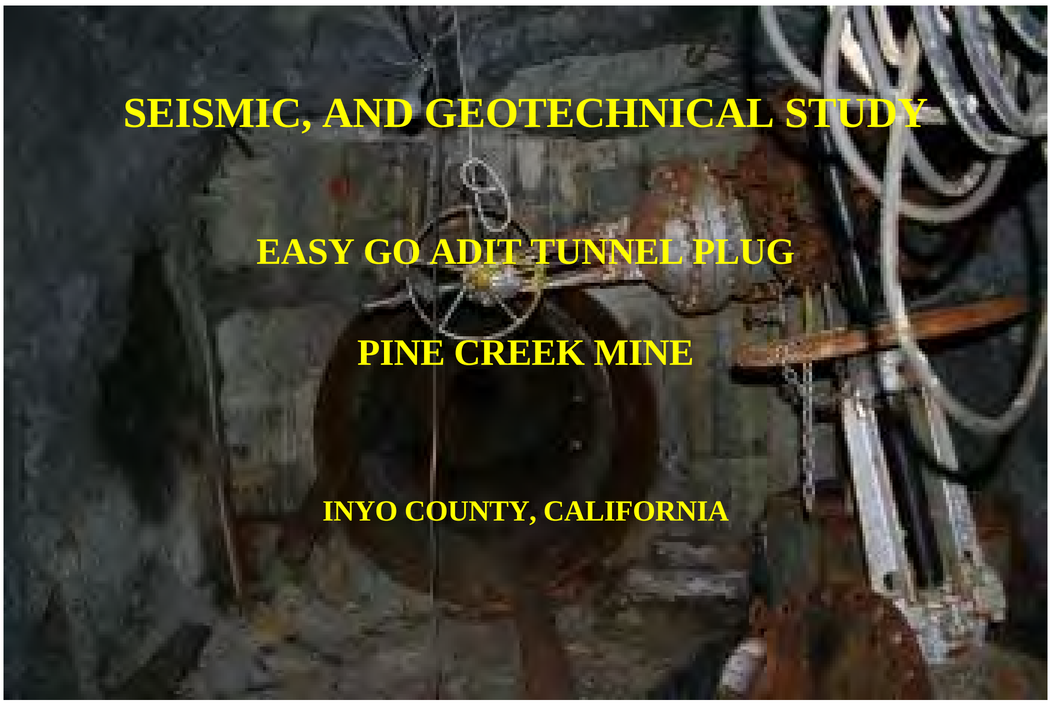
Appendix I

Seismic, and Geotechnical Study and Review of Plug Conditions

SIERRA GEOTECHNICAL SERVICES INC.

MATERIALS • GEOTECHNICAL • GEOLOGY • HYDROGEOLOGY • ENVIRONMENTAL



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1.0. INTRODUCTION

This report presents the findings of Sierra Geotechnical Services, Inc. (SGSI) geotechnical and seismic study on the existing Pine Creek Mine concrete tunnel plug, in Inyo County, California.

1.1. PURPOSE and SCOPE

This study is prepared in response to a letter issued by the United States Forest Service (USFS); Inyo National Forest District Ranger dated February 16, 2005, which centers on the present condition of the plug and the suitability of the plug for service as part of a water reservoir. Included in the USFS letter was a list of recommended studies, two of which are addressed herein:

Seismic Study – Perform a standard seismic safety evaluation using updated information and standard assessment procedures as applied by a California registered geologist or engineer. This work will follow all practices and procedures identified in Guidelines for Evaluating and Mitigating Seismic Hazards in California – Special Publication 117 (Adopted March 13, 1997 by the State Mining and Geology Board in Accordance with the Seismic Hazards Mapping Act of 1990).

Geotechnical Study – Perform a geotechnical evaluation using underground mine maps, underground geologic mapping, and standard assessment procedures as applied by a California registered geologist or engineer with a certification as a geotechnical engineer.

