RED ROCK
CONSULTING

Report of Geotechnical Investigation

OF THE

BACKUP GENERATOR BUILDING
MUSKOGEE, OKLAHOMA

Prepared For:

Spur Design
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Oklahoma City, Oklahoma 73116

Prepared By:

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March 25, 2013
Project No. 13026

RED ROCK
CONSULTING

March 25, 2013

Spur Design
7700 North Hudson Avenue, Suite 9
Oklahoma City, Oklahoma

Attention: Mr. Brian Mays

Re: Report of Geotechnical Investigation
Backup Generator Building
Muskogee, Oklahoma
Project No. 13026

Dear Mr. Mays,

I am pleased to submit herewith this report entitled "Geotechnical Investigation, Backup Generator Building, Muskogee, Oklahoma".

In an effort to provide a more environmentally friendly service, this report has been provided electronically. If you wish to receive a hard copy of this report, please contact our office.

It has been our pleasure to assist you with this project. Should you have any questions regarding the contents of this report, please contact Red Rock Consulting.

Yours very truly,
RED ROCK CONSULTING, LLC
CA No. 5707 Exp. 06/30/13

Jason T. Unruh

Jason T. Unruh, EI
Project Specialist

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REPORT OF GEOTECHNICAL INVESTIGATION

BACKUP GENERATOR BUILDING
MUSKOGEE, OKLAHOMA

PROJECT NO. 13026

INTRODUCTION 1

GENERAL 1
PROPOSED CONSTRUCTION 1
SCOPE OF WORK 1

FIELD AND LABORATORY INVESTIGATIONS 3

FIELD EXPLORATION 3
LABORATORY TESTING 4

SITE DESCRIPTION 5

SURFACE CONDITIONS 5
SITE GEOLOGY 5
SUBSURFACE CONDITIONS 6
GROUNDWATER CONDITIONS 7

CONCLUSIONS AND RECOMMENDATIONS 8

FOUNDATION RECOMMENDATIONS 8
LATERAL EARTH PRESSURES FOR RETAINING WALL DESIGN 10
CONSTRUCTION CONSIDERATIONS 14
ENVIRONMENTAL CONSIDERATIONS 15

CLOSURE 16

APPENDICES

APPENDIX A – Field Investigation
APPENDIX B – General Notes

REPORT OF GEOTECHNICAL INVESTIGATION

BACKUP GENERATOR BUILDING
MUSKOGEE, OKLAHOMA

PROJECT NO. 13026

INTRODUCTION

General

This report presents the results of the geotechnical investigation performed for the proposed construction of a standby full facility backup generator building for the Jack C Montgomery VA Medical Center, which is located at 1011 Honor Heights Drive in Muskogee, Oklahoma. The purpose of this investigation is to evaluate the subsurface conditions at the site and to provide recommendations pertaining to the geotechnical aspects of the proposed project.

Proposed Construction

The project will include the construction of a 4,340 square foot two-story generator building and retaining wall. The type and size of the retaining wall is unknown at this time. No below grade construction is anticipated for the project.

Precise loading information has not been provided, but is anticipated not to exceed 500 kips. Exact grade changes (cut/fill requirements) for the site have not been provided at this time, but are anticipated to be less than 5 feet across the site.

Scope of Work

The scope of this investigation includes the following:

1. Review of previous geotechnical and geological information of this site and sites near this site. This was augmented with data obtained during the field investigation phase of the project.

2. Investigation of the foundation suitability of the subsurface soils by drilling and sampling a total of 3 boreholes within the planned project area

3. A laboratory testing program consisting of Atterberg limits determination, moisture content and sieve tests on the soils encountered

4. Recommendations regarding foundation support of the proposed building

5. Lateral earth pressure recommendations for the design of the retaining walls

6. Recommendations regarding construction and earthwork

7. Sustainability recommendations in regard to site construction and construction materials

FIELD AND LABORATORY INVESTIGATIONS

Field Exploration

Subsurface exploration was performed on March 5, 2013. The boring locations were staked in the field by a representative of Red Rock Consulting as per guidance provided on site by a representative of the VA hospital. A boring location diagram showing the boring locations is included in Appendix A.

