DESIGN CONSIDERATIONS FOR THE DAMANG OPEN PIT EXPANSION

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ABSTRACT

Abosso Goldfields Limited (AGL) is in the process of developing an expansion of their Damang Open Pit gold mine in Ghana. AGL acquired the Damang mine in January 2002, with SRK participating in the geotechnical investigation and design for the project since 2001. A deep resource drilling program, completed during 2004, intersected an extension of the orebody below the then planned final pit floor. A feasibility study has been completed to determine the economic extraction of this deep ore with an expanded pit shell.

The Damang mine is in a stockwork sulphide deposit at the culmination of the Damang anticline. The deposit occurs as hydrothermal mineralisation associated with dominantly east dipping thrusts and sub-horizontal quartz veins. Primary gold mineralisation also occurs in the conglomerates of the Tarkwaian Formation. The geological units consist of steep easterly dipping dolerite, sandstone, siltstone and phyllite beds, with the weathering profile comprising saprolites overlying a relatively thin joint oxidized horizon.

It is planned to deepen the current open pit to a final depth of up to 270 m, primarily by the development of a push-back of the eastern and western walls. The edge of the Eastern Tailings Storage Facility (ETSF) is currently located within 80 to 100 m of the crest of the existing pit.

The geotechnical design focused on the optimization of the bench face angle and finally recommended a “Steep Slope Strategy”, involving 24m high near vertical benches. Limiting bench stack heights of 96m at a maximum of 66° were derived from the influence of major structures and the variation in rock mass conditions. The overall slope angles in each pit sector were developed around the requirement for haulage ramps.

One of the main design challenges was to optimise the standoff distance between the final pit crest and the ETSF. This process had to consider maximum access to the resource without undue influence on or from the nearby tailings dam. A planned increase to the height of the dam wall was an additional factor in the design.

Dewatering of the pit walls was also a major consideration in this tropical environment.
1 INTRODUCTION

Aboso Goldfields (AGL) are undertaking a pit expansion of their current operations at the Damang Goldmine near Tarkwa, Ghana. The proposed pit expansion was devised following the results of a 2004 deep drilling program that identified a significant extension to the resource. A feasibility study was undertaken to consider the viability of mining the increased resource. The proposed cutback of the east and west walls will deepen the pit to a maximum depth of 270 m. The increase in the pit footprint from the expansion will bring the eastern pit crest closer to the western embankment of the existing Eastern Tailings Storage Facility (ETSF). The envisaged cutback involves several controlling geotechnical issues related to the stability of the weathered and fresh rock slopes requiring the development of practical and aggressive geotechnical solutions.

2 PROJECT BACKGROUND

Mining at Damang dates back to 1989 when Ranger Minerals began exploration in the area. Exploration and plant construction between 1990 and 1995 led to the first gold poured at Damang in 1997. The Damang operation has primarily been mined from a single large main pit with the primary orebody in an unweathered hydrothermal steeply dipping setting.

In January 2002 the mine was acquired by AGL and operated as a contract-mined truck and shovel operation. SRK became involved with the project in 2001 and have continued to provide AGL with geotechnical advice on operation and design issues.

The previous life-of-mine plan scheduled the main Damang pit for completion in October 2004, with subsequent near term gold production from treatment of low-grade stockpiles blended with oxide ores from the conglomerate deposits at Kwisie and Lima, and the hydrothermal Rex Orebody. The primary challenge facing Damang was to sustain quality mill feed beyond the scheduled completion of mining from the existing pit.

The Damang Extension Project (DEP) was initiated in December 2003 to accelerate Mineral Resource discovery and Reserve conversion, predominantly through the evaluation of a number of satellite deposits located within the concession areas. This work has successfully brought additional Mineral Resources and Reserves to account from the palaeoplacer Tomento North and Tomento East orebodies, and the hydrothermal Amoanda and Rex Orebodies.

The original mining plan assumed a 21 Mt of ore production with tailings retained within a 15 m high ETSF. The ETSF was constructed using a downstream technique forming a paddock to the east of the pit formed by a zoned earthfill embankment with an upstream sloping, low permeability core. The east wall of the pit lies adjacent to the western embankment of the ETSF. Both upstream and downstream embankment shoulders were
protected using mine waste buttresses. The dam core was extended to form an engineered keyway within the weathered bedrock to restrict seepage through local higher permeability superficial deposits. A shallow interceptor drain was constructed along the downstream toe to capture any surface run off or shallow alluvial groundwater seepage.

