Technical Memorandum

Midnite Mine
Geotechnical Investigations and Existing Waste Rock Piles and Open Pit Highwalls Stability Evaluations

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Prepared for:
US Environmental Protection Agency
Region 10

by
URS Corporation
8181 East Tufts Avenue
Denver, CO 80237

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1.0 INTRODUCTION

1.1 Purpose

The purpose of the geotechnical investigations at the Midnite Mine site is to collect geotechnical information to support evaluations of remedial alternatives for the waste rock piles and open pit highwalls in the remedial investigation/feasibility study (RI/FS). This includes evaluating the slope stability of the waste rock piles and open pit highwalls at the level of detail needed for the RI/FS purposes. Moreover, these geotechnical investigations gathered groundwater level data and information concerning the geologic characteristics of fractures that are useful for the hydrologic characterization and modeling activities needed for the RI/FS. This technical memorandum discusses the activities related to the geotechnical investigations conducted by URS in September and October, 2000 as part of the RI/FS, the results of laboratory testing conducted on material samples obtained during the geotechnical investigations, and the results of stability evaluations performed for the South Spoils and Hillside Dump waste rock piles at the site. In addition, it provides the results of a geologic reconnaissance to assist in evaluating the stability of the open pit highwalls and potential on-site borrow locations for cover materials, which are needed to evaluate remedial alternatives for the RI/FS.

Previous trenching, drilling, and sampling of the waste rock and stockpiles performed by other organizations was conducted for purposes other than geotechnical investigations.

1.2 Scope

The scope of work for the geotechnical portion of the Midnite Mine RI/FS consists of collection and evaluation of information pertaining to the physical stability of the South Spoils, Hillside Dump, and open pit highwalls. Activities conducted included:

- Excavation of test pits in the South Spoils and Hillside Dump to observe subsurface conditions, collect samples for laboratory testing, and measure in-situ density;
- Drilling of test holes and installation of nine piezometers in the South Spoils and Hillside Dump to observe subsurface conditions, collect samples for laboratory testing, perform penetration resistance testing, and collect groundwater level information;
- Laboratory geotechnical testing on selected samples collected from the test pits and test holes to assist in classification and evaluation of engineering properties relevant to earthmoving and stability of the remedial alternatives. The testing included moisture content, grain-size, Atterberg limits, specific gravity, compaction, and triaxial shear strength tests;
- Performing stability analyses for the South Spoils and Hillside Dump waste rock piles using suitable computer programs;

- Geologic mapping of the Pit 3, Pit 4 and Adit Pit highwalls to assist in evaluating stability for closure options and to provide information on fracture characteristics relevant to the hydrologic modeling;
- Preparation of a technical memorandum that summarizes the results of the geotechnical investigations, laboratory testing and stability evaluations.

In addition, to support the RI/FS, a preliminary off-site soil borrow reconnaissance was performed in April 2000 and a "desktop" geotechnical evaluation of waste dump Detail Areas 1, 2, 3, and 4 was conducted in 2001. The results of these investigations and evaluations are provided in references listed in Appendix F.

Historical site documents including letters, reports, and photographs were reviewed during our evaluations. These reports were prepared by various organizations including the Bureau of Land Management (BLM), Dawn Mining Company (DMC), and Shepherd Miller, Inc. (SMI). A list of the documents is provided in the appendices.

1.3 Report Organization

This report is organized into seven sections of which this introduction is Section 1. Section 2 presents a description of the mine site geology. Section 3 presents the results of field and laboratory investigations and geologic mapping of the open pit highwalls. Section 4 presents the results of the stability evaluations. Sections 5, 6 and 7 present conclusions/recommendations, general information and credits, respectively.

2.0 GEOLOGY

The Midnite Mine is located in the northern portion of Washington State and the Columbia Plateau physiographic province. The plateau and cover of basalt flows have been dissected by the Columbia River and tributaries in the area around the mine site to form deep valleys and rugged terrain. Bedrock underlying the basalt has been exposed in the area around the mine, and includes Precambrian metamorphic rocks of the Togo Formation that were intruded by Cretaceous age quartz monzonite. The region contains a general north-south to north-30°-east structural trend, including large folds, shears, and fault zones.

