The Wallaby Mine: Maintaining Pit Wall Stability for Continued Underground Mining

E. Jones  Barrick Gold, Perth, Australia
P. Andrews  Barrick Gold, Perth, Australia
S. Holley  Barrick Gold, Perth, Australia

Abstract

In 2006 three instabilities developed on the western and southern walls of the Granny Smith Wallaby Open Pit. These instabilities are known as Alpha, Beta, and Scallop slips. These slips continued to expand during pit mining, however at that time they were generally considered controlled due to the pumping undertaken. Prism and photogrammetry monitoring of these slips since the initial failure has revealed that all three are still currently active and moving at very slow rates.

Open Pit mining finished in 2006 and in the same period the underground mine increased its development and production rates. The Open Pit continues to service the underground mine by hosting critical pieces of infrastructure and with the current underground mine life until 2018 the pit will continue to be important to the underground operation.

The continued deterioration of these slips from large rainfall events and poor bore pump performance has recently raised concerns about the long term serviceability of the pit for access to some critical pieces of underground infrastructure. In light of these concerns two important questions was raised: “In the case of a large slip failure would these accesses be functional to evacuate underground personnel?” and “Will these walls last the duration of the underground mine life?”

An investigation into the current condition of the slips in conjunction with a back analysis was undertaken. In light of these results four options were assessed to enable continued use of the pit for the underground operation.

This paper summarises the results of the back analysis, the risk assessment and monitoring program employed that led to the decision to establish another portal. The paper also details the design process and execution of establishing the portal in this dynamic environment. Radar data collected during this process is also analysed the results summarised here.

1 Introduction

The Granny Smith complex is located in the north eastern goldfields of Western Australia, approximately 950 kilometres northeast of Perth and 23 kilometres south of Laverton. The Wallaby Open Pit commenced operations in December 2001 by Placer Dome Asia Pacific Group. The Wallaby Open pit straddles both a salt lake to the west and the in-situ land form to the north, east and south.

The pit was mined over four stages. The final “3B” cut-back was mined along the Eastern wall to provide permanent access from the surface to the 260mRL bench of the western wall. Open Pit mining was completed during December 2007. Since this time the Wallaby Pit has been used to provide vehicular access to the Wallaby Underground Gold mine and the location of several key pieces of infrastructure.

In 2006 three instabilities developed on the western and northern walls of the Wallaby Open Pit. These instabilities are known as Alpha, Beta, and Scallop slips.
Although the slips continued to expand during pit mining, they were generally considered controlled due to the pumping regime undertaken. Prism and photogrammetry monitoring of these instabilities since the initial failure verified the continued deterioration of these slips. The results of monitoring to date have revealed that all three are still currently active and moving at very slow rates.

Wallaby Underground has a reserve mine life until 2018 with potential to expand as the ore body has not been closed at depth. The underground has a long mine life which has initiated the wall stability project to ensure continued stability to the end of the underground mine life. Figure 1 illustrates the location of the slips in relation to the Wallaby Pit and underground portal.

Figure 1. (a) Plan view of Wallaby Pit and instability locations have been highlighted.
2  Local geology of Wallaby Pit

The Wallaby gold deposit is located in the southern portion of the Lancefield-Wallaby Basin, which is a tectonically late and fault-bounded basin. The deposit consists of a stacked series of multiple shallow dipping ore Zones of mineralisation, hosted by a thick matrix-supported mafic, (Wallaby), conglomerate. This is intruded by a suite of fractionated carbonatitic syenite and monzonite dykes. The Wallaby deposit host is part of the eastern limb of the Margaret Anticline with these sediments dipping moderately to the SE (Granny Smith Geology Department, 2010).

From the surface down the stratigraphy of the pit comprises a 20m to 30m thick blanket of Tertiary aged transported sediments. These are red-brown silty clays. Beneath these transported sediments lies an Archean Paleochannel. This paleochannel is of varying thickness, but is between 30m and 60m thick with the western end being thicker and hydraulically connected to the salt lake. (Name of lake). The Paleochannel is comprised of mixed gravels, sands, and clays and is the major conduit of ground water into the pit.