Figure 1: Site layout showing Damang Pit and Tailings Storage Facility (west embankment).

The existing pit is aligned to mine grid and forms an excavation 1200 m (North - South) by 300 m (East – West) with a current maximum depth of 200 m. Pit access is via a...
single ramp on the East wall and includes a single switch back. The ramp only passes beneath the west wall near the current pit floor.

3 SITE CONDITIONS

The mine site is located in south western Ghana, close to the town of Tarkwa and approximately 250 km northwest of the Ghanaian capital, Accra. The site receives about 2000 mm of annual rainfall delivered mainly in relatively short high intensity storm events concentrated in wet seasons between May to June and September to November. The daily temperature is relatively uniform with an annual average of 26°C.

The site was developed from a waterlogged region within the gently undulating Ayaasu River valley. The broad river flows northwards at a gradient of about 1:600. Along the western ETSF embankment alignment, the foundations intersected low ridges and valleys containing old alluvial channels. During the original construction of the ETSF in these areas, mine waste working platforms often had to be placed to maintain trafficability.

4 SITE GEOLOGY

The site comprises a deeply weathered profile with a transition from a Residual Soil or Saprolite to generally fine grained metasediments. Within the Ayaasu river channel the weathered strata have been locally eroded and are overlain by a thin layer (reportedly < 2.5m) of a low strength mixture of cohesive and mainly granular alluvial sediments.

The generally sub-horizontal tropical weathering profile has overprinted the original topography to a typical thickness of 10m to 40m. Depth development varies and is less extensive in areas of inferred originally low elevation, where near surface groundwater has reduced the potential for oxidation processes.

4.1 Lithology

The palaeo-proterozoic metasediments of the Tarkwaian Group comprise conglomerates, sandstones and subordinate shale layers lying within the Southern Ashanti Belt (Figure 1).

- Huni Sandstone is a fine to medium grained sandstone that becomes increasingly silty towards the base. The unit is thickly bedded in pit exposures and contains discontinuous siltstone lenses and beds. The unit contains well defined primary bedding planes and weak cross beds and displays well developed sub-vertical jointing. Up to two additional moderately dipping joint sets may be locally present. These are variably developed, however where they occur they exert a strong influence on structural stability, particularly in the upper, semi oxidised sections of the wall.

- Tarkwa Phyllite is a fine grained, thinly bedded siltstone and mudstone unit with strongly developed and highly continuous bedding planes. Sub-vertical and sub-
horizontal joints are also well developed, and strongly influence the structural stability of exposures.

- Banket Series sandstones are a series of medium to coarse grained sandstones and conglomerates which host the majority of the mineralisation. The unit is thickly bedded with weakly cross bedded sandstone between the major bedding planes. Sub-vertical jointing is well developed in the lower sections of the wall. Locally, sub-horizontal jointing occurs as a result of low angle faults.
- A series of dolerite dykes and sills intrudes the sediments. The dolerite units often have strongly developed sub-horizontal jointing, and jointing parallel to the regional stratigraphy. Joint surfaces are normally rough. These are generally basaltic structures ranging from < 1 m to >>100 m in thickness.

Figure 2: Geology of the Ghana and the Ashanti Belt.

4.2 Primary Geological Structure
The Tarkwaian sediments dip steeply (on average 70°) east and in the area of the Damang mine are tightly to isoclinally folded into the gently NNE plunging Damang anticline. The anticlinal hinge is not exposed within the current pit excavations.
Major structure mapping identified two major East-West faults - the Northern Fault and the Damang Central Fault. Neither of these faults appears to adversely influence the pit slope stability although they do act to concentrate groundwater seepage on the eastern wall. Dolerite intrusives occupy some sections of the fault. A sinistrally slickensided shear fabric was identified in the pit exposures associated with the North-South striking Damang Fault. This fabric forms a steeply dipping set of mineralised defects observed to locally influence batter stability in the central section of the pit.