2.0. PINE CREEK MINE

The Pine Creek Mine is located adjacent the confluence of Morgan Creek and Pine Creek, in Pine Creek Canyon, just below the glaciated crest of the Sierra Nevada at elevations between 8,100’ and 11,700’ in Inyo County, Section 8, Township 7 South, Range 30 East, and Mount Diablo Meridian. The mine is accessed by Pine Creek Road which is located west of the community of Rovana and approximately 17 miles west of Bishop, California (Appendix A, Figure 1 and Plate 1).

2.1 Mining Activities

The mine is presently owned by the Pine Creek Mine LLC which also controls the existing claims (Appendix A, Plate 2). Mining of tungsten began in 1918, stopped briefly following WWI and were reinitiated from 1936 through the mid 1990’s. Mining originally began around 11,000’ but as the reserves were depleted it progressed downward into the ore body.

In the 1940's a tunnel was driven to gain access to the ore at the 9,500' (1500 level/ Zero Adit). In 1962, a new adit was drilled at the 8,000' level (EZ-Go Adit) near the milling facilities. EZ-Go is near 12,000' in length and oriented in a northwest direction (Appendix A, Plates 3 and 4).

2.2. Design and Construction Review, Easy Go Plug

The tunnel plug is located within the EZ-Go Adit portion of the mine, approximately 2,700' from the portal entrance behind the existing mill (37.3604º;-118.7041º). The adit runs generally north from the portal and overburden above the plug is estimated at 1200'. In 2002, an approximate 12'7" x 12'7" x 30' long reinforced concrete tunnel plug with impoundment plumbing was installed (Appendix A, Plate 3). The Easy Go Adit plug is located at an elevation of 8083' feet at coordinates of N 27,730 and E 37,660. The plug was designed by Jim Thompson and Andrew Nasser. Construction was performed by Pine Creek Mine staff and oversight was provided by Jim Thompson (Appendix B).

The plug was designed to retain approximately 2,000' of water head (867 psi), and the plumbing within was designed to allow for proposed hydroelectric power generation. The plug length was determined based upon a shear capacity of 101 psi at the undulating rock to concrete interface and a compressive stress of 1642 psi.

The plug was constructed with approximately 175 cy of 4000 psi concrete. Contact grouting was performed after the 28 day concrete cure to fill void spaces and any fractures.

3.0. SITE GEOLOGY

The project site is located along the base of the Sierra Nevada eastern escarpment near the western edge of Owens Valley. The escarpment serves as the boundary between the Great Basin and Sierra Nevada geologic provinces (Appendix A, Figure 2). The Sierra Nevada province is a north-northwesterly trending, asymmetric, tilted fault-block. Predominant basement rock types of the Sierra Nevada include Cretaceous granitics with associated Paleozoic roof pendant rocks.

More specifically, the site is located at the western boundary of the Excelsior-Coaldale section of the Walker Lane Belt (WLB) (Appendix A, Figure 3). The WLB is approximately 700 km long and 100 to 300 km wide and is characterized by Quaternary faults extending from the Garlock fault northward into northeastern California.

3.1 Pine Creek Mine

The Pine Creek Mine is located at the northwest end of the Pine Creek roof pendant in a contact zone between metamorphosed limestone and intrusive granite (Appendix A, Plate 1, Figure 2). The pendant is a raft of metasedimentary and metavolcanic rock intruded by Sierran granitoids. It is almost 7 miles long and 1mile wide, extending from Mt. Tom to Wheeler Crest. The southern one-third is mostly unmineralized metavolcanic rock. The northern two-thirds is metasedimentary rock divisible into three distinct units. The oldest unit is composed of pelitic hornfels, micaceous quartzite, and vitreous quartzite. The next youngest is light gray marble, which in turn is overlain by a unit of micaceous quartzite. They are folded into a tight syncline, whose limbs are nearly vertical at the north end and shallow to the south. The rocks have been correlated with those in the Mt. Morrison pendant to the north, which have been dated by fossils as Pennsylvanian and Permian (?) (Bateman, 1965).

