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PHASE 2
GEOTECHNICAL FIELD AND LABORATORY TEST PROGRAM
PIÑON RIDGE PROJECT
MONTROSE COUNTY, COLORADO

Submitted to:

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1.0 INTRODUCTION

Energy Fuels Resources Corporation (EFRC) is in the process of designing and licensing a new conventional uranium mill, the Piñon Ridge Uranium Mill, located in Montrose County, Colorado. The site is located in the Paradox Valley, approximately 15 miles northwest of the town of Naturita on Highway 90.

Previously, Kleinfelder West Inc. (Kleinfelder) conducted a Phase 1 geotechnical investigation at the site to develop an initial geotechnical characterization of the property. Subsequently, Phase 2 geotechnical investigations were conducted, a joint effort between Golder Associates Inc. (Golder) and Kleinfelder, to support geotechnical designs of the tailings cells, evaporation ponds, ore stockpiles, mill, roads, surface water controls, borrow sources, and facility closure. These geotechnical designs will be incorporated into the Environmental Report (ER) that will be submitted by EFRC to the Colorado Department of Public Health and Environment (CDPHE).

The facilities to be included in the ER are as follows: (1) three tailings cells with areal extents of approximately 30 acres each; (2) a series of evaporation ponds with an initial aerial extent of approximately 40 acres, with expansion capacity to an ultimate aerial extent of approximately 80 acres; and (3) ore stockpiles adjacent to the mill comprised of a concrete-lined ore pad of approximately 1 acre in size in contact with an approximately 5–acre lined ore pad. Golder has prepared this report to summarize results from Golder's portion of the Phase 2 geotechnical field program.

2.0 PHASE 2 FIELD EXPLORATION PROGRAM

Golder conducted a geotechnical investigation (Phase 2) to evaluate the subsurface conditions present in the proposed areas for the evaporation ponds, tailings cells, and ore pads at the Piñon Ridge Project. Kleinfelder conducted a similar investigation for the proposed mill area as part of the Phase 2 field work, and had previously conducted the Phase 1 portion of the geotechnical investigation as part of the baseline studies. Prior to the Phase 1 program, no geotechnical investigation had been conducted on the Piñon Ridge Project site. The results from Kleinfelder's Phase 1 and Phase 2 investigation are presented separately.

Golder's Phase 2 geotechnical investigation program consisted of drilling 48 drill holes (GA-BH-01 through GA-BH-48) and excavating 11 test pits (GA-TP-01 through GA-TP-11). The locations of the drill holes and test pits are shown on Drawing 1. Golder's work plan for the Phase 2 field investigation program is provided in Appendix A.

This section summarizes the results from Golder's portion of the Phase 2 field exploration program for the Piñon Ridge Project site. Results from the Phase 2 field exploration program, including drill hole logs, core photos, packer test data, and test pit logs are included in Appendix B.

2.1 Drill Hole Exploration Program – Equipment & Methods

Geotechnical drill holes were advanced by Dakota Drilling (Dakota) of Denver, Colorado from October 23 through December 15, 2007. Forty-eight drill holes were completed, including 17 in the proposed evaporation pond area, 26 in the proposed tailings cell areas, 5 in proposed ore pad areas. The boundaries for the proposed areas mentioned above and illustrated on Drawing 1 were approximations at the time of drilling in advance of design layouts. Drill hole logs and core photos are presented in Appendix B-1 and Appendix B-2, respectively.

Geotechnical samples in the form of Shelby tubes, Modified California Barrel (MCB) drive samples, and Standard Penetration Test (SPT) split spoon samples were obtained while drilling. Typically, MCB samples were collected every 5 feet. At the onsite engineer's discretion, some MCB samples were replaced with Shelby Tube or SPT samples.

The MCB sampler consists of a 3-inch outside diameter (O.D.), 2.4-inch inside diameter (I.D.) split barrel shaft that is driven up to 16 inches into the soil at the base of the borehole. A combination of

3- and 4-inch long, 2.4-inch diameter rings are located inside the split barrel shaft and are used to retain soil for laboratory tests as well as field soil classification.