The hinge of the Damang Anticline is nearly co-planar with the north – south trending sub-vertical Damang Fault which has been largely mined out during past operations. The fault appears typically as a narrow silicified and altered structure at depth but forms a wider fracture zone, observed to be about 20m wide, nearer to ground surface.

4.3 Mineralisation

There are 2 distinct types of Au mineralisation at Damang:

- Free gold deposits associated with sedimentary conglomerates within the Banket Series, widely interpreted as a palaeo-placer deposit and representing possibly 20% of the Damang resource.
- In addition there are horizontal to shallowly dipping late stage hydrothermal quartz veins and mineralised faults mostly developed in the Banket Sandstone and the Dolerite.

The mineralisation is hosted in Tarkwaian metasediments as well as the intruded dolerites with 85% of the economic Au mineralisation occurring below the Tarkwa Phyllite (Tunks et al., 2004).

4.4 Hydrogeology

The site groundwater regime is recharged by the Ayaasu river system, although the fine grained, low permeability, alluvial sediments and upper weathered profile restrict the seepage velocity.

Conceptually, the recent sediments appear as saturated local aquifers within the saprolitic weathered profile which forms a flat lying aquitard. The saprolite in the lower expression is influenced by the steep easterly-dipping bedding. The orientation of the structure and the fine grained sedimentology have the effect of restricting flow towards the pit and therefore maintaining locally elevated hydraulic gradients.

Previous groundwater studies (Knight Piésold, 1997 and 1998) have identified the flow within the fresh rock to be mainly fracture controlled. A zone of enhanced permeability in the central and western portion of the pit was postulated and field observations this zone is related to the intersection of 2 East-West striking faults.
Groundwater entering the pit is pumped from a sump at the pit floor. Surface water is currently intercepted by a diversion drain that runs along the pit perimeter. There are no active dewatering measures in place for the pit slopes. Groundwater drains from the base of weathered material in the middle of the east wall. Elsewhere on the eastern side there is evidence of past instability due to localised pore pressure elevation but this appears temporal and to stabilise following small scale slumping.

![Figure 3: Typical geological cross section showing the current and proposed final cutback profiles superimposed on the lithological model and major North-South Faults.](image)

The predominant source of water inflow into the pit is direct rainfall and associated run-off. Minor quantities of ground water seepage from the pit walls also contribute to the overall inflow. Pumping from the pit bottom resumed in advance of the proposed cutback. The current dewatering system consists of a two-lift arrangement with mains electric powered submersible pumps and a combination of 200mm and 350mm diameter discharge pipelines located on the eastern wall of the pit. The lower pumps consist of two 55kW pumps while the stage pumps located part way up the wall consist of two 75kW pumps. Power cables pass through a service drill hole from the surface to a location near the upper pumps.

5 MINING CONSTRAINTS

It was proposed to deepen the pit to a maximum final depth of 270 m over a 4 year period. Preliminary studies indicated some 700,000 oz Au could be available in an optimised pit shell. An aggressive design was implemented to optimise the ore recovery
whilst accommodating the recognised geotechnical risks. The geotechnical risks to achieving the design profile are a function of the strength of the residual soil and the rock mass, the presence of geological structures and the near-pit piezometric (hydrogeological) conditions. The proposed cutback is shown in Figure 4.

The key mining constraints controlling the design are summarised below:

- Minimum mining width is 37 m based upon the Caterpillar 777D haul trucks. The area within 6.5 m from the crest of the existing pit is considered a high-risk area and will be cordoned off during mining leaving approximately 30 m for equipment access in the cutback on the western side of the pit, which is just sufficient to allow the turning of dump trucks (with a turning circle of 28.4 m).
- The final design has single ramp which enters the pit in the north-eastern corner, avoiding the need to traverse the waste dump with haul trucks. The minimum ramp width is 23.5 m for double lane and 14 m for single lane, with a majority of the ramp having a gradient of 1 in 10. The gradient is steepened to 1 in 9 for the bottom 90 m of the pit. In addition 3 temporary ramps on the west wall will be used for the top 36m of the pit to reduce haulage distances.
- The distance required for turning of the 777D trucks, which are the largest trucks employed on the mine, is 28.4 m. The width of the cut back pit is therefore just sufficient for this purpose.
- The northern wall of the pit is formed by an in-pit waste dump which currently stands at 130 m high. The new design looks to form a stable slope through this material at an angle of 35° using the same 24m bench height geometry with a 5 m wide berm. The waste dump will be excavated in 3m flitches to achieve the final profile.
- ETSF stand-off distance - The proposed pit deepening and the easterly dip of the orebody will reduce the distance between the dam and the pit. The tailings from the proposed mining operations will be stored in an expanded ETSF with the Stage 1 crest raised 15 m to a final height of 30m. The ETSF has been raised using downstream construction by extending the zoned earthfill design whilst maintaining a downstream waste rock buttress. Alternative construction options are being considered for the top 5 m of the dam.
- The ETSF foundations and the crest of the East Wall cutback are in the variably weathered saprolite profile. The saprolite slopes were designed to an overall FoS of 1.2, as well as ensuring that the FoS of the ETSF embankment was ≥ 1.35 to account for the increased consequences of failure. This level of FoS was considered suitable given the expected short duration of mining operations in the pit.
GEOTECHNICAL DESIGN ISSUES

An economic cutback of the existing pit requires several geotechnical issues to be mitigated. To accommodate the mining design constraints a geotechnical investigation was carried out for the cutback, and especially the ETSF stand-off distance.

The geotechnical design criteria included:
- Development of suitable saprolite slope angles for the East wall cutback recognising the influence of the ETSF. Slope angle selection is crucially
important in the Residual soils and weathered rock profile. The consequences of slope failure within these materials could be insurmountable for the project. The slope angles along the East Wall need to reflect not only the relatively low strength of the weathered materials but also the varying levels of saturation (significant in terms of effective stress) and potential destabilising influence of the earthworks of the ETSF western embankment.

- Optimisation of slope angles to evaluate unweathered rock on the east and west walls. In the unweathered rock, the selection of steep slope angles would provide access to a potentially larger resource. The slope angles were influenced by the presence of secondary geological structures within the pit walls. Analysis of the kinematic stability of measured and statistically representative discontinuity sets enabled a suitable face angle to be selected and appropriate spill berm widths to be calculated. The overall stability of the proposed slope was analysed using numerical modelling and considered the effective rock mass strength.

- Verification of slope geometry design assumptions for in pit waste dump. In the northern in-pit waste dump the slope design required an estimate of waste rock material properties. This was carried out using the results of a literature review backed up by empirical observation of on site performance.

- Development of groundwater management plan including slope depressurisation strategy for fresh and weathered rocks. Pit dewatering using sump pumping occurs currently and will be maintained to avoid water ponding at the base of the pit. In addition, the stability analysis shows that to achieve the design FoS the natural groundwater profile must be modified by the use of horizontal drains to reduce the magnitude of the pore pressures that can build within the slope.

7 GEOTECHNICAL DESIGN SOLUTIONS

The geotechnical solutions developed to mitigate the mining constraints were based on SRK’s 4 years of site specific experience which was supplemented by the “total engineering geology” methodology (Fookes et al., 2000). The construction of a conceptual engineering geological model for the project allowed the identification of potential geo-hazards at the beginning of the project and the subsequent ground truthing with observations from field investigations.

7.1 Weathered rock

The weathered profile presented the most significant geotechnical challenge to the geotechnical classification and design. The site history indicated a variable thickness could be expected because of the pre-mining topography, as well as the expected variable development of oxidation between the better drained higher elevations and the poorer drained lower elevations, as illustrated in Figure 5 below:
7.1.1 East Wall Investigations

An investigation was designed to sample the subsurface properties of the weathered profile. The area available for drilling was restricted by the location of waste dumps within 50 m to 100 m of the current pit crest, as well as the need to maintain the integrity of the zones of the existing tailings dam. In addition the available geotechnical drilling equipment was limited both at the mine and locally. The investigation area and the design sectors are shown in Figure 6.

A cable tool rig was sourced locally with the capability to take Standard Penetration Tests (SPT's) and undisturbed samples. A drill programme was designed and was complemented by geotechnical mapping of local exposure, with particular attention to material distribution and the presence of groundwater. The site geological database was also interrogated to provide the most detailed geological model of the site.