The subsurface exploration program consisted of drilling 3 borings under the full time supervision of an engineer. The borings extended to depths ranging between 25 and 27.5 feet using an all-terrain-vehicle mounted CME 75 drill rig. Draft boring logs of the subsurface conditions encountered were developed in the field. Representative samples were obtained using the split-barrel sampling procedures (Standard Penetration Test, SPT) in general accordance with ASTM D-1586. After refusal was attained, the hardness of bedrock was evaluated using a Texas Cone Penetrometer in accordance with the AASHTO Manual on Subsurface Investigation and as modified by the Oklahoma Department of Transportation.

The SPT test uses a standard, 2-inch O.D., split-barrel sampling spoon that is driven into the bottom of the boring with a 140 pound automatic drive hammer falling 30 inches. The blows per foot, N, is the number of blows required to advance the sampling spoon the last 12 inches, or less, of an 18-inch sampling interval. The N value is used to estimate the in-situ relative density of granular soils, the consistency of cohesive soils and the hardness of weathered bedrock, when encountered.

The Texas Highway Department cone penetrometer test is a standard test developed by the Texas Highway Department to evaluate the consistency or hardness of the bedrock material. The Texas Cone Penetrometer (TCP) test drives a penetrometer cone into the bedrock material with a 140 pound automatic drive hammer that falls 30 inches. The TCP is driven for a series of blows, the first 10 blows being seating blows, followed by two 50 blow counts. After 50 blows of the automatic hammer, the distance the TCP has advanced is measured and recorded. The distance the TCP is driven is used to estimate the hardness of bedrock.

After performing SPT and TCP tests, the holes were backfilled with grout and cuttings as required by the Oklahoma State Statutes for Geotechnical drilling.

Samples were collected and transported back to the office for further classification and testing. The final boring logs were developed from the draft logs and observations and test results of the samples returned to the laboratory. The stratigraphic contacts indicated are only for the specific dates and locations reported and, therefore, are not necessarily representative of other locations and times. The boring logs, presenting conditions encountered at each location explored, are included in Appendix A.

Laboratory Testing

Representative soil samples were tested to refine the field classifications and evaluate physical properties of the soils which may affect the geotechnical aspects of project design and construction.

The laboratory testing program included the following:

- Soil Classification in general accordance with ASTM D2487
- Liquid and Plastic Limits of soils in general accordance with ASTM D4318
- Washed No. 200 US Standard Sieve test in general accordance with ASTM Method D1140
- Moisture content tests in general accordance with ASTM Method D2937

The results of the physical laboratory tests conducted are shown on the boring logs in Appendix A.

SITE DESCRIPTION

Surface Conditions

At the time of drilling activities there were parking lots located around the project site. West of the project site was an undeveloped wooded area. To the east of the project site was a large hospital building. To the south of the project site was a hospital building. To the north of the project site was another parking lot and a wooded area. Across the site were several medium to large trees and the site was covered with short grass and small to large sandstone cobbles. There were small to moderate sized pieces of concrete rubble present at the surface throughout the site. There was an apparent to be a greenhouse located in the southwest corner of the site. Located north of the greenhouse were several concrete barricades and a sand pile that was 3 feet high and 10 feet by 10 feet in area. Concrete stairways were located on the north and south ends of the site.

At the time of the field investigation, the locations of the borings were relatively flat. The site appeared to drain to the west. The east side of the project site was approximately 25 feet higher in elevation than the west side of the project site. The site was dry at the time of drilling activities and the drilling rig did not experience difficulty maneuvering around the site.

Several underground utilities were present across the project site. The boring locations were selected considering these utilities.

The finished concrete slab of a building to the north of the project site was used as the benchmark and was assigned an elevation of 100 feet. Based on the benchmark, the elevations of the borings ranged from 97.8 to 98.8 feet. The approximate elevation at each boring location is shown on the boring logs in Appendix A. If a more accurate elevation for the concrete slab is know or obtained, please contact Red Rock Consulting for revised boring elevations.

A boring location diagram showing the locations of the borings is included in Appendix A.

Site Geology

Division One of the “Engineering Classification of Geological Materials”, published by the Oklahoma Department of Transportation (ODOT) indicates the project site is underlain by the Bluejacket Unit (Pbj).

This unit consists of sandstone and shale. The sandstone is soft to extremely hard, brown to gray in color, and the beds are from a few inches thick to 20 feet thick, with zones of sandstone, separated by shale stringers, as much as 150 feet thick. The shales are gray to black, generally fissile, and in zones up to 300 feet thick. The thick zones of shale have thin stringers of siltstone, sandstone, and minor amounts of limestone.

The Bluejacket unit ranges in thickness from approximately 300 feet to 400 feet.