Ore deposits at the Midnite Mine were located along the steeply-dipping contact between the quartz monzonite intrusion and the phyllite and calc-silicate host rocks. Pits 3 and 4, and the Adit Pit (see location on Figure 1) are oriented roughly in a north-south direction along the contact, and have a western highwall composed mostly of quartz monzonite and an eastern highwall composed of phyllite and calc-silicate rocks. The quartz monzonite consists of a gray, porphyritic, medium-grained, and typically strong rock that is altered to highly altered and weak in many areas along the intrusive contact.

The metamorphic rocks of the Togo Formation include phyllite, consisting of gray to black, fine-grained, strong rock interbedded with calc-silicate rocks consisting of gray to white, fine grained, thin-bedded, and strong to very strong rocks. The phyllite was observed to locally have a schistose texture near portions of the intrusive contact and was often also highly altered and mineralized. The calc-silicate rocks contain beds of marble, quartzite, and hornfels. Bedding is present in the phyllite and calc-silicate rocks and is oriented generally north-south to north-30°-east and dips about 45° to 70° southeast.

Mineralization in the rock mass includes sulfides that are currently in the process of weathering. The rock mass contains heavy iron oxide stains and coatings, and other weathering products such as sulfate salts. Clay materials are present in portions of the rock mass that have been sheared and faulted.

3.0 INVESTIGATIONS

3.1 General

Geotechnical investigations of the South Spoils, Hillside Dump, and open pit highwalls were performed at the Midnite Mine in September and October, 2000. The field program included: (1) test pit excavation, sampling and density testing; (2) Becker test hole drilling, penetration testing, sampling and piezometer installation; and (3) geologic mapping of the open pit highwalls. The laboratory testing program included index property, compaction, and triaxial shear strength tests on material samples obtained from the test pits and test holes.

3.2 Test Pit Excavation, Sampling and Testing

A total of nine test pits were excavated as part of the Midnite Mine test pit field effort. Test pits were excavated in both the South Spoils waste rock pile located near the southern portion of the mine and in the Hillside Dump waste rock pile located southwest of Pit 4.

During test pit excavation, the geotechnical engineer logged the materials encountered in the test pit, performed in-situ density (sand cone) tests, collected waste rock samples, and photographed features exposed within the test pit. In addition, maximum particle size, and the percentage and distribution of cobbles and boulders were visually estimated. The test pit data and field density test results are provided in Table 1. Additional test pit information is provided on the test pit logs included in Appendix A. The test pits were backfilled with excavated materials upon completion.

3.2.1 South Spoils Test Pits

Test pits TPSS-01 through TPSS-06 were excavated in the South Spoils waste rock pile (Figure 1). The test pits were excavated to depths ranging from 10 to 14 feet below the ground surface. In general, the test pits encountered coarse-grained materials consisting of sand, gravel, cobbles, and boulders. Test pit TPSS-01 encountered clay from a depth of 8 feet to the bottom of the pit (14 feet).

In-situ density testing was conducted on the waste rock in the test pits using the sand cone method. The testing indicated dry densities ranging from 95 to 124 pounds per cubic foot (pcf), and wet densities ranging from 101 to 138 pcf. One in-situ density test in Test Pit TPSS-02 encountered an open void in the coarse waste rock resulting in loss of sand and did not provide valid results. Natural moisture contents of the waste rock in the test pits ranged from 6 to 11 percent.

The wide range of grain sizes of the materials encountered in the test pits, and the presence of an open void, indicate the highly heterogeneous nature of the waste rock, which significantly restricts remedial alternatives that involve materials segregation.

3.2.2 Hillside Dump Test Pits

Test pits TPHD-01 through TPHD-03 were excavated in the Hillside Dump waste rock pile (Figure 1). The test pits were excavated to depths ranging from 11.5 to 12 feet below ground surface. In general, the test pits encountered coarse-grained materials consisting of gravel and boulders.

