A Phased Modelling Approach to Identify Passive Drainage Requirements for Ensuring Stability of the Proposed West Wall Cutback at Ok Tedi Mine, Papua New Guinea

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Abstract

The Ok Tedi Copper-Gold mine in Papua New Guinea is situated within a seismically active, mountainous region of extremely high rainfall. The current open pit is transected by several large faults, and the rock mass conditions are complex. Material permeabilities are variable, with considerable contrast within the major rock types and fault zones. The pit is being progressively deepened with ongoing mining, and a cutback of the West Wall is being considered which would result in a final wall height of nearly 1000m.

A phased modelling approach was devised in order to make an assessment of the stability of the cutback design for the West Wall, and to investigate passive drainage requirements for ensuring its stability. The modelling involved an iterative series of numerical analyses. The stability of the cutback design was evaluated using first the existing hydrogeological model, and subsequently by using the pore pressure distributions generated from 2-D and quasi 3-D hydrogeological modelling with the inclusion of horizontal drainholes. The modelling was conducted for a range of drainhole designs with carefully selected spacings and lengths. Pit slope stability evaluations were conducted using small-strain finite element analyses, whilst hydrogeological modelling of pore pressure distributions was carried out using unsaturated finite element flow modelling.

The drainhole designs required to maintain stability of the current cutback design at a suitable factor of safety were identified. The results of the modelling are to be used as input for confirmatory slope stability analyses using distinct element methods.

1 Introduction

Ok Tedi mine is situated in the highlands of the Western Province of Papua New Guinea, close to the border with Indonesian territory (see Figure 1). The orebody lies within Mt Fubilan and contains copper, gold and silver mineralisation. The mine is currently an open pit operation in which about 78,000 tonnes of ore and 80,000 tonnes of waste rock are mined each day from a pit covering approximately 3km by 2km in plan area.

1.1 Background to the study

The current large open pit at Ok Tedi is being progressively deepened, with open pit mining originally intended to cease in 2013. Extension of the mining activities was planned, by means of underground mining of the lower mineralised skarn bodies flanking the central monzodiorite and monzonite porphyry orebodies. Pre-feasibility studies for underground mining beneath the east and West Walls of the pit were completed in 2008. The risk assessments conducted during these studies indicated that underground mining would be feasible beneath the east wall only. The skarn body on the western margin of the intrusions contains the greatest tonnage of ore, and in order to support ongoing mining it was deemed necessary to access this material by means of a significant cutback of the West Wall. This would involve pushing the wall back 200m to 300m horizontally, resulting in a final height of nearly 1,000m. A photo of the current West Wall is included in Figure 2.
Data obtained from ongoing geotechnical drilling and mapping, structural mapping, laboratory testing and hydrogeological testing over the years has been used to progressively update the geotechnical domain and conceptual hydrogeological models. These models have been utilised to conduct numerical analyses during the current feasibility study for assessment of the stability of the West Wall cutback design.

1.2 Site geology

The geology at Ok Tedi consists of siltstones and limestones, into which large monzonite porphyry and monzodiorite bodies have been intruded. The pit is centred within these intrusive bodies, as the monzonite porphyry has formed the major ore type and makes up the majority of economic mineralisation tonnage. Skarns have been formed on the eastern and western margins of the intrusive bodies, and are of two main types. The endoskarns are of igneous protolith, and have only minor ore grade mineralisation. The skarns of sedimentary protolith lie immediately outside the endoskarns and form major skarn orebodies, and these are a principal target of ongoing mining operations. The endoskarns and skarns present highly variable, often weak rock, except the skarns that are magnetite-rich which present very strong and sparsely-jointed rock.

Two major thrust zones are recognised within the mining area: the Parrot’s Beak Thrust and the Taranaki Thrust. These are well exposed in the West Wall of the pit. The thrust faults contain highly fractured and altered fault gouge, pyrite, magnetite skarn lenses, brecciated monzodiorite, and brecciated siltstone hornfels. A fracture zone of generally 20-30 m thickness is associated with each thrust; however the Parrot's Beak thrust has been modelled with a thickness of up to 80 m in places. Recent mapping studies have identified a steeply-dipping major fault on the West Wall of the pit which has been termed “The Gleeson’s Fault”. An associated fracture zone of brecciated siltstone and highly fractured limestone is present to the immediate west of this fault. Figure 3 presents an illustrative west-east cross section through the Ok Tedi geology.
Figure 2. The current West Wall at Ok Tedi under typically cloudy conditions.

