Stabilization of a Rock Pile Moving on a Pre-sheared Foundation at Questa, New Mexico

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Abstract

In 2003 concerns were raised by both state regulators and the nearby town of Questa, about the potential mobility of the 300 metre high Goathill North rock pile at the Questa Molybdenum Mine, located in the Rocky Mountains of New Mexico. The 3.4 million m$^3$ rock pile was constructed between 1969 and 1973, when foundation movements were initiated at the toe of the pile underlain by pre-sheared material that had been creeping for over 30 years. This paper outlines the extensive investigations and instrumentation needed to verify the movement mechanism and to develop a mitigation program acceptable to all stakeholders. The mitigation program was divided into four phases involving the placing of a rock under drain and toe buttressing prior to moving equipment onto the moving rock pile for regrading and construction of surface water control features. Considerable effort was made to balance over 760,000 m$^3$ of cut and fill and ensure that the sequencing of material movement maintained stable and safe conditions. Integral to the mitigation, was a comprehensive instrumentation and monitoring program that continued for over a year after completion.

Introduction

The work at Goathill North was initiated in June, 2003 after concerns were raised by the state appointed Stability Review Board regarding potential down-valley risks associated with foundation shearing movements. After an initial inspection and meetings with all stakeholders, immediate concerns were allayed and a conceptual mitigation plan was proposed that was subject to a comprehensive program of investigation and monitoring, as well as detailed geotechnical evaluations.

Data gathering and mitigation designs were completed and approved in 2004 and construction activities continued from July 2004 to August 2005. The nominal one year post-construction monitoring period occurred between August 2005 and September 2006 and an as-built stability report issued in April 2007 confirmed that all defined success criteria has been met. Instrumentation continues to be read and visual inspections are carried out every six months.

Rock Pile Setting and History

The Questa Mine is located near Taos in northern New Mexico and at the initiation of this project, the mine was owned and operated by Molycorp Inc. (now Chevron Mining Inc.). The Goathill North rock pile is situated at the head of the Goathill drainage, about 3 km upstream from Red River valley.
There are several actively eroding alteration scars in the valley that deposit materials at the base of the scar slopes as talus or colluvial deposits. Storm activity within the steep mountain watershed has mobilized these materials resulting in naturally occurring debris flows. Flows that have reached the bottom of the valley at the confluence with the Red River, were deposited within a horizontally layered debris fan.

To gain a full appreciation of the potential factors contributing to the slide movements at Goathill North, a detailed review of historical aerial photography was conducted. Stereo pair photographs were available from 1962 and 1963 before the open pit was developed, through 1969 to 1974 during construction of the rock pile, to post construction, failure and creep during the period 1976 to 1997. The review of aerial photography concluded that Goathill North rock pile was constructed in an area characterized by alteration scars. There were indications of historical slide activity underlying the area of slide movement, suggesting that the basal zone was pre-sheared. However, the pre-mine air photo did not show any signs of active landsliding. At the head of the valley drainage a local rock ridge runs up the valley separating the slide from stable rock pile material.
Foundation movements associated with the initial development of the current slide occurred between 1969 and 1973. These movements were associated with lateral and down valley displacement in the colluvial bench at and under the toe of the north side of the rock pile. These movements continued to occur for more than 30 years after their initiation.

**Investigation**

Most of the information used to evaluate the slide and develop the mitigation plan was collected during a field investigation program carried out from July to October 2003. Investigations included 23 test holes located on the Goathill North rock pile underlain by alteration scars, and 5 test holes located on the adjacent Capulin rock pile, where no pre-mining alteration scars were evident. This information was supplemented by previous investigation information from 2 test holes at Goathill North and 2 test holes at Capulin that were drilled in 2002.

The geotechnical investigation included a number of drilling methods to advance the test holes to depths ranging from 17 to 114 metres, through mine rock, colluvium and underlying bedrock. Truck mounted Becker hammer and CME-75 drill rigs and a track mounted ODEX/core rig were used to drill the test holes, conduct sampling, conduct in situ testing, and install instrumentation in the test holes. In situ testing was conducted in some of the test holes to measure permeability using open hole test methods and packer test equipment.

A total of 36 vibrating wire piezometers were installed to measure pore pressures at select locations within the test holes, such as at the bottom of the mine rock, within the colluvium, adjacent to possible slide failure surfaces and within the bedrock. Slope inclinometer casing was installed in 18 of the test holes to provide an indication of where the slide was moving and also confirm that the apparently stable part of the rock pile was not moving. In addition, slope movement monitoring prisms, tilt meters, extensometers, and crack meters were installed by the mine to provide an indication of surface movements.
Figure 1: Goathill North rock pile with drill hole locations

Laboratory testing was carried out on samples obtained from the test holes to provide index and engineering properties of the materials, including natural moisture content, density, grain size analyses, Atterberg limits, specific gravity, direct shear tests, and X-ray diffraction.