The Bluejacket unit outcrops in Haskell, McIntosh, Muskogee, Pittsburg, and Wagoner Counties within Division One. Generally the outcrop of the Bluejacket unit is an east-facing ridge, which trends in a north-south direction. The sandstone caps the ridge, and the slope below is formed on the underlying shale. On top of the ridge, a slighty rolling surface is formed on the gentle westward dip-slope of the sandstone beds.

According to the Geologic Map of the "Hydrologic Atlas 8 of Oklahoma," Reconnaissance of the Water Resources of The Fort Smith Quadrangle, East-Central Oklahoma," by Robert B. Morton, U.S. Geological Survey, 1980, indicates that the project site is located over the Boggy Formation (IPbg) of the Pennsylvanian. The geologic formation is described therein as follows:

Shale, sandstone, and coal; includes Bluejacket Sandstone Member at base. Yields limited amounts of water of poor quality.

Subsurface Conditions

Information collected during the field investigation indicates that the overburden materials consisted of various combinations of sand, silt and lean clay. Within the overburden, small to moderate sized sandstone cobbles and gravel were encountered at varying depths in all of the borings. The overburden materials extended from the surface to the boring termination depths of 17 feet in boring B-1, 13.5 feet in boring B-2, and 16.5 feet in boring B-3. The bedrock material encountered was well cemented to very well cemented shaley sandstone that extended to the boring termination depths that ranged from 25 to 27.5 feet.

More detailed soil information can be found on the boring logs in Appendix A.

Groundwater Conditions

Groundwater conditions were monitored during the advancement of the borings and immediately after the completion of drilling operations. At these times, groundwater was not encountered in any of the borings. All of the borings remained open following drilling activities.

To obtain more accurate groundwater level information, long-term observations in a well or piezometer that is sealed from the influence of surface water would be needed. Fluctuations in groundwater levels can occur due to seasonal variations in the amount of rainfall, runoff, altered drainage paths, and other factors not evident at the time borings were advanced. Consequently, the contractor should be aware of this possibility while constructing this project.

CONCLUSIONS AND RECOMMENDATIONS

Foundation Recommendations

Recommendations pertaining to the building pad, floor slab subgrade, foundation systems and lateral earth pressures pertaining to retaining wall design are discussed below.

Building Pad Preparation

Building pad preparation for the proposed structure should include removal of the sandstone cobbles, concrete rubble, vegetation, topsoil and any other unsuitable materials which may be encountered. Removal depths should be determined at the time of construction by a representative of Red Rock Consulting.

Floor Slab Subgrade

Structures such as the ones proposed for this site are generally designed for post-construction vertical floor slab movements of less than 1 inch. Based on Atterberg limits test results of the on-site soils and assuming a minimum natural dry in-situ soil condition and a zone of influence (average depth of relatively constant moisture) of 8 feet below the existing ground surface, the evaluation indicates a PVR of less than 1 inch. The weight of the structure was not included in the potential vertical heave estimation.

The in situ soils at the project site are adequate to provide direct support of the floor slab in the existing soil condition. Procedures are recommended below for developing a properly compacted soil zone beneath the concrete slab.

- The floor slab area for the structure plus approximately 5 feet in each horizontal direction must be stripped of all asphalt, concrete, vegetation and topsoil.

- The work area should then be proofrolled with a loaded, tandem-axle dump truck weighing at least 25 tons to locate any areas that are soft or unstable. The proofrolling should involve overlapping passes in mutually perpendicular directions. Where rutting or pumping is observed during proof rolling, the soft and/or unstable soils should be excavated and replaced with a low volume change soil as described below.

- After proofrolling and completing any corrective work, the work area should be scarified to a depth of 8 inches, moisture conditioned and compacted. The moisture content of the scarified soil should be adjusted to its optimum value or above, as determined by a standard Proctor test (ASTM D-698), prior to being compacted to at least to 95 percent of its maximum dry density.

- The minimum recommended moisture content must be maintained in the building pad materials until the floor slab is constructed. Drainage must be developed sloping away from the building to prevent water from ponding along the perimeter and affecting future floor slab performance.

- The geotechnical engineer or a representative of the geotechnical engineer should be present to verify the above recommendations are implemented successfully.

The use of a vapor retarder is recommended beneath concrete slabs-on-grade that will be covered with wood, tile, carpet or other moisture sensitive or impervious coverings, or when the slab will support equipment sensitive to moisture. When using a vapor retarder, the slab designer and slab contractor should refer to ACI 302 for procedures and cautions regarding the use and placement of a vapor retarder.

