High Rock Slope Cutback Geotechnics: A Case Study at Ok Tedi Mine

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Abstract

Ok Tedi copper-gold mine which has been operating in the remote highlands of Papua New Guinea since 1982 proposes to cut back its West Wall by 200m to 300m over a crest length of 1500m. The final slope will be almost 1000m high. The cutback will take 4 years and will be completed in parallel with mining operations in the existing pit. The terrain around the pit is rugged, and rainfall is 9m to 11m per year. Earthquakes of 4 to 6 on the Richter scale occur in the region. Pit geology and structure is complex. Key factors impacting pit slope design and stability at the mine have been identified, and the challenges associated with this high rock slope cutback project discussed. The results and conclusions of field and laboratory investigations to characterize the slope rock mass and divide it into domains are presented. Step-path and Hoek-Brown methods have been used to derive rock mass strength parameters for slope stability assessment using a variety of numerical modelling techniques. The results and conclusions are presented.

1 Introduction

This paper presents the geotechnical design considerations for cutback of a high rock slope at Ok Tedi mine. The proposed cutback will extend 1500m along and 200m to 300m back behind the current slope face; and will create a slope almost 1000m high.

Ok Tedi is a world class open pit copper-gold mine (Figure 1) located in the remote western highlands of Papua New Guinea near its border with Indonesia. The terrain is rugged, with the pit wall crest approximately 2100m above sea level. Regional earthquake risks are moderate, typically ranging from 4 to 6 on the Richter scale, however more severe tremors do occur occasionally. The mining environment is challenging. Annual rainfall ranges from 9m to 11m, with the intensity varying seasonally. Cloud often shrouds the mine workings.

Mining operations commenced in 1982. Daily pit production is approximately 80,000t of ore and a similar tonnage of waste rock. The ore is crushed and milled to a concentrate which is slurry piped 150km to the port at Kiunga. It is then sent 700km by barges to the mouth of the meandering Fly River and loaded onto ships for transport to overseas smelters. Each year, the mine produces about 150,000t of copper metal, 550,000 ounces of gold and 1,100,000 ounces of silver.

The current open pit covers an area of about 3000m by 2000m and is 800m deep. The end of mine life for the current pit design is 2013 when the pit depth will reach 900m. At the end of this pit design, a large volume of ore worth several billion USD will remain in pit slopes and below the floor.

A concentrated study and design effort has been in progress since 2006 to extend mine life by a decade or longer. Options include open cut mining by cutback of the existing slopes, underground block and sublevel caving or some combination of the two options. The latter option is favoured; where ore in the western slope is mined by cutback of the existing slope and ore in the eastern slope is mainly recovered by underground sublevel caving. The final design is still being developed.
2 Design considerations

The design was underpinned by the usual objectives to maximize ore recovery; to minimize waste rock volumes and to optimize financial return on investment. However, these were not the only factors. Other key considerations included the need to:

- achieve the cutback mining within a 4 year timeframe;
- guarantee continuity of ore feed to the mill;
- satisfy working-area constraints on the number of shovels that could be deployed in the cutback footprint;
- limit the volume of waste rock to that which could be accommodated within a long-term waste dump;
- minimize waste haulage distances and truck fleet numbers;
- minimize interference between existing pit mining operations and those in the proposed cutback; and
- dewater the cutback slope to the required distance behind the slope face within the cutback period.

The mine technical staff understand the basic geotechnical factors impacting slope stability of the existing 800m high slope, and are aware of deficiencies in the existing design. Ground conditions to be exposed in the new slope face are not expected to significantly differ from those in the existing face; although the projected location of weak zones would be somewhat offset from their current daylighting positions and/or be mined out.

The following cutback slope design is proposed:

- Double-benching will be adopted, increasing bench height from current 15m to 30m; but retaining existing batter / face angles of 65 degrees.
- The existing berm width of 8m will be retained; but with every fourth berm widened to 34m. This will create a wide catch-bench every 120m down the slope face. This design will allow major equipment long-term drive-on access to the slope face for installation and maintenance of groundwater dewatering measures, surface water runoff discharge drains and ground control monitoring stations. Potential interramp slope face instabilities will be decoupled and rendered more easily remediable.
- Berm gradients will be 1: 200 to channel water runoff to drainage shaft installed in the floor of the existing pit.
- Separate from the 34m wide catch-berms, a 40m wide haul road will be constructed down the cutback slope face
- The overall slope angle in the cutback design is 38° to 39°. The inter-ramp design angle between the 34m wide catch-berms is 56 degrees. The latter angle will be flattened where the haul road traverses the slope.

