Design, Implementation and Monitoring of Novel Rehabilitation Measures in an Open Pit Gold Mine

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Abstract

Heavy rainfall increased water inflow from a paleochannel and caused a large scale flow of completely weathered material from a section of wall of an open pit gold mine. The debris flow completely overtopped the haul ramp at a time when mining was nearly completed yet a large amount of broken, high value ore remained in the base of the pit. A number of slope and ramp rehabilitation measures were proposed and evaluated. After the detailed evaluations, a rehabilitation technique with a tactical construction procedure was implemented. One of the key components of the rehabilitation was extensive monitoring of the failed material prior to, during and following implementation. Extraction of the remaining ore was completed successfully and the effectiveness of the rehabilitation measures were demonstrated by their ability to sustain subsequent heavy rainfall and a large earthquake that caused significant structural damage in the nearby town of Boulder.

1 Introduction

In the Kalgoorlie area of the Eastern Goldfields region in Western Australia there are many relatively shallow open pit mines. Some of these mines have walls formed in completely weathered rock and sometimes intersect highly permeable, water bearing paleochannels with large catchment areas. While the Kalgoorlie region may be classified as being “dry”, it can sometimes be subject to heavy downpours of rain as the consequences of cyclones formed off the north-west coast that move in a south-easterly direction and pass over the goldfields area. The adverse effects of water on slope stability in weathered materials are well known. The consequences of large scale failures include sterilisation of ore or complete loss of access to recover ore.

In the case study to be described, a failure initiated in the wall of a pit formed in paleochannel material. The saturated material flowed in an uncontrolled manner to cover the haul ramp and prevent access to valuable broken ore in a crown pillar at the base of the pit. The writers formed part of a team asked to provide possible options for re-establishment of the haul ramp so that the ore could be recovered in a safe manner with minimal risk to personnel and equipment. The options considered and complementary investigations will be described in detail. The solution adopted, the method of its implementation and some of the problems that arose and how they were addressed will also be described. Finally, an assessment will be made of the rehabilitation measures.

2 Background

The Alacer Gold South Kalgoorlie Operations are located 25 kilometres south of Kalgoorlie in Western Australia. The operations, formerly owned by Dioro Exploration, comprised several conventional open cut mines, some of which intersect inactive underground workings. The south-western sector of the Mt Marion open pit intersected a major paleochannel. Maintenance of stable wall conditions in this sector was problematic almost from initiation of mining in the mid 1990s. Intermittent slumping and minor failures within the south-western sector of the Mt Marion pit were typically managed with remedial earthworks and excavation. A major slump
from this sector occurred in October 2008, with debris from the paleochannel and adjacent highly weathered pit batters flowing to the base of the pit as shown in Figure 1. The presence of this failure material prevented access to the ore zone located at the base of the pit. The south wall was monitored using electro-optical distance measurement (EDM) techniques, and data showed that the failure debris remained mobile after the time of the initial failure.

As part of an ongoing mining geotechnical review, investigations were commenced to assess the likely viability of restraining debris movement sufficiently to enable resumption of operations in the pit. The objective was to restrain the upslope failure debris to permit re-establish ramp access to the base of the pit.

Figure 1. Photo of slump from south-west corner of Mount Marion pit.

3 Design considerations for rehabilitation of slope

The general design considerations were to:

- Minimise the movement of the upslope failure material.
- Effectively manage the upslope water situation.
- Provide medium-term stability to allow re-entry to the pit floor.
- Provide safe containment of all potential rock falls.
- Provide monitoring features for ongoing monitoring of position.
3.1 Options considered for rehabilitation

A number of potential methods for achieving the design objectives were considered. These included:

- Major cutback to the southern wall, including modification to the waste dump close to the pit crest.
- Terraced retaining wall.
- Partial terraced retaining wall with steel and concrete piles replacing the lower wall segments.
- Sequential removal of failed material and construction of a rock gabion wall restrained by ground anchors.
- Installation of sheet piling restrained by ground anchors with subsequent removal of failed material.
- Excavation of a trench to be back filled rock and removal of failed material to expose the haul ramp.
- Sequential construction of an anchored concrete (shotcrete) pad.
- Sequential removal of failed material and construction of a rock buttress.
- Hybrid solution involving two or more of the above solutions.