Drilled Pier and Grade Beam Foundation System

It is understood that a drilled pier and grade beam foundation system is anticipated of this project.

Straight shaft drilled piers that derive their support from end bearing could be used to support the proposed building. Drilled piers should extend at least 10 feet into the sandstone bedrock. The drilled piers should be designed using an allowable end bearing pressure of 58,000 psf and skin resistance of 6,000 psf. Skin friction in the top 2 feet of the bedrock should be neglected. The allowable end bearing and skin resistance have a safety factor of 2 and 3, respectively, included.

Grade beams should be structurally connected to the top of the piers. Grade beams should extend at least 2.5 feet below the final exterior adjacent grade. Excavations for the grade beams should be free of loose material. The straight shaft piers should have a minimum diameter of 24 inches. The piers should be provided with enough steel reinforcement to provide adequate structural integrity.

An earth auger was used to penetrate the overburden soils and the bedrock materials. The pier drilling contractor should be made aware of these subsurface conditions. Temporary casing may be needed to prevent caving of the excavation sides in the piers due to the cohesionless overburden materials. However, the final determination should be made at the time of construction.

Groundwater was not encountered in the borings. Therefore, groundwater seepage into pier excavations is not anticipated to occur. The need for dewatering will depend on the actual groundwater conditions at the time of construction.

Prior to placing concrete, any water or sloughed material should be removed from the base of the drilled piers. If water is encountered and cannot be removed, concrete should be pumped or placed using a tremie pipe and placed from the bottom of the pier excavation to the top, displacing the water to the surface. To facilitate pier construction, concrete should be onsite and ready for placement as pier excavations are completed. In no event should a pier excavation be allowed to remain open overnight.

Long-term settlement for straight shaft piers bearing within the hard sandstone bedrock and constructed as recommended should be less than ¾ inch. Differential settlement should be negligible.

Lateral Earth Pressures for Retaining Wall Design

Details pertaining to the retaining wall for this project are not yet available. General recommendations for LEPs for retaining wall design are presented below.

Lateral soil pressures on retaining walls depend on several factors including drainage provisions, amount of wall movement (rotation) that is allowed and the retained soil type.

The following is recommended for the retaining wall drainage design and construction:

- Granular backfill should be clean, free-draining sand or crushed stone
- A drainage system should be constructed with perforated pipe under drains
- Perforated pipe should be surrounded by at least 4 inches of ASTM C-33 No. 57 stone or equivalent with the stone and pipe encased in an approved filter fabric to resist the migration of fines into the drain system.
- The exterior ground surface should consist of a minimum of 12 inch compacted clay cap or pavement section sloped to drain from the wall.

If adequate drainage is not possible, then combined hydrostatic and lateral earth pressures should be calculated for granular backfill using an equivalent fluid weighing 80 and 90 pcf for active and at-rest conditions, respectively. For lean clay backfill, an equivalent fluid weighing 90 and 100 pcf should be used for active and at-rest, respectively. These pressures do not include the influence of surcharge, equipment or additional loading, which should be added. Heavy equipment should not operate within a distance closer than the exposed height of retaining walls to prevent lateral pressures more than those provided.

Walls with unbalanced backfill levels on opposite sides should be designed for earth pressures at least equal to those indicated in the following table. Earth pressures will be influenced by structural design of the walls, conditions of wall restraint, methods of construction and/or compaction and the strength of the materials being restrained. Active earth pressure is commonly used for the design of free standing cantilever retaining walls. Passive pressure is usually associated with soil at least 3 feet in depth that is located at the toe of the wall. Active and passive pressures assume some wall movement. The “at-rest” condition assumes no wall rotation.

Basic active, passive and at-rest conditions on a wall are illustrated in the diagram below. If a footing foundation were added, the pressure diagrams would extend to the depth of the footing. The recommended design lateral earth pressures <u>do not include a factor of safety and do not provide for possible hydrostatic pressure on the walls.</u>