The pendant is in contact with three granitic intrusives. The two most important are the Tungsten Hills quartz monzonite, dated as Triassic (Bateman, l978) and the Wheeler Crest quartz monzonite, 96 m.y. (Kistler and others, 1965). Most of the tungsten mineralization in the Bishop District is thought to be related to the Tungsten Hills quartz monzonite because of its close association to the Pine Creek ore body and numerous other tungsten deposits (Bateman, 1965). An older body of quartz diorite has little or no associated tungsten mineralization. The Pine Creek ore deposit occurs along the western margin of the pendant, at the northernmost contact between the marble unit and the Tungsten Hills quartz monzonite. It is a contact metasomatic deposit of a scheelite-bearing garnet-pyroxene rock called tactite. The scheelite usually occurs in the tactite as disseminated crystals. Tactite occurs only along the northernmost area of the contact between the quartz monzonite and the marble.

3.2 Geology at the Plug

Rock cover directly over the plug area is estimated at 625’ (100’ of decomposed/soil, and 525’ of rock). The geologic log sheets (Appendix A, Plate 1 and Plates 7A-8) indicate that the plug is anchored in quartz diorite (granite) along a solid part of the adit. The nearest zone of unstable rock is approximately 470’ upstream from the plug in an area denoted as “timbered”. Fracture-water is noted in the Easy Go adit approximately 210’ downstream from the plug and approximately 400’ upstream.

4.0. HYDROGEOLOGIC SETTING

The project site is located within the Wheeler Crest hydrogeologic divide between the Pine Creek and Rock Creek hydrologic sub-basins. Pine Creek and its two tributaries, Morgan Creek and Gable Creek, flow from the John Muir Wilderness area northeastward to its juncture with the Owens River. Rock Creek flows from Mount Morgan due north through Little Lakes Valley to its juncture with the Owens River.

Prior to mining operations the Pine Creek/Rock Creek hydrologic system existed in equilibrium. The ground water flow systems had existed in steady state and the long term potentiometric surface in the basins and along the drainage divide (Wheeler Crest) were relatively constant. Groundwater below the water table occupied a large storage reservoir and was not contributed to the surface systems but contained water available for groundwater development (HCI, 1990). Following mining operations, groundwater has been discharging from the storage reservoir such that

A detailed hydrogeologic investigation of the project site was conducted by Hydrologic Consultants, Inc. (Appendix B), the results of which indicate that the Pine Creek Mine currently produces an average of groundwater flow of 5.8 cubic feet per second (cfs) from the fractured rock aquifer, and if left in its present state may decline to 1.8 cfs by the year 2040. Flows in Morgan Creek and the upper part of Rock Creek are expected to decline due to this withdrawal.

If impoundment is pursued and groundwater flows regulated then the rate of decline from storage would be substantially reduced.

5.0. FAULTING

Our discussion of faults on the site is prefaced with a discussion of California legislation and state policies concerning the classification and land-use criteria associated with faults. By definition of the California Geological Survey, an "active fault" is a fault that has had surface displacement within Holocene time (about the last 11,000 years); hence constituting a potential hazard to structures that might be located across it. This definition is used in delineating Earthquake Fault Zones as mandated by the Alquist-Priolo Geologic Hazards Zones Act of 1972, which is detailed in the California Geological Survey Special Publication SP-42 (Hart and Bryant, 1999). The intent of this act is to assure that unwise development does not occur across the traces of active faults. Based on our review, the site is **not** located within any “Earthquake Fault Zones” or Alquist-Priolo Hazard Zones as identified in this document (Appendix A, Figure 4).

Faults considered to be significant potential sources for seismic events that are likely to impact the site are presented in Appendix C. Recent faulting (surface rupture less than 11,000 years ago) and historic faults (surface rupture less than 200 years ago) are located regionally near the site. Regional faults in this report are considered to be those faults within a 62 mi radius of the site. At least 14 major active fault zones are located within this radius with the Round Valley fault being the closest at 4.3 mi from the site. A brief description of the Round Valley fault is included herein.