The SPT split spoon sampler consists of a 2-inch O.D., 1.4-inch I.D. split barrel shaft that is driven up to 24 inches into the soil at the base of the borehole (per ASTM D1586). Similar to the MCB sampler, soil is retained inside the split barrel shaft (without rings or liners), is visually classified, and then retained in a bag for laboratory testing.

Both the MCB and SPT samplers were driven into the soil using a 140-pound hammer free-falling a vertical distance of 30 inches. All Dakota drill rigs used on this project were equipped with manually-operated hammers using a rope and rotating cathead. The total number of hammer blows required to drive the sampler the final 12 inches is termed the "blow count." The procedures employed in the field are generally consistent with those described in ASTM D1586.

Shelby tubes were used to collect relatively undisturbed samples of the soils encountered. The thin-walled tubes were pushed into the soil hydraulically and sealed on each end of the tube with plastic caps to maintain the in-situ moisture content of the soil sample. Thin walled sampling was performed in general accordance with ASTM D1587. The Shelby tubes used for this field investigation were 30 inches long and 3 inches in diameter. Tubes were pushed the full 30 inches unless refusal was met due to either dense soil or the presence of gravel and cobbles.

While coring, packer tests were conducted at intervals selected by the field engineer to measure the in-situ horizontal permeability of the bedrock units encountered. The packer test method is used for testing rock in which the walls of the open borehole are stable. Inflatable packers are generally used to isolate the interval of the core hole to be tested. Single packer tests were performed by sealing off the core hole at a specified depth, and injecting water into the section of the hole between the packer and the base of the core hole. Typically, single-packer testing is conducted as the drill hole is advanced. As water is injected into the testing interval, the flow into the formation is measured along with the constant water pressure being applied. Based on these measurements, the horizontal permeability is calculated. The location of the packer tests are listed in the 'remarks' column of the drill hole logs, and the packer test data reduction calculations are presented in Appendix B-3.

Seventeen drill holes (GA-BH-01 through GA-BH-17) were completed in the proposed evaporation pond area using either a CME-55 drill rig or a Diedrich-50 drill rig with hollow-stem auger (HSA).

Drill hole depths in the evaporation pond area ranged from 25 to 70 feet below ground surface. Drilling in these explorations typically stopped at the lesser of 50 or 70 feet (per the exploration work plan, see Appendix A), or when bedrock was encountered.

Of the 26 drill holes completed in the proposed tailings cell areas (GA-BH-18 through GA-BH-43), 14 were drilled to the bedrock contact using HSA and sampling as described for the proposed evaporation pond area drill holes. The remaining 12 drill holes (GA-BH-21, GA-BH-22, GA-BH-24, GA-BH-26, GA-BH-27, GA-BH-29, GA-BH-31, GA-BH-33, GA-BH-36, GA-BH-38, GA-BH-39, and GA-BH-42) in the tailings cell areas were generally drilled with HSA until bedrock was reached, followed by NX wireline coring to a total depth of 100 feet. Sampling during the HSA portion of these holes was performed as per the evaporation pond area holes, and continuous core was collected during the NX wireline coring method. The 12 core holes listed above as well as GA-BH-18 were augered and cored using a Diedrich-120 drill rig. Continuous coring techniques were used, and all recovered core was shipped to Golder's geotechnical testing laboratory in Lakewood, Colorado.

The five drillholes (GA-BH-44 through GA-BH-48) advanced in the proposed ore pad areas were drilled and sampled in the same manner as those drilled in the proposed evaporation pond area. Drillhole depths ranged from 39 to 51 feet and were terminated prior to contacting bedrock.

With the exception of drill hole GA-BH-42, all drill holes were typically completed by backfilling with cuttings, with the final (upper) 6 to 12 inches sealed with bentonite chips. In drill hole GA-BH-42, a standpipe piezometer was installed and developed to determine if naturally occurring water existed within the formation. Subsequent monitoring of the piezometer in GA-BH-42 indicated a dry hole, consistent with all other drill holes that were part of this Phase 2 geotechnical investigation.