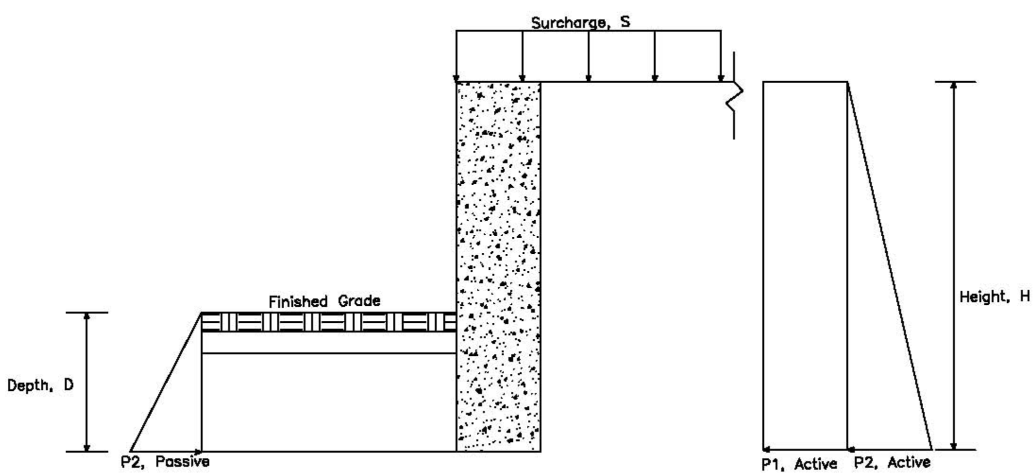


Table 1 – Lateral Earth Pressure Parameters

EARTH PRESSURE CONDITIONS	COEFFICIENT FOR BACKFILL TYPE	EQUIVALENT FLUID PRESSURE (psf)	SURCHARGE PRESSURE, P_1 (psf)	EARTH PRESSURE, P_2 (psf)
Active (Ka)	Granular – 0.33	40	(0.33)S	(40)H
	Lean Clay* – 0.49	60	(0.49)S	(60)H
At-Rest (Ko)	Granular – 0.50	50	(0.50)S	(50)H
	Lean Clay* – 0.66	80	(0.66)S	(80)H
Passive (Kp)	Granular – 3.0	315	-----	(315)D
	Lean Clay* – 2.0	240	-----	(240)D+3.2c**

* low plasticity cohesive soils

** c = cohesion of clay soil

Conditions applicable for the above table include:

- The resultant force of the surcharge pressure, P_1, acts a distance of H/2 from the bottom of the wall. To determine the resultant active force per unit length of wall, $P_s = (P_1)(H)(\text{Width of Wall})$.
- The resultant force of the earth pressure, P_2, acts a distance of H/3 or D/3 from the bottom of the wall. To determine the resultant per unit length of wall, $P_a = ½(P_2)(H^2)$.
- Passive pressure in the top 3 feet should be neglected. The top three feet is where the soil is exposed to the elements and is subjected to freeze-thaw and wet-dry cycles. The interactive forces between the soil and water and/or the soil and concrete are disrupted when the soil is frozen or dried away from the concrete.
- Uniform and level surcharge, where S is surcharge pressure
- A maximum in-situ soil backfill weight of 120 pcf
- Granular soil assumes a 30° friction angle and weight of 110 pcf; lean clay assumes a 20° friction angle and weight of 120 pcf
- Horizontal backfill, compacted to at least 95 percent of the material's maximum dry density as determined by test method ASTM D-698

- Loading from heavy compaction equipment, proposed roadway barriers, pavement and traffic not included

- No groundwater acting on wall

- No safety factor included

For active earth pressure to be mobilized, the soil mass must strain laterally 0.002H to 0.004H for sand and 0.02H to 0.05H for clay, where H is the total height of the wall (including the buried portion). For passive earth pressure to be mobilized, the soil mass must strain laterally 0.005D to 0.01D for sand, and 0.02D to 0.05D for clay, where D is the total depth below grade at the toe of the wall.

The amount of translation required for mobilization is related to the densification of the sand or compaction of the clay. Walls which are retained in some manner or are built with too stiff a basal stem *may not deflect sufficiently* to mobilize the shear strength of the soil about them. These walls must be designed to withstand the full hydrostatic pressure of the soil/water mixture they hold in retention.

Backfill placed against the retaining walls should consist of select backfill as per the ODOT Standard Specifications for Highway Construction (2009), Section 705. Granular backfill (sand) is anticipated directly behind the wall and similar select backfill materials are anticipated beyond the reinforcing straps of the wall. For the granular values to be valid, the granular backfill must extend out from the base of the wall at an angle of at least 45 and 60 degrees from vertical for the active and passive cases, respectively.