The design has been developed as robust to suit variable conditions. Slope design in weak zones will not be altered, but will include ground support (cable dowels, mesh and fibrecrete) to minimize slope face ravelling.

3 Pit geology

Ok Tedi is a porphyry copper deposit. The geology comprises a layered sedimentary sequence of siltstone, mudstone and limestone that was regionally thrust-faulted and then domed / drag-folded, low-grade metamorphosed, clay-altered along some contacts and then economically mineralized by two or more phases of intrusive activity. Mineralization is variously disseminated throughout the intrusive and sedimentary rocks; with more massive concentrations in skarn bodies along the perimeter of the intrusions. The geology is illustrated in section and plan in Figure 2 and Figure 3, respectively.

Geological structure is complicated (see Figure 4). Due to up-doming, strata dip gently into the pit walls. Mineralization is truncated by a regional basal unconformity / thrust fault. Two thrust faults (Taranaki and Parrot’s Beak), also dipping into the pit walls, slice through the strata. Major normal / transverse faults cross the pit workings. Defects co-aligned with thrust, normal, and transverse faults are superimposed on bedding partings and orthogonal joint sets. Several sets of geological defects, spaced 0.05m to 0.3m, exist at many locations (see Figure 4 insets).
Figure 2. WNW - ESE section through the Ok Tedi geology with 970m high West Wall cutback profile.

Figure 3. Plan view of the geology exposed with the current pit.
Figure 4. Plan showing the projected exposures of major structures within the walls of the pit design, with inset (A) closely spaced jointing and (B) intense fracturing in siltstone on the West Wall.
4 In-situ stress and regional seismicity

Down-hole hydraulic fracturing (1991), acoustic emission testing (2007) on rock cores, and over-coring of down-hole strain-cells (2010) were used to assess the in-situ stress regime. Stress interpretation is complicated by the rugged topography. Based on the latest work, the principal stress is interpreted as sub-horizontal, aligned NNE-SSW and 1.0 to 1.8 times the vertical overburden load. The minor stress is sub-horizontal, aligned ESE-WNW and 0.5 to 0.7 times the vertical overburden load.

Earthquake loadings were thoroughly reviewed by mine staff in 1994 to establish peak ground acceleration (PGA) values relevant for a 25 to 30 year pit slope design. Subsequent review work was completed in 2002-2004 for dredged materials storage facilities. A new study is in progress (2011) for long-term stability of the waste rock dump.

A 0.07g horizontal seismic loading has been adopted for pit slope design evaluations. The authors are aware that some geotechnical practitioners consider that earthquakes do not significantly impact the stability of hard rock slopes in mines. Until this viewpoint is substantiated by convincing field evidence, however, it is prudent geotechnical practice to continue to consider seismic loadings in pit slope design.

5 Groundwater and surface water

Groundwater flow in upper sections of pit slopes is controlled by cavernous limestone and strata that dips away from the pit face. Flow away from the pit occurs at higher slope elevations where mine workings are above creek channels that drain the surrounding rugged terrain. With greater pit depth, flow into the pit takes place from the main recharge area located in the high topography to the northwest. Significant groundwater inflows to the open pit occur on subvertical major structures. Shallow dipping thrust faults are generally infilled with low permeability materials which act as aquitards.

The current groundwater regime in the pit slopes is a product of the drawdown achieved by natural drainage of the rock mass over the last 30 years and induced drainage due to 150m to 250m long subhorizontal dewatering holes drilled in last 10 to 15 years (typically on 50m to 70m centres) into the slope face. Data from piezometers suggest that the general groundwater level in pit slopes is currently being lowered by 10m to 15m per year.

The subject slope will be cutback 200m to 300m behind the existing slope face. This new face is expected to be saturated below the uppermost (Taranaki) thrust fault horizon. A significant dewatering effort will be required to dewater the pit walls to a horizontal distance of 200m to 250m behind the slope face, which is considered necessary for maintaining pit wall stability within the 4 years mining-down period. The dewatering effort is the subject of another paper to be presented at this conference.