2.2 Drill Hole Exploration Program – Observations

This section describes the conditions encountered in the various facility areas based on the results of the geotechnical field exploration program. The geotechnical conditions encountered in the geotechnical exploration program were highly variable across the site, due to the structural geology of the Paradox Valley area derived from formation and subsequent collapse and erosion of an anticline.

2.2.1 Evaporation Pond Area

The geotechnical conditions encountered in drill holes GA-BH-1 through GA-BH-17 drilled in the evaporation pond area were highly variable, with shallow bedrock encountered at a depth of 7.0 feet in GA-BH-15, but not encountered in several borings (i.e., GA-BH-8, GA-BH-9, GA-BH-10, GA-BH-12, GA-BH-13, GA-BH-14, GA-BH-16, and GA-BH-17) with exploration depths ranging from 50 to 70 feet.

The overburden soils generally consist of windblown loess (i.e., ML, SM, CL) with occasional layers of alluvium (i.e., SM-GM). Bedrock encountered generally consists of claystone and gypsum of the Hermosa Formation, with claystone, siltstone and sandstone of the Cutler and Moenkopi Formations interpreted in some locations. Blowcounts in the overburden materials underlying the evaporation pond area ranged from 16 to refusal (i.e., greater than 50 blows per 6 inches).

2.2.2 Tailings Cell Areas

The geotechnical conditions encountered in drill holes GA-BH-18 through GA-BH-43 drilled in the tailings cell areas consisted of bedrock depths ranging from 13 feet (GA-BH-29) to 103 feet (GA-BH-31). Bedrock was not encountered in several borings (i.e., GA-BH-20, GA-BH-25, GA-BH-35, GA-BH-40, GA-BH-41, and GA-BH-43) at exploration depths ranging from 44 to 70 feet. The overburden soils generally consist of windblown loess (i.e., ML, SM, SW) with occasional layers of alluvium (i.e., GW, ML, SM). Bedrock encountered generally consists of claystone, shale, gypsum and anhydrite of the Hermosa Formation, with conglomerate and sandstone of the Cutler Formation and sandstone, claystone, and conglomerate of the Moenkopi Formation interpreted in some locations. Blowcounts in the overburden materials underlying the tailings cell areas ranged from 9 to refusal (i.e., greater than 50 blows per 6 inches).

Eight packer tests were attempted in drill holes drilled within the proposed tailings cell areas (i.e., GA-BH-24, GA-BH-26, GA-BH-33, GA-BH-36, GA-BH-38, and GA-BH-42). The Rock Quality Designation (RQD) of the tested intervals ranged from 0 to 92 percent (though generally less than 40 percent), indicating that the permeabilities obtained are largely a function of the rock structure (i.e., fractures and joints), and not representative of the intact rock. The three most prevalent bedrock types encountered while coring (i.e., gypsum, claystone, and sandstone) were tested. The horizontal bedrock permeabilities estimated using the packer test method are as follows: gypsum –

$2.0x10^{-7}$ centimeters per second (cm/sec) to $1.9x10^{-5}$ cm/sec; claystone – approximately $2.4x10^{-5}$ cm/sec; and sandstone – approximately $4.3x10^{-4}$ cm/sec.

2.2.3 Ore Pad Areas

Drill holes GA-BH-44 through GA-BH-48 were advanced in proposed ore pad areas to depths ranging from 39 to 51 feet below the ground surface. Bedrock was not encountered in any of these borings. Overburden soils generally consist of windblown loess and alluvium (i.e., ML, SM, CL, SC). Blowcounts in the overburden materials underlying the proposed ore pad areas ranged from 11 to refusal (i.e., greater than 50 blows per 6 inches).