Foundation Conditions

Mid-Tertiary age rhyolite tuffs and andesite flows with porphyry intrusions underlie Goathill North. The intrusions are cut by a later mineralizing stock that was responsible for the formation of pyrite, molybdenite and secondary silicates. At the Questa Mine, higher pyrite contents are commonly associated with quartz-sericite-pyrite halos around the molybdenite orebodies, which are prone to low temperature weathering leading to clay formation. Field mapping and drill core visual estimates clearly indicate higher pyrite concentrations directly beneath Goathill North.

A clayey weathered bedrock zone at the top of the bedrock forms the interface with surficial soils in the valley. The photo below shows the weathered bedrock exposed in the gully below the scar, which in the slide area is typically 3 to 6 metres thick. The very deep weathering at this location was likely associated with high angle faulting.
Sounding of old exploration drillholes in the Goathill area, as well as the 2003 geotechnical drilling, indicated a bedrock phreatic surface 150 to 180 metres below the drainage. This deep system is separate from a shallow perched water table on top of the bedrock surface that slopes gently down valley sub-parallel to the ground surface.

**Geotechnical Properties**

Mine rock material at Goathill North exhibits a wide variation in gradation ranging from cobble sized material to silty sands. Most of the material is a sandy gravel with less than 20% fines and clay contents less than 10%. A lower bound shear strength of c’=0 and phi’=36° was supported by triaxial testing and by the angle of repose of the rock pile slopes at the Questa Mine, which are mostly in the range of 36° to 38°.

The Goathill North colluvium also shows a wide variation in gradation ranging from cobble sized material to sandy clay. Road cuts and drillhole logs showed that the colluvium could be divided into an upper zone of sandy gravel material and a lower zone of mixed sandy gravel and weathered bedrock containing the higher silt and clay sized materials. The upper colluvium was a sandy gravel material with a similar texture to the finer grained mine rock. Based on the results of direct shear testing and considering the similarity to the mine rock, the upper colluvium was assumed to have a shear strength of c’=0 phi’=36°.

The weak shear zone in the lower colluvium was located within a 6 to 9 metre thick zone above the weathered bedrock. The contact with the upper colluvium was gradational and characterized by higher clay contents and the first appearance of white to light grey clayey gravel. In many cases blow counts reduced at the top of the weak zone and in other cases there were rafted weathered bedrock blocks within the zone. While most movements took place on a fairly discrete shear plane, descriptive information on the logs showed that pre-shearing has occurred throughout the zone. The lower contact was defined by distinctly higher blow count values and for the purpose of stability analysis, the lower contact was considered to be a hard layer forming a lower boundary for potential slip surfaces. A shear strength of c’=0 phi’=23.8° was assigned based on back analysis and direct shear testing.
Slide Geometry And Mechanism

Figure 3 shows a plan view of the slide with key information on the sliding surface derived from the extensive network of slope inclinometers that permitted detailed evaluation of slide movements. The contours on the drawing represent elevations of the shear surface derived directly from the well defined shears evident on the slope inclinometer profiles. The total slide volume, based on the slip surface and topographic contours was 1.9 million m$^3$, comprised of 1.3 million m$^3$ of mine rock and 0.6 million m$^3$ of valley colluvium.

Movement vectors for the slide plane were derived from the slope inclinometer data, with lengths corresponding to rates of movement in inches per month. Movement rates indicated three different zones, a) a compression zone (5 to 18 mm/month) adjacent to the western boundary of the slide where most of the rotation is taking place as the slide changes direction towards the south, b) an over-thrusting zone (30 mm/month) where there is an upper zone of shearing at about 9 metres above the main shear plane, c) the main body of the slide (30 to 80 mm/month) where the shape of the shear plane and generally higher movement rates along the southeast edge are consistent with a rotating slide mass.
Cross sections through the centre of the slide oriented normal to the direction of movement showed that the slide zone was about 15 to 25 metres thick and that sliding was occurring along a lower surface dipping at about 20° beneath the colluvium bench and about 30° along an upper surface beneath the rock pile. The geometry and displacement vectors indicated that the landslide could be classified as a “translational slide” and that the velocity class was “very slow” according to criteria proposed by Cruden and Varnes (1996).

![Section B-B](image_url)

**Figure 4 Cross section B-B through the centre of the slide**

Back analyses to calculate the shear strength on the shear surface were carried out to recover an “average” value operating on the whole surface. Shear strength values were found to vary between 23° and 25° with the slightly higher strengths towards the south. This is consistent with the slide rotation and suggests there is some significant internal shear resistance within the mass. These results compare well with the average friction angle of 23° measured from the direct shear testing.

