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October 31, 2008
File 89241-2

Ms. Kimberly Finke-Morrison
Golder Associates, Inc.
44 Union Boulevard, Suite 300
Lakewood, Colorado 80228

Subject: Geotechnical Design Recommendations
Mill and Infrastructure
Piñon Ridge Project
Montrose County, Colorado

Dear Ms. Morrison,

Kleinfelder West, Inc. (Kleinfelder) is pleased to present geotechnical design recommendations for the planned mill and infrastructure facility as part of the Piñon Ridge Project. Kleinfelder's work consisted of subsurface exploration, laboratory testing, design team meetings, engineering analyses, and preparation of this report.

Kleinfelder appreciates this opportunity to be of service to you, and look forward to future endeavors. If you have any questions regarding this report or need additional information or services, please contact Kleinfelder's Golden office.

Respectfully submitted,

KLEINFELDER WEST, INC.

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10/31/08
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Cc: Energy Fuels Resources Corporation
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DHA/ADT/jw

Enclosures

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GEOTECHNICAL DESIGN RECOMMENDATIONS
MILL AND INFRASTRUCTURE
PIÑON RIDGE PROJECT
MONTROSE COUNTY, COLORADO

Prepared By:

10/31/08

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October 31, 2008

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1 INTRODUCTION

1.1 GENERAL

This report presents geotechnical design recommendations for the mill and infrastructure of the Piñon Ridge Project near Bedrock, Colorado. The Vicinity Map (Figure A-1, Appendix A) shows the location of the project. Kleinfelder performed this investigation as a subconsultant to Golder Associates, Inc.

The report includes Kleinfelder's recommendations relating to the geotechnical aspects of project design and construction. The conclusions and recommendations stated in this report are based on the subsurface conditions found at the locations of Kleinfelder's exploratory borings at the time Kleinfelder's exploration was performed. They also are subject to the provisions stated in the report section titled **Limitations**. Kleinfelder's findings, conclusions, and recommendations should not be extrapolated to other areas or used for other projects without Kleinfelder's prior review. Furthermore, they should not be used if the site has been altered, or if a prolonged period has elapsed since the date of the report, without Kleinfelder's prior review to determine if they remain valid.

1.2 PROJECT DESCRIPTION

Energy Fuels Resources Corporation (EFR) proposes to construct and operate a conventional acid leach uranium mill at the Piñon Ridge Mill site. Project improvements will consist of a 12-acre mill site, tailings cells, evaporation ponds, and ore stockpile pads. The mill will be designed for a production capacity of 1,000 tons of ore per day and will be licensed initially for production up to 500 tons per day. The expected operating life of the mill is 20 to 40 years. The mill site, which will occupy an area roughly 850 feet by 850 feet, is shown on Figure A-2. The mill will include the following main elements:

- **SAG Mill** – The SAG mill will be contained in a metal building with a peaked roof with an overall height of about 100 feet. Ore will be conveyed to the mill via a feed hopper structure constructed about 25 feet below finished grade and east of the mill. The building will include a 14-foot diameter grinder and leach tank circuit. A 25-ton bridge crane will span the width of the 100-foot wide building. The building will contain numerous vessels, hoppers, and pumps. A concrete finished floor elevation of 5547 feet is planned. Elevated pulp storage and pre-leach tanks will be located immediately west of the building. Cone bottom thickeners supported on a concrete pad are to the north of the building. Tailings will move to the tailings cells through two, 8-inch diameter HDPE pipes in a concrete-lined trench about 3 feet deep.

- **Solvent Extraction** – Solvent extraction will occur in an approximate 140-foot by 370-foot clear span metal building with a peaked roof. The building will contain multiple mixers measuring 12 feet by 36 feet and 4 feet high, and numerous above ground storage tanks. A concrete finished floor elevation of 5544 feet is planned.
- **Precipitation/Packaging Warehouse** – This will be a metal building having plan dimensions on the order of 150 feet by 250 feet, with a 40 to 100-foot high peaked roof. The precipitation portion of the building will have filters, pumps, and tanks. The packaging area is split into vanadium and uranium areas and will contain dryers, filters, conveyors, and drum storage. Two recessed truck docks are planned along the west side of the building. A concrete finished floor at elevation 5546 feet is planned. Storage, process, and return water tanks are located north of the building.
- **South Perimeter** – Facilities along the south perimeter of the mill include an electrical substation, 125-foot by 175-foot laboratory and change room building, 100-foot square warehouse building, reagent facility with tanker truck unloading concrete apron, and gravel-surfaced truck access drives and parking areas.
- **West Perimeter** – Facilities along the west perimeter of the mill include a 75-foot square truck maintenance building, gravel-surfaced truck access apron, and above ground storage tanks for propane, ammonia, and kerosene.
- **East Perimeter** – Sulfuric acid storage tanks and pumps and a diesel fuel oil storage tank with pump are planned south of the feed hopper along the east perimeter.
- **Pipe Rack Corridors** – The mill facilities will be connected with extensive piping contained within defined corridors. The pipes will be supported above ground on pipe racks. These racks may be up to 40 feet above ground.

Estimated structure loads for the various mill elements were provided by the mill designer, CH2M Hill, in an email dated May 6, 2008 and are included in the following table.