In-situ density testing was conducted on the waste rock in the test pits using the sand cone method. The testing indicated dry densities ranging from 104 to 113 pcf, and wet densities ranging from 110 to 125 pcf. The density test in Test Pit TPHD-01 at 3 feet depth indicated a dry density of 57 pcf and a wet density of 61 pcf. A void in the coarse waste rock may have been encountered at the location of this test, thereby resulting in the low density calculated. Natural moisture contents of the waste rock in the test pits ranged from 6 to 11 percent.

3.3 Test Hole Drilling, Testing, Sampling and Piezometer Installation

A total of nine (9) test holes were drilled using an AP 1000 drill rig fitted with a Becker hammer and a six-inch open bit. Test holes were drilled in both the South Spoils and Hillside Dump waste rock piles. Samples of the test hole drill cuttings were generally collected every 5 feet for visual logging and laboratory testing using a cyclone and bucket sampling apparatus. Standard penetration testing (SPT) was conducted at approximately 10-foot intervals in the test holes. Open-bit Becker penetration resistance was recorded throughout drilling. The test hole data is provided in Table 2. The test hole logs are included in Appendix B.

3.3.1 South Spoils Test Holes

Test holes THSS-01 through THSS-06 were drilled in the South Spoils waste rock pile (Figure 1). The material logged in the test holes generally consisted of sand and gravel. The Becker drilling method resulted in some mechanical breakage of the waste rock during drilling. The SPT blow counts for each test hole are summarized in Appendix C. The SPT blow counts ranged from 9 to 83 blows per foot. At several depths, the SPT yielded poor recovery and/or was obstructed by rock fragments during testing. A 1-foot thick void was encountered in Test Hole THSS-06 at a depth of 18 feet.

3.3.2 Hillside Dump Test Holes

Test holes TPHD-01 through TPHD-03 were drilled in the Hillside Dump waste rock pile (Figure 1). Here also, the material logged in the test holes consisted mainly of sand and gravel, and mechanical breakage of the material resulted from the Becker drilling method. N_{becker} and SPT blow counts are summarized in Appendix C. The SPT blow counts ranged from 16 to 92 blows per foot. The SPT yielded poor recovery and/or was obstructed by rock at several depths.

3.3.3 Groundwater

During drilling of the South Spoils and Hillside Dump test holes, groundwater was encountered in only one test hole, THHD-02 at the Hillside Dump, at an approximate depth of 99 feet below the ground surface.

Upon completion of drilling, all test holes were completed as piezometers with protective steel casing and locking cap. The screen intervals of the piezometers varied in depths from 77 to 141 feet at the South Spoils, and from 90 to 99 feet at the Hillside Dump. The piezometer installation records are included in Appendix B.

Groundwater was measured in the piezometers in August, 2001. No free water was found in any of the piezometers. However, traces of mud were observed on the tip of the measuring probe upon retrieval from the bottom of the piezometers at Test Holes THHD-02 and THHD-03. The groundwater observations are summarized in Table 2.

3.4 Laboratory Testing

The subsurface samples obtained from the test holes and test pits were logged, labeled, sealed and delivered to Advanced Terra Testing laboratory in Lakewood, Colorado, for further examination and testing. Index and engineering property tests were conducted on selected representative samples to assist in classification and to provide information on engineering properties. The laboratory testing was conducted following applicable ASTM standards. The testing consisted of moisture content determinations (ASTM D 2216), specific gravity tests (ASTM D 854), Atterberg limit tests (ASTM D 4318), particle size analyses (ASTM D 422), compaction tests (ASTM D 698), and triaxial shear strength tests (ASTM D 4767). The laboratory test results are included in Appendix D, and described below.