5.1 The Round Valley (Wheeler Crest) Fault Zone

The Round Valley fault zone is the nearest active fault zone and is approximately 6.5 miles due east of the Pine Creek Mine (Davis, 1985). The Round Valley fault zone marks the exact boundary between the Sierra Nevada and the Basin and Range geomorphic provinces (Bryant, 1984). Also known as the Wheeler Crest fault, the Round Valley fault zone is well-defined along the eastern range front of the Sierra Nevada eastern escarpment with 6,700’ of topographic relief between Round Valley floor and the Wheeler Crest ridge top. Regional gravity and seismic-refraction profile studies indicate that the thickness of alluvial valley-fill in Round Valley is at least 2,000’ thick suggesting that about 8,700’ of vertical displacement has occurred since the Quaternary to the present at this locality (Pakiser, 1964; Bateman, 1965). Late Pleistocene activity on the Round Valley fault is indicated by offset Tioga-age glacial moraines. Historical activity on the fault is evidenced by the M_L = 6.1, 1984 Round Valley earthquake (Smith et al., 1988). A slip rate of 1 mm per year is estimated based on offset Tioga moraines (Clark et al., 1984).

6.0. SITE SEISMICITY

Site coordinates of latitude 37.3604º N and 118.7041º W were acquired using the computer program **GoogleEarth**. A deterministic seismic analysis was performed within a 62.2 mi (100 km) radius of the site using the computer program **EQFAULT** (Blake, 2001). The results of the analysis indicate that the peak ground acceleration estimated for a maximum earthquake event within the specified radius is 0.33g. This acceleration represents deterministic peak ground accelerations and could occur from a magnitude 6.8 (Mw) earthquake on the Round Valley fault located approximately 4.3 mi (7.0 km) east of the site. The Hilton Creek fault, located approximately 7.1 mi (11.5 km) from the site could produce a magnitude 6.7 (Mw) earthquake resulting in a peak ground acceleration of 0.24g at the site. The tabulated results of the deterministic seismic analysis are presented in Appendix C.

The computed maximum site acceleration within a 62.2 mi (100 km) radius of the site was derived from **EQSEARCH** (Blake, 2001) during the time period of 1800 to 2011. The largest estimated site acceleration was 0.159g, which occurred in 1980. This earthquake was located approximately 9.7 mi (15.6 km) from the site. The Modified Mercalli Intensity and earthquake magnitude were VIII and 6.3 (M_w) respectively. The tabulated results of the historical analysis are presented in Appendix C. The Earthquake Epicenter Map, which depicts the epicenters and magnitudes of historical earthquakes that have affected the site, an Earthquake Recurrence Curve, and a plot depicting Earthquake Events versus Magnitude also presented in Appendix C.

A site-specific probabilistic seismic hazard analysis was performed to evaluate anticipated peak ground accelerations (PGAs) for the site using the computer program **FRISKSP** (Blake, 2001). A probabilistic analysis incorporates uncertainties in time, recurrence intervals, size, and location (along faults) of hypothetical earthquakes. This method thus accounts for likelihood (rather than certainty) of occurrence and provides levels of ground acceleration that might be more reasonably hypothesized for a finite exposure period. **FRISKSP** calculates the probability of experiencing various ground accelerations at a site over a period of time and the probability of exceeding expected ground accelerations within the lifetime of the proposed structure from the significant earthquakes within a specific radius of search. For the present case, a search radius of 62 miles (100 km) was selected. The earthquake magnitudes used in this program are based on the current CGS fault model.

The 2010 California Building Code recommends that the design of structures be based on the horizontal PGA having a 2-percent probability of exceedance in 50 years which is defined as the Maximum Considered Earthquake (MCE). The statistical return period for PGA MCE is approximately 2,500 years. In evaluating the seismic hazards associated with the subject site, we have used an attenuation relation proposed by Boore, et al. (1997) for rock. The PGA MCE for the site was calculated as 0.50g. The Probability of Exceedance versus Acceleration graph is presented in Appendix C.