2.3 Test Pit Exploration Program

The Phase 2 test pit investigation was conducted from November 1 to November 2, 2007. Using a Caterpillar Model 430 D backhoe, High Desert Construction excavated the test pits to depths ranging from 9.5 to 11 ft below the ground surface. Eleven (11) test pits were excavated, as follows: four test pits located in the proposed evaporation pond area (GA-TP-01 through GA-TP-04); five test pits located in the proposed tailings cell area (GA-TP-05 through GA-TP-8, and GA-TP-11); and two test pits in the proposed ore pad area (GA-TP-09 and GA-TP-10). Test pit locations are shown on Drawing 1, and test pit logs are included in Appendix B-4.

In general, test pits revealed little to no topsoil over silty sand (SM) foundation soils. Softer material was found in GA-TP-09 and GA-TP-10, near the southern edge of the site. Gravelly and cobbley layers were often encountered in the lower 5 feet of the test pit excavations.

3.0 PHASE 2 GEOTECHNICAL LABORATORY TESTING PROGRAM

Soil samples collected from the Phase 2 geotechnical field exploration program were transported to Golder's Lakewood, Colorado laboratory for the majority of the laboratory testing program. Geosynthetics samples (i.e., geosynthetic clay liner [GCL], geomembrane, and drainage geocomposite) and select soil samples were shipped to Golder's Atlanta, Georgia laboratory for interface shear testing. Select soil samples were sent to the University of Wisconsin-Madison for unsaturated hydraulic conductivity testing.

Soil samples were classified using the Unified Soil Classification System (USCS) (ASTM D2487). Laboratory test methods generally followed American Society for Testing and Materials (ASTM) procedures. Laboratory analyses conducted on the various soil or geosynthetic samples consisted of the following:

- Sieve Analysis – ASTM C117/C136;
- Hydrometer/Sieve/Specific Gravity – ASTM D422;
- Atterberg Limits – ASTM D4318;
- Natural Moisture Content – ASTM D2216;
- Natural Density and Moisture Content – ASTM D2937/D2216;
- Standard Proctor Compaction Testing – ASTM D698;
- Swell / Consolidation Testing – ASTM D4546, modified;
- Constant Head Flexible-Wall Permeability Testing – ASTM D5084;
- Consolidated-Undrained (CU) Triaxial Compression – ASTM D4767;
- Interface Shear Testing (ASTM D5321); and
- Unsaturated Hydraulic Conductivity Testing (ASTM D6836).

Laboratory test results for the Phase 2 testing program are presented in Appendix C. The test results are discussed below.

3.1 Index Testing

Sieve analyses were conducted on samples collected during the Phase 2 geotechnical field program. These analyses were conducted in order to evaluate the gradation of the in-situ materials and their potential for use in the construction of the proposed facilities. The sieve analyses indicate that the in-situ materials have gravel fractions ranging from 0 to 59 percent (with an average of approximately 5 percent), sand fractions ranging from 23 to 85 percent (with an average of approximately 56 percent), and fines fractions (passing the No. 200 sieve) ranging from 8 to 77 percent (with an average of approximately 39 percent).

Atterberg limits were conducted on the portion of select samples which passed the #200 sieve. Results from the Atterberg limits tests indicate that the fines portion of a majority of the samples are non-plastic (NP). Of the 77 samples tested for Atterberg limits, 29 exhibited some plasticity with liquid limits ranging from 19 to 37 and plasticity indices ranging from 4 to 21. Accordingly, the in-situ materials investigated generally classify as silty sand (SM) with occasional lenses of well-graded silty gravel (GW-GM), clayey sand (SC), and low plasticity silt (ML) and clay (CL). Although only one tested sample classified as gravel, several gravelly and cobbley layers were encountered while drilling. Sampling in these coarse-grained layers rarely returned significant recovery as the small diameter sampler barrels are not conducive to collecting samples of coarse-grained materials.

The natural moisture contents of the site soils ranged from 1.5 to 13.1 percent, with an average of 4.5 percent. The majority of the site soils had natural moisture contents below the plastic limit (PL), as shown in Table 1, indicating that the in-place soils are relatively dry and will require moisture conditioning to become workable for construction.

Index test results are presented in Appendix C-1, and summarized in Table 1. A summary of the gradations of in-situ soil materials is provided in Figure 1.

