EXHIBIT S

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HOLDREGE & KULL

CONSULTING ENGINEERS • GEOLOGISTS

Project No. 2890-01
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Richard Sykora
P.O. Box 622
Foresthill, California 95631

Reference: ***Big Seam and Red Ink Maid Mining Claims***
Foresthill, California

Subject: ***Proposed Stockpile 5 Plan Sheets and Stability Review***

Dear Mr. Sykora,

At the request of Jeff Huggins of the California Regional Water Quality Control Board, we have prepared this revision of our January 26, 2007 report regarding proposed wasterock Stockpile 5. The revision has been prepared to provide additional information regarding site seismicity. The additional site seismicity information was requested by Mr. Huggins during a recent telephone conversation to facilitate review of the stability analysis of the proposed wasterock stockpile.

The enclosed plan sheets depict two alternate wasterock configurations for proposed Stockpile 5. One of the proposed wasterock configurations includes the construction of a gabion basket wall at the toe of the slope to allow increased wasterock storage volume. The plans are intended to facilitate the review and permitting process associated with the existing mine operation onsite. The enclosed plan sheets, as well as the corresponding stability analysis results, have been provided to the Placer County Planning Department for distribution to associated reviewing agencies.

Our plan sheets depict anticipated finished wasterock stockpile configurations based on the existing topography at the proposed stockpile location as well as the recommended maximum finished slope gradient. The finished dimensions of the stockpile are expected to vary, depending on the actual slope gradient used, the optional construction of a gabion basket retaining structure at the toe of the slope, and the variation of the natural topography. We anticipate that, during wasterock placement, temporary slope gradients approaching the friction angle of the material will occur, particularly at the location of dumping. However, it is critical that the finished slope gradient at the end of wasterock placement not exceed the recommended slope gradient of 33 degrees unless further stability analysis and site review is performed to confirm stability.

Site preparation, wasterock placement and eventual reclamation of the stockpile should incorporate the recommendations presented by the USDA Forest Service in their recommended Mitigation Measures for this project. We can provide additional site specific erosion control and reclamation recommendations for the project, if requested.

One concern associated with the placement of wasterock on steeply sloping sites is the increased likelihood of wasterock and fine grained sediments being transported from the stockpile locations to downgradient streams. Please note that the plan sheets depict redundant debris or sediment barriers to be constructed at locations downslope from the proposed toe of the wasterock stockpile. These barriers are intended to be installed prior to wasterock placement, and will need to be maintained and functional during the course of wasterock placement. Please note that if the gabion wall option is used, the gabion wall should be constructed prior to the initial stages of wasterock placement to serve as a debris barrier. Following wasterock placement, we anticipate that coarse rock fragments will be located on the lower portions of the stockpile surface, serving as slope armor and reducing the need for the sediment and debris barriers. The need for continued maintenance of the barriers should be evaluated following wasterock placement.

SITE SEISMICITY

Our site seismicity assessment was performed in general accordance with California Division of Mines and Geology (CDMG) Special Publication 117, Guidelines for Evaluating and Mitigating Seismic Hazards in California (1997), California Fault Parameters (California Geological Survey 2002), and CDMG Note 42, Guidelines for Geologic/Seismic Reports, (1986).

REGIONAL SEISMIC SOURCES

The 2001 California Building Code (CBC) Seismic Zone Map designates zones of relative earthquake hazard from 1 to 4, with 4 being the greatest hazard. According to the CBC map, all sites in California are rated as either Zone 3 or Zone 4, and there are no areas in California where seismic hazards are considered non-existent. The project site is located within Seismic Zone 3.

The California Geological Survey report and map, *California Fault Parameters* (2002), categorizes faults as Type A, B, or C. Type A faults are capable of producing large magnitude events, and have a high rate of slip. Type C faults are not capable of producing large magnitude earthquakes, and have a relatively low slip rate. Type B faults are all other type faults.