6.1 Seismic Design Criteria

Table 1 presents the seismic parameters for use in preparing a Design Response Spectra for the site. The site class is A, “hard rock”.

TABLE 1

2010 CBC	SEISMIC PARAMETER	RECOMMENDED VALUE
1613.5.2	Site Class	A
1613.5.3(1)	Seismic Coefficient F_a	0.8
1613.5.3(2)	Seismic Coefficient F_v	0.8
	Mapped Spectral Acceleration, S	1.500
	Mapped Spectral Acceleration, S_1	0.531
	Spectral Acceleration Adjusted For Site (SMs)	1.200
	Spectral Acceleration Adjusted For Site (SM1)	0.425
	Design Spectral Acceleration (SDs)	0.800
	Design Spectral Acceleration (SD1)	0.283

7.0. SECONDARY EARTHQUAKE HAZARDS

The potential for secondary geologic hazards that can be associated with a relatively large earthquake include ground rupture, lurching, liquefaction, dynamic settlement, water hammer, slope failures, landslides, cave-ins.

7.1. Ground Rupture

Ground surface rupture results when the movement along a fault is sufficient to cause a gap or break along the upper edge of the fault zone on the surface. Our review of available geologic literature indicated that there are no known active, potentially active, or inactive faults that transect the subject site. Further, a review of California Geological Survey Special Publication 42 (Hart, 1999) indicates that the site is not in an Alquist-Priolo active earthquake fault zone. The nearest known active regional fault is the Round Valley fault located 4.3 mi from the site.

7.2. Lurching

Ground lurching refers to the rolling motion on the ground surface generated by the passage of seismic surface waves. Effects of this nature are likely to be most severe where the thickness of soft sediments varies appreciably under structures. In its present condition, the potential for lurching in the tunnel and around the plug is considered nonexistent due to the lack of compressible soil that would attenuate seismic waves.

7.3. Liquefaction

Liquefaction of cohesionless soils can be caused by strong vibratory motion due to earthquakes. Research and historical data indicate that loose granular soils below a near-surface groundwater table are most susceptible to liquefaction. Liquefaction is characterized by a loss of shear strength in the affected soil layers, thereby causing the soil to behave as a viscous liquid. This effect may be manifested at the ground surface by settlement and, possibly, sand boils where insufficient confining overburden is present over layers. In order for the potential effects of liquefaction to be manifested at the ground surface, the soils generally have to be granular, loose to medium-dense and saturated relatively near the ground surface, and must be subjected to ground shaking of a sufficient magnitude and duration. The potential for liquefaction to occur is considered non-existent, given the lack of a water table and lack of soils on-site.

7.4. Dynamic Settlement

Geologic materials of significantly different properties (i.e. cohesion, cementation, density, moisture, constructed cut/fill contacts, etc.) can experience differential movement and dynamic settlement in response to earthquakes events. Poorly consolidated materials, may experience settlement (dynamic compaction) due to seismic shaking. The potential for dynamic settlement in response to an earthquake is considered non-existent, given the lack of soils in the area of the plug.

8.0. PLUG ASSESSMENT

Design and construction of the tunnel plug as well as the characteristics of the surrounding environment in the mine were reviewed to determine the applicability of the plug to meet its intended purpose.

8.1 Site Reconnaissance

SGSI performed four site visits of the tunnel plug within Easy Go Adit between April 2006, and August 2010. The first site visit involved the observation for any sediment “trapped” behind the bulkhead (SGSI, 2006). The second visit included observing and photographing the tunnel plug and adjacent adit bedrock. The third and fourth visits involved the detailed mapping of the adit walls for a distance of 100’ upstream and downstream from each bulkhead face.