3.2 Natural Moisture-Density Testing

In-situ wet and dry unit weights were measured on 27 samples. The wet unit weight of the in-situ materials ranges from 86.4 to 144.4 pounds per cubic foot (pcf), with an average of 105.3 pcf. The dry unit weight of the in situ material ranges from 81.1 to 135.6 pcf, with an average of 100.7 pcf. Natural moisture contents measured on these soil samples ranged from 1.7 to 14.0 percent, with an average of 4.7 percent. The results of the natural moisture-density testing are presented in Table 2.

3.3 Compaction Testing

Standard Proctor compaction tests (ASTM D698) were performed on eight (8) bulk samples collected during the field program. The compaction analyses indicate that the surficial soils (within the upper 12 feet) at the project site have maximum dry densities ranging from 116.5 to 120.4 pcf, with an average dry density of 118.0 pcf. The optimum moisture content for these soils ranges from 11.3 to 13.1 percent, with an average value of 12.2 percent. Compaction test results are presented in Appendix C-2, and summarized in Table 1.

3.4 Swell/Consolidation Testing

One-dimensional swell/consolidation testing was performed on 12 in-situ soil samples and one remolded soil sample collected during the Phase 2 geotechnical field program. A bulk sample obtained from GA-TP-09 at a depth of 1 to 11 ft, classifying as silty sand (SM), was tested under remolded conditions, with the sample remolded to a dry density of 111.4 pcf at a moisture content of 12.9 percent.

Six of the in-situ soil samples classified as silty sand (SM); two as lean clay (CL); two as lean silt (ML); and two as lean clay and silt (CL-ML). The samples were incrementally consolidated to pressures ranging from 2.0 to 8.0 kips per square foot (ksf) based on the maximum anticipated loads on the respective samples. Samples were then saturated, allowed to collapse (or swell), and continued to be incrementally loaded to maximum pressures between 8.0 and 32.0 ksf. Samples were then unloaded in stages following the prescribed loading.

The results of these tests provided measured collapse potentials ranging from 3.9 to 16.7 percent for the in-situ soil samples. The measured collapse of the fine-grained soil samples (i.e., CL, ML, CL-ML) ranged from 5.1 to 16.7 percent, averaging about 9 percent. The collapse potential of the sandy soils (i.e., SM) ranged from 3.9 to 8.4 percent. The remolded sample exhibited a collapse potential of 1.7 percent. The collapse potential does not exhibit a clear trend with depth (e.g., decreasing). The swell/consolidation test results are presented in Appendix C-3 and summarized in Table C-3-1.

3.5 Permeability Testing

Flexible-wall permeability testing was conducted on 3 undisturbed soil samples and 13 remolded soil samples. The three undisturbed samples were derived from Shelby Tubes advanced in boreholes

GA-BH-08, GA-BH-42, and GA-BH-47 at approximate depths of 45, 10.5, and 3 feet, respectively. Bulk samples obtained during the test pit investigation were used for the remolded permeability testing. Permeability test results are presented in Appendix C-4.

The dry densities of the undisturbed soil samples ranged from 73.2 pcf to 89.9 pcf, with initial moisture contents ranging from 1.7 to 13.5 percent. The samples were tested under an effective stress of 10 or 15 pounds per square inch (psi). Based on the results of these permeability tests, the permeability of the in-situ soils ranges from 3.4×10^{-5} to 5.9×10^{-4} cm/sec. Results of the permeability testing on undisturbed samples are summarized in Tables C-4-1.

Flexible-wall permeability testing was conducted on two soil samples (GA-TP-02 and GA-TP-03) remolded to approximately 85 percent of the standard Proctor maximum dry density within 1.1 to 1.6 percent dry of the optimum moisture content to estimate the permeability of loosely-compacted cover soils. Four soil samples (GA-TP-04, GA-TP-05, GA-TP-07, and GA-TP-09) were remolded to approximately 95 to 100 percent of the standard Proctor maximum dry density at approximately the optimum moisture content, and tested to evaluate their use as compacted soil liner. Additional permeability testing was conducted by amending various soil samples with one to three percent by dry weight of bentonite to obtain an estimate of the resulting reduction in permeability.

