTIALLY WEATHERED, GRAY AND BLACK, CHERT.						
931.4	2.0	VERY SOFT TO SOFT, DRY, WEATHERED TO PARTIALLY WEATHERED, LIGHT GRAY TO LIGHT OLIVE GRAY AND DARK GRAY, CHERT.						ROLLER CONE BORING ADVANCEMENT
929.4	4.0	HARD, DRY, WEATHERED TO PARTIALLY WEATHERED, LIGHT GRAY TO LIGHT OLIVE GRAY AND DARK GRAY, CHERT.			13% --- 8%			NX SIZED ROCK CORING BEGINNING.
919.4	14.0	BORING TERMINTAED AT 14.0 FEET. NO GROUNDWATER ENCOUNTERED AT THE TIME OF DRILLING.						BOREHOLE BACKFILLED WITH AUGER CUTTINGS AT TIME OF COMPLETION.

N-VALUE FROM DYNAMIC CONE PENETROMETER TESTING (ASTM STP 399)
PP - POCKET PENETROMETER
CR – 15% CORE RECOVERY, NX OR BX DESIGNATES BIT SIZE (ASTM D-2113)
RQD – ROCK QUALITY
SYMBOLES DESCRIBED BELOW:
UNDISTURBED SAMPLE (ASTM D-1587)
WATER TABLE, TIME OF BORING
WATER TABLE, 24 HR READING
LOSS OF DRILLING FLUID

5		
13	70	
20	100	

BORING NUMBER: B-5
DATE DRILLED: 2/4/19
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TEST PIT RECORD

ELEV.	DEPTH	DESCRIPTION	DEPTH	N	CR --- RQD	S	POCKET PENETROMETER (TSF)	REMARKS
933	0.0	TOPSOIL AND ROOTS. (6")						
932.5	0.5	VERY SOFT TO HARD, DRY, WEATHERED TO PARTIALLY WEATHERED, GRAY AND BLACK, CHERT.						
929	4.0	TEST PIT REFUSAL AT 4.0 FEET. NO GROUNDWATER ENCOUNTERED AT TIME OF EXCAVATION.						

N-VALUE FROM DYNAMIC CONE PENETROMETER TESTING (ASTM STP 399)
PP - POCKET PENETROMETER
CR – 15% CORE RECOVERY, NX OR BX DESIGNATES BIT SIZE (ASTM D-2113)
RQD – ROCK QUALITY
SYMBOLES DESCRIBED BELOW:
UNDISTURBED SAMPLE (ASTM D-1587)
WATER TABLE, TIME OF BORING
WATER TABLE, 24 HR READING
LOSS OF DRILLING FLUID

5		
13	70	
20	100	

BORING NUMBER: TP-1
DATE DRILLED: 2/4/19
LAB. NO.: 318194

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TEST PIT RECORD

ELEV.	DEPTH	DESCRIPTION	DEPTH	N	CR --- RQD	S	POCKET PENETROMETER (TSF)	REMARKS
932	0.0	TOPSOIL AND ROOTS. (4")						
931.7	0.3	SOFT, VERY MOIST TO WET, REDDISH TAN, SANDY, SILTY CLAY MIXED WITH CHERT.						
930.5	1.5	VERY SOFT TO HARD, DRY, WEATHERED TO PARTIALLY WEATHERED, GRAY AND BLACK, CHERT.						
930	2.0	TEST PIT REFUSAL AT 2.0 FEET. NO GROUNDWATER ENCOUNTERED AT TIME OF EXCAVATION.						

N-VALUE FROM DYNAMIC CONE PENETROMETER TESTING (ASTM STP 399)
PP - POCKET PENETROMETER
CR – 15% CORE RECOVERY, NX OR BX DESIGNATES BIT SIZE (ASTM D-2113)
RQD – ROCK QUALITY
SYMBOLES DESCRIBED BELOW:

5
13 70
20 100

UNDISTURBED SAMPLE (ASTM D-1587)
WATER TABLE, TIME OF BORING
WATER TABLE, 24 HR READING
LOSS OF DRILLING FLUID

BORING NUMBER: TP-3
DATE DRILLED: 2/4/19
LAB. NO.: 318194

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TEST PIT RECORD

ELEV.	DEPTH	DESCRIPTION	DEPTH	N	CR --- RQD	S	POCKET PENETROMETER (TSF)	REMARKS
928	0.0	TOPSOIL AND ROOTS. (4")						
927.7	0.3	SOFT TO HARD, SLIGHTLY MOIST, LIGHT GRAY AND BLACK, CHERT.						
924	4.0	TEST PIT REFUSAL AT 4.0 FEET. NO GROUNDWATER ENCOUNTERED AT TIME OF EXCAVATION.						

N-VALUE FROM DYNAMIC CONE PENETROMETER TESTING (ASTM STP 399)
PP - POCKET PENETROMETER
CR – 15% CORE RECOVERY, NX OR BX DESIGNATES BIT SIZE (ASTM D-2113)
RQD – ROCK QUALITY

5		
13	70	
20	100	

SYMBOLES DESCRIBED BELOW:
UNDISTURBED SAMPLE (ASTM D-1587)
WATER TABLE, TIME OF BORING
WATER TABLE, 24 HR READING
LOSS OF DRILLING FLUID

BORING NUMBER: TP-4
DATE DRILLED: 2/4/19
LAB. NO.: 318194

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TEST PIT RECORD

ELEV.	DEPTH	DESCRIPTION	DEPTH	N	CR --- RQD	S	POCKET PENETROMETER (TSF)	REMARKS
933.4	0.0	TOPSOIL AND ROOTS. (6")						
932.9	0.5	VERY SOFT TO HARD, DRY, WEATHERED TO PARTIALLY WEATHERED, GRAY AND BLACK, CHERT.						
931.4	2.0	VERY SOFT TO SOFT, DRY, WEATHERED TO PARTIALLY WEATHERED, LIGHT GRAY TO LIGHT OLIVE GRAY AND DARK GRAY, CHERT.						ROLLER CONE BORING ADVANCEMENT
929.4	4.0	TEST PIT REFUSAL AT 4.0 FEET. NO GROUNDWATER ENCOUNTERED AT TIME OF EXCAVATION.						

N-VALUE FROM DYNAMIC CONE PENETROMETER TESTING (ASTM STP 399)
PP - POCKET PENETROMETER
CR – 15% CORE RECOVERY, NX OR BX DESIGNATES BIT SIZE (ASTM D-2113)
RQD – ROCK QUALITY
SYMBOLES DESCRIBED BELOW:
UNDISTURBED SAMPLE (ASTM D-1587)
WATER TABLE, TIME OF BORING
WATER TABLE, 24 HR READING
LOSS OF DRILLING FLUID

5		
13	70	
20	100	

BORING NUMBER: TP-5
DATE DRILLED: 2/4/19
LAB. NO.: 318194

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DAYTON INDUSTRIAL PARK
WATER STORAGE TANK
DAYTON, TENNESSEE

APPENDIX II

ASFE

Important Information About Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

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