CONSTRUCTION CONSIDERATIONS

Construction considerations such as construction in working in wet weather and construction monitoring are discussed in this section

Wet Weather Earthwork

During or after wet weather, it may be necessary to import granular materials to protect open subgrade soils. It may also be necessary to install a granular working pad to support construction equipment. Delays in site earthwork activities should be anticipated during periods of heavy rainfall. Additionally, site clearing and stripping activities may expose subgrade material that may be damaged if subjected to disturbance from construction traffic.

When a granular working base is used to protect open subgrade material and construction equipment, the base should consist of a suitable thickness of crushed rock or ballast placed by end-dumping off an advancing pad of rock fill. Because construction practices can greatly affect the amount of rock required, we recommended that if conditions require the installation of a granular working blanket, the design, installation and maintenance be made the responsibility of the contractor. After installation, the working blanket should be compacted with a minimum of four overlapping passes with a smooth-faced steel drum or grid roller.

Construction Monitoring

Red Rock Consulting should be retained to provide construction monitoring services during earthwork activities and foundation construction. The purpose of field monitoring services is to confirm that site conditions are as anticipated, to provide field recommendations as required based on conditions encountered and to document the activities of the contractor to assess compliance with the project recommendations provided by Red Rock Consulting.

ENVIRONMENTAL CONSIDERATIONS

The environmental effect of construction projects is a growing concern in our industry. Some points for consideration of the environment regarding site construction and construction materials are summarized in the following paragraphs. These points should be incorporated into the design and construction of this project for a more environmentally friendly result. The following is only a summary. For a more in-depth discussion on sustainable design and construction, please contact Red Rock Consulting.

SITE CONSTRUCTION

Sedimentation and Erosion Control

Reduce pollution from construction activities by controlling soil erosion, waterway sedimentation and airborne dust generation. This can be accomplished most efficiently by using seeding or mulching and silt fence.

- Seeding or Mulching – If, for some reason, the excavated site is left open for an extended amount of time, soil erosion should be retarded by using seeding or mulching to cover and hold the soils.
- Silt Fence – Prevent sedimentation of the storm sewer or receiving streams by constructing silt fence (posts with a filter fabric media) around the project site. The silt fence is used to remove sediment from stormwater that may runoff the construction site.

CONSTRUCTION MATERIALS

Local Materials

Increase the demand for building materials and products that are extracted and manufactured within the region, thereby supporting the use of indigenous resources and reducing the environmental impacts resulting from transportation of materials. Examples of local materials that could be considered in the construction of this project include cement, fly ash, water, recycled concrete and/or aggregate and sand.

Recycled Materials

Reuse building materials and products in order to reduce demand for virgin materials and to reduce waste, thereby reducing impacts associated with the extraction and processing of virgin resources. Examples of recycled materials that could be considered in the construction of this project include recycled concrete and aggregate.

CLOSURE

The analysis, conclusions, and recommendations presented in this report are based on the scope of work defined in the proposal and site conditions as they existed at the time of the field exploration, and it is further assumed that the conditions encountered in the exploratory borings are representative subsurface conditions within the study area. If conditions differ from those described in this report are encountered or appear to be present beneath the excavations, Red Rock Consulting should be advised at once so that additional recommendations may be provided where necessary.

This report was prepared for the exclusive use of Spur Design and their agents and consultants. It should be made available to prospective contractors for information factual data only and not as a warranty of subsurface conditions similar to those interpreted from the boring logs or discussions presented herein.

APPENDIX A

Benchmark: Concrete Slab of Generator to the North of the site
Elevation = 100' (assigned)

RED ROCK CONSULTING
P.O. Box 30591
Edmond, Oklahoma 73003
Phone: (405) 562-3328

BORING LOCATION DIAGRAM
BACKUP GENERATOR BUILDING
MUSKOGEE, OKLAHOMA

Project Mngr:	JTU	Project No.	13026
Designed By:	JTU	Scale:	NOT TO SCALE
Checked By:	KKB	Date:	3/11/13
Approved By:	KKB	Drawn By:	JTU

RED ROCK CONSULTING
7042 Highwater Circle, Suite B
Edmond, Ok 73034

BORING NUMBER B-1

CLIENT Spur Design
PROJECT NAME Backup Generator Building
PROJECT NUMBER 13026
PROJECT LOCATION Muskogee, Oklahoma
DATE STARTED 3/5/13 **COMPLETED** 3/5/13
GROUND ELEVATION 98 ft **HOLE SIZE** 6 in
DRILLING CONTRACTOR DSO - Drilling Services of Oklahoma
GROUND WATER LEVELS:
DRILLING METHOD 4.5" augers - CME 75
DURING DRILLING none
LOGGED BY JTU **CHECKED BY** KKB
0 hrs AFTER DRILLING none
NOTES North Boring
Cave In Depth open