The merit of each solution needed to consider the cost, likely effectiveness and schedule for supply of materials, preparation and construction. Preliminary calculations showed steel beams within concrete piles, within practical and economic limits of construction, were not able to resist the required moment loads. Modified steel placement within the piles and tiebacks were suggested as potential solutions to improve the capacity. However, a trial of installation of ground anchors with subsequent pull testing showed that drilling and installation of ground anchors would be problematic and the depth required to penetrate to fresh rock below the failed material would be excessive. It was also considered that it would be very difficult to get suitable equipment access on to the failed material for installation of sheet piling. Finally, it was decided that it would be more economic to construct a rock buttress compared with the use of rock gabions and to be able to achieve the following objectives:

- Maximise the permeability of the buttress to prevent hydrostatic pressure building behind the wall
- Maximise the resistive force achieved by the rock buttress
- Allow for (easy and effective) future additions and modifications to the buttress

Accordingly, only the design, construction and monitoring of the waste rock buttress will be presented and discussed in detail.

3.2 Geometric design of rock buttress

Previous geological investigations (Osborne 2009) were used to estimate the lower boundary of the failure volume. This was assumed to be at the top of weathered rock which was known to have reasonable strength compared with the completely-weathered, failed material. It was also known that the depth of completely-weathered materials was deeper towards the western side of the failure due to its closer proximity to the influence of the paleochannel.

It was decided early in the design process that, irrespective of the actual rehabilitation measure, it would be necessary to estimate the demand on the buttress imposed by the failed material. A number of circular failure mode stability analyses were performed using the program SLIDE (Rocscience 2008). Simple laboratory tests and back analysis of the failure were used to estimate appropriate values for the material physical and mechanical properties given in Table 1.

A lower bound force was calculated using a simple retaining wall model and Rankine theory. The model assumed a smooth, vertical, water-permeable retaining structure (wall) and cohesionless soils with an uphill debris slope of 10°. The lower bound force was calculated to be just under 1000kN per metre strike length.

An upper bound force was calculated using a simple “block on a slope” model. The model assumed the failed material acted as a solid block sliding down a slope with no deformation or lateral dissipation of load. Friction between the block and the layer below was assumed to be equal to the estimated internal friction angle of the failure material (28°). The upper bound force was found to be ~ 2800kN per metre width.
Table 1. Methodology to estimate and assess values of failed material properties.

<table>
<thead>
<tr>
<th>Process of Force Calculation</th>
<th>Result</th>
<th>Accuracy of Method</th>
<th>Assessment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Estimated a range of realistic friction angles from Atterberg test results</td>
<td>$27^\circ \leq \phi \leq 29^\circ$</td>
<td>Acceptable</td>
<td>Reasonable</td>
</tr>
<tr>
<td>Unit weight of soil assumed to be 20 kN/m$^3$</td>
<td>$\gamma = 20$ kN/m$^3$</td>
<td>Acceptable</td>
<td>Reasonable</td>
</tr>
<tr>
<td>Assumed a failure surface from the failed cross-section</td>
<td>Circular failure</td>
<td>Could be improved</td>
<td>Conservative</td>
</tr>
<tr>
<td>Back calculated a value for cohesion assuming Factor of Safety $= 1.0$</td>
<td>$0 \text{ kPa} \leq c \leq 5$ kPa</td>
<td>Relies on failure surface being realistic</td>
<td>Reasonable</td>
</tr>
<tr>
<td>Increased cohesion within a computer model until Factor of Safety $= 1.2$</td>
<td>13 kPa</td>
<td>Acceptable</td>
<td>Variable due to ground</td>
</tr>
<tr>
<td>Change in cohesion used to calculate the total force required to retain debris</td>
<td>2300 kN/m width</td>
<td>Acceptable</td>
<td>Within bounds</td>
</tr>
</tbody>
</table>

The force (2300kN/m) given in Table 1 is between the upper and lower bound estimations of 1000kN/m and 2800kN/m, respectively. It was considered to be reasonably conservative and was adopted for the design of the waste rock buttress.

The failure zone was a complex system containing various mixtures of clay, sand and rock fragments with a wide range of sizes. There were also varying degrees of saturation and deformation within the failed material as well as an undulating surface geometry that was relatively steep towards the rear and side scarps of the failure.

It should be noted that all the calculations of the total force from the failed ground were based on simplified two-dimensional models of the failure. The real failure was three-dimensional and contained the following complexities that may not have been fully (or suitably) incorporated into each of the simplified models:

- Internal deformations and forces.
- Variable water pressures.
- Variable failure surface and material properties.
- Three dimensional reactions (beneficial or adverse effects from lateral confinement).
- Tension cracks.
- Frictional losses.
- Potential for composite failure mechanisms/surfaces.
- Continued ground movement.