Beneath the Paleochannel lies the oxidised bedrock, which is commonly 10m to 20m in thickness. Relic rock structures are pervasive throughout this zone. The infill of these structures has been weathered to clay. The transitional material between the oxide zone and the bedrock is between 5m and 10m thick and grades from highly weathered to fresh rock.

2.1  Paleochannel channel

The location and thickness of the paleochannel and oxide material was established during open pit mining and exploration drilling. Figures 2 illustrate the instances where the paleochannel intersects the open pit and Figure 3 illustrates the depth of the paleochannel, highlighting the locations of the borehole pumps that are currently being used to control water seepage into the pit.

Seepage through the contact between superficial formations and upper transported clays continues to contribute to the deterioration of the slip and slip material, with undermining of slopes at the contact.

Figure 2.  Cross-section looking North of Wallaby Pit and intersection of Paleochannel and partial oxide contact
3 Background of previous assessment completed for wall instabilities

Deterioration of the three slips continued after the commencement of the underground mine. Concerns about this ongoing deterioration and the potential effect on the long term serviceability of some critical pieces of underground infrastructure within the pit were raised. In the case of a large slip failure would these accesses be functional to evacuate underground personnel.

At the start of 2009, a joint project between mine site personnel and the Yilgarn Shared Services (YSS) technical team was undertaken to establish the current stability of the walls and assess various options based on risk assessment outcomes. The project included a back analysis of the Scallop slip and a Risk Assessment of various options. The following are a summary of the results of these.
3.1 Geotechnical investigation of Scallop Slip

In 2009 poor pumping performance was thought to be the initiator for a period of rapid deterioration. A site investigation was made to observe wall conditions and establish slip mechanism. The following is a summary of key observations made during the investigation (Andrews, 2009a).

- The failure has propagated up through the sands to the contact of the paleochannel sands and the Tertiary clays.
- The failure has a circular failure plane, indicating a rotational type failure.
- Water is still flowing through the sediments and was before failure, indicating that the sediments were saturated.
- In the back and side walls of the slip a series of cracks were observed. The potential for further ongoing slips of material is very high.

3.2 Initial back analysis

Further work from the investigation was carried out as part of the wall stability project. A back analysis on the Scallop slip was completed to define material strength parameters of the oxides (Andrews, 2009b).

The initial design of the pit in the vicinity of the Scallop slip had an overall slope angle of 33° from the crest to the toe of the paleochannel units. This part of the slope was 100m high from surface to the base of the paleochannel units. The initial design was divided into the following domains.

- Surface clays: 25m high, slope angle 43°
- Sandy clays (upper most paleochannel unit) 40m high slope angle 31°
- Gravelly sands (lower paleochannel unit) 35m high slope angle 33°

The results of the back analysis revealed that material properties used in the design of the pit wall were a single value. A range of material values were used in modelling to provide a FoS of 1.04. The following final material properties were derived from the results.

- Sandy clay unit: Cohesion = 42kPa +/- 3kPa and a friction angle of 25°
- Gravelly sand unit: Cohesion = 36kPa +/- 3kPa and a friction angle of 25°

These values were within expected material variations. It was also believed that the original design Factor of Safety may not have accounted for variations in ground water levels. These fluctuations would have had a large influence on slope stability.

3.3 Initial risk assessment

From the back analysis work and geotechnical investigation of the Scallop instability the YSS technical group concluded that the Scallop slip and others will continue to fail through the sandy clay and surficial clay units. Both Slide® and Phase2® results correlated well to determine the area of potential failure.

A time scale of this failure could not be determined from this back analysis work, but at in 2009 observations indicated deterioration had decelerated. The reason for this was that the failure has progressed to the surface clays and historically in this region it has been observed that surface clay failures are slower than the sand failures.

To tackle this deterioration, several short term and long term options were put forward by the technical group aiming at short term controls and also long term options that would impact the underground operation. The short term options included:

- Retention bund to capture the failure material before reaching haul ramp
- Determine extra dewatering requirements
Ongoing monitoring of slip movement

Four long term options were proposed including:

- A new escape way
- potential pit cutback
- portal relocation
- rock armouring

A Semi Quantitative Risk Assessment (SQRA) was completed onsite in August 2009 in aim to identify and assess the risks associated with the four long term options. This was facilitated by Trevor N Little (TNL Consultants Pty Ltd).