Geotechnical domains were defined using the field investigation results and interpretation was based upon the conceptual engineering geological model. The domains were related to the site geological model and boundaries interpreted based upon ground and groundwater conditions using engineering geological judgement. Areas A and C at the northern and southern ends of the project represent similar deep developments of oxidised Huni sandstone to a depth of at least 30 m. The intervening area B was subdivided into B1 and B2 to reflect the change in base of weathering despite the common reduction to that seen in the other areas. Area B1 and B2 were characterised by deposits of alluvium.
7.1.2 Laboratory testing
Geotechnical classification testing was carried out at the laboratories of the Tarkwa School of Mines. Also, Consolidated Undrained Triaxial (CUT) testing was performed by the Civil Engineering Research Laboratory at the Kumasi National University of Science and Technology (KNUST).

Classification testing indicated the alluvial and weathered material to be of low to moderate plasticity (generally Liquid Limit < 35%) with a low potential for volumetric shrinkage with desiccation. The results confirm the field assessments and observations. The particle size distribution analyses indicate the sampled materials to be fine grained soils, comprising predominantly silt and sand sized particles with up to 8% clay.

CUT testing on 4 U100 samples obtained from the geotechnical drilling in Areas A and C showed the drained friction angle of the saprolite lay within a broad range of 27° to 35°, as would be expected for a material formed from the in situ weathering of the Huni Sandstone.
The triaxial test results are compared in Figure 7 with the effective friction angle calculated from standard empirical analyses available for SPT test analysis, as well as standard empirical soil classification tests. The selected drained strength assessment approaches were Terzaghi & Peck (1948), Thorburn (1963) and Peck et al. (1974). Also empirical SPT correlations (e.g. Appolonia et al. 1970) and analyses based upon the results of the grading analysis (Lang & Huder, 1994) were calculated. The results show good agreement between all methods, allowing a design curve to be developed for the slope stability analyses for each design sector (Figure 7).

**Figure 7: A comparison of CUT test results with field and lab testing empirical soil strength relationships.**

### 7.1.3 Stability modelling

Stability modelling was approached using the limit equilibrium program SLIDE to evaluate the stress distribution and potential failure surfaces, and FLAC to consider the deformation analysis of the oxide slopes and the tailings dam.

The SLIDE analyses identified the principal stability modes which needed to be satisfied as illustrated in Figure 8.
Figure 8: Typical Stability Analysis Results for Stand-Off Without Waste Buttress.

- **Mode 1**: The red to green region near the crest. A design FoS of 1.25 is considered suitable for these slopes i.e. failure influencing the surface of the saprolite slope.
- **Mode 2**: A zone marked by a long blue dashed line including the dam crest and the downstream face and daylighting at the base of the weathered profile. A higher design FoS of at least 1.35 is recommended against failure endangering the dam. A Mode 2 failure described larger failure surfaces that intersected the saprolite slope and the ETSF dam crest.
- **Mode 3**: A zone marked by a shorter red dashed line extending from the dam crest and daylighting between the downstream toe and the pit crest. A design FoS of 1.35 is applicable. For conditions of weak foundation materials, a Mode 3 failure was identified, which intersects the ETSF dam crest and part of the Saprolite profile (i.e. not as extensive as Mode 2). The design safety factor for all failure modes intersecting the dam crest was 1.35.
- **Mode 4**: A deeper seated, light and dark blue region failing through the tailings and upstream dam face and daylighting at the base of the weathered profile. A design FoS of 1.35 is applicable.

Most critical to stability in Area A is Mode 2. The presence of a weaker surface layer in Area B showed that Mode 3 is the controlling mechanism here. Analyses of the influence of the waste rock downstream buttress revealed that the western toe of the waste rock buttress must be at least 20m from the crest of the weathered slopes in all areas. In Area B the waste rock buttress is required to achieve the design safety factor.
The results of the SLIDE analyses are summarised in Table 1. The results represent Mode 2 Failure surfaces only (i.e. passing through the dam crest). The Mode 1 failure surfaces (shallow failures of the crest) tend to have the lowest FoS (refer Figure 8), but the stability can be improved by the drainage measures discussed hereafter.