The buttress was designed with the following assumptions:

- Required to resist approximately 2300kN/m.
- Water-permeable formation to prevent development of hydrostatic pressure.
- Base to be sufficiently keyed into in situ material to prevent sliding failure.

The buttress was designed using the geometry shown in Figure 2 with various values of parameters appropriate to the different materials and the interfaces between them as summarised in Table 1.

Assuming the maximum force for each 5m interval and using a design force of 2300kN, the buttress dimensions detailed in Table 2 were chosen, subject to the following provisos:

- The buttress dimensions were a best guess estimate of the minimum requirement and provided as a guide only. It was recognised that the geometry may need modification during construction.
• Failed material should be completely removed and excavation should then continue ≥ 1m into undisturbed ground.
• The buttress should follow the horizontal curvature of the failure (arching out to the south).
• The buttress should make contact with competent pit wall at the east and west extremities.

If the recommended buttress width could not be accommodated, the width was to be made as wide as permitted and the height then increased to ensure the same total volume of rock was included.

Figure 2. Geometry used for design and stability assessment of rock buttress.

Table 2. Summary of buttress design.

<table>
<thead>
<tr>
<th>Interval (m)</th>
<th>Estimated %</th>
<th>Driving Force (kN)</th>
<th>Recommended Width (m)</th>
<th>Recommended Height (m)</th>
<th>Volume of Waste Rock (m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-5</td>
<td>10</td>
<td>230</td>
<td>4</td>
<td>4</td>
<td>100</td>
</tr>
<tr>
<td>5-10</td>
<td>30</td>
<td>690</td>
<td>8</td>
<td>5</td>
<td>200</td>
</tr>
<tr>
<td>10-15</td>
<td>65</td>
<td>1495</td>
<td>11</td>
<td>8</td>
<td>450</td>
</tr>
<tr>
<td>15-20</td>
<td>90</td>
<td>2070</td>
<td>12</td>
<td>10</td>
<td>600</td>
</tr>
<tr>
<td>&gt; 20</td>
<td>100</td>
<td>2300</td>
<td>14</td>
<td>10</td>
<td>700</td>
</tr>
</tbody>
</table>

3.3 Water management

In recognition of the detrimental effects of water on stability, a Water Management Plan (WMP) was developed. The WMP included a comprehensive set of measures to manage the surface and sub-surface water. These measures included:

• Temporary decant towers to reduce the volume of water within the failure material.
• Diversion channels and bund walls formed within the upslope failure material to encourage water towards the decant towers and minimise infiltration into the failed material. These channels were also configured to intercept all major flow paths of water from the paleochannel and also from rainfall runoff. Interception channels developed behind the failure to reduce inflow and ponding.
• Relief wells in the form of vertical boreholes with perforated casings located towards the rear of the failure and pumped to remove water. It was recognised that inflow from the paleochannel could be larger than the pump rate of a relief well.
3.4 Monitoring

A comprehensive monitoring programme was developed to monitor for any potential problems during and after construction. The monitoring included:

- Installation of EDM prisms and automated survey of to record movement of the buttress and upslope failure material.
- Visual inspections of the buttress, adjacent areas and southern pit crest.
- Inspections of decant towers and other peripheral water management installations to ensure correct operation.

Acceptable qualitative movement and water parameters were developed as the information was gathered.

4 Risk assessment

A risk assessment was conducted given the importance of safety for personnel working in the confined location at the base of the relatively small pit. Once completed, the planned buttress was a single layer of rock fill spanning the width of the slope failure at about the mid depth of the pit. For the risk assessment, “failure” was considered to be any change to the buttress, surrounding walls or upslope failure material that would render conditions unsafe to continue mining within the pit. The purpose of the assessment was to examine the potential risks and consequences of such a failure.

This assessment examined the following aspects:

- Possible causes of failure.
- Failure modes.
- Probability of failure.
- Consequences of failure.

A “qualitative” Risk Assessment of the buttress design was completed since a “quantitative” risk assessment would have required more complex and detailed analysis with many assumptions regarding a large number of unknown parameters. The possible causes of failure considered and their likelihood of occurrence are summarised in Table 3 and Table 4 describes the probability and consequence of each failure mode. A response plan was also developed for emergency situations and the occurrence of unforeseen natural disasters such as minor and major earthquakes and extreme rainfall events.