After evaluating the results using Barrick’s Risk Matrix and SWOT analysis, site management and YSS completed a financial study on the options that appeared more favourable from the risk assessment. The results led the group to decide to proceed with option 3 which was a new escape-way/portal. The group also agreed to implement all the short term controls in conjunction with this long term option.

The short term options of the retention bunds and monitoring were in operation by mid 2010. The retention bunds were constructed in 2009 and they continue to retain any failed material from reaching the active bench below. Material build up is monitored regularly by site personnel and removed as required.

The only outstanding short term project was the de-watering option. This project was split into two phases. The first phase focused on fixing the current bore pumps so that they were operating at full capacity and then maintaining these so that pumping was consistent. Parallel to this external consultants were approached to put together a design a larger de-watering project. A proposal was completed in mid September 2010.

The long term project of the new escape portal commenced work at the end of 2009. At the start of 2010 a design for a new escape-way was approved. This design included development from the underground decline to the new escapeway/portal.

4 Escapeway/Portal

4.1 Geotechnical assessment

This escapeway/portal was to provide a secondary means of egress for the underground mine. A geotechnical investigation on several proposed locations was carried out in June 2009. The chosen option was selected based on both a financial and geotechnical assessment.

The new portal was designed to break into the pit on the 245mRL bench, which is one bench lower then the current portal location. Figure 4 illustrates the location of the new escape-way portal in relation to the current portal and slip locations.

The geotechnical assessment carried out on this area concluded that the ground was mostly a massive, competent conglomerate and syenite intrusive. Logging data from old drill holes and three new drill holes were analysed for dominant discontinuity sets and any major structures. The structural analysis revealed two sub-vertical sets and one flat set which were consistent in this area. The drilling also identified some fractured, mud-filled, zones.

The surface condition of the wall showed the effect of weathering over time. Surface run off had resulted in salt build up on the wall and over time a thin layer of weathered material to form. This layer was scaled off and fresh rock encountered below. Figure 5 shows the wall condition in portal location.

Ground support design for portal accounted for the presence of the mud filled structure and other discontinuity sets identified during mapping and drilling. The ground reinforcement design comprised of cable bolts and resin bolts which were located specifically where wedge formation was identified. Fibrecrete was used as the surface support.
Figure 4. Location of new portal in relation to slip.

Figure 5. Wall condition of Portal location. Photograph highlights syenite intrusion location, faults including the mud structure and location of carbonate veins.
4.2 Fault tree risk assessment

As a part of the portal assessment a fault tree/event tree risk analysis was undertaken to account for potential hazards that would affect the portal works.

The main hazards identified in the assessment were the slips, in particular the Beta Slip which is located above the new escapeway/portal. This risk was quantified using a fault tree/event tree risk assessment and was undertaken at the start of 2010 (Andrews, 2009c).

A combination of site based personnel and external consultants conducted the fault tree analysis on site. The purpose of the analysis was to determine the probability of failure of the pit wall above the proposed portal using numerical models recent slip monitoring results. Prior to the risk assessment various numerical modelling analyses including Slide® and Dan-W® software were completed to determine both failure planes and run-out analysis of failure.

The results from the fault tree were then entered into four separate event trees, to determine the probability of damage to equipment and infrastructure, the economic impact and also the probability of injury or fatality to workers.

The results (Andrews, 2009c) indicated that for the worst case scenario the annualised mean probability of failure (PoF) of the pit wall was 9.35%, whilst the annualised PoF for the most likely case scenario was 5.94%.

From the event trees it was determined that the probability of equipment being destroyed was 0.02% and 0.01% for the worst case and most likely case scenarios respectively. The probability of a force majeure was determined to be 0.13% for the worst case and 0.08% for the most likely cases.

The analysis also revealed that these probabilities could be further lowered by implementing some extra controls namely:

- Pit dewatering must be continued to at least pump at values equivalent to extraction rates during the mining of the open pit with new rapid change out of broken or blocked pumps/boreholes procedures.
- Installation of a continuous slope monitoring system to be implemented to lower ongoing exposure risks of personnel driving under the Beta slip.

These controls were included in the Portal project.