The models for CP3, CP4 and CP5 indicated that the Mode 3 failure FoS was less than the Mode 2 value because of the greater thickness of the lower strength near-surface materials. The stand-off distance for these sections will depend upon the inclusion of a downstream waste rock buttress.

<table>
<thead>
<tr>
<th>Stand-off Distance (D)</th>
<th>CP1</th>
<th>CP2</th>
<th>CP3*</th>
<th>CP4*</th>
<th>CP5*</th>
<th>CP6</th>
</tr>
</thead>
<tbody>
<tr>
<td>30 m</td>
<td>1.117</td>
<td>1.129</td>
<td>1.289</td>
<td>1.252</td>
<td></td>
<td></td>
</tr>
<tr>
<td>50 m</td>
<td>1.243</td>
<td>1.278</td>
<td>1.463</td>
<td>1.313</td>
<td>&gt; 1.8</td>
<td>1.438</td>
</tr>
<tr>
<td>70 m</td>
<td>1.611</td>
<td>1.550</td>
<td>&gt; 2</td>
<td></td>
<td></td>
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</tbody>
</table>

* Mode 3 Failure has a lower FoS for this section because of the thickness of low strength material. Downstream buttress important for stabilisation to achieve FoS > 1.35.

The FLAC (ITASCA, 2000) models were used to consider the development of strain within the oxide slopes that could impact upon the integrity of the tailings dam. The calculated strains were compared with those published by Brox et al. (2003) as well as the results of observed slope deformation at current project sites, and found to give good agreement between the magnitude of strain, observed slope conditions and the computations.

Pit slope deformation observations and published case history data (Brox et al., 2003) indicate that tension cracking can be induced at < 0.2% strain. A deformation of 0.6% strain is associated with onset of progressive failure, and 2% strain is associated with actual failure. These criteria were used as a guide to modify the stand off distance obtained from the SLIDE (Rocscience, 2004) analyses.
Saprolite depressurisation

The effectiveness of horizontal depressurisation drains in the saprolite was studied using FEFLOW (WASY, 2005) finite element modelling. The saprolite bulk hydraulic conductivity was estimated by performing falling head tests in the existing piezometers. In the field the structure dipped steeply to the east (i.e. away from the pit) indicating that there was likely to be hydraulic anisotropy present, with the maximum conductivity oriented parallel to bedding.

Saprolite recommendations

The Saprolite slopes on the East Wall form a relatively low strength, low permeability foundation for the ETSF and were the focus of a separate investigation to assess the suitable slope design parameters. The investigation indicated geotechnical conditions varied along the approximately 600 m crest length and could be attributed to 3 general design sectors.

The geotechnical recommendations for the weathered (saprolite) material were based on the results of the site investigation and laboratory testing. A geotechnical model was developed from the site investigation results to study the influence of the pit crest proximity on the stability to the TSF. The potential geotechnical interaction of the 2
structures was assessed using limit equilibrium stability analyses to study the slope instability characteristics. The model was also studied using non-linear finite difference code to study the role of foundation deformation (strain). There was a concern that small strains associated with elastic foundation deformation could provide potential seepage pathways that could lead to piping failure of the ETSF and endanger pit personnel and operations. The results of the numerical analyses were compared with published slope deformation data (Brox et al., 2003) as well engineering judgement based upon performance of similar unpublished cases. The result of these studies lead to the recommendation of a minimum stand-off distance between the ETSF and the pit crest (Table 2).

<table>
<thead>
<tr>
<th>North sector (Area A)</th>
<th>Central sector (Area B1 to Area B2)</th>
<th>South (Area C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CP1 60</td>
<td>CP3 55</td>
<td>CP6 50</td>
</tr>
<tr>
<td>CP2 60</td>
<td>CP4 55</td>
<td></td>
</tr>
<tr>
<td></td>
<td>CP5 55</td>
<td></td>
</tr>
</tbody>
</table>

The design required a minimum stand off distance of between 50 m and 60 m with the stability to be enhanced by the installation of horizontal drains at the base of the oxides to provide pit slope depressurisation and groundwater control within the Saprolite.