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		**LEAN CLAY** with small to moderate-sized sandstone cobbles, brown to reddish brown, medium stiff to stiff	SPT	9	10				
			SPT	8	15				
5		**CLAYEY SAND** with small to moderate-sized sandstone cobbles, reddish brown, very loose to loose	SPT	3	18	27	18	9	39.2
			SPT	9	11				
		SILTY SAND with small to moderate-sized sandstone cobbles, reddish brown, loose to medium dense							
10			SPT	11	11				
		SILTY, CLAYEY SAND, reddish brown, medium dense							
15			SPT	27	12	23	16	7	38.1
		SILTY SANDSTONE, tan to reddish brown, iron stains, cemented to very well cemented	SPT	50/3"	7				
		SHALEY SANDSTONE, gray, well cemented to very well cemented	TC	50/0.3" 50/0.3"					
20									
			TC	50/1.3' 50/0.3'					
25									
		Boring Termination Depth = 27.5 feet Boring Completed and Grouted on 3/5/13	TC	50/0.3" 50/0.1"					

RED ROCK CONSULTING 7042 Highwater Circle, Suite B
Edmond, Ok 73034

BORING NUMBER B-2

CLIENT Spur Design
PROJECT NAME Backup Generator Building
PROJECT NUMBER 13026
PROJECT LOCATION Muskogee, Oklahoma
DATE STARTED 3/5/13 **COMPLETED** 3/5/13
GROUND ELEVATION 98.8 ft **HOLE SIZE** 6 in
DRILLING CONTRACTOR DSO - Drilling Services of Oklahoma
GROUND WATER LEVELS:
DRILLING METHOD 4.5" augers - CME 75
DURING DRILLING none
LOGGED BY JTU **CHECKED BY** KKB
0 hrs AFTER DRILLING none
NOTES Center Boring
Cave In Depth open

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
0		**CLAYEY SAND** with small to moderate-sized sandstone cobbles, dark brown to reddish brown, very loose to medium dense	SPT	2	15				
			SPT	16	12				
5		**CLAYEY SAND**, brown, loose	SPT	7	11				
		SILTY SAND, iron stains, light brown, loose to medium dense	SPT	9	17				
10			SPT	14	19				
15		**SHALEY SANDSTONE**, weathered, light brown to brown, well cemented to very well cemented	SPT	33 25 50/2"	12	25	17	8	40.9
			TC	50/1" 50/0.3"					
20			TC	50/1.3" 50/0.8"					
25		Boring Termination Depth = 25 feet Boring Completed and Grouted on 3/5/13	TC	50/0.4" 50/0.4"					

RED ROCK CONSULTING
7042 Highwater Circle, Suite B
Edmond, Ok 73034

BORING NUMBER B-3

CLIENT Spur Design
PROJECT NAME Backup Generator Building
PROJECT NUMBER 13026
PROJECT LOCATION Muskogee, Oklahoma
DATE STARTED 3/5/13 **COMPLETED** 3/5/13
GROUND ELEVATION 97.8 ft **HOLE SIZE** 6 in
DRILLING CONTRACTOR DSO - Drilling Services of Oklahoma
GROUND WATER LEVELS:
DRILLING METHOD 4.5" augers - CME 55
DURING DRILLING none
LOGGED BY JTU **CHECKED BY** KKB
0 hrs AFTER DRILLING none
NOTES South Boring
Cave In Depth open

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0	**SANDY LEAN CLAY**, dark brown to brown, soft	SPT	3	18				
	CLAYEY SAND with small to moderate-sized sandstone cobbles, reddish brown, loose to medium dense	SPT	16	17				
5		SPT	8	12				
	SILTY SAND with small to moderate-sized sandstone gravel, greenish brown, loose to medium dense	SPT	6	13	21	18	3	26.7
10		SPT	12	14				
	SILTY, CLAYEY SAND, light brown with thin black layers, very dense							
15		SPT	8 11 50/5"	13				
	SHALEY SANDSTONE, dark grey, well cemented to very well cemented	TC	50/0.3" 50/0.3"					
20		TC	50/1.3" 50/0.5"					
25								
	Boring Termination Depth = 26.5 feet Boring Completed and Grouted on 3/5/13	TC	50/0.8" 50/0.3"					