With 9m to 11m of annual rainfall and water generated by systematic dewatering of slopes, control of surface water runoff is a major design consideration. Previous loss of surface drains resulted in water cascading over slope bench crests, with major erosion chasms developing in places. Considerable remedial action continues to be needed to arrest ongoing expansion of these chasms on pit slopes.

In the cutback design, runoff problems are being addressed by incorporating 34m wide drainage benches at 120m intervals down the slope face. Wide benches allow long-term maintenance access to surface drains. Runoff is channelled to the pit floor where it flows into shafts connected to a 3km long drainage tunnel beneath the floor.

6 Blasting

Previous blasting and large equipment excavation practices have in places damaged the integrity of final slope faces. This situation was compounded by the high intensity of adversely orientated geological defects and uncontrolled water runoff at some locations. More benign mining practices will therefore be used to establish the cutback slope face, including pre-splitting techniques. For stability analysis, a 50m wide zone of blasting-
disturbed ground behind the slope face was assumed. Sensitivity check of 4 to 5-fold wider blasting disturbed zone resulted in a 15% reduction in factors of safety.

7 Rock mass conditions

7.1 Geological structure

Geology (see Figures 2 and 3), structure (orientation and intensity of discontinuities) and ground conditions (combination of intact rock strength, structure and argillitic alteration) are variable across the pit.

Slope photography, structural and geotechnical mapping of slope faces, targeted geotechnical cored drilling investigations and laboratory testing were used to investigate the rock mass. Many of the holes drilled were orientated for determination of structural discontinuity orientations.

Prior to 1997 about 35,000 geological defects were mapped in the upper half of the pit slopes and 6,000 were measured from drill cores. Selective pit slope mapping has been ongoing since then.

The structural geologic model adopted for numerical stability analyses reflects a “simplified” reality, with defect spacing being grossly exaggerated to allow executable models to be constructed, whilst still allowing for simulation of the likely effect of structure on slope failure mechanisms.

7.2 Geotechnical domains

Geotechnical domains were developed for each rock type in the subject cutback slope on basis of intact rock strength and structural intensity. Table 1 and Figure 5 detail the five geotechnical domains (A to E where A represents the best class rock mass and E the worst).

Table 1. Description of rock types and geotechnical conditions defining geotechnical domains.

<table>
<thead>
<tr>
<th>Domain</th>
<th>Descriptor</th>
<th>Rock types</th>
<th>Blockiness conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Very good</td>
<td>Monzonite porphyry, local magnetite skarn, Monzodiorite in south east of pit</td>
<td>Strong to moderately strong, massive to large blocky rock. Occasional small fault zones.</td>
</tr>
<tr>
<td>B</td>
<td>Good</td>
<td>Porphyritic monzondiorite, skarn, east wall endoskarn, unaltered monzodiorite, limestone</td>
<td>Strong to moderately strong, large and medium blocky rock (locally small blocky). Occasional small fault zones.</td>
</tr>
<tr>
<td>C</td>
<td>Fair</td>
<td>Siltstone, limestone</td>
<td>Strong to moderately strong (locally weak or brittle), medium and small blocky rock. Localised Fault zones containing brecciated or weak, altered rock.</td>
</tr>
<tr>
<td>D</td>
<td>Poor</td>
<td>Altered monzodiorite, altered endoskarn, oxide skarn, contact breccias.</td>
<td>Weak (locally very weak or moderately strong), very highly fractured or brecciated, and/or slightly plastic rock. Significant fault/contact zones and highly altered rock.</td>
</tr>
<tr>
<td>E</td>
<td>Very poor</td>
<td>Major thrusts, Lower West Wall Fault zone, West Wall brecciated zone, localised lesser faulted &amp; brecciated zones</td>
<td>Highly brecciated, granular and/or highly plastic fault breccia and gouge. Significant fault zones of very poor quality.</td>
</tr>
</tbody>
</table>
7.3 Rock mass strength methodology

Historically, two approaches were used to develop rock mass strength parameters at Ok Tedi Mine.

- STEPSIM4: a statistical “Monte-Carlo” step-path method that considers both defects and rock bridges between defects along potential failure paths through slopes. This approach was used during a risk-based slope design optimization study for the current pit design (1997-2000); and

- Hoek-Brown rock mass strength method: for the proposed mine life extension design (2006-2011). The authors believe that the Mohr-Coulomb model overestimates rock mass strength with respect to shallow-seated failure immediately behind the slope face and underestimates the strength with respect to deep-seated failure of the overall slope. Therefore, non-linear Hoek-Brown strengths were adopted for most material types in the cutback slope.