7.2 Unweathered rock

The relatively poor geotechnical characteristics of the Saprolite material required an aggressive approach for the unweathered rock slopes in order to optimise ore recovery. This part of the design was assisted by the relatively high strength rockmass resulting from a combination of high UCS and a well interlocked structural fabric. The geotechnical design was carried out using structural and geotechnical angled holes drilled and logged by mine geotechnical and geological staff. The data was used to generate structural and empirical data sets for use in slope design. Laboratory testing of selected samples was performed to provide representative design properties.

7.2.1 Empirical and structural analysis

Empirical and structural analyses were performed to provide an envelope of suitable bench stack angles (BSA) and spill berm width (SBW). The analyses were based on:

- the application of Rock Mass Rating (RMR) and Mining Rock Mass Rating (MRMR) to slope design established in Laubscher (1990) and Haines & Terbrugge (1991);
- the analysis of the kinematic stability of wedges resulting from the structural logging of the investigation boreholes e.g. using SWEDGE (Rocscience, 2002) and accepted berm geometry formulae, e.g. Piteau & Martin (1977).

This envelope of slope angles can then be tested using numerical models to refine the geometrical design whilst accounting for stress redistribution and rock mass deformation.
7.2.2 Numerical analysis

Overall or large-scale slope stability was assessed using FLAC (ITASCA, 2000) by approximating the rock mass as a continua defined by equivalent Mohr Coulomb strength parameters. The slope was represented at the stack scale and included geotechnical berms (GB) and ramps to define the overall breaks in slope (Figure 10).

![Figure 10: Geological model for West Wall Cutback showing lithologies and major structures.](image)

Intact rock properties for design were developed from laboratory testing and these were used to estimate appropriate rock mass strengths.

The Factor of Safety (FoS) for the FLAC analyses was calculated by systematically reducing the model shear strength until failure occurred. Unlike the SLIDE computations, these analyses considered the effective shear strength of the rock mass, and predominantly the unweathered section of the proposed pit slope.

Pit slope dewatering and depressurisation was included in the design assumptions to provide the required design FoS. The analyses showed that rock mass depressurisation was required equivalent to a lateral groundwater profile push back of 100 m to 75 m (relative to the final profile). This was modelled using nominal 100 mm diameter drains lined over the collar length to prevent obstruction from blast damaged rock.
7.2.3 Unweathered rock recommendations

Following geotechnical investigations new geometric specifications were developed for the proposed cutback pit design in the rock (Table 3):

Table 3: Proposed cutback pit design summary for unweathered rock with pre-existing BFA and SBW values shown in brackets.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>East Wall</th>
<th>West Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bench height</td>
<td>24 m</td>
<td>24 m</td>
</tr>
<tr>
<td>Bench Face Angle (BFA)</td>
<td>80° (75°)</td>
<td>75° (68°)</td>
</tr>
<tr>
<td>Spill Berm Width (SBW)</td>
<td>8.5 m</td>
<td>6.5 m (8 m)</td>
</tr>
<tr>
<td>Ramp</td>
<td>23.5 m</td>
<td>15 m</td>
</tr>
<tr>
<td>Geotechnical Berm (GB)</td>
<td>≤ 17 m</td>
<td>13 m</td>
</tr>
<tr>
<td>Stack Height</td>
<td>96 m</td>
<td>96 m</td>
</tr>
<tr>
<td>Bench Stack Angle (BSA)</td>
<td>66° (64°)</td>
<td>65° (63°)</td>
</tr>
</tbody>
</table>

The Overall Slope Angle (OSA) varied on each wall due to the location of the ramps. On the east wall the maximum OSA reached 66.5° over a 126 m vertical height but was locally as low as 55° over 76 m. On the west wall the OSA varied between 62° and 53° over 142 m and 246 m respectively. The steepening of the slope angle for the cutback can be seen clearly in Figure 3.

The investigations enabled a steepening of the bench face angle by 5° and 7° for the east and west walls respectively, and included a reduction in the SBW by 1.5 m on both walls whilst improving wall stability against mid-scale instability by the inclusion of a geotechnical safety berm between each stack.

7.3 Groundwater Control Measures

Pit slope drainage and depressurisation are important in the maintenance of stable slopes. The approach is different in unweathered and weathered rock as explained below.