The laboratory test results indicate that the waste rock materials from the South Spoils pile and the Hillside Dump pile appear to have similar physical characteristics. The moisture contents of the waste rock material generally range from 2 to 9 percent with two samples as high as 23 percent. The specific gravity ranges from 2.75 to 2.84. Gradation tests of test pit samples indicated gravel, sand and fines (silt and clay) contents of 5 to 65 percent, 21 to 43 percent, and 11 to 29 percent, respectively. Gradation tests of test hole samples indicated gravel, sand, and fines contents of 5 to 72 percent, 16 to 55 percent, and 9 to 48 percent, respectively. Atterberg limit test liquid limits and plasticity indices ranged from 19 to 52, and 1 to 29, respectively. Nine (9) samples tested were non-plastic. The laboratory test results are summarized in Tables 3, 4 and 5, and on Figures 2 through 7.

Bulk samples of the waste rock materials obtained from the test pits excavated at the South Spoils and Hillside Dump piles were used for the triaxial shear strength tests (consolidated-undrained with pore pressure measurements). Materials from the test pits with similar index characteristics (gradation and Atterberg limits) were combined into one composite sample and tested. A total of four composite samples were developed and tested for shear strength. Index tests were also performed on the four composite samples. The test specimens were remolded to dry densities at approximately 90 percent of the standard proctor maximum dry densities and near the natural moisture contents. The diameter of the test specimens was 6 inches for composite sample C and 4 inches for composite samples A, B and D. The triaxial test results indicated effective cohesion (c') values ranging from 0 to 650 pounds per square foot (psf) and effective friction angles (ϕ') ranging from 32.6 to 43.7 degrees. The results of the index and shear strength laboratory tests on the composite samples are summarized in Table 6.

3.5 Geologic Mapping of the Open Pit Highwalls

A geologic reconnaissance was performed at the Midnite Mine to assist in evaluating the stability of the open pit highwalls for closure options. Work included geologic mapping, collection of other geologic information, and documentation of stability conditions at Pit 3, Pit 4, and the Adit Pit. Stability evaluations using this information and other information are planned as part of closure option evaluations.

The geologic reconnaissance included observation of the highwalls stability conditions, mapping of geologic units exposed in the highwalls, and collection of geologic information regarding weathering, mineralization, and the location and orientation of geologic structures. In addition, potential borrow areas for cover materials (relatively unmineralized materials) were investigated on a reconnaissance level.

Results of the geologic mapping at Pit 3, Pit 4, and the Adit Pit are shown on Figures 8, 9, and 10, respectively. Figures 8 and 9 include the general locations of the pit limits and the pit benches. These figures also show the distribution of rock types and large geologic structures (shears and faults) exposed in the highwalls. Areas on the highwalls observed to have evidence of instability are also shown. Information about rock mass conditions (rock type, degree of weathering and alteration, fracture spacing, joint set orientations, bedding orientations, and additional geologic notes) at representative locations in the highwalls is summarized in Table 7. Locations on the highwalls where the information was obtained are shown on Figures 8 through 10 with numbers corresponding to the area numbers referenced in Table 7.

3.5.1 Highwall Stability Conditions

Highwalls within Pit 3, Pit 4 and the Adit Pit (including overall highwalls and multiple benches) appear to currently be generally stable. Instability of portions of individual benches and a general tendency for the rock mass to ravel were observed and are related to geologic structures and mechanical and chemical weathering of the rock mass. These two general stability conditions are discussed below.

Overall highwall stability appears to be favorable because the pits are generally oriented such that large geologic features that might contribute to highwall instability are favorably oriented and located. For example, the shear and fault zones that parallel mineralized contacts in the curved north highwalls in Pits 3 and 4 are oriented normal to the highwall slope. In addition, bedding within the metamorphic rocks exposed in the east highwalls dips into the highwalls. Bedding is present in the phyllite and calc-silicate rocks and is oriented generally north-south to north-30°-east and dips about 45° to 70° southeast.

Jointing within the rock mass forming the highwalls typically consists of 3 to 4 joint sets, with joint continuity (areal extent of joint surface with trace length visible on the surface) of up to 20 to 50 feet. Joints with this relatively low continuity (compared to bedding of faults with continuity of hundreds of feet) may result in individual bench failures, but not multiple bench or overall highwall failure. The joints could form a stepped failure surface through the rock mass; however, the stepped surface would include portions through intact rock, which is relatively strong. This mode of failure is possible, but would require a loss of strength of the rock mass at depth within the highwall.

