Slope Stability Study of the Main Pit Footwall, Koolan Island, Mt Gibson Iron

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Abstract

The Main Pit at Koolan Island consists in general of a bermless footwall from 26 mRL to -86 mRL and a benched hanging wall containing the haul roads. The footwall above 26 m RL was benched at 20 m intervals and the top crest of the footwall ranges from 140 m RL to 110 m RL. The current pit is approximately 2000 m long and 500 m wide. The batter angle of the footwall varies from 45° at the western end to 60° at the eastern end.

Mount Gibson Iron Limited (MGI) is currently re-opening the operation with a plan to deepen the pit to around -186 mRL. The width of the hanging wall cutback will be about 80 m and the footwall will be continued without any berms. Extensive zones of extremely weak, friable material and the tabular nature of the footwall formation are the major challenge of overall footwall stability.

This paper will discuss the analysis of the footwall slope stability of the proposed pit including rehabilitation requirements to the existing footwall. The numerical modeling package Phase 2® has been used for the slope stability study. The impacts of dewatering; friable material and bedding planes within the slope; and ground reinforcement have all been studied.

The existing footwall rehabilitation includes draping mesh, catch fences, cable bolting and application of self drilling anchors. To achieve a stable footwall, shotcreting and cable bolting will have to be continued in some areas for the extended pit.

1 Introduction

Koolan Island is located in Yampi Sound, 130 km north of Derby in the Kimberley region of Western Australia. Previous mining on the island included the Main Pit (main orebody) and four satellite pits. BHP (now BHP-Billiton) started mining on the island in 1964 and continued until the early 1990’s. When mining stopped, the Main Pit was at -86 mRL (86 m below the mean sea level). The hanging wall was breached and the Main Pit was flooded when the BHP operation was decommissioned.

The Main Pit consisted in general of a bermless footwall from 26 mRL to -86 mRL and a benched hanging wall containing the haul roads. The footwall above 26 mRL was benched every 20 m and the top crest of the footwall ranges from 140 mRL to 110 mRL.

The current pit is approximately 2000 m long and 500 m wide. The batter angle of the footwall varies from 45° at the western end to 60° at the eastern end.

MGI is currently re-opening the operation with a view to deepening the Main Pit to around -186 mRL. To deepen the Main Pit a further 100 m, the width of the hanging wall cutback will be about 80 m and the footwall will be continued without conventional berms.
2 Geology

The hematite has been mined from the base of the Yampi Member of the Pentecost Sandstone, the top most formation of the Kimberley Group. The Main Pit orebody and other satellite deposits on the island are the surface exposures of a series of synclinal and anticlinal folds which in general plunge gently to the northwest. The Main Pit orebody is located on the overturned southerly dipping limb of an isoclinal syncline.

In addition to bedding/foliation, a regional fault and jointing pattern trending perpendicular to the strike of the orebody and dipping at approximately 70° northwest comprise the major structural features of the deposit. Strike-slip offsets of 2 m or less have been recorded.

The footwall slope is located within the lower unit of the so called footwall formation which consists of interbedded quartzite and sandstone with varying amounts of hematite. The footwall formation is approximately 50 m wide. Within the footwall, individual beds such as massive quartzite vary in thickness or even lenses out both along strike and down dip. Friable zones exist within the footwall formation. The extensive zones of extremely weak, friable material and tabular nature of the footwall formation is a major concern to overall footwall stability.

The main synclinal aquifer is located behind the footwall formation. A series of thin bands of low permeability schist act as an aquitard and separate the footwall formation and the aquifer.

The main rock units of importance are shown on the cross section in Figure 1.

![Figure 1: The cross section showing the main rock units of importance.](image)

3 Existing footwall stability and rehabilitation

The footwall of the Main Pit was mined bermless for over 100 vertical metres below the 26 mRL. Following abandonment, the slope below the sea level has been under water for over 10 years. In some sections, the slope had been undercut by blasting during previous mining and suffered erosional degradation from cyclonic events.

The footwall stabilization methods that were adopted by BHP consisted of fibrecrating/shotcreting and meshing to prevent excessive erosion of friable material and tying the surface support back to deeper horizons in the more friable zones using twin strand cable bolts predominantly in the eastern section of the wall where the slope angle varies from 55° to 65°. As the pit has been abandoned for over 10 years, deterioration to the wall and support system is expected. The detailed wall condition under water is largely unknown as attempts to visually inspect
the wall using robot mounted cameras have only been partially successful. The actual condition will only be revealed as the pit is drained.