ESTIMATED STRUCTURE LOADS

DESCRIPTION	SUPPORT REACTIONS	FOUNDATION TYPE IN ESTIMATE
SX BUILDING		
SX Building Foundation	P=172 kips V= 84 kips	Cont. wall footing with 10' x 10' spread footing & pedestal at long. wall columns
Tanks 610-TKL-01,02,03	P= 80 kips	Elevated tanks in steel structure; highest column load 80 kips. Est. 5' x 5' spread footing
Tanks 610-TKL-04/05 & TKH-01	Oper. Wt=596 kips	Circular cone bottom tank with 10 legs. Circular spread footing 26' mean dia. x 3'-6" wide & 10 pedestals.
MILL & LEACH BUILDING		
Mill & Leach Bldg. Foundation	P= 140 kips V= 50 kips	Cont. wall footing with 6' x 6', 7' x 7', & 10' x 10' spread footings & pedestal at wall columns
Leach Tanks		Elevated tanks in steel structure; est. highest column load 345 kips.
SAG Mill Foundation	P= 743 kips V=204 kips (seismic)	Mass Mat Foundation- 50' x 35' x 4'-9"
WAREHOUSE BUILDING		
Warehouse Bldg. Foundation	P=60 kips V=28 kips	Cont. wall footing with 7' x 7 & 5' x 5' ' spread footing & pedestal at wall columns.

ESTIMATED STRUCTURE LOADS - Continued

DESCRIPTION	SUPPORT REACTIONS	FOUNDATION TYPE IN ESTIMATE
PRODUCT PACKAGING BLDG.		
Product Package Bldg	P= 51 kips V=11 kips	Cont. wall footing with 7' x 7 & 6' x 6' & 9' x 9' spread footing & pedestal at wall columns.
MISC. STRUCTURES		
Pre-Leach Clarifier		Elevated tanks in steel structure; est. highest factored column load 146 kips. Est. 6'-8" sq. spread footing
Pre-Leach Thickener		Column loads= 224 kips + 183 kips. For estimate used 9 combined trapezoidal footings 20'-2" long x 8'-0' x 4'-9"
CCD Thickeners		For est. interior columns bear on 8'-6" octagon foundation and exterior columns bear on ring fdn. @ approx. 3 ksf.
Pipe Rack	P= 66 kips V= 2.5 kips	5' x 5' spread footings

Most of the equipment will be structurally supported independent of the building floor slabs. Some equipment such as propane and ammonia storage tanks will be pad supported.

Grading to develop the mill pad area will be moderate consisting of cut along the south access road and substation to a maximum depth of 14 feet at the substation and fills to

13 and 17 feet at the north end of the solvent extraction building and thickeners, respectively. The majority of the pad will be filled and graded to slope to drain to the northwest where the west stormwater pond will collect most mill area runoff. The east stormwater pond will collect runoff from the ore pad and the extreme eastern side of the mill area. The mill pad drainage and grading plan designed by Kleinfelder is presented on Figure A-3. Grading around the individual structures and features within the mill will be provided by the mill designer.

1.3 PURPOSE AND SCOPE

The purpose of Kleinfelder's investigation was to explore and evaluate subsurface conditions at various locations within the general mill area as part of the Phase 2 geotechnical investigations for the project and, based on the conditions found, develop recommendations relating to the geotechnical aspects of mill design and construction. Kleinfelder's conclusions and recommendations in this report are based on analysis of the data from Kleinfelder's field exploration and laboratory tests, Kleinfelder's experience with similar soil and geologic conditions in the area, and discussions with the design team and EFR. Kleinfelder also evaluated field and laboratory data from the Phase 1 baseline characterization investigation conducted by Kleinfelder as part of the initial geotechnical characterization of the property as well as field and laboratory data from Golder Associates conducted during Phase 2 geotechnical investigations for the tailings cells, evaporation ponds, and ore stockpile pads [1].

Kleinfelder's scope of services included:

- A subsurface exploration program consisting of 20 exploratory borings drilled at the approximately locations designated on Figure A-2.
- Laboratory testing performed on selected samples obtained during exploration to evaluate pertinent engineering properties including moisture content, dry density, shear strength, swell/settlement, gradation analysis, Proctor, pH, water-soluble sulfates, and electrical resistivity.
- Evaluation and engineering analysis of the field and laboratory data to develop Kleinfelder's geotechnical conclusions and recommendations.
- Preparation of this report, which includes a description of the proposed project, a description of the surface and subsurface site conditions found during Kleinfelder's

investigation, Kleinfelder's conclusions and recommendations as to foundation design and related geotechnical issues, and appendices which summarize Kleinfelder's field and laboratory investigations.

- Percolation testing was conducted for absorption field sizing. A report dated July 2, 2008 was issued for Absorption Field Septic System Design.

2 FIELD EXPLORATION AND LABORATORY TESTING

2.1 FIELD EXPLORATION

A field exploration performed between November 2 and 6, 2007 included drilling 20 exploratory borings at the approximate locations indicated on the Boring Location Plan (Figure A-2) to the maximum depth drilled of 82 feet. All borings were advanced using a truck-mounted Dietrich 50 drill rig equipped with continuous-flight, hollow-stem auger. Drive samples were obtained during exploration using either a modified California sampler (2.5-inch I.D.) or standard split-spoon sampler (1.375-inch I.D.) driven into the strata with blows from a 140-pound hammer falling through a 30-inch drop in substantial accordance with local practice. The blows required to drive the sampler in six-inch increments were recorded. This blow count is an indication of the relative density or consistency of the strata.