Pit 3, Pit 4 and the Adit Pit contain a number of geologic structures important to stability conditions in the highwalls. Each pit contains portions of the contact between the Togo formation and the quartz monzonite intrusion. This contact zone has been weathered, altered, sheared, and mineralized with the result that the rock mass along the contact is relatively weak. These relatively large geologic structures are exposed in the north highwalls of all three pits.

Unstable conditions observed in the highwalls of Pit 3 and Pit 4 include a number of individual bench failures and a general loosening and raveling of the rock mass exposed on the benches. Individual bench failures include benches that appeared to have failed during mining, benches that partially failed during mining, and benches that have probably loosened and are currently in the process of failing (Figures 8 and 9). These bench failures occurred where joints within the rock mass were unfavorably oriented, and the failed portion of the bench slid on the joint surface. An example of this type of failure is located in the northwest corner of Pit 4 where three benches failed (apparently during mining) due to unfavorably-oriented bedding and foliation within the phyllite. At this location, the highwall contains numerous large bedding and foliation planes exposed in the rock mass that dip 45° to 70° toward the pit. In general, these joint-controlled failures are not common in the benches, probably because the intact rock is strong to very strong. On many of the bench faces, the rock mass is in the process of mechanically weathering (ice wedging) and chemically weathering (sulfide weathering). The result is that the highwalls tend to loosen and ravel onto the bench below. Some of the benches are completely filled with rock debris and allow rock to fall to the next bench below. These weathering processes are quicker in portions of the rock mass that are mineralized, altered, closely fractured, sheared, and faulted. Loosening, raveling, and accumulation of debris probably will continue until the highwall slopes reach the angle of repose of the broken materials.

3.5.2 Cover Borrow Materials

Potential borrow locations for cover materials were identified at a reconnaissance-level during the field mapping conducted at Pit 3. A potential borrow area, located in soil and weathered rock on top of the east highwall (Figure 8) was selected as possible suitable material. The soil consists of clayey material estimated to be up to 15 to 20 feet thick. The soils probably were derived from underlying highly weathered to weathered calc-silicate bedrock. The area of this potential borrow source extends to the east, beyond the limits of Pit 3, into a forested area that slopes to the southeast. Additional data to evaluate the acid-generating potential and radiological characteristics of this material will be necessary to ascertain whether it is suitable for use as cover material.

4.0 SLOPE STABILITY EVALUATIONS

4.1 General

Utilizing the topographical data provided to us for this project by SMI and the geotechnical information from the field and laboratory investigations described in Section 3, slope stability analyses were performed for the South Spoils and Hillside Dump waste rock piles. This section describes the analyses performed to evaluate the physical stability of the piles under both static and pseudo-static (seismic) loading conditions, and provides the results of the analyses.

4.2 Analysis Model

Current static and pseudo-static (seismic) slope stability analyses were performed using limit equilibrium method slope stability evaluations that compute the theoretical factor of safety. By definition, the theoretical factor of safety is the factor by which the shear strength within the slope must be divided to place the slope into a static state of limit equilibrium where the slope is just stable.

Slope stability computations were performed using the computer program UTEXAS3 (Wright 1991). Spencer's procedure of slices (1967) was used for computing the factors of safety. UTEXAS3 is a two-dimensional limit equilibrium slope stability computer program. The Spencer procedure satisfies conditions of static equilibrium including horizontal and vertical force imbalance and moment imbalance. Search routines, available within UTEXAS3, were used for both circular and non-circular trial shear surfaces to locate the most critical shear surface.

4.3 Geometry

Slope stability analyses were performed along three cross-sections at the South Spoils and two cross-sections at the Hillside Dump. The locations of these cross-sections are shown on the plans of the waste rock piles on Figures 11 and 12, respectively. The five cross-sections illustrate various aspects of the piles geometry including steepest slope, highest pile area, and the presence of the Pollution Control Pond (PCP) at the toe of one section.