7.3.1 Unweathered rock

Beneath the existing ETSF the interaction of the North and Central Fault seeps water at point sources of typically 1 to 5 l/min. This zone will experience an increase in hydraulic head of approximately 20% from the tailings dam raise and so inflows are expected to increase proportionally. In addition, the act of steepening the pit walls and deepening the pit will lead to increased rock mass dilation in the slope vicinity which is expected to increase the storativity and secondary permeability of the rock mass, further promoting the increase in anticipated inflow.

The proposed control measure for groundwater within the unweathered rock is to construct a series of horizontal drains along the geotechnical safety berms i.e., 96 m apart...
vertically and spaced laterally approximately 75 m. The drains will be installed as early as possible during the cutback mining so as to maximise the stabilisation effect of the drainage measures (Figure 11).

![Figure 11: Cross section through West Wall showing horizontal drain locations in unweathered rock.](image)

**7.3.2 Weathered rock**

The depressurisation of the saprolite or oxide slopes is key to the success of the cutback project and can be achieved by the use of surface and groundwater control measures (Figure 12). The profile is known to be of low permeability with adversely dipping structure restricting drainage potential. The material is locally dispersive posing additional challenges for the management of surface water discharge and the control of groundwater drainage.
The surface water must be managed to control run off erosion and ponding by maintenance of a perimeter diversion drain. This will be assisted by a trench drain along the perimeter of the western embankment of the ETSF. The drain will intercept near surface groundwater that is flowing along pre-existing fluvial features and potential seepage from the tailings dam. This drain will reduce the western development of the groundwater profile.

Deeper groundwater and associated pore pressures within the oxides will be drained by the drilling of shallow 30 m long horizontal drains at the base of the oxidised zone. These will be lined and closely spaced to allow for the low permeability of the weathered layer.

8 PROJECT OUTCOMES

The following outcomes were developed from the cutback design process:

- Use a Steep Slope Strategy in the unweathered materials to maximise the slope angle - The field investigations and site geotechnical procedures indicated the rock mass quality was measured in terms of the RMR and to lie within the range RMR 60 to 70 i.e. good quality rock. The relationship between slope angle and RMR can be judged using accepted relationships such as Haines & Terbrugge (1991).

- ETSF construction to minimise encroachment towards the pit - The current design for the ETSF calls for crest raises to be formed using the downstream construction method. The ETSF downstream footprint can be altered by choosing an upstream method for the top 5 m of the dam. Alternatively, a secondary TSF could be developed presenting the potential to limit the maximum crest height and make available a further section of the east wall.
A geological engineering model has been developed to assist in providing effective geotechnical design responses to the particular mining problems posed at Damang. The model comprises a variably tropically weathered profile, overlying steeply dipping fresh sedimentary rocks with a low lying undulating and waterlogged topography. Understanding the evolution of the topography provides an explanation behind the local development of the weathering profile exposed in the mining profile. The same model provides guidance on the hydrogeological characteristics of the recent sediments and the fresh rock. Recognition of the different sources of information that input the conceptual model allows predictions to be made of the expected performance of different aspects of the system.

Geological conditions and slope behaviour have to be measured and analysed to test the proposed geological engineering model for the design to be validated. The validation process is envisaged to be incorporated in a slope management programme.

Slope monitoring programme - The slopes should be regularly surveyed to monitor for areas of deformation. This will test the adequacy of the slope design geometry. This can best be achieved with an automatic permanent survey network of monitoring prisms. Areas of concern can be defined by increased reading frequency, survey station density and by real time displacement monitoring if necessary. Survey should measure the performance of both fresh and weathered rock slopes, as well as the ETSF embankments.

During mining to the design profile the additional wall exposure should be geologically and geotechnically mapped, making particular note of structures and the presence and quantity of groundwater seepage. The regular mapping of geological and groundwater seepage will provide early warning of the relevance of the design assumptions, and alert the site to the onset of hazardous conditions.

The construction and installation of pit drainage and depressurisation measures will enable management of the site groundwater conditions. The flows and pore pressures should be monitored to assess the conformance of field conditions with the design requirements.

Pre-split blasting techniques will be employed where necessary, to ensure that berm crests and pit walls are not damaged during production blasting.

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