Appendix B to this report includes logs describing the subsurface conditions. A legend to the boring logs including a summary of the Unified Soil Classification System used to describe the soils is presented at the front of the appendix. The logs of the borings are shown in profile at the end of the appendix. The lines defining boundaries between soil types on the logs are based on drill behavior and interpolation between samples, and are therefore approximate. Transition between soil types may be abrupt or gradual.

2.2 LABORATORY TESTING

Laboratory tests were performed on selected soil samples to estimate general engineering properties. The following tests were performed in general accordance with local practice and recognized standards-setting bodies:

Classification of Soils for Engineering Purposes
Unit Weight and Moisture Determination
Sieve Analysis of Fine and Coarse Aggregates
Liquid Limit, Plastic Limit, and Plasticity Index of Soils
One Dimensional Swell-Settlement
Moisture-Density Relationship Determination (Modified Proctor)
Direct Shear Strength

Water Soluble Sulfates
pH
Electrical Resistivity

Selected results of the laboratory tests are shown on the boring logs in Appendix B and presented graphically in Appendix C. Test results are also tabulated in Table C-1 at the beginning of Appendix C.

3 SITE CONDITIONS

3.1 SURFACE

The Piñon Ridge Project occupies 880 acres south of State Highway 90 in the Paradox Valley, Montrose County, Colorado. The project site is approximately 14 miles northwest of Naturita, Colorado. The legal description is the Southwest ¼ of the Southeast ¼ of Section 5, all of Section 8, the North ¼ of Section 17, and the Southeast ¼ of the Northwest ¼ of Section 17, Township 46 North, Range 17 West, of the New Mexico Principle Base and Meridian. The project site is located on both the Davis Mesa Quadrangle and Bull Canyon Quadrangle 1:24,000 United States Geological Survey (USGS) topographic/geologic maps.

The mill site is situated at the south end of the project parcel between about elevations 5530 and 5562 feet. The ground surface topography slopes gently down to the north. Vegetative cover consists of sparse grasses and sagebrush.

3.2 GEOLOGIC SETTING

The site is located within the Canyonlands Section of the Colorado Plateau Physiographic Province[10], which is characterized by mesas, plateaus, deep canyons, pediments, barren badlands and mostly arid climatic conditions[11]. The site is situated along the south side of the eastern end of Paradox Valley. Paradox Valley is cut into a well developed, faulted anticline with a diapiric salt and gypsum core.

Bedrock found in boreholes consisted of gypsum, anhydrite and shale of the Pennsylvanian Hermosa formation and inter-bedded siltstone and sandstone of the Chinle formation. The majority of the site is covered by Holocene deposits of sand, silt, and clay which may be up to 80 feet thick. Alluvial stream deposits occur along ephemeral stream or wash channels.

3.3 GEOLOGIC HAZARDS

No geologic hazards were identified that would significantly impact site development. The natural sloping terrain appeared to be stable and no evidence was found to indicate potential future instability. No evidence was found to indicate that ground subsidence has occurred on the site.

Faults mapped in the southwest corner of the project site during Phase 1 investigations trend to the northwest, appear to be normal, and may be down-thrown to the northeast or southwest. Based on seismic reflection and refraction surveys conducted in Phase 1, this same general fault trend is observed in the covered bedrock surface. Three trenches excavated at the site did not find evidence of fault offset and Kleinfelder believes the mapped faults are not active. Kleinfelder does not believe fault rupture or liquefaction are credible hazards at the site. Seismic design parameters based on the 2006 International Building Code (IBC) are presented in Appendix F[12].

The site soils are susceptible to erosion. All permanent cut and fill slopes should be re-vegetated or protected by other means. Surface drainage is designed to divert runoff from slopes or direct runoff into properly designed drainage features.

3.4 SUBSURFACE

The general subsurface profile encountered in Kleinfelder's borings consisted of an upper layer about 15 to 20 feet thick comprised of silty to clayey sand with sandy clay and silt lenses overlying predominately silty sand extending to bedrock encountered in one boring at a depth of 78 feet. No groundwater was encountered during Kleinfelder's investigation. The subsurface profile is discussed in greater detail in the following sections.

3.4.1 Overburden

The upper 15 to 20 feet of overburden soil is a variable combination of silty sand (SM), clayey sand (SC), sandy clay and silt (CL/ML), and sandy clay (CL). The sands have a loose to medium dense relative density based on field penetration resistance testing, while the finer grained clay/silt has a stiff to very stiff consistency. Particle size gradation characteristics typical of the upper soil layer are presented in Appendix C, Figures C-1 through C-10.

A porous soil structure was observed in several of the samples taken within the upper layer. Extensive swell-settlement testing was conducted. Samples were inundated with water under a 1,000 psf surcharge pressure. This pressure was selected to better simulate foundation-induced loads and to allow direct data comparison. Test data indicates the collapse potential is greatest near the ground surface (approximately upper 8 feet), then reducing steadily with depth. The material in this upper layer typically exhibits high dry strength under initial loading, but collapses when wetted and rapidly loses strength with continued loading.

The upper layer of soil was generally about 4 to 6 percent below the optimum moisture content as determined by modified Proctor tests. Moisture density relationship determinations by modified Proctor are presented on Figures C-32 and C-33.

Direct shear tests were conducted on samples of clayey sand remolded to approximately 94 percent of maximum modified Proctor dry density near optimum moisture content. Test results, shown on Figures C-34 and C-35; indicate an ultimate angle of internal friction of about 23 degrees with cohesion of 360 pounds per square foot (psf) at the maximum strain tested for a sample with moderate plasticity and 37 degrees with no cohesion for a sample with low plasticity.