Prior to compaction of the amended soil samples, Wyo-Ben Envirogel 200 bentonite was added (see Appendix D). The soil was thoroughly mixed, moisture-conditioned, and remolded to the required density.

Results of the permeability tests on remolded samples are summarized in Table C-4-2. The measured permeability of loosely compacted soils ranged from 7.0×10^{-5} to 8.2×10^{-5} cm/sec, decreasing by more than one order of magnitude by addition of three percent by dry weight of bentonite. Soil samples tested for potential use as soil liner (i.e., compacted to 95 percent or greater of the standard Proctor maximum dry density) exhibited permeabilities ranging from 4.9×10^{-6} to 1.3×10^{-4} cm/sec, decreasing by up to 1.5 orders of magnitude depending on the amount of bentonite added. The lowest measured permeability using added bentonite was 5.4×10^{-7} cm/sec, which does not meet the maximum permeability requirement of 1×10^{-7} cm/sec necessary to act as a compacted clay liner for the tailings cells. However, three percent bentonite amendment to the native soils does appear to meet the requirements of a soil liner with a less stringent permeability requirement of 1×10^{-6} cm/sec, which may be applicable to other site facilities.

3.6 Strength Testing

Two series of consolidated-undrained (CU) triaxial compression tests were conducted on remolded soil samples obtained from test pits GA-TP-9 (1 to 11 feet) and GA-TP-7 (1.5 to 9 feet). These samples were remolded to a target dry density of approximately 95 percent of the standard Proctor maximum dry density at the optimum moisture content.

Two series of CU triaxial compression tests were conducted on undisturbed Shelby tube soil samples obtained from drill holes GA-BH-42 (10 to 11 feet) and GA-BH-47 (2 to 3.5 feet). The initial dry unit weight of the undisturbed soil samples ranged from 83.8 to 89.9 pcf.

The sample obtained from GA-TP-9 was consolidated at pressures of 10, 40, and 60 pounds per square inch (psi), representative of stresses anticipated under a height of about 95 feet of tailings, assuming an average tailings total density of 90 pcf. The sample obtained from GA-TP-7 was consolidated to higher pressures of 25, 50, and 100 psi. The undisturbed soil samples were tested with confining pressures ranging from 10 to 80 psi.

Test samples were placed in a triaxial compression chamber, subjected to a confining pressure, and then loaded axially to failure. In the CU test, the test specimen is permitted to drain and consolidate under the confining pressure until the excess pore pressures dissipate. The axial stress is then slowly applied to failure, but drainage is not permitted.

For the remolded samples, the effective stress strength parameters yielded an effective friction angle of 30.3 to 35.5 degrees with zero to 2.0 psi effective cohesion. The total stress strength parameters for the remolded samples yielded a friction angle of 26.8 to 30.1 degrees with zero to 1.7 psi cohesion. The effective stress strength parameters for the undisturbed materials tested included an effective friction angle of 33.7 to 37.1 degrees with zero cohesion. The total stress strength parameters for the undisturbed samples yielded a friction angle of 29.0 to 31.1 degrees with zero cohesion. The results of strength testing are presented in Appendix C-5.

3.7 Interface Shear Strength Testing

Interface shear testing (ASTM D5321) was conducted to ensure stability of the proposed liner systems under the anticipated loading conditions. The focus of the interface shear testing program was on the liner systems for the tailings cells and the five-acre ore pad. Though double-lined, the

evaporation ponds will be shallow, and liner stability is not considered a major concern. Interface shear testing was conducted to evaluate the following interfaces:

- 60-mil textured high density polyethylene (HDPE) geomembrane manufactured by Poly-Flex Inc. (Poly-Flex) versus drainage geocomposite (representative of the primary or secondary geomembrane liner in the tailings cells in contact with the Leak Collection and Recovery System [LCRS] layer on the cell slopes);
- 60-mil textured HDPE geomembrane manufactured by Poly-Flex versus the following geosynthetic clay liners (GCL):
    - Bentomat CLT manufactured by CETCO Lining Technologies (CETCO), with clay side against the geomembrane (a liner system considered for the tailings cells secondary composite liner); and
    - Bentomat ST manufactured by CETCO, with woven side against the geomembrane (a liner system considered for the tailings cells secondary composite liner).
- GCL materials versus native compacted soil materials, as follows:
    - Bentomat CLT manufactured by CETCO, with laminated HDPE side against soil (a liner system considered for the five-acre ore pad); and
    - Bentomat DN manufactured by CETCO, with white nonwoven side against soil (a liner system considered for the five-acre ore pad, but also representative of the GCL adjacent to the subgrade for the tailings cells).

Data sheets on the various geosynthetic materials tested are included in Appendix D. A double-composite liner system is being considered for construction of the tailings cells and evaporation ponds, while a single liner system is being considered for the 5-acre ore pad.

It is important that the tests are conducted based on the anticipated conditions in the field, i.e., the test specimen should be representative of final construction by using the soils planned for underliner construction, as well as the specific geomembrane liner material planned for construction. Interface shear testing is used to evaluate the friction angle with respect to translational failure along the liner system (i.e., geomembrane / GCL; geomembrane / drainage geocomposite; or GCL / compacted soil interface). The following section presents the results of the interface shear testing as well as the sample construction, test procedures, and test results.

Soil Materials

Where applicable, soil materials were air dried prior to testing. After air drying, the soils were moisture treated to approximately the optimum moisture content as determined by a standard Proctor test performed for this material and allowed to hydrate. The soils were compacted to approximately 95 percent of the standard Proctor maximum dry density.

Test Procedure

The interface shear testing was performed in general accordance with the ASTM Standard Test Method D5321, "*Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method*." All tests were conducted at normal stresses of 20, 40, and 80 psi in a direct shear device containing an upper and lower shear box.

Testing Program

The interfaces tested were as described above. The interface shear strength test is performed by constructing the configuration in the laboratory, placing a normal load onto the configuration, and shearing the sample at a rate of 0.04 to 0.2 inches per minute (in/min). The tests involving use of soil materials were allowed to consolidate under the normal load for 12 hours prior to shearing, while the geosynthetic-only tests were consolidated only 15 minutes prior to shearing.

Test Results

Results of the interface shear tests are summarized as follows:

- Textured geomembrane versus drainage geocomposite: Peak friction angle of 21.2 degrees with residual friction angle of 14.8 degrees;
- Textured geomembrane versus GCL: Peak friction angles ranging from 20.0 to 23.2 degrees with residual friction angles ranging from 12.7 to 13.1 degrees;
- HDPE-laminated GCL versus soil: Peak friction angle of 25.1 degrees with residual friction angle of 13.3 degrees; and
- Reinforced GCL versus soil: Peak friction angle of 35.5 degrees with residual friction angle of 30.5 degrees.

The results of interface shear strength testing are presented in Appendix C-6.

3.8 Soil-Water Characteristic Curve Testing

Soil samples from test pits GA-TP-02, GA-TP-03, and GA-TP-09 were shipped to the University of Wisconsin-Madison (UW) laboratory for unsaturated/saturated hydraulic conductivity testing and development of soil-water characteristic curves (SWCC). The results of the SWCC testing program are included in Appendix C-7.

SWCC testing included the following tests and methods:

- Constant Head Flexible-Wall Permeability Testing (ASTM D5084); and
- Unsaturated Hydraulic Conductivity Testing (ASTM D6836 – Methods B and D).

The soil samples tested by UW were remolded to a target dry unit weight of 100 pcf at a water content of 10 percent. This unit weight was selected based on 85 percent of the average standard Proctor maximum dry density, which approximates the anticipated condition of the loose cover soils to be placed over the tailings cells at closure. The moisture content of 10 percent is approximately two percent drier than the average optimum moisture content obtained from standard Proctor tests.