Inputs to the STEPSIM4 method are statistical models for geological defect set types (fault, joint, bedding, etc), including:

- orientation and persistence of defect sets
- probability of defect set occurrence in the rock mass
- probability of defect cut-off by other defects
- rock bridge lengths for non cut-off defects
- infill materials and strength characteristics
- small scale defect surface roughness, large defect surface undulations
- intact rock / rock mass strength for the rock bridges.

The bridge strength parameters are computed by the Hoek-Brown method. STEPSIM4 considers the directional strength of rock masses. Strength in directions co-aligned with defect orientations is less than in directions perpendicular to defects. For example, in sedimentary strata, strength in directions subparallel to bedding planes and subparallel to near-vertical jointing is significantly less than in directions across these defect sets. Key inputs to STEPSIM4 must be obtained from slope face mapping data; this information cannot be determined from drill core logging data alone, which limits the application of this method.

Inputs to the Hoek-Brown method are well known. They include statistical models for: intact rock compressive strength (UCS); Geological Strength Index (GSI, a function of structural intensity and defect condition - closely related to Bieniawski RMR classification rating); mi constant (a function of rock type, being lowest for soft
sedimentary rocks and highest for brittle igneous rocks); and blasting-disturbance factor (D). The Hoek-Brown strength can be assessed on the basis of slope face mapping and/or core logging data. However, this method is not especially suited to assessing directional strength through the rock mass.

Irrespective of the method used for rock mass strength determination, the stability impact of major structures must be evaluated individually, as even a single, adversely orientated structure may impose a strong control on slope stability. A kinematic analysis of slope failures along all defined major structures was conducted in 2010, during which several potential problem zones were identified.

7.4 West Wall rock mass strength

Because geotechnical drilling information provides the great majority of the geotechnical data within the West Wall cutback, the Hoek-Brown method was used for deriving rock mass shear strength parameters. The Hoek-Brown input parameters are summarised in Table 2. Figure 6 provides comparisons of strength models developed via STEPSIM4 and Hoek-Brown methods.

It is important to note that the two sets of curves are not comparing strength estimates for identical rock masses. The STEPSIM4 strength is largely based on slope face mapping data for the upper half of the existing West Wall. The Hoek-Brown strength is largely based on drilling and mapping data for the lower half of the existing slope. Ground conditions in the upper half of the slope may perhaps be inherently better because the investigated locations are further away from the brecciated and locally clay-altered intrusive contact. Poorer conditions nearer the slope toe are also due to the occurrence of the subvertical shear zone bounded by the Gleeson Faults that traverse the slope face. Ground conditions in this zone had been more favourably interpreted when the STEPSIM4 parameters were developed in the late 1990s. Conversely, the STEPSIM4 strength for the dominant rock types is directionally reduced by consideration of the defects co-aligned with overall slope face.

Table 2. Input parameters for deriving the rock mass shear strength using the Hoek-Brown method.