The overburden soils transition to predominantly silty sand below the upper 15 to 20-foot layer. The sands have medium dense to dense relative density. Cemented layers were logged in 13 of the 20 borings, most commonly at depths from about 20 to 30 feet, with some cemented layers extending to nearly 50 feet. The cementation appears to be discontinuous and in some borings formed gravel size particles. Practical drill rig refusal was met in cemented material in borings TB-12, TB-14, and TB-16.

Gravel was identified in borings TB-1 and TB-2 in the northwest corner of the mill site at depths of 37 and 21 feet, respectively. Practical drill rig refusal was met in the gravel in both of these borings.

3.4.2 Bedrock

Sandstone bedrock was encountered in boring TB-18 along the south mill site boundary at a depth of 78 feet. Groundwater monitoring wells MW 5 and MW 6, installed by Kleinfelder for the Phase 1 investigation at the south end of the site, measured bedrock at depths of 80 and 85 feet, respectively. The bedrock in these borings was logged as siltstone and sandstone. Golder borings GA-BH-39 and GA-BH-42 located north of the mill site identified siltstone at 55 feet and claystone at 56 feet, respectively.

3.4.3 Groundwater

Groundwater was not encountered in the mill site geotechnical borings during drilling or when checked at the completion of all drilling. Groundwater was encountered in monitoring well MW 6 at a depth of approximately 407 feet and in MW 5 at a depth of 282 feet during the Phase 1 baseline investigation. Soil moisture levels and shallow groundwater levels commonly vary over time depending upon seasonal precipitation, irrigation practices, land use, and runoff conditions. Deep groundwater levels are less influenced by these factors. Considering the intended land use, low seasonal precipitation, no irrigated features, and carefully designed drainage within the mill area, Kleinfelder believes development of shallow groundwater at the mill site is unlikely.

4 CONCLUSIONS AND RECOMMENDATIONS

4.1 GEOTECHNICAL CONSIDERATIONS

Kleinfelder found no subsurface conditions during this investigation that would preclude development of the site essentially as conceived, provided the recommendations in this report are incorporated into the project design. The presence of collapsible soils within the upper layer of overburden soils is the main geotechnical characteristic affecting mill design.

Kleinfelder believes the collapse potential can be mitigated by removal of some of the collapsible soils and creating a relatively impermeable compacted soil layer beneath the facility. This can be accomplished by moisture conditioning the on-site soils and replacement as compacted engineered fill. Conventional spread footing foundations placed on the compacted engineered fill could then be used for support of mill structures, equipment and slabs (Section 4.4.1).

Several foundation types and ground improvement techniques were evaluated for foundation support as an alternative to conventional spread footing foundations supported on compacted engineered fill. Augered Pressure Grouted (APG) piles and Rammed Aggregate Piers (Geopiers) have been reviewed and appear to be feasible foundation alternatives. APG piles would bear in the dense sand well below the upper collapsible layer. Structural loads from buildings, equipment, mats supporting equipment and pipe racks would be supported by APG piles. Building floor slabs would remain susceptible to movement from the collapsible soil layer and would require a layer of compacted engineered fill beneath slabs.

The geopier system is a proprietary system that involves drilling 24 to 36-inch diameter holes to depths ranging from 10 to 25 feet and filling the holes with aggregate rammed in place with specialized equipment. The piers would be installed at variable spacing, but would likely be required on narrow spacing due to the collapse potential downdrag load.

Kleinfelder also reviewed deep dynamic compaction, soil lime amendment, and pre-wetting as alternatives for mitigation of the collapse potential. None of these options appear appropriate for the existing subsurface conditions.

- Because of the high dry strength in the upper soil layer, Kleinfelder does not believe deep dynamic compaction would be effective to an adequate depth and would require repeated applications and field testing.
- Lime amendment is not applicable to stabilization of sandy soils, has seasonal construction limitations, and requires specialty contractors.
- It is Kleinfelder's experience that surface pre-wetting is only effective to shallow depths in these types of soils and would require injection wells at close spacing in order to pre-wet to sufficient depth. This would be costly, require significant water, reduce soil bearing capacity, and require field testing.

4.2 CONSTRUCTION CONSIDERATIONS

4.2.1 General

All site preparation and earthwork operations should be performed in accordance with applicable codes, safety regulations and other local, State or federal guidelines. All references to maximum dry densities are established in accordance with standard Proctor, ASTM D-698, unless noted otherwise. Laboratory testing presented in this report was conducted using modified Proctor, ASTM D-1557 procedures; however, subsequent to testing the design team decided to use standard Proctor methods in order to standardize the Quality Control/Quality Assurance testing methodology throughout the project.

4.2.2 Excavation Considerations

Excavation of the overburden soils should be possible with conventional heavy earthmoving equipment to the depths anticipated. The feed hopper will require a cut of about 25 feet. Below depths of approximately 20 feet, areas of cemented material were encountered that could affect excavation and require ripping in the feed hopper area.

All excavations must comply with the applicable local, State, and federal safety regulations, and particularly with the excavation standards of the Occupational Safety and Health Administration (OSHA)[13]. Construction site safety, including excavation safety, is the sole responsibility of the Contractor as part of its overall responsibility for the means, methods, and sequencing of construction operations. Kleinfelder's recommendations for excavation support are intended for the Client's use in planning the project, and in no way relieve the Contractor of its responsibility to construct, support, and maintain safe slopes. Under no circumstances should the following recommendations be interpreted to mean that Kleinfelder is assuming responsibility for either construction site safety or the Contractor's activities.