The CGS report indicates that no Type A faults are located within 100 kilometers (62.5 miles) of the site, and that the Genoa fault, located approximately 50 miles east of the site, is a Type B fault. The Genoa fault is an east dipping, normal fault that is estimated to have produced up to 50 feet of displacement within the last 2000 years. Type C-fault zones within 100 kilometers of the site are categorized as areal source zones with the hazard distributed over a large area instead of along a single fault trace, and include the Foothills Fault System, the Mohawk-Honey Lake Fault Zone, and the Western Nevada Zone.

The site is located near the eastern edge of the Foothills Fault System, a seismic zone composed of pre-Quaternary to Quaternary faults. The Volcano Canyon fault, mapped approximately 1.5 miles west and north of the site, and the Foresthill fault, mapped approximately 4 miles west of the site, are included within the Foothills Fault System (CDMG, 1992). Both the Volcano Canyon fault and the Foresthill fault are mapped as pre-Quaternary faults (between 2 million and 7 million years before present).

The Western Nevada Zone is a shear zone composed of strike slip and dip slip faults within the eastern portion of the Sierra Nevada and the western portion of Nevada. The Mohawk-Honey Lake Zone distributes shear carried from the Western Nevada Zone.

Historic Seismicity

We used EQSEARCH (Blake, 2004) to search multiple earthquake data records for information about historic earthquakes between 1850 and 2004. The records search indicate that 66 earthquakes with estimated magnitudes greater than 5.0 have occurred within 100 kilometers of the site since 1850, and that 12 earthquakes exhibited magnitudes greater than 6.0. The search indicated that the nearest historic earthquake was approximately 20 miles northwest of the site, and had a magnitude 5.0. The largest earthquake had a magnitude of 6.4 and was located approximately 60 miles (99 km) east of the site. The largest acceleration recorded during these historic events was 0.053g.

EARTHQUAKE GROUND MOTION

Ground motion for the site can be estimated by performing either a probabilistic or deterministic seismic hazards analysis in an effort determine the maximum horizontal acceleration during a significant seismic event.

The maximum probable earthquake (MPE), for the purposes of review of proposed land disposal, is defined as the maximum earthquake that is likely to occur during a 100 year interval. MPE is also known as the design basis earthquake in probabilistic analysis, and is typically designated as possessing a 10 percent probability of ground motion being exceeded in 50 years, or having a return period of 475 years. Alternately, the upper bound earthquake (possessing a return period of 949 years) or the maximum considered earthquake (with a return period of 2,475 years) can be considered in the analysis.

The maximum credible earthquake (MCE) differs from the MPE in that it is defined as the maximum earthquake that appears capable of occurring given the presently known geologic conditions. The determination of the MCE does not take into consideration the likelihood of the occurrence of the earthquake during a specified period of time. We understand that the use of the MCE is required for review of Class II waste units. However, for Class III units, the use of MPE is appropriate.

We performed both a deterministic and probabilistic analysis for comparison purposes. Summary descriptions of the methods and results are provided in the following sections.

Deterministic Seismic Hazard Analysis

The deterministic seismic hazard analysis allows the estimation of the MCE at the subject site based on a specific fault that is assumed to present the greatest seismic hazard. The MCE analysis considers the estimated earthquake magnitude, fault type, and distance of a specific fault, without consideration of the effects of other faults. The deterministic analysis is performed on specific faults identified by California fault database catalogs within a 100 kilometer radius of the site, with the hazard calculated for each fault. We performed a deterministic ground motion analysis using Version 3.0 of EQFAULT, a computer program developed by Thomas F. Blake, updated 2004.

Because of the variability in fault zones, the ground motion level used for analysis may be either the median (50th percentile) for faults of low rates of seismicity, or one standard deviation above the median (84th percentile) for faults with high rates of seismicity. The following table provides deterministic values, slip rates, and fault class activity ratings for individual faults within 62.5 miles (100 kilometers) of the site.