The cross-section profiles are illustrated on Figures 13, 14 and 15. These profiles show the existing waste rock pile surface, the pre-mining ground surface, and the locations of the URS test holes (projected to the plane of the section). The existing and pre-mining ground surfaces are based on the topographical surveys available for the waste rock piles supplied by SMI. The depths to the natural ground surface measured during drilling of the test holes generally agree with the elevation of pre-mining ground surface on the topographic map.

For each waste rock pile cross-section, the minimum factor of safety against failure was computed. In some cases, the failure surface was deep-seated, occurring near the bedrock interface. In other cases, the failure surface was of intermediate depth, occurring within the waste rock material. Both of these modes of failure are considered massive and would undermine the overall physical safety of the waste rock piles. In addition, shallow non-plane and plane shear surfaces were analyzed for each of the waste rock piles.

4.4 Material Properties

No appreciable groundwater was found in the test holes drilled in the piles during the field investigations or was measured subsequently in the installed piezometers. Effective shear strength parameters for the waste rock were used in the stability analyses because of the lack of groundwater and the granular nature of the waste rock material.

Beneath the waste rock, natural weathered bedrock was encountered in eight of the nine URS test holes. This material was conservatively modeled in the stability analyses as a 10-foot thick foundation layer using effective shear strength parameters.

Material unit weights and shear strength parameters used in the stability analyses were based on the results of the geotechnical field and laboratory testing programs, and on URS's past experience with similar projects. The field testing program included standard penetration testing (SPT) and open-bit Becker penetration resistance testing in the test holes, and in-situ density testing in the test pits. The laboratory testing program included triaxial shear strength tests on composite samples from the test pits. The results of the triaxial tests are considered to be conservative because: (1) only the waste rock fraction passing the 1½-inch sieve was used for the tests, (2) the remolded specimens received only nominal compaction, and (3) the specimens were saturated, which does not occur in the field at this site. Figure 16 shows a summary of the triaxial test results for the waste rock material and the design shear strength parameters used in the analyses.

Shear strength parameters recommended for rockfill in published literature (Barton and Kjaernsli 1981 and Corps of Engineers 1970) are shown in Figure 17. Figure 17 also shows the shear strength parameters adopted by SMI in their slope stability analyses of the South Spoils waste rock pile (1996). Comparing these parameters with those selected by URS for this study, and considering the specimen conditions during the triaxial tests as mentioned above, it can be concluded that the design shear strength parameters used in the stability analyses are conservative.

The material properties used in the computer analyses are presented in Table 8.

4.5 Phreatic Surface

As indicated in Section 3.3.3, groundwater was encountered near the bottom of only one test hole during drilling, and none was found in the piezometers 10 months after their installation (Table 2). A phreatic surface was conservatively modeled in the slope stability analyses at 5 feet above the pre-mining ground surface.

4.6 Seismic Acceleration

Pseudo-static (seismic) conditions modeling the effects of earthquake loading were considered in the slope stability analyses. The pseudo-static stability analysis approximates earthquake loading as a constant horizontal force represented by an equivalent pseudo-static acceleration. This type of analysis is applicable to embankments and structures that do not exhibit liquefaction or significant shear strength loss due to the build up of large pore water pressures resulting from seismic shaking (Seed 1979). The waste rock piles meet these conditions.

The project site seismicity was evaluated using the national seismic-hazard maps developed by the United States Geological Survey (USGS 1996). Using these maps, the seismic horizontal peak ground acceleration for the Midnite Mine site was estimated to be about 0.06 g for a 10 percent probability of exceedence in 50 years. The horizontal seismic acceleration used by SMI in their pseudo-static stability analyses of the South Spoils waste rock pile (1996) was 0.1 g. Based on this information, a horizontal seismic acceleration of 0.1 g was used in the pseudo-static stability analyses in this study.

4.7 Results

The results of the stability analyses are summarized in Table 9. Cross-sections showing the individual shear surfaces and their corresponding computed static and pseudo-static (seismic) factors of safety are shown on Figures 18 through 22. In all but one case, the computed theoretical static factors of safety for the cross-sections evaluated were 1.3 and greater, and the computed theoretical pseudo-static factors of safety were greater than 1.0. These computed factors of safety are consistent with accepted engineering practice.