**Table 1. Summary of Back Analyses for Section’s A, B and C**

<table>
<thead>
<tr>
<th>Section</th>
<th>Limit Equilibrium Friction Angle for constant shear strength along slip surface</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>22.8°</td>
</tr>
<tr>
<td>B</td>
<td>23.7°</td>
</tr>
<tr>
<td>C</td>
<td>25.0°</td>
</tr>
<tr>
<td>Average</td>
<td>23.8°</td>
</tr>
</tbody>
</table>

Although slide movements were not entirely in a down-valley direction, back analyses were carried out on Section E-E to assess the frictional resistance in this direction. Results indicated a weak zone friction angle of 22.8°, very close to the average value of 23.8° for Sections A, B and C. This suggested some significant resistance accounting for the rotational movement at the toe and preventing the whole mass from sliding in the down valley direction.
In order to further address the potential for down valley movements and internal shearing within the mass, three dimensional stability analyses were also carried. The back analyzed friction angle for three dimensional limit equilibrium conditions was 20.5°, somewhat lower than the 23° to 24° values derived from two dimensional analyses. Assuming that the higher values are more representative of the actual mobilized weak zone shear strength, results in a three dimensional factor of safety in the down valley direction of 1.16 to 1.20. This was further evidence of a resistant ridge at the toe of the slide preventing large scale down valley movement.

Mitigation Design

Several options were considered during the development of the mitigation plan, all of which were directed towards achieving adequate geotechnical stability based on two prime goals:

- Removal of driving mass from the upper part of the slide
- Increasing the shear resistance in the lower part of the slide.

Removal of driving mass from the head of the slide was shown to be effective due to the steeper slope and foundation under this part of the slide. The extent of removal was limited by practical considerations such as access and disposal options, as well as economics. Also, the current distribution of mass leads to reducing incremental gain in stability as the unloading progresses down slope. The most effective way to increase the shear resistance in the lower part of the slide, was to place unloaded material and additional material from the stable part of the rock pile, where it will either increase resistance in the zone of shearing or lengthen the path of any potential shearing through higher strength granular material.

Other mitigation options were considered, but these were discounted on the basis of cost, short term or interim stability concerns, or access limitations. These included construction of a shear key at the toe of the slide, injection grouting and groundwater depressurization. Different sizes of buttress fills were also evaluated to determine the incremental gain in down valley stability with increasing mass. This showed that in order to achieve significant benefit in stability, a very large incremental volume of fill was required in the toe berm area and this was therefore difficult to justify.

The mitigation plan had four phases, from installation of the under drain through to the construction of surface water management facilities. In accordance with the design criteria, final slope profiles were designed to increase the stability of the overall sliding mass by a minimum of 20% to achieve a factor of safety of at least 1.2 at the end of major earthworks construction.

Phase 1 involved placing the rock drain in the erosion gully at the toe of the slide area. The purpose of the drain was to prevent build up of groundwater pressure beneath the rock pile and to maintain the phreatic surface at pre-mitigation levels. The drain was constructed with processed, inert, coarse granular material, with a gradation compatible with the rock pile and colluviums.

Phase 2 involved an estimated 390,000 m³ cut on the stable part of the pile with the corresponding fill placed as an initial toe buttress in the erosion gully where the shear surface daylighted. This phase was designed to achieve some initial stabilization of the original slide movements before starting work on the slide itself. The cut also significantly improved the stability of the adjacent areas of the rock pile.
Figure 6  Phase 2 Mitigation

Phase 2 dozer push
Phase 3 Mitigation

Phase 3 involved unloading the slide by pushing an estimated 336,000 m$^3$ from the upper portion of the slide down to the 2945 m elevation and then re-grading the slope below this point by dozing down an additional 57,000 m$^3$ to achieve an overall gradient between 2H:1V and 2.5H:1V. The upper unload removed all the material down to the base of the shear surface leaving a final slope in original ground at an angle of 1.5H:1V. Phase 4 involved constructing an interim drainage system on the mitigated surface to handle the 100 year 24-hour precipitation event and control erosion until final reclamation activities would commence.

Monitoring

Monitoring data was collected from slope inclinometers and piezometers before and during the mitigation earthworks and for a period of one year following the completion of construction.