Kleinfelder believes the majority of the overburden soils on this site will classify as Type C material using OSHA criteria. OSHA requires that unsupported cuts be no steeper than 1½:1 (horizontal to vertical) for Type C material for unbraced excavations up to 20 feet in height. In general, Kleinfelder believes these slope inclinations will be temporarily stable under unsaturated conditions. Please note that an OSHA-qualified "competent person" must make the actual determination of soil type and allowable sloping in the field.

4.2.3 Cut and Fill Slopes

Based on Kleinfelder's experience with similar site and subsurface conditions, Kleinfelder does not expect major slope stability problems with cuts and fills, if the site grading recommendations presented in this report are followed. Permanent cuts and fills in the overburden soils should be designed with slopes of 3:1 (horizontal to vertical) or flatter for heights up to 10 feet. The ground below the fill areas should be properly prepared prior to fill placement and the fill constructed as discussed in Section 4.2.5. The overburden soils are susceptible to wind or water erosion and protection by re-vegetation or other means is advised.

4.2.4 Suitability of Site Soil

The native overburden soils may be used as engineered fill anywhere on the mill site provided they are processed and moisture conditioned as discussed in this report. The in situ moisture content of the overburden soils is well below optimum moisture content and the addition of water will be required in order to meet the moisture content specifications in the following section and to obtain a homogeneous mixture.

4.2.5 Site Preparation

All vegetation should be properly stripped. A stripping depth of about 3 inches can be used for planning purposes. In areas to be filled, the exposed native soils should be scarified to a depth of at least 8 inches and moisture conditioned and compacted to the same specification as the overlying fill provided below.

Engineered fill within the mill pad area should be placed in nominal 8-inch compacted lifts, adjusted to moisture content within 2 percent of optimum moisture and compacted to 98 percent of the maximum dry density as determined by ASTM D 698 (standard Proctor). Kleinfelder recommends a disc be used to process the fill in order to provide uniform fill moisture content prior to compaction.

4.2.6 Fill Shrinkage/Bulking

Moisture density relationship determinations (modified Proctor) were made on two bulk samples representative of the overburden soils. The maximum dry density ranged from 122.7 to 127.7 pcf. The average in-place dry density of the upper 20 feet of overburden soils was 98 pcf. Kleinfelder estimates the overburden soils will shrink about 20 to 25 percent when compacted as recommended in Section 4.2.5.

4.2.7 Corrosion and Cement Type

The corrosion potential of the soils was determined by conducting pH, electrical resistivity, and water-soluble sulfate testing. Test results are summarized below.

LOCATION	pH	ELECTRICAL RESISTIVITY (ohms-cm)	WATER SOLUBLE SULFATES (%)	SOIL TYPE
TB-6 @ 6'	7.8	1,912	0.015	Silty Sand
TB-10@9'	7.8	1,757	0.015	Silty Sand
TB-18@9'	7.7	1,157	0.021	Silty Sand

The soil pH is slightly basic. Electrical resistivity less than 2,000 ohm-centimeters generally indicates an aggressive environment for corrosion; however, many factors influence corrosion potential. Kleinfelder recommends a qualified corrosion engineer review the data to determine appropriate levels of protection for buried metals.

The concentration of water-soluble sulfates represents a Class 0 exposure to sulfate attack, based on Chapter 2 of the *Guide to Durable Concrete*[14]. No special cement type is required for this exposure.

4.2.8 Construction in Wet or Cold Weather

It is important to avoid ponding of water in or near excavations. Promptly pump out or otherwise remove water that accumulates in excavations or on subgrades, and allow these areas to dry out before resuming construction. Use berms, ditches, and similar means to prevent stormwater from entering the work area and to convey it off site efficiently.

If the structures are constructed during cold weather, do not install the foundations or slabs on frozen soil. Frozen soil should either be removed from beneath these elements altogether, or thawed and recompacted. To avoid soil freezing, minimize the amount of time passing between excavation and construction. Use blankets, soil cover, or heating as required to prevent the subgrade from freezing.

4.3 DRAINAGE

Surface drainage is critical to the performance of the facility. Grade the ground surface on and around the mill pad and all structures so that surface water will quickly flow off the pad and away from the structures. Kleinfelder recommends minimum gradients of 5 percent away from each structure for a distance of at least 10 feet. Roof drainage should be collected and not allowed to drain onto the pad. Water from precipitation should drain away from the structures as rapidly as possible and not be allowed to stand or pond on the pad. A maintenance program should be developed to routinely monitor surface drainage conditions and to repair any areas inhibiting flow off the pad. Mill employees should be trained regarding the importance of surface drainage.

4.4 FOUNDATIONS

4.4.1 Spread Footing Foundations

The primary characteristic influencing foundation support of spread footings on shallow soils at the proposed mill is the collapse potential of the upper layer of soils. Several options for managing the collapse potential were examined, including deep foundations and construction of a layer of compacted engineered fill. After careful evaluation of these subsoil conditions, consideration of the proposed construction, and discussions with the design team and EFR, Kleinfelder believes that conventional spread or drilled footing foundations bearing on compacted engineered fill are feasible for support of structural loads.

The soils within the mill area will be protected from infiltrating moisture by a variety of measures including surface grading and drainage to conduct water away from the mill, lined collection ponds, and roofs and other covers over mill structures. Water is used in the mill process, but containment and leak detection features are built into the mill circuit, and given the semi-arid climate, the probability of infiltration of mill liquids to the foundations grades is very low.