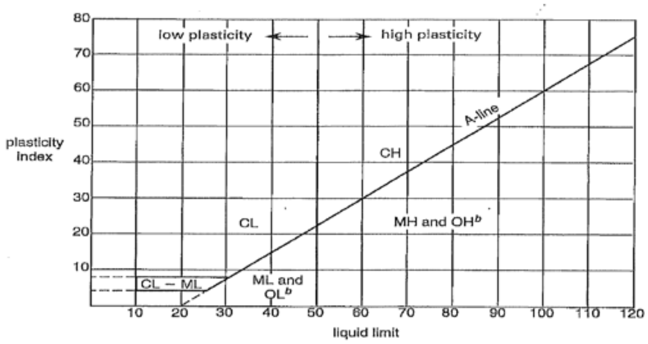
APPENDIX B

RED ROCK CONSULTING

GENERAL NOTES

The Unified Soil Classification System is used to identify the soil unless otherwise noted.

UNIFIED SOIL CLASSIFICATION SYSTEM ASTM D 2487



[b] Distinguishing between M and O classifications requires identifying organic components by observation, odor, or other testing.

SOIL PROPERTY SYMBOLS

N	Standard "N" penetration: Blows per foot
Qu	Unconfined Compressive Strength, tsf
Qp	Penetrometer value, tsf
Mc	Water Content, %
LL	Liquid Limit, %
PI	Plasticity Index, %
DD	Natural Dry density, pcf
∇	Apparent groundwater levels

DRILLING AND SAMPLING SYMBOLS

BS	Bag Sample
SPT	Split Spoon – 1 3/8" I.D., 2" O.D., except where noted
ST	Shelby Tube – 3" O.D., except where noted
AU	Auger Sample
TC	Texas Cone Penetrometer
DCP	Dynamic Cone Penetrometer

RELATIVE DENSITY AND COSNISTENCY CLASSIFICATIONS

Major Divisions			Group Symbol	Typical Names
Course-Grained Soils More than 50% retained on the No. 200 sieve	**Gravels** 50% or more of course fraction retained on the No. 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 50% or more of course fraction passes the No. 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-Grained Soils More than 50% passes the No. 200 sieve	**Silts and Clays** Liquid Limit 50% or less		ML	Inorganic silts, very fine sands, rock four, silty or clayey fine sands
			CL	Inorganic clays of low to medium plasticity, gravelly/sandy/silty/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 or high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat, muck, and other highly organic soils

Prefix: G = Gravel, S = Sand, M = Silt, C = Clay, O = Organic **Suffix:** W = Well Graded, P = Poorly Graded, M = Silty, L = Clay, LL < 50%, H = Clay, LL > 50%

DEGREE OF PLASTICITY OF COHESIVE SOILS

Degree of Plasticity	Plasticity Index	Swell Potential
None	0 to 4	Very Low
Slight	5 to 9	Low
Medium	10 to 19	Low to Medium
High	20 to 39	Medium to High
Very High	40+	Very High

MOISTURE CONDITION OF COHESIVE SOILS

Description	Condition	Moisture Content
Absence of moisture, dusty, dry to touch	Dry	0 to 10%
Damp but no visible water	Moist	10 to 30%
Visible free water	Wet	30 to 70%

COHESIVE SOILS

CONSISTENCY	SPT	Qu – (tsf)
Very Soft	<2	0.00 – 0.25
Soft	2 to 4	0.25 – 0.50
Medium Stiff	5 to 8	0.50 – 1.00
Stiff	9 to 14	1.00 – 2.00
Very Stiff	15 to 30	2.00 – 4.00
Hard	31+	4.00+

COHESIONLESS SOILS

RELATIVE DENSITY	SPT
Very Loose	<4
Loose	4 to 10
Medium Dense	11 to 30
Dense	31 to 50
Very Dense	51+

QUALITY OF ROCK CORE

CORE QUALITY	R.Q.D.	CONDITIONS
Excellent Quality	90 – 100%	Unweathered
Good Quality	75 – 90%	Slightly Weathered
Fair Quality	50 – 75%	Moderately Weathered
Poor Quality	25 – 50%	Highly Weathered
Very Poor Quality	<25%	Completely Weathered

PARTICAL SIZE

DESCRIPTION	SIZE
Boulders	11.81 in.
Cobbles	2.95 in.
Gravel	0.19 in.
Course Sand	0.08 in.
Medium Sand	0.02 in.
Fine Sand	0.003 in.
Silt	0.0002 in.