<table>
<thead>
<tr>
<th>Cause of Failure</th>
<th>Probability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reactivation of failure (during construction)</td>
<td>Low – due to the relatively short period of construction</td>
</tr>
<tr>
<td>Reactivation of failure (post construction)</td>
<td>Very Low – once buttress is installed, stability will be increased</td>
</tr>
<tr>
<td>Load larger than estimated</td>
<td>Low – due to conservative assumptions</td>
</tr>
<tr>
<td>Buttress resistance lower than estimated</td>
<td>Low – due to conservative assumptions</td>
</tr>
<tr>
<td>Seismic Event</td>
<td>Very Low –“low risk” based on earthquake acceleration map (Geoscience Australia 2011) and not analysed</td>
</tr>
<tr>
<td>Extreme Rainfall Event</td>
<td>Low – if successful WMP is implemented</td>
</tr>
<tr>
<td></td>
<td>High – if WMP is not implemented</td>
</tr>
</tbody>
</table>
### Table 4. Probability and consequence of failure mode.

<table>
<thead>
<tr>
<th>Failure Mode</th>
<th>Probability</th>
<th>Consequence</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sliding Failure</td>
<td>Low</td>
<td>Moderate – Sliding failure could reduce the resistance of the buttress</td>
</tr>
<tr>
<td>Minor Collapse</td>
<td>Moderate</td>
<td>Mild – Buttress can be reformed</td>
</tr>
<tr>
<td>Major Collapse</td>
<td>Low</td>
<td>Major – Depending on location and size of collapse</td>
</tr>
<tr>
<td>Differential Movement</td>
<td>Low</td>
<td>Mild – Buttress can be improved</td>
</tr>
<tr>
<td>Total Movement</td>
<td>Very Low</td>
<td>High – Prevent safe access to base of pit</td>
</tr>
<tr>
<td>Foundation Failure</td>
<td>Very Low</td>
<td>Extreme – Buttress will not be able to be rectified</td>
</tr>
</tbody>
</table>

### 5 Implementation of adopted rehabilitation measures

The buttress was implemented in several stages. The first stage involved a trial of construction to between ~5 and 20m from the eastern edge of the failure. The effectiveness of the buttress was then assessed on the basis of EDM monitoring results and visual observation of performance. Following the successful construction and demonstrated effectiveness, construction was continued to completion. The stages of construction are listed in Table 5. Stages 1 and 2 were constructed using large size fresh waste rock (open pit waste). The waste rock was screened for Stage 3 construction to minimise the fines content. The success of the completed buttress was to be decided on the basis of EDM monitoring results and visual observation of performance.

### Table 5. Construction sequence.

<table>
<thead>
<tr>
<th>Construction Stage</th>
<th>Start Date</th>
<th>Completion Date</th>
<th>Supervisor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stage 1 (Trial)</td>
<td>9 October 2009</td>
<td>22 October 2009</td>
<td>Dioro</td>
</tr>
<tr>
<td>Stage 2 (Trial)</td>
<td>11 November 2009</td>
<td>18 November 2009</td>
<td>Peter O’Bryan &amp; Associates</td>
</tr>
<tr>
<td>Stage 3 (Final)</td>
<td>7 January 2010</td>
<td>19 January 2010</td>
<td>Peter O’Bryan &amp; Associates</td>
</tr>
</tbody>
</table>

### 5.1 Trial construction

The trial design covered the initial 20 linear metres of construction, progressing westward from the eastern edge of the failure, and assumed that the full driving force of the failure material was developed by this chainage.

#### 5.1.1 General description

The buttress was formed into a wall of waste rock designed to resist the movement of the upslope failure material. The buttress featured the following design elements:

- **Rock Fill** - The rock used for the buttress construction needed to be large in size and contain minimal fines, appropriate material was sourced as waste rock from previous open pit mining.
- **Keyway Trenches** – Keyway trenches were required to interlock the base of the rock buttress with the in situ material. This was to prevent the base of the buttress sliding over the in situ ground and to allow full engagement of the rock fill resistive force.
- **Temporary Decant System** – Large diameter (~1 to 3m) slotted towers surrounded by coarse filter rock were required immediately upslope of the construction to reduce the volume of water within the failure material. Water from the towers was to be pumped or connected to a passive water pipeline discharging...
collected water into the decommissioned underground workings. Polyethylene was used for the tower construction given the anticipated limited service requirements.

- Temporary Diversion Channels – Provision was made during construction for diversion channels to prevent water entering the construction area.
- Monitoring System – EDM monitoring prisms were installed on the crest and face of the rock buttress when construction was completed. Prisms were surveyed at least daily for the following three (3) weeks.