Table 2. Summary of Slope Inclinometer Data

<table>
<thead>
<tr>
<th>Zone</th>
<th>Slope Inclinometers</th>
<th>Target Maximum Velocity</th>
<th>Measured Velocity</th>
<th>Meets Target?</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outside the slide area</td>
<td>SI-25, SI-26</td>
<td>Should show no evidence of basal shearing</td>
<td>No basal shearing</td>
<td>✓</td>
</tr>
<tr>
<td>Compression/ buttress zone</td>
<td>SI-23, SI-31</td>
<td>1.8 mm/month</td>
<td>0.025 mm/month</td>
<td>✓</td>
</tr>
<tr>
<td>Slide zone</td>
<td>SI-33, SI-34, SI-35</td>
<td>7.6 mm/month</td>
<td>0.076 mm/month</td>
<td>✓</td>
</tr>
</tbody>
</table>
By April 2007, based on the slope inclinometer data it was confirmed that the slide had been arrested. Movements that ranged from 5 to 80 mm/month prior to mitigation were reduced by the regrading activities to near zero (a factor of about 1000). Some small creep movements were detected that were attributed to consolidation of non-compacted soil fill. These were expected to decrease to rates similar to those measured in other stable rock piles at the mine.

Data from the piezometers installed in the rock pile were compiled following the one year monitoring period and generally showed that groundwater levels were steady or slightly declining within the rock pile. During construction, there were spikes in the pore water pressure noted in several piezometers, caused by a combination of loading during the push activities and precipitation. However, these elevated levels returned to normal following the completion of construction.

**Stability Analysis of the As-Built Rock Pile**

The 2005 as-built topography was combined with the existing foundation information to create the updated sections through the rock pile. The geotechnical properties of the materials used in the analysis were the same as the original design and the sections were analyzed with the same piezometric surfaces used in the original design, given that no significant changes in water table elevation were recorded. The results of the stability analysis met or exceeded the target factors of safety.

**Table 3. Summary of 2-D Stability Analysis of the As-Built Rock Pile**

<table>
<thead>
<tr>
<th>Cross-Section</th>
<th>Actual As-Built FOS</th>
<th>Design Target FOS</th>
<th>Meets Target</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Specified Slip Surface</td>
<td>Critical Slip Surface</td>
<td>Specified Slip Surface</td>
</tr>
<tr>
<td>Section A</td>
<td>1.21</td>
<td>1.16</td>
<td>1.17</td>
</tr>
<tr>
<td>Section B</td>
<td>1.41</td>
<td>1.37</td>
<td>1.36</td>
</tr>
<tr>
<td>Section C</td>
<td>1.44</td>
<td>1.36</td>
<td>1.42</td>
</tr>
<tr>
<td>Average of A, B, &amp; C</td>
<td>1.35</td>
<td>1.30</td>
<td>1.32</td>
</tr>
</tbody>
</table>

**Ongoing Assessment**

Visual inspections and monitoring of instrumentation are continuing on a six monthly basis. The latest results indicate that while movement velocities remain well below the set targets, in one inclinometer a small discrete displacement of about 10mm has developed near the original shear zone under the upper part of the slide. No such discrete movement has been detected downslope or anywhere else under the rock pile. This movement has been attributed to ongoing minor settlement and consolidation of the upper rock pile that is compressing upslope behind the stabilized colluvial bench.

Seepage from the toe of the rock pile has been observed over the last few years. Although there was no evidence of a seep or blockage of the under drain observed in 2010, potential geochemical processes that may be affecting the performance of the drain are currently under investigation.

**Conclusions**

The extensive review of all available historical information, together with comprehensive geological, geotechnical and hydrogeological investigations were instrumental in fully characterizing the causes.
and the configuration of the Goathill North rock pile movement. This allowed an appropriate mitigation program to be put in place that was acceptable to the owner, the regulators and other local stakeholders.

After a full year of monitoring following completion of the mitigation, a detailed review of all information gathered from the instrumented rock pile concluded the following:

1. The Goathill North rock pile was performing as intended and was geotechnically stable based on assessment of instrumentation readings, factors of safety and visual inspection.
2. In areas adjacent to the slide, monitoring indicated there were no movements initiated by the mitigation activities.
3. Slope inclinometers within the slide mass indicated only barely perceptible movements near the weak failure plane and no discrete shearing noted at the failure plane.
4. There was no evidence of sustained acceleration along the basal shear plane in the buttress/compression zone or the main slide zone.
5. The calculated factor of safety for the basal shear surface and the critical slip surface of 1.35 and 1.30, respectively, show that the design targets were achieved by the mitigation work.
6. Visual inspection of the rock pile revealed that the rock pile was performing as intended one year after mitigation work was completed. There were no signs of any deep seated instability.
7. Pore water pressures within most of the rock pile were confirmed to be at or below pre-mitigation elevations.

Ongoing monitoring of the performance of the rock pile has indicated a small displacement near the weak failure plane that is not accelerating and is well below the target velocity. This is attributed to residual lateral compression in the upper part of the rock pile.

References