Figure 3. A WNW - ESE cross section through the Ok Tedi geology illustrating the West Wall cutback.
1.3 Focus of this study

The main factors affecting stability of the West Wall cutback are:

- the quality of the various materials within the wall;
- the position and nature of major structures; and
- the pore water pressure distribution within the wall

Of the above factors, only the pore pressure distribution within the pit wall can be adjusted to enhance stability. A phased modelling approach was devised in order to make an assessment of the stability of the cutback design for the West Wall, and to investigate passive drainage requirements for ensuring stability. The modelling involved an iterative series of numerical analyses. The stability of the cutback design was evaluated using first the existing hydrogeological model, and subsequently by using the pore pressure distributions generated from 2-D and quasi 3-D hydrogeological modelling with the inclusion of horizontal drainholes. The modelling was conducted for a range of drainhole designs to test effectiveness of various spacings and lengths. Pit slope stability evaluations were conducted using small-strain finite element analyses, whilst hydrogeological modelling of pore pressure distributions was carried out using unsaturated finite element flow modelling.

The drainhole designs required to maintain stability of the current cutback design at a suitable factor of safety were identified. The results of the modelling are to be used as input for confirmatory slope stability analyses using distinct element methods.

The phased modelling approach included the following steps:

1) Initial slope stability analyses of two sections by means of Phase² © small-strain finite element analysis to determine the approximate groundwater pressure distributions required in order for a Factor of Safety (FoS) of approximately 1.3 or greater to be maintained within the West Wall cutback design.

2) 2-D and 3-D hydrogeological modelling by means of FEFLOW® software to determine the drainhole configurations required to achieve the groundwater pressure determined in step 1 that will provide a stable West Wall cutback. Identification of these requirements allowed for an assessment to be made whether the necessary drainage requirements were possible to implement within the time periods under consideration during the cutback.

3) Confirmatory Phase² © analyses, using the results from the FEFLOW® modelling to provide better groundwater inputs and to confirm the lateral (i.e. out of section) spacing of drainholes required to maintain suitable wall stability. The lateral spacing assessment was performed using the groundwater conditions in sections halfway between drainholes, as indicated by 3-D FEFLOW® modelling.

Of the two sections selected for the initial Phase² © and FEFLOW® analyses, the southernmost section (Section 1) was selected in the pit area known as Paris. Section 2 was selected to assess the pit wall behaviour further to the north, in the pit area known as Berlin.

2 First-pass Phase²© analyses

The stability of the West Wall cutback in Sections 1 and 2 was analysed to study the effect of variation in groundwater conditions on pit wall stability. For each analysis, plots of total displacement and maximum shear strain were used to assess slope performance. The FoS in each case is determined by means of the strength reduction process and thus is expressed by means of the Strength Reduction Factor (SRF). It must be noted that the total displacement shown for the critical SRF plots (the SRF at which failure occurs) does not necessarily represent the actual displacement, but provides an indication of the scale of destabilisation. The first-pass stability results are summarised in Table 1.
Table 1. FoS results for first-pass Phase\textsuperscript{2} analyses.

<table>
<thead>
<tr>
<th>Section</th>
<th>Groundwater</th>
<th>Critical SRF</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Worst case groundwater level (piezometric surface almost at pit wall)</td>
<td>1.02</td>
</tr>
<tr>
<td>1</td>
<td>150m pushback of piezometric surface behind wall</td>
<td>1.36</td>
</tr>
<tr>
<td></td>
<td>200m pushback of piezometric surface behind wall</td>
<td>1.46</td>
</tr>
<tr>
<td></td>
<td>300m pushback of piezometric surface behind wall</td>
<td>1.55</td>
</tr>
<tr>
<td>2</td>
<td>Worst case groundwater level (piezometric surface almost at pit wall)</td>
<td>0.88</td>
</tr>
<tr>
<td></td>
<td>150m pushback of piezometric surface behind wall</td>
<td>1.21</td>
</tr>
<tr>
<td></td>
<td>200m pushback of piezometric surface behind wall</td>
<td>1.32</td>
</tr>
<tr>
<td></td>
<td>300m pushback of piezometric surface behind wall</td>
<td>1.41</td>
</tr>
</tbody>
</table>

These results show that the piezometric surface needs to be pushed back to around 150m behind the pit wall in Paris and 200m behind the pit wall in Berlin, in order for suitable FoS to be achieved. As an illustration, the plots of total displacement and maximum shear strain for the 150m pushback of the piezometric surface in Section 1 (Paris) are provided in Figure 4 and Figure 5 respectively. All plots for critical SRF and thus all FoS are for failure at a large scale (i.e. the whole or greater part of the pit wall).