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Domain</th>
<th>Density (KN/m3)</th>
<th>UCS</th>
<th>GSI</th>
<th>mi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pnyang Siltstone</td>
<td>C</td>
<td>26.1</td>
<td>35</td>
<td>39</td>
<td>9</td>
</tr>
<tr>
<td>Upper Limestone</td>
<td>B</td>
<td>27.0</td>
<td>90</td>
<td>35</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>25.7</td>
<td>90</td>
<td>38</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>D</td>
<td></td>
<td>50</td>
<td>30</td>
<td>7</td>
</tr>
<tr>
<td>Upper Siltstone</td>
<td>B</td>
<td></td>
<td>90</td>
<td>41</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td></td>
<td>90</td>
<td>38</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>D</td>
<td></td>
<td>50</td>
<td>30</td>
<td>7</td>
</tr>
<tr>
<td>Limestone</td>
<td>B</td>
<td>26.9</td>
<td>90</td>
<td>41</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td></td>
<td>60</td>
<td>38</td>
<td>10</td>
</tr>
<tr>
<td>Lower Siltstone</td>
<td>C</td>
<td>25.7</td>
<td>90</td>
<td>43</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>D</td>
<td></td>
<td>50</td>
<td>30</td>
<td>7</td>
</tr>
<tr>
<td>Skarn</td>
<td>B</td>
<td>43.7</td>
<td>100</td>
<td>57</td>
<td>17</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td></td>
<td>100</td>
<td>42</td>
<td>17</td>
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<tr>
<td></td>
<td>D</td>
<td></td>
<td>100</td>
<td>40</td>
<td>17</td>
</tr>
<tr>
<td></td>
<td>E</td>
<td></td>
<td>35</td>
<td>21</td>
<td>17</td>
</tr>
<tr>
<td>Endoskarn</td>
<td>D</td>
<td>31.9</td>
<td>35</td>
<td>23</td>
<td>17</td>
</tr>
<tr>
<td></td>
<td>E</td>
<td></td>
<td>35</td>
<td>15</td>
<td>17</td>
</tr>
<tr>
<td>Monzodiorite</td>
<td>B</td>
<td>25.0</td>
<td>75</td>
<td>40</td>
<td>24</td>
</tr>
<tr>
<td></td>
<td>D</td>
<td></td>
<td>40</td>
<td>25</td>
<td>24</td>
</tr>
<tr>
<td></td>
<td>E</td>
<td></td>
<td>20</td>
<td>15</td>
<td>24</td>
</tr>
<tr>
<td>Monzonite porphyry</td>
<td>A</td>
<td>25.0</td>
<td>90</td>
<td>55</td>
<td>24</td>
</tr>
</tbody>
</table>
Figure 6. Comparisons of strength models developed via STEPSIM4 (assessed in 1997) and Hoek-Brown (assessed in 2010) methods (where D is the disturbance factor applied to Hoek-Brown curves only).
8 Stability analysis approach and results

The slope design evolved progressively, and a multi-pass and multi-method approach was used to assess stability. The aim was to achieve a factor of safety (FoS) for the overall slope of 1.3 or greater without seismic loading (“static” conditions) and 1.1 or greater with seismic loading of 0.07g horizontal ground acceleration. The target FoS for the inter-ramp slopes was 1.2 under static conditions, and >1 under seismic loading.

The underlying premise has been that, apart from the risk of occurrence of an unknown adversely-orientated major fault, geotechnical and mining factors impacting the stability of the cutback slope were likely to be similar to those affecting the existing slope. Variability in ground conditions is visible along the existing 3km long West Wall face, and a similar degree of variability could reasonably be expected to exist in the cutback slope. While many of the geotechnical risks identified during the 1997-2000 pit slope design optimization apply to the proposed cutback, the understanding of the slope had been refined by the additional investigative work completed in the interim and by use of more sophisticated, real-time, ground movement and groundwater monitoring technology on the slopes.

The following approach to stability evaluations was conducted:

1) It was accepted that the groundwater drawdown / slope depressurisation requirements identified for slope stability will be practically achieved in the cutback period.

2) The geotechnical domain model was progressively updated as new investigations were completed.

3) Potential failure modes for overall and inter-ramp slopes and benches were reviewed. The smaller scale failures are often controlled by structure or locally weak materials, whereas circular and/or quasi-circular rock mass failure modes are viewed as being appropriate for the overall slope.

4) Two-dimensional limiting equilibrium slope stability analyses were carried out using both Slide© and GALENA software. These were performed on seven regularly-spaced cross sections through the West Wall in 2010. Analyses for three key sections were repeated during 2011 for a modified slope design of increased height. Various algorithms (Bishop, Spencer-Wright, Sarma, Morgenstern-Price and Janbu) were utilised for thorough assessment of stability with respect to circular failure. The sensitivity of slope stability to groundwater, blasting disturbance, seismic loading, and unexpectedly poor ground conditions was tested during initial analyses.

5) Limiting equilibrium analyses was supplemented with two-dimensional finite element analyses (using Phase2© software) in 2010 and distinct element analyses (using UDEC software) in 2011.

6) Basic geotechnical premises and stability analysis results were critically reviewed by external peer consultants.

During a 2007-2008 pre-feasibility study for underground block caving of the ore beneath the West Wall, the stability and stress development was also assessed using UDEC, and FLAC3D and Map-3D numerical codes. Although this work is no longer directly relevant to the proposed slope cutback, it provided useful insights into likely ground responses and failure modes if the toe of the slope was steepened and undercut.