4.3 Portal outcome

Development of the 245mRL drive took 4 months to complete. A couple of delays were encountered which slowed down the development rates and the planned day of breakthrough. There were no rock mass issues during the development towards the pit wall. The recommended development cycle was followed closely by mine site personnel and as a result the profile and ground support installation was excellent.

To comply with recommendations from the risk assessment a MSR300 radar was installed and utilised throughout the establishment. The radar monitored slip movement throughout the development successfully. The radar revealed no fluctuations of slip movement as a result to the underground work.

In January 2011 the final breakthrough cut was completed. The ramp material in place confined the rock mass well during the blast. There was no fly rock as a result and minimal blast damage around the portal itself.

Once the ramp material was removed it was revealed that the portal profile was excellent with minimal over break. The final result is illustrated in Figure 6.
5 MSR-300 radar

Up to September 2010 these slips were monitored using monthly photographs and survey pick-up of prisms. Since the occurrence of the slips it was observed that prism placement was not ideal as these could not be located on the slips themselves because of safety issues. So any recorded movement was of the surrounding wall adjacent to the slip and not the slip itself and therefore no quantitative monitoring of actual slip acceleration.

As mentioned previously, the deficiency of a continuous slope monitoring system was identified in Risk Assessment for the new portal. In 2010 further options for wall monitoring were investigated. These options included simple prism installation to the more advanced radar options.

Prisms or skid mounted GPS units could not be safely installed within these wall instabilities. Only a remote optical monitoring device would achieve the minimum requirements for slope performance monitoring.

The outcome from this work was to trial the Reutech MSR300 radar for six months. The short term arrangement was for the purpose of creating a baseline of movement magnitudes of the slips themselves and the adjacent ground. The results of this monitoring was to help determine future slip controls and/or new underground decline access.

5.1 Radar installation and triggers.

The radar was set up on a pad on the eastern side of the pit to focus the monitoring on the three slip areas. An example of the main software screen exemplifies these areas in figure six.

The radar software allows the user to set up smaller areas within the instability zone to monitor. The advantage here is that the radar doesn’t have to wait for a large area to move before an alert is activated. This is also an advantage if areas within the insatiability are more susceptible to movement or are moving more quickly than other areas. Figure 7 also illustrates these areas which have been delineated to monitor within the slip.
The Reutech MSR-300 was set up to send alarms if movement or acceleration breaches the set trigger levels. The current alerts and alarms set for the Wallaby Pit are

- Yellow and Grey Alert: No data – system has gone down
- Orange Alert: 5mm/hr acceleration (based on cumulative) - notification to tech staff only
- Red Alert: 20 mm/hr acceleration (based on cumulative) – Follow Emergency Procedure

6 Data analysis

6.1 Dewatering history

During the pit and underground production borehole pumps have been maintained on the surface of the Wallaby Pit. There are two areas where these pumps are concentrated which is illustrated in figure three. The majority of the pumps are situated in the southern end of the pit, close to the Scallop and Alpha slips.

After the completion of open pit mining, there was steady decrease in the total volume of water abstracted from the combined dewatering program in the Wallaby pit. The combination of boreholes, pit sumps and underground dewatering were below the required 250L/sec that was achieved during open pit mining.

These levels dropped to below 150L/sec from September 2007 to January 2009 as are shown figure seven. This time period also saw a rise in ground water levels and an increase in ongoing slip failures in the pit.

As part of the risk assessment recommendations the health of these pumping bores was assessed. It was revealed that these pumps were underperforming which was having a direct effect on the movement and deterioration of the slips. In August 2010 a concerted effort to increase the productivity of these bore pumps was carried out. The resultant observations indicated less movement on the slips.
The total target abstraction rate for the pit is 241 L/sec which is from both the bores and pit sumps. The bore pumps on the surface alone aim for an abstraction rate of 165 L/s per month.

In August the borehole target was reached as illustrated in Figure 8. Pumping rates are still not consistent and can vary.

Figure 8. Total monthly groundwater abstraction rates from URS Monthly Report for December 2010.

6.2 Combined radar and pumping analysis.

By using a combination of the radar data, rainfall events and pumping data the wall stability performance was analysed. The aim of this analysis is to identify any correlation of instability movement with rainfall events and borehole pump rates.