Figure 4. Plot of total displacement for the 150m pushback of the piezometric surface in Section 1 (Paris).
Initial 2-D FEFLOW® seepage and depressurisation analyses were carried out to investigate the passive drainage requirements (horizontal drainholes) that must be employed to obtain piezometric surfaces of approximately 150m and 200m behind the West Wall cutback in Paris and Berlin respectively. The analyses were performed on the same Sections 1 and 2 used for the initial Phase 2 analyses. Separate sets of analyses were carried out for each section, in order to assess the effectiveness of various drainhole configurations. Vertical drainhole spacings of 120m (i.e. drainholes at the base of each bench stack) were assessed, with drainhole lengths of 150m and 300m. Using the latest groundwater level measurements and scientific judgement, the boundary and initial conditions were defined. Seepage boundary conditions were assigned for the pit wall face to allow seepage flow. As the volume of flow is expected to be low through drainholes, seepage boundary conditions were used for simulating drains. The precipitation recharge was assumed to be low due to surface run off along the back slope, the low permeability of the uppermost rock mass (Pnyang Siltstone), and planned measures for active dewatering and cut-off of recharge from the northwest. However, the potential effect of any recharge can be investigated in subsequent stages of modelling.

Based on the approximate expected mining period of the West Wall cutback, four excavation stages of one year period each were defined for this modelling exercise. The analysis section for Paris (Section 1) is shown in Figure 6. Horizontal drains at different levels were sequentially activated as the excavation progressed. For comparison, a model with no drains was also analysed. The analyses were carried out for Section 1 under transient conditions with both saturated and unsaturated flow modes for comparison. It is strongly suggested that this type of analysis is more accurate under unsaturated conditions, as saturated analyses overestimate the flow and show unrealistically rapid depressurization. Therefore, only unsaturated conditions were used for the
Section 2 analyses. Because field monitoring data was relatively sparse, unsaturated conditions were defined using a linear parametric model. The results of the modelling in each case were assessed using plots of the pore pressure distribution for the final West Wall cutback (after 4 years, represented by the 4th stage of excavation). For illustration, the plots for Section 1 analyses are presented in Figure 7, comparing results for both saturated and unsaturated flow. It must be noted that pore pressure distributions of over 500kPa are shown using a single colour in order to simplify the plots.

In general, the unsaturated flow results indicate that vertical spacing of horizontal drains may need to be decreased for effective depressurisation. As expected, longer horizontal drains are more effective for the selected vertical spacing.

It can be seen from the unsaturated flow analyses that drainholes of any length at 120m vertical spacing will result in an effective (average) piezometric surface that is much nearer the pit wall than the length of the drainhole. For example, drainholes of significantly greater than 150m length are required to provide an effective piezometric surface that is 150m behind the pit wall. In order to effectively assess the influence on slope stability of the various drainage configurations, the pore pressure distributions provided from each of the analyses were imported into Phase2© and a second round of slope stability analyses were performed. These analyses provided a more accurate but still simplistic assessment of slope stability in the plane of the drainholes, and are described Section 5.

In order to assess the out-of-plane (horizontal) spacing requirements for drainholes, quasi 3-D FEFLOW analyses were performed. These analyses provide for an assessment of the drainage and resulting pore pressure distributions within sections (planes) in between the drainholes, and also provide a more accurate assessment of the drainage and pore pressure distributions in the plane of the drainholes, by taking 3-D flow into account instead of being limited to 2-D flow. These 3-D FEFLOW analyses are described in Section 4, and the results have been utilised for further Phase2© stability analyses, also included in Section 5.

Figure 6. FEFLOW® Section 1 showing the excavation stages and drainhole positions used for modelling.
Figure 7. Plots of pore pressure distribution for Section 1 with drainholes at 120m vertical spacing, comparing saturated and unsaturated modes of flow for no drainage and for 150m and 300m long horizontal drains.
4 3-D FEFLOW® Modelling

The 2-D vertical sections analysed for Paris and Berlin were projected horizontally to form a 300m wide 3-D block (comprising 21 slices of 15m thickness). The model is described as quasi 3-D because, for simplicity, it was constructed of these slices (3-D numerical elements) that are extrapolated from individual sections, rather than being constructed from the detailed 3-D geological and hydrogeological models. Model conditions were interpolated linearly between sections. The same hydraulic properties, and boundary and initial conditions used for each slice of the model described in Section 3 were used for this analysis. Default no-flow boundary conditions were applied to the lateral boundaries perpendicular to the section.