Saturated hydraulic conductivity testing (ASTM D5084) conducted by UW resulted in measured soil permeability values ranging from 2.7×10^{-5} to 1.8×10^{-4} cm/sec, tested at an effective stress of 5 psi. The saturated permeabilities measured by UW were comparable to those obtained from Golder's remolded permeability testing (see Section 3.5).

UW conducted soil-water characteristic curve testing (ASTM D6836) on the soil samples using the pressure plate extractor (Method B) and chilled mirror hygrometer (Method D) methods. The measured soil-water characteristics curves were used to determine van Genuchten modeling parameters. Plots of moisture content versus negative pore water pressure (i.e., suction), also called SWCCs, are presented in the report by UW included as Appendix C-7. These plots can be used to estimate infiltration rates during design of the tailings cell closure cover.

4.0 LIQUEFACTION ASSESSMENT

The potential for liquefaction of the Piñon Ridge foundation overburden soils was evaluated using data obtained from Standard Penetration Tests (SPT) conducted as part of the Phase 2 geotechnical field investigation. The liquefaction analyses are presented in Appendix E.

Liquefaction is defined as a reduction in the shear strength of a saturated or nearly saturated soil when the pore pressure builds up to the magnitude of the overburden pressure, and as a result the effective stress is reduced to zero. During a seismic event, an increase in pore pressure can occur when the soil is subjected to cyclic shear stresses induced by the ground motions. When the soil pore pressures increase, the effective stress in the soil decreases; thereby reducing the soil shear strength, resulting in flow failures and large deformations. Liquefaction generally occurs most often in saturated, loose to moderately dense granular soils with poor drainage, such as silty sands or sands and gravels (Youd et al., 2001). The consequences of liquefaction can include global instability or flow failures, ground loss, settlement, and localized differential movements.

The design ground motions for the Piñon Ridge Project site were identified by Kleinfelder (2008), including a moment magnitude **M** 4.8 earthquake occurring at a distance of 15.5 kilometers (km) from the site. The design peak ground acceleration (PGA) is 0.11g. The Maximum Credible Earthquake (MCE) event corresponds to a PGA of 0.16g. These values were derived from the International Building Code (IBC).

The results of the field investigation indicate that the overburden soils are unsaturated and therefore not prone to liquefaction. The current phreatic levels (i.e., deep groundwater) are assumed to be representative of phreatic conditions during operations and post-closure of the mill. Therefore, these materials are unlikely to become saturated.

The selected method to assess liquefaction potential, conservatively assuming saturation of the alluvial soils, is based on the Simplified Method using correlations to blow counts from SPTs as set forth in Youd et al. (2001) and the design seismic event. The factor of safety against liquefaction for the site soils was generally evaluated to be 10 or greater under both the design earthquake and MCE events. The minimum factor of safety against liquefaction under the design earthquake and MCE events were evaluated to be 4.3 and 2.9, respectively, in drill hole GA-BH-36 at a depth of 10 feet where the unadjusted blow count was 9. Therefore, the liquefaction analyses indicate that

liquefaction of the overburden soils is not predicted for any earthquake event that may be expected at the site, up to the MCE, even in the event of saturation.

5.0 USE OF THIS REPORT

This report has been prepared exclusively for the use of Energy Fuels Resources Corporation (EFRC) for the specific application to the Piñon Ridge Project. The engineering analyses reported herein were performed in accordance with accepted engineering practices. No third-party engineer or consultant shall be entitled to rely on any of the information, conclusions, or opinions contained in this report without the written approval of Golder and EFRC.

The site investigation reported herein was performed in general accordance with generally accepted Standard of Care practices for this level of investigation. It should be noted that special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions. Even a comprehensive sampling and testing program implemented in accordance with a professional Standard of Care may fail to detect certain subsurface conditions. As a result, variability in subsurface conditions should be anticipated and it is recommended that a contingency for unanticipated conditions be included in budgets and schedules.

Golder sincerely appreciates the opportunity to support EFRC on the Piñon Ridge Project. Please contact the undersigned with any questions or comments on the information contained in this report.

Respectfully submitted,

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KFM/dls

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