5.1.2 Construction process

The rock buttress was constructed incrementally to minimise the exposure of the failure debris at any time. Initial buttress construction commenced from the eastern end and incorporated an allowance for wet conditions. The construction method was as follows:

- A working platform was formed, being just large enough to enable the construction process.
- Failure material was removed downwards until in situ material was exposed.
- Keyway trenches were excavated running perpendicular to the direction of slip movement.
- Temporary water diversion trenches were excavated within the upslope failure material as necessary;
- Fresh waste rock fill was placed and shaped in the keyway trenches and the rock buttress.
- The process was repeated until the buttress was extended sufficiently to the west.
- EDM monitoring prisms were installed on the crest and face of the rock buttress.

5.1.3 Construction supervision and quality control

Initial buttress construction at the eastern side of the failure was considered to be safe and acceptable without engineering supervision. However, as the wall construction continued to the west, it was expected that the depth to in situ material would increase, as well as the mobility of the failure material as a result of decreased stability due to the adverse effects related to water. As such, it was recommended that full-time construction supervision by a qualified geotechnical engineer be provided during construction of the buttress as it extended into that less stable zone. This was to ensure that the design objectives were achieved.

The geotechnical engineer’s duties included:

- Ensuring the construction undertaken meets the design intent.
- Selecting materials to be used for construction.
- Implementing design modifications and field changes as required.
- Ensuring works were conducted as specified.
- Monitoring progress and keeping mine personnel fully informed.
- Ensuring health, safety and environmental procedures were adhered to.
- Issue of a construction report.

5.1.4 Performance assessment

The trial installation was completed on 18 November 2009. The performance of the trial buttress after construction was assessed as sound on the basis of observation and measurement. Displacement data from EDM prisms on the buttress and surrounding areas for over three weeks indicated that the movement of the failure debris had been retarded. Monitoring was continued.

The assessed performance of the trial buttress was such that it was recommended that construction be continued to aim to bridge the failure. The recommended buttress geometry given for the Interval > 20 in Table 2 was considered to be acceptable for the remainder of the buttress construction.
5.2 Final construction phase

The Stage 3 construction of the rock buttress comprised of a single material fresh rock buttress with keyway trenches. Additional decant towers were installed during and after Stage 3 construction with a pipeline to enable water to discharge into underground workings. Construction followed the sequence shown in Figure 3.

The Stage 3 section of the buttress (Figure 4) varied in height from ~12 m to 17 m and had a crest width of 13 m. The base of the buttress varied between ~15 and 20 m and the keyway trenches varied in number and dimension depending on base width and access during construction. The entire stage was constructed using screened waste rock sourced from several local waste dumps within the pit.

A safety berm was formed out of rock on the down slope side of the crest as part of the construction process. The safety berm was maintained for the entire construction process and remained in place post-construction.

![Sequential construction of the buttress.](image)

a. Pull back of Stage 1  
b. keyway trench  
c. buttress with dozer  
d. bund at crest of buttress

Figure 3. Sequential construction of the buttress.
5.3 Field modifications to design

Modifications to the design carried out onsite during construction are summarised in the following sections.

5.3.1 Buttress geometry

The buttress design was modified during construction to maximise effectiveness while remaining within the available space. The rear face, originally designed at 45° was steepened to approximately 60° once the failure material was found to have higher than expected short-term strength (i.e. longer stand-up time). This modification significantly increased the amount of rock fill that was able to be placed within the construction area.

The width of the buttress was reduced by a small amount due to the moment of the rear wall between excavation and backfill. This effect was minimal and can be resolved if necessary by adding rock to the outer face at a later stage.

The height of the buttress ended up being several metres higher than designed due to the in situ material being at a lower than expected elevation. To maintain realistic rock fill quantities, the crest was re-aligned to slope down towards the west. Even with this modification to the crest, the height of the buttress for the Stage 3 section remained higher than designed. This is beneficial to the effectiveness of the buttress and more than compensates for the small loss of width.

5.3.2 Keyway trench dimensions and spacing

Keyway trenches were originally designed to be approximately 2m deep and 2m wide with a spacing of approximately 2m. The practical construction of the trenches were wider, deeper and with larger spacing. This was due to the size of the machinery and ground conditions encountered.