8.2 Geologic Mapping of Plug Area

SGSI performed detailed structure logging of the Easy Go Adit bedrock for a distance of 100' out from each tunnel plug face, with particular emphasis on any discontinuities, including joints, joint infill character (i.e. soluble, erodible, tight, etc.), shears, faults, seepage, fractures, and lithology. The structure log (Appendix A, Plate 8) includes a portion of the original geologic log that was prepared by the Union Carbide Corporation (Appendix A, Plate 6, and Plates 7A-7P).

Plate 8 illustrates a total of approximately twenty-three relatively minor shears, two of which intersect the plug, and three of which produce seepage up-gradient from the plug. The two shears that intersect the plug displayed no seepage, no measurable offset, no thickness, and no infilling. All other shears were located outside the plug and exhibited varying thicknesses ranging between negligible and one-quarter-inch maximum. Infill character was noted on only four shears, one with calcium carbonate (non-soluble, tight, and non-erodible) and the others with silt gouge (non-soluble, erodible, and tight). The results of the structure log confirm that the plug was constructed at a stable location in the adit, within solid diorite that is relatively free of significant shearing.

8.3 Strength of Rock Mass

Laboratory testing was performed on several core samples taken from the rock mass in the area of the plug. The rock mass at the tunnel plug is quartz diorite (Gray et al, 1960). Laboratory tests included ASTM D2936 - Standard Test Method for Direct Tensile Strength of Intact Rock Core Specimens; ASTM D3967 - Standard Test Method for Splitting Tensile Strength of Intact Rock Core Specimens; ASTM C293 - Standard Test Method for Flexural Strength (Using Simple Beam with Center-Point Loading); ASTM C7012 (Method C) - Uniaxial Compressive Strength of Intact Rock Core Specimens. The strength test results according to each method are provided in Appendix D.

8.3.1 Shear and Wedging Failure

The factors of safety for resistance to shear previously calculated by Nassar (2002), were based on estimated rock quality in a static system, and did not take into account site seismicity factors. The design values and factors of safety and have been subsequently reviewed and recalculated based upon actual rock strength results acquired from rock cores in the plug area (Appendix B), and from site specific seismic factors.

In the original design of the plug Nasser (Appendix B) assumed the following:

2000’ head of water (867 psi)
500 psi effective external hydro pressure
5 psi safe shear based on 525’ overburden @ 10x factor of safety
101 psi safe shear strength of rock

The tunnel plug was constructed as planned. Rock core samples were tested to verify in-situ shear strengths. The more recent assessment of rock quality indicated the following:

1,336 psi bedrock splitting tensile strength
1,360 psi bedrock direct tensile strength
2,495 psi bedrock flexural strength
12,418 psi average uniaxial compressive strength

Based on the calculations the plug is sufficiently constructed to withstand increased shear and wedging pressures from both static and seismically effected (dynamic) water forces.

8.4 Hydraulic Jacking

The plug location was chosen because of the quality of the rock mass, which is monolithic, impermeable and has little to no jointing and fracturing. The original design process is documented in Appendix B. The bulkhead was designed to withstand a pressure force of 867 psi (Nasser 2002). Impound test data from 2003 showed water levels reached a maximum recorded height of approximately 1,219 feet of head (528 psi, 250 af), which is approximately 281-feet below the maximum impoundment height where water can exit to daylight from the adit 1,500 above the bulkhead (Plate 5, Appendix E). From the data it appears that the pressure force will not exceed the design parameters.

Based on the Selmer-Olson (1969) paper on unlined pressure shafts, failure and/or leakage of these systems are generally associated with inadequate depth of the burial of a tunnel. For an average rock mass, the vertical stress due to the weight of the rock increases 1 psi for every depth of foot. Consequently, in order to contain a water pressure of 528 psi the minimum cover should be 528 psi. In the case of EZ-go the cover over the top of the tunnel plug is estimated at 1200’. Though there are fracturing systems in the cover above the tunnel, the amount of cover over the tunnel is sufficient to withstand the pressure force and therefore a blowout failure due to jacking above the plug is unlikely.