Kleinfelder recommends any facilities within the mill circuit (including buildings, slab-supported equipment, stand-alone equipment, and pipe racks) be placed on at least 10 feet of compacted engineered fill as detailed in sections 4.2.4 and 4.2.5. To accomplish this compacted fill layer, over-excavation of portions of the native soils may be required. The purpose of the compacted fill layer is to manage the collapse potential and provide uniform support and reasonable bearing capacity for building foundations. For those facility elements outside of the mill circuit that can tolerate more settlement, the depth of compacted fill may be reduced appropriately, but should not be less than 5 feet. The compacted engineered fill boundaries should extend at least 10 feet beyond the horizontal limits of these structures.

Design and construction criteria are presented below for spread footing foundations. The construction details should be considered when preparing project documents.

1. Footings placed on compacted engineered fill with an embedment depth of at least 30 inches and the fill depth is at least 10 feet (within mill circuit) may be designed for a maximum allowable bearing pressure of 3,000 psf. This allowable bearing pressure should be reduced to 2,500 psf for structures out of the mill circuit with a 5-foot compacted fill layer.
2. The allowable bearing capacity may be increased for drilled footings bearing on compacted engineered fill depending upon the bearing depth. Drilled footings should bear no deeper than 5 feet below grade, assuming a 10-foot layer of compacted fill, and may be designed for an allowable bearing pressure of 5,000 psf at this depth. Linear interpolation can be used to determine the maximum allowable soil pressures of footings with depths between 30 inches and 5 feet. Uplift loads may be resisted using a skin friction of 20 psf per foot.
3. The feed hopper may be founded directly on the native soils, provided the bearing elevation is below elevation 5525 feet, and designed for an allowable bearing pressure of 5,000 psf. The bag house and tramp adjacent to the feed hopper should be founded on compacted engineered fill.
4. The above bearing pressures may be increased by one-third for transient loads.
5. Lateral loads and overturning moments may be resisted using a coefficient of friction for sliding of 0.4 for engineered fill and a passive earth pressure of 350 pounds per cubic foot (pcf). These are ultimate values and appropriate safety factors should be applied, particularly for the passive case.
6. Kleinfelder estimates total movement for footings designed as recommended in this section and considering the precautions for controlling sources of wetting as discussed in the report will be about 1-inch. Differential movement is anticipated to be ¾ of the total movement.
7. The minimum embedment depth of 30 inches will satisfy requirements of Montrose County for frost protection.
8. Footings should have a minimum size of 16 inches for continuous footings and 24 inches for isolated pads.
9. Continuous footings should be reinforced to simply span at least 10 feet.
10. It may be necessary to compact the bearing surface with a plate-type compactor if the compacted engineered fill becomes disturbed during forming of foundations.
11. Concrete should be placed in drilled footings immediately upon completion or the holes should be covered prior to concrete placement to prevent loose material from falling into the drill hole.

12. Foundation excavations should be observed per the Technical Specifications and CQA Plan.

The potential for infiltration around or through the foundation soils at the mill facility is limited by the site drainage design and by the naturally low net infiltration rates of the native soils, as evident in the fact that no zones of saturation in the site soils have been found in any of the many exploration borings drilled on the site. Natural evapotranspiration rates are also high at the site. Kleinfelder believes the key element in managing the collapse risk at this site is control of potential sources of wetting. The recommendations in Section 4.3 Drainage should be carefully followed. In addition, Kleinfelder recommends the mill septic system be designed as a lined evapotranspiration system. All buried pipes should be double contained and cleanouts or sumps incorporated into the design to allow inspection. A maintenance program should be developed to inspect double containment at frequent, regular intervals. Mill employees should also be briefed on risks of leakage.

The guard house and administration building will be located outside of the mill footprint. Kleinfelder believes these lightly loaded structures may be founded on spread footings or reinforced slab foundation with turned down edges bearing directly on the native soils designed for a low allowable bearing pressure of 1,000 psf. Footings should be at least 24 inches wide and reinforced to simply span a distance of at least 12 feet.

4.4.2 Augered Pressure Grouted Piles

A feasible foundation alternative is Augered Pressure Grouted (APG) piles bearing in the dense sand well below the upper collapsible soil layer. Structural loads from buildings, equipment, mats supporting equipment, machinery, tanks and pipe racks would be supported by APG piles. APG piles are cast-in-place concrete or grout piles installed by using a continuous-flight hollow-stem auger and a two-stroke grout pump. Concrete grout is pumped under pressure through the auger stem as the auger is slowly withdrawn at a constant steady rate from the hole. Kleinfelder has discussed this foundation system with Berkel & Company Contractors, Inc. (Berkel) in Bonner Springs, Kansas and they believe APG piles can be drilled into the overburden soils, including the cemented materials, and can support the anticipated structural loading.

Building floor slabs and other non-structural features within the mill footprint would remain susceptible to the collapsible soil layer. Kleinfelder recommends at least 5 feet of compacted engineered fill beneath these features within the mill circuit and 2.5 feet in other areas.

Allowable capacity curves for axial and uplift loads for 16, 18 and 24-inch diameter piles are presented in Appendix D. Downdrag loads have been estimated for each pile diameter, but are not included in the allowable capacity curves and actual capacity should be reduced by the estimated downdrag load.

Kleinfelder recommends the following design and construction details for APG piles.