TABLE 1 - FAULT PARAMETERS FOR DETERMINISTIC SEISMIC HAZARD ANALYSIS

FAULT NAME and GEOMETRY[1]	FAULT CLASS[1]	SLIP RATE[1]	MAGNITUDE[1]	CLOSEST PROJECTION OF RUPTURE AREA[2,3]		PEAK GROUND ACCELERATION[3]
		(mm/yr)	(Mmax)	(mi)	(km)	(g)
Foothills Fault System 3 n, rl, o, 75 E	C	0.05	6.5	0.0	0.0	0.34
Foothills Fault System 2 n, rl, o, 75 E	C	0.05	6.5	10.4	16.8	0.14
Foothills Fault System 4	C	0.05	6.5	20.4	32.8	0.08

TABLE 1 - FAULT PARAMETERS FOR DETERMINISTIC SEISMIC HAZARD ANALYSIS

FAULT NAME and GEOMETRY[1]	FAULT CLASS[1]	SLIP RATE[1]	MAGNITUDE[1]	CLOSEST PROJECTION OF RUPTURE AREA[2,3]		PEAK GROUND ACCELERATION[3]
n, rl, o, 75 E						
Foothills Fault System 1 n, rl, o, 75 E	C	0.05	6.5	20.9	33.6	0.08
Western Nevada Zone 1 rl-ss	C	4.00	7.3	29.9	48.1	0.08
Mohawk-Honey Lk Zone 5 rl ss	C	2.00	7.3	36.4	58.6	0.07
Western Nevada Zone 2 rl-ss	C	4.00	7.3	43.9	70.7	0.06
Mohawk-Honey Lk Zone 4 rl ss	C	2.00	7.3	48.3	77.8	0.05
Genoa (Carson Range Zone) n, N60E	B	2.00	6.9	49.5	79.6	0.05
Western Nevada Zone 3 rl-ss	C	4.00	7.3	56.9	91.0	0.05
Mohawk-Honey Lk Zone 3 rl ss	C	2.00	7.3	60.9	98.0	0.05

[1]California Geological Survey, 2002, California Fault Parameters, on-line document and map, http://www.consrv.ca.gov/CGS/rghm/psha/fault_parameters/pdf/2002_CA_Hazard_Maps.pdf

[2] Blake, T.F., 2004, EQFAULT, Deterministic Estimation of Peak Horizontal Acceleration from Digitized Faults

[3] The fault hazard over areal source zones is distributed within the zone, and does not truly indicate a single rupture or fault trace.

EQFAULT indicates a maximum moment magnitude (M_{MAX}) of M_w 6.5 on Foothills Fault System 3. Because the site is mapped near the eastern boundary of the Foothills Fault System, we elected to consider the rupture area as occurring at the project site for purposes of the evaluation. Thus, the MCE peak ground acceleration would be 0.34g. The US Geological Survey document, *Potential Sources for Earthquakes Larger than M6 in Northern California* (2000), indicates the effective recurrence interval for a M_w 6.5 earthquake is 12,500 years.

Probabilistic Seismic Hazard Analysis

Probabilistic seismic hazard analysis allows estimation of the MPE with consideration of the probability of the ground motion being exceeded in a certain time period. The resulting peak ground acceleration (PGA) is an aggregate of the contributions of possible earthquakes from all of the faults within the specified area. The probabilistic analysis considers the expected magnitude, fault structure, movement, hazard level, return period, and distance to the site relative to other regional faults.

We used FRISKSP Version 4 (T.L. Blake, 2004) for our probabilistic seismic hazard analysis. The analysis results are summarized in Table 2 below.