8.5 Sediment Accumulation

Erosion within the mine system and therefore sediment accumulation behind the bulkhead, excluding sediment and debris dislodged during seismic events, is very limited due to the geology. Water discharge records from 2004 corroborate the low sediment levels within the system. Turbidity levels were measured at less than 5 NTU each day during drainage indicating that sediments in the water discharged are minimal.

The bulkhead was designed to withstand a pressure force of 867 psi against sliding, which is equivalent of approximately 2,000 feet of impounded water. Impound test data from 2003 showed water levels reached a maximum recorded height of approximately 1,219 feet of head (528 psi), which is approximately 281-feet below the maximum impoundment height where water can exit to daylight from the adit 1,500 above the bulkhead.

Although initial structural design calculations did not include additional pressures from sediment accumulation, we have included additional pressures from a theoretical full width and height sediment buildup behind the plug to see if the additional load would have a negative impact on the bulkhead. An accumulation of sediment to 15'x 15' x 100' behind the bulkhead would impose a pressure force of roughly 35 psi or an added 4.0% of design limit (see attached calculations).

If sediment were to accumulate to roughly this assumed theoretical value while water was fully impounded to the maximum recorded level, the maximum pressure force exerted on the bulkhead would be approximately 563 psi or 65% of the design value (Appendix B).

Periodic monitoring of turbidity in the mine drainage water can serve as a warning signal for signs of any materials being mobilized in the mine by excessive water flows or internal failures.

8.6 Water Hammer

Once water is impounded behind the plug it will have a static pressure head and therefore there will be no air gaps, or free water surfaces to slosh and create water hammer effect during a seismic event.

8.7 Cave-ins

Potential hazards from cave-ins are likely to occur in disturbed areas and or in areas where the rock is naturally weak. A cave-in could occur upstream or downstream of the plug but is unlikely to occur in the area where constructed as the rock is massive and relatively stable.

A variety of mining methods were used in the Pine Creek Mine including timber supported open stopes, cut and fill, and sub level caving. In the event that the wooden staging and framing gives way, a slide of unconsolidated material could enter the mine water reservoir. The worst case is that the slide falls from above water level into the reservoir, gaining momentum as it falls freely. However, it is unlikely that the leading edge of the slide could act as an effective piston across the entire opening to transfer all of the slides momentum directly to a pulse in water pressure. Gravity would cause the slide to occupy only the lower portion of the inclined stope, causing primarily displacement, rather than impact forces. In this case the magnitude of the pressure wave at the Easy Go Adit plug would equal only the additional height of water displaced by the slide.

8.8 Concrete and Reinforcement

The design life of the concrete plug is not known although extensive measures were taken to ensure that its service life was as long as possible. Long life design details include the use of 4000 psi cement, (3000 psi concrete was specified on the design plans) and perimeter grouting around the plug. At present the plug does not show signs of any significant deterioration.

#6 rebar @12" ew was used for reinforcement. Concrete stresses and radial pressures acting against the plug were recently reevaluated (Appendix B, Nasser 2011). Homogeneous concrete stresses were deemed acceptable in its present configuration.

Concrete durability with regard to acid attack has not been assessed at this time. Water quality testing should be performed to ensure that pH levels will not affect concrete performance. If water pH is found to be acidic, a resin coating or equivalent may need to be applied to all exposed concrete faces.

8.9 Shear Failure between Pipes and Plug

The design of the plug anticipated possible weaknesses of the pipe-concrete bond and was designed to reduce seepages and to restrain the pipes from being pressed outwards under pressure.

A potential failure mode of the pipes could be initiated by corrosion, leading to a failure of the pipe within the plug. An inspection of the piping was outside the scope of this study; however corrosion of the piping was observed during our review. We recommend that a thorough inspection of all piping be performed prior to impoundment of water.

9.0 CONCLUSIONS

Based upon the results of this study, The Easy Go Adit plug appears unlikely to fail in any catastrophic mode as it is adequate in length, the walls were well roughened, the stress in the rock is applied uniformly, and the tunnel walls in the area of the plug are tapered, putting much of the contact area into compression.