5.3.3 Diversion channels

The exact hydrology of the area was undefined during design, so diversion channels were installed onsite as required. The construction area had significantly higher water inflows than previous stages of construction. This placed greater emphasis on adequate water management during construction.
5.3.4 *Shear pins*

On the basis of previous experience in slope rehabilitation (Thompson et al. 2004), shear pins were designed to retain the rock at the toe of the buttress. The shear pins were constructed with 250mm diameter by 5mm thick wall steel pipes and fully encapsulated with cement grout. The shear pins in upper primary row were 6 metres in length and protruded about 1m above the base of the buttress. These shear pins were used mainly to retain the upper level of the buttress. A secondary row of shear pins was installed along the edge of the ramp. The lower row of shear pins were 12 metres long, protruded between 1 and 2 metres above the ramp and passed beyond the failed material into fin situ material. Both rows of shear pins are shown in Figure 5.

![Primary row of shear pins](image1)

![Secondary row of shear pins](image2)

Figure 5. Shear pins used to retain the toe of the buttress to the failure material.

5.3.5 *Additional construction*

Debris material (up to ~ 4,500m³) was also removed from above the buttress to reduce the disturbing force on the buttress. Further fresh waste rock was added to the lower (down slope) face of the buttress. Additional excavation from the failure mass was carried out in early-mid March to reduce the active pressures acting on the failure mass. The active load had been increased by debris from crest collapse on the western side of the failure scarp. The entire mass of failure debris remained mobile; however, the buttress remained effectively in place. The buttress and failure zone as at 19 March 2010 are shown in Figure 6. Two decant towers (draining to the lower pit) were located behind the buttress, one within the corridor where greatest movement had occurred; the other immediately adjacent to the western edge of the failure. Excavation of debris from the lower pit had reached 255mRL (in the mid pit area) on 19 March 2010 with broken floor stocks exposed at the eastern end of the pit.

6 Assessment of rehabilitation measures

6.1 General comments

Following construction, the monitoring showed the failure debris was mobile. Greatest movement and highest movement rates were associated with the central flow zone/ corridor of the original failure; however, it was also
clear that the eastern abutment of the failure was moving towards the central corridor and down slope. It was inferred that the loss of confinement caused by excavation associated with buttress construction initiated the movement which caused cracking within the buttress. The buttress design was based on expected static loads; however, the movement initiated by the construction process could not be arrested as readily as previously anticipated.

However, the EDM data did show subtle signs of regression (slowing of movement rates). While the extent to which regression would continue was unknown, past movement of the failure rill had occurred in a stick-slip manner, as attested by markings on the failure scarp. On the eastern side there were signs of recent water flow over the failure debris and of upper material bearing on and displacing the underlying mass. It was inferred that observed cracking of in situ material was associated with this eastern side movement. Cracking was also evident in the western abutment to the failure zone and around the scarp of the failure.

Cracks behind the scarp were located close to the crest; there were no obvious signs of deeper disturbance. Future falls were anticipated from the western abutment and fretting from the scarp will continue with time and future rainfall. Surficial failures from these areas were expected to have only a minor (to negligible) adverse influence on movement of the total failure debris mass. Significant cracking was detected and further waste rock was added to the buttress and a row of shear pins installed (at ~ 305mRL) to (aim to) retard movement in late January 2010. Debris and buttress movement/cracking continued in early February 2010, driven by the load of the overlying debris (and exacerbated by ingress of groundwater/paleochannel flows) and promoted by excavation of material down slope of the buttress which, as well as reducing confinement, also exposed the basal clay zone/sliding surface.

6.2 Stability conditions

Maximum movement rates were restricted to the upper debris slope. The upper scarp continued to fret in the vicinity of the crest (Figure 6). However, steady conditions were observed around the eastern and western edges of the failure area. The upper row of shear pins provided various levels of restraint to the failure, depending on their location and the applied disturbing force and anchorage conditions into weak or competent material. The lower shear pins appeared to work well in constraining the face of the buttress.

Figure 6 South wall failure and completed buttress.
7 Concluding remarks

The construction of the Mt Marion trial rock buttress was completed successfully. The rock buttress was constructed in accordance with the design intent and specifications. Strict material selection combined with careful operation of equipment resulted in a high quality rock buttress installation. Construction was completed in a realistic time frame, considering the experimental nature of the work and the delays encountered. In summary, the rehabilitation was deemed to be economically viable and was successful in that the broken ore was recovered in a safe and efficient manner.

8 Acknowledgements

The writers wish to thank the management of Alacer Gold for giving permission to publish this paper and to acknowledge the valuable contributions by the various personnel that assisted in the successful implementation of a novel and challenging slope rehabilitation project.

9 References