TABLE 2 - EXCEEDENCE PROBABILITY vs. PEAK GROUND ACCELERATION			
Magnitude (M_w)	6.5	6.5	6.5
PGA (g)	0.07	0.09	0.13
Exceedence Probability	10% in 50 years	10% in 100 years	2% in 50 years
Return Period	475	949	2475
Probability Type	Design Basis Earthquake	Upper Bound Earthquake	Maximum Considered Earthquake

The probabilistic analysis procedure considers aggregate risk from earthquakes of different magnitudes and source locations within the region. The probabilistic analysis results are not associated with an individual earthquake magnitude or source. Deaggregation can be performed to determine which earthquake source and magnitude contributed most to the resulting ground motion, allowing further evaluation of seismic hazards. For example, the nearest fault to the site is Foothills Fault System 3, with an estimated moment magnitude (M_w) of 6.5, and an annual slip rate of .05 mm/year. However, deaggregation of the hazard and seismic activity contributions revealed little hazard contribution from Foothills Fault System 3. Based on a deaggregation of the fault data, the most significant contribution to hazards at this site are associated with the faults located more than 45 kilometers from the site.

Summary of Seismicity for Slope Design

Based on our probabilistic analysis, the expected peak ground acceleration resulting from the MPE would be 0.07g. However, if a longer return period of 2,475 years is considered, the peak ground acceleration increases to 0.13g.

The deterministic approach indicates that the peak ground acceleration associated with the MCE (derived from the Foothills Fault System, with the source at the site) would be 0.36. Considering the probability of the seismic source within the Foothills Fault System occurring at the site, no historic earthquakes greater than M_w=5.0 having been reported within 30 kilometers of the site, and slip rates of less than 5 millimeters per year, our opinion is that the use of the MCE in this case is overly conservative. The peak ground acceleration associated with a source within the foothills fault system approximately 17 kilometers from the site (Foothills Fault System 2) would be 0.14g.

For the purposes of this project, we elected to consider a horizontal acceleration of 0.2g for the pseudo-static stability analysis.

Summary of Stability Analysis for Stockpile 5

We performed a computer-assisted slope stability analysis to evaluate the existing stockpile configurations. The slope models used were based on the proposed finished wasterock slope gradient of 33 degrees (equivalent to a 1½:1, horizontal to vertical slope). Our stability analysis used the laboratory test results obtained during our previous geotechnical review of the existing stockpiles onsite, as described in our November 1, 2006 report entitled *Wasterock Stability Evaluation and Initial Characterization.* Our analysis was performed using Stabl6™ software utilizing the Janbu and Bishop's simplified methods of slices.

The stability of a slope is evaluated by calculating its "factor of safety". The factor of safety is a ratio obtained by dividing the resisting forces (i.e., the shear strength of the material comprising the slope) by the driving forces (resulting from the slope gradient, the weight of the material, groundwater, and surcharge loading). If the factor of safety is greater than 1, the slope is theoretically stable. A factor of safety equal to or less than 1 means the slope is theoretically unstable.

Required factors of safety are selected in an effort to address uncertainties in the conditions as well as the anticipated consequences of slope instability. Higher design factors of safety are often appropriate where slope instability would threaten a critical facility or create a hazard to health and safety. In some cases a more thorough investigation of subsurface conditions, including extensive laboratory testing to reliably establish lower bound shear strength and accurately identify material properties, allows the use of lower factors of safety. In general, we use minimum required factors of safety of 1.5 to account for variability in groundwater, subsurface soil and rock conditions, and laboratory test results when analyzing slopes associated with critical facilities, inhabited structures, and other locations where the consequences of a slope failure would be high. Factors of safety as low as 1.2 are often employed for slopes of relatively low risk and where conditions can be readily observed and confirmed by laboratory testing such as cut slopes for driveways and rural roads. In addition, the use of lower factors of safety may be justified for existing slopes where information regarding past performance is available. One reason for this is that the degree of uncertainty regarding shear strength and piezometric levels can be reduced through back analysis.

Furthermore, reduced factors of safety are often used when the stability analysis considers short term seismic loading, rapid change in groundwater elevation, or other events of relatively short duration or infrequent occurrence.

Title 27 employs a two-stage slope stability evaluation process which first utilizes a pseudo-static analysis and then, if necessary, employs a more rigorous deformation analysis. The pseudo-static analysis is a screening evaluation to determine if the proposed slope configuration possesses a factor of safety Of 1.5 or greater when considering the peak horizontal ground acceleration at the site. If the resulting factor of safety is less than 1.5, then the deformation analysis is performed to determine if the expected deformation exceeds acceptable values. If the deformation criteria is not met, then the proposed slope configuration would need to be revised.