1. Piles should penetrate the native soils at least 35 feet. This will result in pile lengths on the order of 40 to 50 feet.
2. The minimum pile diameter should be 16 inches and the total pile length should not exceed 30 times the pile diameter.
3. Minimum center-to-center spacing should be 3 pile diameters.
4. Pile load tests should be performed in the field in accordance with ASTM D-1143[15]. Test piles should be loaded to 200 percent of the design working load plus downdrag load.
5. Concrete or grout should have a 28-day compressive strength of 4,000 pounds per square inch (psi).
6. Pile reinforcement must be hand placed immediately after auger removal and grout placement.
7. After a given hole is augered to its design depth, the auger should be withdrawn at a slow uniform rate as the grout is pumped in place. The grout pressure should be sufficient to prevent sloughing or heaving of the hole and the formation of a non-continuous pile.
8. Pile installation should be observed per the CQA Plan. If APG piles are selected as the foundation type, the CQA Plan will need to be modified to include APG pile installation.

Lateral load response of APG foundations can be calculated with the computer programs LPILE or COM624 or beam-on-elastic-foundation type analysis. The stiffness of the pile and the stress-strain properties of the surrounding soils determine the lateral resistance of the pile system. For beam-on-elastic-foundation analysis, Kleinfelder recommends a modulus of horizontal subgrade reaction of 20Z tons per cubic foot (tcf), where Z is the soil depth in feet. This modulus value is for a long, one-foot diameter pile and must be factored by the reciprocal of the pile diameter (in feet). For example, a 24-inch diameter pile would use a modulus of 10Z tcf for design. Suggested criteria for LPILE or COM624 analysis are presented in the following table.

Parameter	Sand
Soil Type	API Sand
Effective Unit Weight (pci)	0.072
Friction Angle (deg.)	30
p-y Modulus k_s (pci)	90

4.4.3 Rammed Aggregate Piers

The Rammed Aggregate System® by Geopier Foundation Company may be considered as a feasible foundation alternate. The geopier system is a proprietary system that involves drilling 24 to 36-inch diameter holes to depths ranging from 10 to 25 feet. Thin lifts of well-graded aggregate are rammed in place and densified using a high frequency hydraulic hammer and a patented beveled tamper. This construction results in a very stiff aggregate pier and improved composite soil conditions surrounding the pier. The installation of the piers improves the allowable bearing pressure for design and provides engineered settlement control. Additionally, when installed at close spacing the piers reduce the potential for collapse.

Piers are typically installed at close spacing ranging from 3.5 to 5 feet on-center beneath isolated footings. Wider spacing ranging from 5 to 12 feet on-center is used for support of light to heavily loaded floor slabs and mats. The pier spacing is determined based on site-specific soil and loading conditions. The design for Rammed Aggregate Pier systems is performed by Geopier Foundation Company. Kleinfelder has discussed this project with Mr. Joe Kerrigan, P.E., regional engineer for Geopier Foundation Company who is evaluating the feasibility of using a Rammed Aggregate Pier system for this project.

4.5 FLOOR SYSTEMS

The mill buildings will have concrete slab-on-grade floors. Due to the collapse potential of the upper overburden soils, Kleinfelder believes non-structural slabs should be placed on compacted engineered fill for slab-on-grade support. If spread footing foundations are used, the compacted fill layer will be at least 10 feet thick within the mill circuit and 5 feet thick in other areas. If APG piles are used for foundation support, Kleinfelder recommends a minimum of 5 feet of compacted engineered fill beneath non-structural slabs within the mill circuit and 2.5 feet elsewhere. If geopiers are used, no over-excavation and replacement of collapsible soils will be required provided the piers are at sufficient spacing to mitigate the collapse potential.

To reduce effects of differential slab movement, slabs should be separated from all bearing walls, columns, and slab-bearing equipment with a positive expansion joint. The slabs should be provided with frequent control joints to reduce damage due to shrinkage cracking. Control joint spacing is a function of slab thickness, aggregate size, slump and curing conditions. The requirements for concrete slab thickness, joint spacing and reinforcement should be established by the designer based on experience, recognized design guidelines and the intended slab use. Placement and curing conditions will have a strong impact on the final concrete slab integrity.

4.6 FOUNDATION WALLS AND RETAINING STRUCTURES

Foundation walls for the feed hopper and loading docks in the packaging warehouse will require design for lateral earth pressure. Kleinfelder is not aware of any other planned retaining walls within the mill pad area; however, considering the variable pad elevations and the grade across the pad site some minor walls may be necessary. Magnitude of the lateral earth pressure depends on the natural and backfill soil types and acceptable wall movements, which affect soil strain and mobilize the shear strength of the soil. More soil movement results in the development of greater internal shear stresses, thereby lowering the lateral pressure on the wall. Soil strain and allowable wall rotation must be greater to mobilize full strength and reduce lateral pressures for fine-grained soils than for cohesionless granular soils. Fine-grained soils also tend to exhibit lower ultimate strengths. In most cases, a triangular pressure distribution is satisfactory for design and is usually represented as an equivalent fluid unit weight or pressure.

The design and construction criteria presented below should be observed for foundation and retaining walls. The construction details should be considered when preparing construction documents.