To continue mining the footwall for another 100 m vertically without conventional berms, MGI has decided to rehabilitate the existing footwall. The rehabilitation program includes:

- draping wire mesh over the existing slopes below the 26 mRL;
- installing a catch fence along the 26 m RL berm above the bermless slope;
- reinforcing identified sections of the wall as it is exposed and undertaken repairs to undercuts and washouts; and
- Installing slope monitoring systems including extensometers, prisms and piezo-meters.

Most of the rehabilitation work carried out already has been and will continue to be carried out on barges moored along the footwall while the pit is dewatered. The pit has been dewatered at a suitable rate to ensure that there are no pockets of high phreatic water.

3.1 Drape mesh

Drape mesh is installed along the footwall to contain and minimise the energy and damage resulting from loose rocks falling. Heavy duty Tecco mesh from Geobrugg has been selected for this project which is made of high tensile steel wire. All the mesh is pinned along the berm and then draped freely or pinned along the slope. Figure 2 shows the footwall with drape mesh being installed over the majority of the slope and Figure 3 is a close up view along footwall showing a barge and the rolls of mesh on the wall.

Figure 2. View along footwall looking west, the draped mesh can be seen being rolled down progressively over the majority of the slope. Access to the wall and to the installation is facilitated off the barges moored along the wall.
3.2 Catch fences and debris flow fences

The wall above the 26 mRL berm has deteriorated due to weathering over the past ten years with potentially unstable surfaces exposed or undercut. Since the slope does not have regular berms below 26 mRL level, it is essential to contain any falling rocks at the 26 mRL level.

Geobrugg catch fences have been built along the 26 mRL berm and other berms where access is available. The fences are 3 m high with a 500 kJ of energy absorption capacity. Figure 4 shows a section of the rock catch fence.

Large washouts developed along the berms from run-off water over the time period exposed. At the bottom of an erosion gully a debris flow fence has been built. The gully occurs in a vulnerable section of the pit wall where the friability of the rock mass is pronounced and jointing is intense. Figure 5 shows the established debris flow fence.
Figure 4. Rock catch fence installed along the footwall berm at -21 mRL.

Figure 5. Established debris flow fence at one of the washouts.
3.2.1 Reinforcement

BHP installed a significant number of 10 m long cable bolts into the footwall to stabilize the near surface bedding. In the exposures to-date, the cables appear to have corroded and most of the cables no longer have barrels or plates attached. Cables located within the tidal zone appear to have been subjected to an accelerated level of corrosion compared to those fully submerged.

To examine the residual capacity of the existing cable bolts, the geophysical method GRANIT was used by applying a controlled impulse axially to the exposed end of a tendon and measuring the response. If a section of a cable is un-encapsulated, the cable will resonate. The location and length of the un-encapsulated cable can be predicted by analysing the received frequency. Based on the geophysical study results (Jones, 2010), 70% of the existing cables are fully encapsulated, enabling them to operate as a dowel. The challenge is that the first 2 m of cable may not prevent surface deterioration thus remediation may be required.

Due to the existence of cracking and voids within the footwall, cables are may not be fully grouted. New cables are installed to reinforce the ground in some sections. All new cables have twin strands with 50 tons of yield strength and 17.5 m imbedded length. The geophysical method was also used to check the encapsulation condition for newly installed cables. The test results indicated that some of the newly installed cables may not be fully grouted.

4 Slope stability of the deep pit footwall

4.1 Potential failure mechanisms

The slope stability of the footwall slope will depend on the friability of the rock mass within the footwall slope, particularly the near surface rock mass. If the surface rock mass is extremely friable, it will have to be consolidated through artificial reinforcement methods such as meshing, bolting and shotcreting.

Since the friable material is relatively thin and tabular in nature, the friability, the thickness and distribution within the footwall are all important parameters of the slope stability. After erosion removing the friable material and exposing blocks, the main failure mechanism is considered to be bedding sliding due to toe failures. Buckling of thin tabular slabs is another potential failure mechanism with increasing height.