Our slope stability analysis was based on a wide variety of assumptions and variables including:

1. Strength data variables - The strength data used in our analysis was based on laboratory test results performed on the sand and finer portions of samples collected from the wasterock onsite. We used the lower internal friction angle and apparent cohesion values obtained during two direct shear tests performed on loose specimens. Based on our laboratory testing, the wasterock was modeled as possessing an internal friction angle of 43.1 degrees and having an apparent cohesion of 110 pounds per square foot. The model also assumed a saturated, approximate 3-foot thick native soil/colluvium layer below the wasterock. The strength properties of the underlying colluvium was estimated with consideration of the native slope gradients, our experience with soil and rock conditions in the area, and the results of back calculations of the past slope instability in wasterock stockpile 4. No direct shear testing was performed on the colluvium and underlying weathered rock onsite.

2. We considered seismic loading (modeled as a horizontal acceleration of 0.2g) in our analysis of the proposed stockpile configuration.

Based on our analysis, we calculated a factor of safety of 1.5 for the proposed wasterock stockpile configuration. The calculated factor of safety is extremely sensitive to horizontal acceleration due to seismic loading. The use of an acceleration of 0.2g, assumed to occur precisely in the out of slope direction, is considered to be conservative. The actual apparent cohesion present in the stockpile materials, as well as the effect of slope armoring due to the accumulation of coarse material on the lower slope surface, will likely cause the actual factor of safety for the configuration to vary. Based on our pseudo-static analysis, we did not perform the more rigorous deformation analysis.

In addition to our stability analysis, we considered the likelihood of rock fall during wasterock placement which would result in individual boulders traveling beyond the toe of the wasterock stockpile and rolling into the steeply sloping canyon below. To evaluate the likelihood of rock fall, we used the Colorado Rockfall Simulation Program (CRSP) distributed by the Colorado Department of Transportation. CRSP models rock fall considering user selected slope and rock properties. Empirically derived functions correlating slope geometry, friction, and rock properties are used in conjunction with conservation of energy principles to calculate the trajectory of individual rocks. The simulation is repeated for hundreds of rock fall events, allowing statistical analysis of probable rock fall behavior for a given slope. CRSP output includes estimates of probable rock fall velocities, bounce heights, and kinetic energies.

To perform our rock fall evaluation, we considered 12-inch boulders dropped on the finished slope surface during the final stages of wasterock placement. Although blasting and excavation of the rock onsite generates subangular and angular rock fragments, the boulders are conservatively modeled as being spherical. It is also assumed that the rock does not break into smaller fragments during the fall. The stockpile slope was modeled as having a 33 degree slope, and a relatively rough surface similar to a talus slope, armored with coarse rock fragments. Furthermore, we considered the placement of a smooth-faced gabion basket retaining wall at the toe of the slope, with fill placement to the top of the wall.

Our CRSP analysis indicated that, with the dropping of 1,000 spherical, 12-inch diameter boulders on the 33 degree slope, one boulder may reach the gabion basket wall. No boulders were calculated to pass beyond the debris barriers or approach the steeper canyon slopes below the proposed stockpile location.

Based on our stability analysis, our opinion is that the proposed wasterock stockpile configuration, utilizing a maximum finished slope gradient of 33 degrees, provides an appropriate factor of safety for the intended use. In addition, the rock fall simulation performed indicated that it is unlikely that individual boulder-sized wasterock fragments will travel beyond the toe of the stockpile onto the canyon slopes below.

Limitations

This letter may be considered to be an addendum to our November 1, 2006 report for the project. The limitations presented in that report apply.

Please contact us if you need any additional information or clarification.

Sincerely,

HOLDREGE & KULL

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attachments: Sheets 1 and 2 - Site Plan

copies: Placer County Planning Department /Attn: Crystal Jacobsen
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