In addition, there is redundancy in the resistance to failure available in the plug configuration. Both longitudinal shear and wedging blowout tension are resisting the downstream movement of the plug. These two resistive mechanisms may be assumed to share the applied load.

The long term stability of the plug may be affected by seepage through the rock if the grouting was not sufficiently effective or if the grout cement has been eroded by acid attack. Monitoring of seepage at the downstream end of the plug and the surrounding rock will provide an effective check of trends in the stability of the plug-rock contact and the tunnel rock against hydraulic erosion and hydraulic pressure changes.

At present no instrumentation has been installed to measure pressures and flows of impounded water. Instrumentation should be installed and testing conducted to measure the pressures and flows involved in filling and draining the reservoir as well as to monitor the seepage response of the plug and the rock mass to the applied hydraulic gradients.

Other instruments should be installed to monitor any relative displacement of the plug within the tunnel rock or any creep of the piping within the concrete. An instrumentation program should be developed by others.

10.0 LIMITATIONS

This report has been prepared for the sole use and benefit of our client. The conclusions of this report pertain only to the site investigated. The intent of the report is to advise our client of the geologic and geotechnical recommendations relative to the future development of the proposed project. It should be understood that the consulting provided and the contents of this report are not perfect. Any errors or omissions noted by any party reviewing this report, and/or any other geotechnical aspects of the project, should be reported to this office in a timely fashion. The client is the only party intended by this office to directly receive this advice. Unauthorized use of or reliance on this report constitutes an agreement to defend and indemnify Sierra Geotechnical Services Incorporated from and against any liability, which may arise as a result of such use or reliance, regardless of any fault, negligence, or strict liability of Sierra Geotechnical Services Incorporated.

Conclusions and recommendations presented herein are based upon the evaluation of technical information gathered, experience, and professional judgment. Other consultants could arrive at different conclusions and recommendations. Final decisions on matters presented are the responsibility of the client and/or the governing agencies. No warranties in any respect are made as to the performance of the project.

The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings within this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

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GEOTECHNICAL • GEOLOGY • HYDROGEOLOGY • MATERIALS TESTING • INSPECTION

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July 18, 2019

Subject: **REVIEW OF CONDITIONS - EASY GO ADIT TUNNEL PLUG**
Pine Creek Mine
Inyo County, California

Reference: **Seismic, and Geotechnical Study**
Easy Go Adit Tunnel Plug
Pine Creek Mine
Pine Creek, Inyo County, California
SGSI Project Number 3.30716; Dated December 2011

Response to State Water Resources Control Board Comments
Pine Creek Mine
Pine Creek, Inyo County, California
SGSI Project Number 3.31321; Dated September 2015

Mr. Francis:

In June 2019, Sierra Geotechnical Services, Inc. (SGSI) performed a visual review of the tunnel plug system to assess any changes in the plug, piping, and/or bedrock since the date of issuance of our above referenced report. Our assessment included visual observation of the bedrock, the concrete plug, the contacts between plug and bedrock at both ends, and the associated piping/plumbing, as well comparative observation of photographs from 2011 and present. At the time of our review the plug manhole and all valves were open allowing water to flow unimpeded.

With exception of the piping, the concrete as well as the bedrock are effectively in the same condition as when the original report was prepared. No signs of instability of the bedrock or degradation of the concrete were noted. The valve pipes and manhole cover however show some signs of rust, which may be superficial, but should be further evaluated prior to impoundment, by the project mechanical engineer.

SGS
SIERRA GEOTECHNICAL SERVICES, INC.

In our opinion the tunnel plug is in substantially the same condition as when the original study was prepared. Therefore, the conclusions and recommendations included in the above referenced report and letter remain applicable.

We appreciate the opportunity to be of service to you. Should you have any questions regarding this report, please do not hesitate to contact us.

Respectfully,

SIERRA GEOTECHNICAL SERVICES, INC.

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