1. Retaining walls that are laterally supported can be expected to undergo only a slight amount of deflection. These walls should be designed for an "at-rest" lateral earth pressure computed on the basis of an equivalent fluid unit weight of 65 pcf for backfill consisting of the overburden soils.
2. Retaining structures, which can deflect sufficiently to mobilize the full active earth pressure condition, should be designed for a lateral earth pressure computed on the basis of an equivalent fluid unit weight of 45 pcf for the overburden soils.
3. Lateral loads may be resisted using a coefficient of friction for sliding of 0.4 and a passive earth pressure of 350 pcf. Due to the relatively large movements required to mobilize the passive pressure, Kleinfelder recommends a suitable factor of safety be utilized.
4. The above lateral earth pressures assume drained conditions behind the walls and a horizontal backfill surface. Kleinfelder can provide recommendations and details related to drainage behind earth-retaining walls if desired.
5. Fill against retaining walls should be properly placed and compacted as recommended in Section 4.2.5 of this report. Care should be taken when placing backfill so as not to damage the walls. Kleinfelder recommends compaction behind walls be reduced to 95 percent of Proctor density. Compaction of each lift adjacent to and near the walls should be accomplished with hand-operated tampers or other lightweight compactors. Over-compaction may cause excessive lateral earth pressures, which could result in wall movements and potential damage to the walls.

4.7 INTERIOR MILL PAVEMENTS

Kleinfelder understands access to maintenance vehicles will be necessary throughout the plant. The heaviest vehicle load will be a 4-wheel, 20-ton crane, while the remainder of traffic will be lightly loaded maintenance vehicles. The heaviest traffic will occur during construction. Kleinfelder understands a gravel surface is preferred. Design of the main access/haul roads will be provided in a separate report.

Kleinfelder anticipates the subgrade soils will be silty to clayey sand with occasional sandy clay. Hveem Stabilometer (R-value) tests were performed on samples of silty sand and sandy clay from borings drilled in the planned haul road area. The results of the testing, presented on Figures C-36 and C-37, indicated R-values of 28 and 14, respectively for the soils.

Kleinfelder evaluated two conditions for pavement. The heaviest traffic will occur during construction. Kleinfelder assumed a traffic mix of 20 concrete trucks, or equivalent, and 2 semi-tractor trailers six days per week for a one-year construction period, which calculates to an 18-kip equivalent single axle load (ESAL) of about 13,000. During operation Kleinfelder has assumed an ESAL of 10,000 for a 20-year design life, which is equivalent to a lightly traveled rural road. If these assumed loadings do not appear appropriate for the plant site, Kleinfelder should be contacted to re-evaluate the proposed pavement sections.

Kleinfelder recommends a graveled surface for those areas within the plant that will be utilized by heavy construction equipment during construction consisting of either 12 inches of CDOT Class 6 aggregate base course or 14 inches of CDOT Class 1 aggregate base course. CDOT specifications indicate Class 6 aggregate base course should have 3 to 12 percent passing the #200 sieve. For a graveled surface Kleinfelder recommends the percent passing the #200 sieve be modified to a range of 6 to 12 percent to allow slightly more fines to provide better binding of the material.

The permanent access areas within the plant should use 4 inches of Class 6 aggregate base course placed above the temporary construction graveled surface or 8 inches of Class 6 aggregate base course placed over prepared native subgrade. Periodic maintenance will be required to maintain a smooth surface. It will likely be necessary to add base course during the life of the plant to rejuvenate the surface.

Prior to placing the temporary or permanent gravel section, the subgrade should be scarified to a minimum depth of 8 inches and compacted as recommended in Section 4.2.5. The entire pavement subgrade should be proofrolled with a heavily loaded pneumatic-tired vehicle after preparation. Areas that deform under heavy wheel loads should be removed and replaced to achieve a stable subgrade prior to paving.

5 LIMITATIONS

The recommendations in this report are based on Kleinfelder's field observations, laboratory testing, present understanding of the proposed construction, and discussions with the design team and EFR. Subsurface conditions can vary between or beyond the points explored. If the conditions found during construction differ from those described in this report, please notify us immediately so that Kleinfelder can review the report in light of those conditions and provide supplemental recommendations as necessary. Kleinfelder should also review the report if the scope of the proposed construction, including the proposed loads or structure locations, changes from that described in this report.

Kleinfelder has prepared this report for the exclusive use of Golder Associates and Energy Fuels Resources Corporation for the Piñon Ridge Mill Facility in Montrose County, Colorado. The report was prepared in substantial accordance with the generally accepted standards of practice for geotechnical engineering as exist in the site area at the time of Kleinfelder's investigation. No warranty is expressed or implied.

This report may be used only by the client, and only for the purposes stated, within a reasonable time from its issuance, but in no event later than 3 years from the date of the report. Land or facility use, on- and off-site conditions, regulations, or other factors may change over time, and additional work may be required with the passage of time. Based on the intended use of the report, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Kleinfelder from any liability resulting from the use of this report by an unauthorized party and the client agrees to defend, indemnify, and hold harmless Kleinfelder from any claim or liability associated with such unauthorized use or non-compliance.

6 REFERENCES

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[2] *American Society for Testing and Materials*, ASTM D 2487 Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)

[3] *American Society for Testing and Materials*, ASTM D 4643 Standard Test Method for Determination of Water (Moisture) Content of Soil by the Microwave Oven Heating

[4] *American Society for Testing and Materials*, ASTM D 2937 Standard Test Method for Density of Soil in Place by the Drive-Cylinder Method

[5] *American Society for Testing and Materials*, ASTM D 422 Standard Test Method for Particle-Size Analysis of Soils

[6] *American Society for Testing and Materials*, ASTM D 4318 Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

[7] *American Society for Testing and Materials*, ASTM D 4546 Standard Test Methods for One-Dimensional Swell or Settlement Potential of Cohesive Soils

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