In the case of shallow shear surface in the slope immediately above the PCP at the South Spoils waste rock pile (Cross-Section SS1), the minimum computed theoretical static and pseudo-static factors of safety for a non-plane shear surface were 1.1 and 0.9, respectively. This indicates that the slope at that location has marginal safety level against shallow, surficial-type failure. The level of safety could decrease with saturation by heavy rains or earthquake loading.

5.0 CONCLUSIONS/RECOMMENDATIONS

The approach to the waste rock pile stability evaluations included the following activities:

- Site reconnaissance;
- Geotechnical field and laboratory investigations;
- Geotechnical model development; and
- Performance of stability analyses.

Based on our reconnaissance of the South Spoils and Hillside Dump waste rock piles, and on the results of the geotechnical investigation and stability analyses, our conclusions and recommendations are provided below.

1. Based on the field and laboratory investigations carried out to date, the waste rock piles consist mostly of coarse sand and gravel material with cobbles and boulders. The investigations and piezometer monitoring information indicate groundwater is of very limited extent within the South Spoils and Hillside Dump waste rock piles. Groundwater was only encountered in one of the nine test hole borings, near the base of the Hillside Dump.

2. Fractures exposed in the highwalls of Pit 3 and 4 are closely spaced and vary widely in orientation.

3. The stability analyses performed did not indicate any mass or large-scale instability of the waste rock piles analyzed under current static and pseudo-static (seismic) conditions. The calculated minimum factors of safety generally meet or exceed currently accepted practice for waste rock piles. Shallow, surficial-type failures are possible within the few and limited steep portions of the piles, such as above the PCP at South Spoils. This could particularly be the case following heavy rains or high magnitude seismic events. The failures would generally consist of sloughing and raveling of surface material, and if properly and promptly repaired, should not affect the overall stability of the piles.

4. Overall highwall stability appears to be favorable as large geologic features that might contribute to highwall instability are favorably oriented and located. Loosening, raveling, and accumulation of debris on benches will continue.

5. Monitoring of the piezometer water levels should be performed.

6. Reclamation efforts should be directed to maintain and control surface water, minimize saturation of exposed slopes, minimize the possibility for development of phreatic surfaces within the piles, and minimize surface erosion.

7. We recommend a site reconnaissance of the waste rock piles be performed twice annually, before and after the winter season. The site reconnaissance should be performed and documented by a qualified geotechnical engineer or engineering geologist. Field observations should include:

    - General area conditions;
    - Site conditions (crest, slope, toe areas);
    - Surface conditions (crest, slope);
    - Cracking;
    - Slides/sloughs/scarps; and
    - Erosion.

6.0 GENERAL INFORMATION

URS represents that our services are performed within the limits prescribed by our clients, with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included in our proposals, contracts, reports, or memorandums.

It is important to note that the condition of a waste rock pile depends on changing internal and external conditions, and is evolutionary in nature. The present condition of the pile may change and may not represent the condition of the pile at some point in the future. Only through periodic inspections can unsafe conditions be detected, so that corrective action can be taken. Likewise, continued care and maintenance are necessary to minimize the possibility of development of unsafe conditions.

7.0 CREDITS

The following URS personnel were involved in the site geotechnical investigations and engineering analyses for the Midnite Mine project. The field exploration was performed under the observation of Chuck Hart, Kirk Palicki, Dale Baures and Chris Williams. The laboratory testing program was managed by Kirk Palicki, Jim Scott and Charles Khoury. The slope stability analyses were performed by Birgit Dixon and Charles Khoury. This report was prepared by Birgit Dixon and Charles Khoury, and was reviewed by Jim Scott, Task Manager.

Tables

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Test Pit Logs

Appendix B

Test Hole Logs and Piezometer Installation Records

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Test Hole Becker and SPT Blow Counts

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Laboratory Test Results

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Field Investigation Photos

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