If the analysis verifies this relationship then the question of if the borehole pumps are controlling wall stability can be answered.

6.3 Monthly total movement

To accurately analyse the movement of these slips, the data was separated by area and proximity to the instability and analysed separately.

To simply the large quantities of data the radar records the total movement for each day was calculated.

As illustrated above the bore pumps are spread along the western wall. To accurately compare the radar data and bore pumping data the bore pumps were assigned to a slip based on their proximity and data imported into the radar database. Rainfall data was obtained from the Bureau of Meteorology website for the Laverton area and was also imported to the same database.

This data was combined for the Beta Slip and the resulting graph displayed in Figure 9. Any records before November 2010 were ignored because of a 3 week gap in the data.
Observations about the Beta Slip data include:

- The data presented in Figure 9 shows areas where pump and/or rainfall data correlate to movement. The areas on the graph that highlight this are B-1, B-2, D-3, E-4, F-5, G-6, H-7, I-8, J-9, K-10 and L-11. Areas B-1, D-3, E-4, F-5, G-6, H-7, I-8, J-9 and K-10 highlight rainfall and movement correlation. Areas B-2, I-8 and K-10.

- There are some areas in this graph where arguably the influence of pumping and/or rainfall events lag and affecting slip movement rates a few days later. Areas where this may be the case include B-2, D-4, E-5&6, G-7, H-8, I-9, J-10 and K-1. However as in figure seven this is not consistent throughout.

- Background movement of the Beta slip ranges between 2mm to 5mm. Between February and Mid-March movement increases and fluctuates. The cause of these fluctuations is the heavy rainfalls that occurred during this period of time. After Mid March background levels are reached once again and remain around these levels.

- There are some periods of relatively constant pumping and no rainfall events. When pumping levels are above 1200 litres background movement levels are achieved. Pumping rates dropping below this increase the amount of movement. It is too hard to determine from the data that the drop in pumping alone has caused the fluctuations of movement as rainfall events coincide around the same time.
7 Data analysis conclusions

Based on the data collected from the Beta slip there is strong evidence to show that pumping and rainfall events appear to have an influence on slip movement. The graph shows a reduction in movement after the rainfall event. A few important conclusions can be made from this relationship, namely:

- The constant rate of pumping mostly restricts wall movement when there are no rain events.
- Data may show minimum pumping requirements to keep movement levels at a constant rate
- The rain events have a significant effect on the wall movement.

However there are several unknowns that may or may not be influential in the above results namely:

- Influence of rain events up stream. This may explain a variation in magnitude of movement in some places where constant pumping is achieved.
- Pump measurements are summarised on a weekly basis which can be misleading. The recorded amount may have all been pumped in one day, allowing 6 days of build up in the system which may result in slip movement. The data should either be taken more frequently or maintenance records compared.
- Pore pressure behind the wall is unknown. Therefore verifying a lag between pumping and movement is inconclusive.

The data analysis has answered several queries regarding the dynamics of the movement of the slip material however it has also raised several new questions, including:

- Is the pumping enough to prevent failure of the wall?
- If so, is it sustainable for the duration of underground mining at Granny Smith? Should we invest in more pumps?
- Is more data needed? Do we need pore pressure data to understand the hydroteology of the slip area in more detail
- What is the ultimate bore pump rates that we need to achieve to maintain a constant movement and/or is there a level where equilibrium of pumping versus movement can be reached?

Analysis completed on the data to date does not give definitive answers to the above questions. More information over a longer time period may enable a better correlation between pumping, rain events and wall movement.

8 Further work

Presently long term wall controls are still being assessed. As addressed in the previous section, a longer period of data collection is required. This data collection and analysis will aid site management in deciding if a new portal should be established on a higher bench or if continue monitoring will be sufficient.

Monitoring movement levels and pumping rates is ongoing. This continued monitoring allows site personnel to better understand critical levels of movement for alarms. At the time of this paper, a scope of works was commencing to answer the questions raised form the data analysis. This work includes:

- Extra bore-pumps to be established on the surface
- A maintenance program to be established so system health and pumping rates can be compared to wall movement
- A piezometer to be installed behind the slips to obtain pore pressure data of the wall
9 References