Based on the results of the 2-D analyses, the model was run with 300m long horizontal drains at a vertical spacing of approximately 120m and horizontal spacing of 60m and 30m. Horizontal drains at different levels were sequentially activated as the excavation progressed. Based on the results of the Section 1 analyses, analyses were performed for Section 2 with horizontal drain spacing of 30m only. As for the 2-D analyses, results are more accurate under unsaturated flow conditions. These analyses were therefore carried out under transient mode with unsaturated flow.

The results of the modelling in each case are illustrated using plots of pore pressure distribution for the final West Wall cutback (after four years, represented by the 4th stage of excavation). For each set of analyses, pore pressure distributions are shown for two slices: the first slice represents the plane of the drainholes (best case drainage); and the second slice represents the plane halfway between the drainholes (worst case drainage). The plots for Section 1 analyses are presented in Figure 8.

Figure 8. Plots of pore pressure distribution for Section 1 with drainholes at 120m vertical spacing, comparing unsaturated flow for 300m long horizontal drains at 30m and 60m horizontal spacings A) in the plane of the drainholes and B) in a plane halfway between the drainholes.
Based on the analyses of Section 1, the unsaturated flow results indicate that the drain configuration with vertical spacing 120m and horizontal spacing of 60m is not effective. A configuration of with 30m horizontal spacing appears to be necessary to get closer to achieving effective (target) depressurisation. However, it is expected that reduced vertical spacing with a horizontal spacing of 30m will be most suitable for effective depressurisation of the West Wall cutback to be achieved in order to obtain target FoS for slope stability.

The pore pressure distributions provided from each of the 3-D analyses were imported into Phase\textsuperscript{2}© for further slope stability analyses to be performed. These Phase\textsuperscript{2}© analyses provided the most accurate assessment of slope stability and corresponding drainhole configuration requirements, and are described in Section 5.

5 Second-pass Phase\textsuperscript{2}© slope stability analyses

Further slope stability analyses with Phase\textsuperscript{2}© were performed for more accurate confirmation of depressurisation requirements using the imported pore pressures from the results of the 2-D and quasi 3-D FEFLOW analyses, described in Sections 3 and 4. These second-pass analyses were conducted using the same sections and corresponding rockmass input properties as were used for the initial analyses described in Section 2. The results of the stability analyses are summarised for comparison in Table 2.

Table 2. Results of second-pass Phase 2 stability analyses.

<table>
<thead>
<tr>
<th>Section</th>
<th>Analysis Type</th>
<th>Flow mode</th>
<th>Drain Configuration (for 120m vertical spacing)</th>
<th>Plane</th>
<th>Critical SRF</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Drain length (m)</td>
<td>Horizontal drain spacing (m)</td>
<td></td>
</tr>
<tr>
<td>2-D</td>
<td>Saturated</td>
<td>150</td>
<td>-</td>
<td>In plane of drainholes</td>
<td>1.36</td>
</tr>
<tr>
<td>1 (Paris)</td>
<td>Unsaturated</td>
<td>-</td>
<td>-</td>
<td>In plane between drainholes</td>
<td>1.04</td>
</tr>
<tr>
<td></td>
<td>Saturated</td>
<td>200</td>
<td>-</td>
<td>In plane of drainholes</td>
<td>1.41</td>
</tr>
<tr>
<td></td>
<td>Unsaturated</td>
<td>-</td>
<td>-</td>
<td>In plane between drainholes</td>
<td>1.16</td>
</tr>
<tr>
<td></td>
<td>Saturated</td>
<td>300</td>
<td>-</td>
<td>In plane of drainholes</td>
<td>1.50</td>
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<tr>
<td></td>
<td>Unsaturated</td>
<td>-</td>
<td>-</td>
<td>In plane between drainholes</td>
<td>1.43</td>
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<tr>
<td></td>
<td>Saturated</td>
<td>300</td>
<td>60</td>
<td>In plane of drainholes</td>
<td>1.48</td>
</tr>
<tr>
<td>3-D</td>
<td>Unsaturated</td>
<td>300</td>
<td>60</td>
<td>In plane between drainholes</td>
<td>1.45</td>
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<tr>
<td></td>
<td></td>
<td>-</td>
<td>-</td>
<td>In plane of drainholes</td>
<td>1.17</td>
</tr>
<tr>
<td></td>
<td>Unsaturated</td>
<td>300</td>
<td>-</td>
<td>In plane between drainholes</td>
<td>1.12</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-</td>
<td>-</td>
<td>In plane of drainholes</td>
<td>1.32</td>
</tr>
<tr>
<td></td>
<td>Unsaturated</td>
<td>300</td>
<td>30</td>
<td>In plane of drainholes</td>
<td>1.28</td>
</tr>
<tr>
<td>2 (Berlin)</td>
<td>Unsaturated</td>
<td>150</td>
<td>-</td>
<td>In plane between drainholes</td>
<td>0.89</td>
</tr>
<tr>
<td></td>
<td>Unsaturated</td>
<td>300</td>
<td>-</td>
<td>In plane of drainholes</td>
<td>1.31</td>
</tr>
<tr>
<td>3-D</td>
<td>Unsaturated</td>
<td>300</td>
<td>30</td>
<td>In plane of drainholes</td>
<td>1.14</td>
</tr>
<tr>
<td></td>
<td></td>
<td>-</td>
<td>-</td>
<td>In plane between drainholes</td>
<td>1.12</td>
</tr>
</tbody>
</table>
From the summarised results, the following is indicated:

For Section 1 (Paris):
- A drainhole length of 150m and vertical spacing of 120m is sufficient to achieve a FoS >1.3 assuming saturated flow in 2-D conditions
- A drainhole length of around 250m and vertical spacing of 120m is necessary to achieve a FoS >1.3 assuming unsaturated flow in 2-D conditions
- A drainhole length of 300m and horizontal drainhole spacing of 30m is necessary to achieve an FoS of ~1.3 where the vertical drainhole spacing is 120m assuming unsaturated flow in 3-D conditions

For Section 2 (Berlin):
- A drainhole length of 300m and vertical spacing of 120m is necessary to achieve a FoS >1.3 assuming unsaturated flow in 2-D conditions
- A drainhole length of 300m and horizontal drainhole spacing of 30m is insufficient to achieve an FoS of ~1.3 where the vertical drainhole spacing is 120m assuming unsaturated flow in 3-D conditions

6 Summary and conclusions

From the hydrogeological and slope stability numerical analyses completed so far, the following must be noted:
- Iterative modelling using geotechnical and hydrogeological codes was conducted to build a logical and orderly sequence of results (and to test sensitivities) in order for preliminary establishment of the most suitable passive drainage requirements for ensuring suitable stability of the West Wall cutback.
- The results of the slope stability analyses performed assuming unsaturated flow in 3-D conditions should be regarded as the most realistic, and these have been used to reach the conclusion below.
- The slope stability analyses and 2-D hydrogeological analyses have been conducted on selected sections and the quasi 3-D hydrogeological analyses have been conducted on blocks of limited extent within the Paris and Berlin areas of the pit, assuming the conditions within each area to be homogeneous in horizontal extent. This will not strictly be the case, and therefore the results of the analyses for each of these areas can be viewed as indicative only.
- Analyses indicate that the conditions in Paris (analysed in Section 1) result in greater pit wall stability than in Berlin (Section 2). This is likely due to the fact that the rockmass properties are in general slightly better for Paris, and the initial groundwater levels are not as high (because the direction of recharge is from the northwest, closer to Berlin). The indicative drainage requirements in Paris are, therefore, less onerous.
- A drainhole length of 300m, horizontal drainhole spacing of 30m and vertical drainhole spacing of 120m is necessary to achieve a FoS of ~1.3 in Paris.
- A drainhole length of 300m, horizontal drainhole spacing of 30m and vertical drainhole spacing of 120m will not be sufficient to achieve a FoS of ~1.3 in Berlin.
- A vertical drainhole spacing of less than 120m (perhaps in the order of 60m) may need to be considered for Berlin, although it is recognised that it may be difficult to easily accommodate surface flow resulting from intermediate level drainholes (i.e. those holes not at the level of the 30m wide access berms at the base of 120m high bench stacks). Alternatively, the effectiveness and practicality of inclined drainholes or fans of holes can be assessed.
These results have been achieved in order to a) identify the most suitable inputs for the final distinct element modelling of the West Wall cutback and b) to achieve an understanding of the likely drainage requirements and assess whether these are feasible to implement.

7 Acknowledgements

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8 References