The schist aquitard limits water from the footwall aquifer flowing into the footwall slope and further to the pit. Water pressure will build up along the bedding planes near the aquitard. It is not possible to depressurise the aquifer as it represents the freshwater supply to the Island. The distance from slope surface to the aquitard is approximately 50 m. In this regard, the footwall slope is considered as a dam and is approximately 180 m deep.

4.2 Numerical modelling

The numerical modeling package Phase 2 from Rocscience is used for this study. Phase 2 is a 2D elastic-plastic finite element modeling package for continuum modeling.

Various scenarios of footwall formation have been modelled including the thickness of bedding planes, thickness of the friable material and slope angles. The footwall sandstone is separated from the main sandstone by 5 m of schist which is modelled as an aquitard.

The friable zone is modelled as Mohr-Coulomb material. The strength parameters are quite difficult to determine since no core samples have been obtained and tested. The friction angle of the friable zone is assumed to be 37° based on observation of rill slopes. For cohesion, low band and high band values were applied. The low band cohesion is assumed to be 0 kPa and the high band value is assumed to be 100kPa.

Sandstone is the main rock material in the footwall. Both Mohr-Coulomb and Hoek-Brown criteria have been used to model the sandstone. In the Mohr-Coulomb model, the cohesion and friction angle were assumed to be 200kPa and 38° respectively. In the Hoek-Brown model, both cohesion and friction angle values were derived by
Mohr-Coulomb curve fitting. The derived cohesion and friction angle were 850 kPa and 35° respectively. The derived cohesion from Hoek-Brown criteria is significant higher than that used in Mohr-Coulomb model.

To model the dam effect, the footwall slope was modelled as a cantilever along a fault beneath the aquitard, which is only subject to gravity load and water force, as shown in Figure 6. When the effect of the water pressure is greater than the gravity load, tensile stresses will be developed inside the slope along the fault and the cantilever will flex towards the pit. Various thicknesses of the footwall slope have been modeled to identify the influence of thickness of the footwall at which tensile stress is developed. This would then provide some guidance as to what to observe in the pit when installing support.

![Figure 6. Model set up for toppling failure mechanism modeling.](image)

The general conclusions from this modelling work of various scenarios are:

- For the proposed extended pit, footwall geotechnical conditions vary along strike. The two major geotechnical factors which will affect the footwall stability are the overall slope angle and the condition of the friable zones.
- The maximum displacement will increase with decreasing bedding thickness.
- Cable bolting using a 4 m x 6 m pattern may increase the safety factor by 3% which is not considered to be significant. Cable bolting is more effective in the extremely friable zones.
- Pore pressure within the footwall formation will have a significantly adverse impact upon the footwall stability. Footwall dewatering and depressurisation will be essential to have a stable footwall. The planned target of depth 20 m for depressurisation appears to be adequate.
- The slope angle has a significant impact upon the footwall stability. For the footwall where there is an overall slope angle of approximately 45° to 50°, the slope will have a safety factor 1.2 or greater, primarily due to the shallow slope angles. In areas where slope angles are higher than 50°, pit walls failures are expected.
- For the dam effect modelling, modelling results have indicated that pressures generated by the synclinal aquifer are not expected to have a significantly adverse effect on footwall stability. The 50 m thick footwall should generate sufficient ‘counterweight’ to the water pressure within the aquifer.
It should be appreciated that the friability, thickness and vertical extent of the friable zone are all variable. Modelling results change significantly with the material properties applied. There are no simple means by which local variations in footwall conditions can be ascertained.

5 Discussions and conclusions

The ultimate Main Pit footwall at Koolan Island will be over 200 m high without conventional berms. The unique footwall slope profile and material properties will make maintaining the footwall stability challenging.

Friable material and the tabular nature of the footwall formation are the two foci of overall footwall stability. How to prevent the friable material from deterioration is critical to maintain the footwall integrity. Furthermore, horizontal to sub-horizontal cracks and blast damage to the bedded slope surface can also cause large scale slides.

Based on numerical modelling, cable bolting may not make a significant impact upon the global slope stability unless the bolting can be constructed to “fuse” thick blocks together. However, cable bolts will increase the integrity of the bedded slope surface and prevent the slope from large scale slides.

Ground water is modelled to have a significant impact on footwall stability. Hence, footwall depressurisation is essential to reduce the pore pressure and keep the bedded footwall formation stable.

Monitoring will have a crucial role for both the footwall and hanging wall. Innovative techniques are under consideration.

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7 References