A simplified summary of the key stability analysis results for the most important sections through the central and south parts of the West Wall is presented in Table 3. It can be seen from these results that the Slide©, GALENA and UDEC analyses yielded generally similar factors of safety and allow for a common set of conclusions to be reached concerning the performance of the West Wall cutback slope design. Where fault zones are exposed in inter-ramp slopes, potential localised failures with factors of safety less than the target have been identified (especially under seismic conditions). These are generally shallow failures that can be supported and/or managed. Selected analysis sections illustrating FoS for overall slope failure are presented in Figure 7.
Table 3. Simplified summary of latest (2011) slope stability analyses results.

<table>
<thead>
<tr>
<th>Analysis Section</th>
<th>Analysis Method</th>
<th>Conditions</th>
<th>FoS for overall slope</th>
<th>Lowest Intermountain FoS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Central Slide</td>
<td>Static</td>
<td>1.39</td>
<td>1.19</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Seismic Loading</td>
<td>1.22</td>
<td>1.09</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Static</td>
<td>1.40</td>
<td>1.20</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Seismic Loading</td>
<td>1.22</td>
<td>1.07</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Static</td>
<td>1.25</td>
<td>1.15</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Seismic Loading</td>
<td>1.10</td>
<td>1.0-1.05</td>
<td></td>
</tr>
<tr>
<td>South Slide</td>
<td>Static</td>
<td>1.35</td>
<td>1.08</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Seismic Loading</td>
<td>1.17</td>
<td>&lt;1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Static</td>
<td>1.32</td>
<td>1.10</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Seismic Loading</td>
<td>1.16</td>
<td>&lt;1</td>
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<tr>
<td></td>
<td>Static</td>
<td>1.30-1.35</td>
<td>1.0-1.05</td>
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</tr>
<tr>
<td></td>
<td>Seismic Loading</td>
<td>1.20</td>
<td>&lt;1</td>
<td></td>
</tr>
</tbody>
</table>

Figure 7. Comparison of overall slope failure in the centre section under static conditions: UDEC FoS = 1.25 (left), GALENA FoS= 1.40 (right).

9 Summary and conclusions

A summary of the key issues and conclusions with respect to the stability of the West Wall cutback is provided below.

- The proposed 38 to 39 degrees overall slope is predicted to be unstable if saturated. The slope needs to be dewatered to approximately 250m behind the slope face in order to achieve the target design factors of safety.
- The overall slope is stable with respect to sliding failure of major fault-defined blocks and wedges, provided that the design groundwater drawdown is achieved. However, some benches will be prone to crest failure due to undercutting by major structures and adverse combinations of minor structures.
- Overall, limiting equilibrium and numerical work yielded relatively similar stability conclusions for the cutback slope. UDEC models considered geological structures as well as the in-situ stress field,
and therefore provided better insight into slope failure modes. UDEC confirmed that the circular failure mode is an appropriate failure mechanism for the overall subject slope.

- A blast-damaged slope face is weaker and inherently less stable than the overall slope. UDEC modelling (not shown in any of the presented figures as numerical analyses were still in progress at time of writing of this paper) suggests that subvertical defects exposed in the slope face tend to open up as the rock mass rebounds after excavation. Such dilated ground will be prone to rapid water recharge and establishment of a perched watertable during heavy rainfall, further impacting slope face stability. Control of surface water runoff and placements of less permeable materials on bench berms will minimize water infiltration. Good blasting practices are needed to minimize the depth of blast-damaged slope face.

- Slope faces in weak materials (such as thrust faults) will need to be supported, as necessary. Support is only intended to ensure local stability and thereby to prevent ravelling of the weak ground and potential undercutting of the slope face immediately above.

- The cutback design attempts to also address other known deficiencies in the existing slope design. Wide catch-berms at 120m intervals down the slope will provide long-term access to the slope face. This access is needed to control surface water run-off, to establish a network of horizontal dewatering drainholes and stability monitoring stations, and to enable rapid clean-up and remediation of any instabilities that might develop in the inter-ramp slopes.

- Ongoing geotechnical mapping of slope faces, geotechnical drilling, and monitoring of the cutback slope are required to confirm design assumptions and to refine the overall design. Prompt interpretation of all new data will be an ongoing requirement. Belated interpretation produces information that is often of academic interest only. Conversely, early recognition and attention to at-risk areas on the slope face allows timely remediation should these areas develop into significant concerns.

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11 References


