The Application of Rock Stress Inputs to Stability Assessments at Ok Tedi Mine, Papua New Guinea
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Abstract
This paper presents the second part of a study of the assessment of rock stress inputs on the performance of various new mining designs at the Ok Tedi large open pit mine in Papua New Guinea. Over the years, four programs of rock stress measurement have been completed at the Ok Tedi, each providing different interpretation of the in situ field stresses. Details of three of these programs and interpretations are provided in the precursor to this paper, entitled ‘Rock stresses at Ok Tedi, Papua New Guinea’ (Lee et al, 2014). This paper explains how the various stress interpretations were utilised in the modelling of the proposed new open pit and underground mining excavations, comparing where possible their expected effects on stability. The modelled performance of three specific mine components have been discussed here as case studies, including:

1. the below-pit drainage tunnel, as the floor of the open pit workings is mined down from 350 m above, to within just 75 m above the crown of the tunnel
2. the stability of the 1000 m high slope of the proposed West Wall cutback
3. the proposed sublevel open stoping extraction sequence in the Gold Coast orebody beneath the East Wall, for which only the likely ‘worst-case’ stress regime was used to assess sequencing and stability issues.

The benefits of scheduling the collection of detailed rock stress measurements and other input data from a range of different sources at various stages of the projects are discussed.

Alternative in situ stress models have less stability impact on open pit slopes than on underground mine excavations.

Introduction
Ok Tedi is a world-class open pit copper-gold mine located in the remote western highlands of Papua New Guinea near its border with Indonesia. The terrain is rugged, and the pit wall crest is 2100 m above sea level. Regional earthquake risks are moderate, typically 4–6 on the Richter scale, however more severe tremors do occur occasionally. The mining environment is fairly challenging. Annual rainfall is 9–12 m, with only a small difference in seasonal intensity. The current mine is shown in the plan view in Figure 1.

Studies for mine life extension, including options for cutback of the west and east pit walls and underground mining beneath these walls, have been ongoing since 2007. In 2013 these culminated in feasibility-level studies for sublevel open stoping and backfill beneath the East Wall in the Gold Coast region of the mine, as well as cutback of the wall in this region, and detailed evaluations for finalisation of the cutback design of the huge West Wall to approximately 1000 m in height. The proposed underground mining has now been abandoned in favour of a more substantial slope cutback of the East Wall. Mining is scheduled to continue for a period of approximately 12 years.

Comprehensive sets of numerical analyses, including 2D and 3D finite element, finite difference and distinct element analyses, were conducted to assess the stability of the various mining options. The in situ stress field is required as input into the models used for these analyses. Over the past 22 years, four programs of rock stress measurement have been completed at Ok Tedi, each providing different interpretation of the field stresses. Details of these programs and interpretations are provided in the precursor to this paper, entitled ‘Rock stresses at Ok Tedi, Papua New Guinea’ (Lee et al, 2014). This paper (essentially part 2) explains how the stress interpretations were utilised in the modelling, comparing where possible their expected effects on stability.
Figure 1: Photograph of Ok Tedi mine in plan view with underlying drainage tunnel superimposed, and (inset) 3D view from the south (note the parallax of the superimposed drainage tunnel in this view)

Stability Modelling Inputs
Rock stress interpretations
The four alternative in situ stress regimes that have been identified at Ok Tedi are described in Table 1, in terms of the relative magnitude and orientations of the principal stresses. These regimes have been taken from measurement and interpretations carried out in recent years by CSIRO (1991), WASM (2007), Mills (SCT, 2010), and Lee (AMC, 2011). The actual vertical stress was estimated as the average unit weight of a single material (25 kN/m³) times the thickness of the overburden.

Table 1: Alternative in situ stress regimes

<table>
<thead>
<tr>
<th>Stress models</th>
<th>σ1</th>
<th>σ2</th>
<th>σ3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lee (Feb 2011)</td>
<td>1.3</td>
<td>1</td>
<td>0.7</td>
</tr>
<tr>
<td>Mills (Dec 2010)</td>
<td>1.7</td>
<td>1</td>
<td>0.5</td>
</tr>
<tr>
<td>WASM (July 2007)</td>
<td>3a</td>
<td>2a</td>
<td>1</td>
</tr>
<tr>
<td>CSIRO (July 1991)</td>
<td>1</td>
<td>0.4</td>
<td>0.4</td>
</tr>
</tbody>
</table>

a – Simplified from initial calculated values (presented in Table 3) to reflect a more adverse stress field for ongoing evaluations.
The CSIRO (1991) stress regime was not directly considered in the 2007–2013 numerical modelling program as the hydro-fracturing measurements were made at only 300–400 m below the natural ground surface and thus were thought not to be representative of the stress regime at much deeper levels (1000–1200 m) where the drainage tunnel was constructed. Lee had considered these CSIRO (1991) hydro-fracturing results in developing his own stress model.

**Rock mass properties**

The geotechnical rock mass characterisation, structural model and conceptual hydrogeological model have been progressively updated since 1997, and have been significantly advanced during the recent studies. The zones of different material properties (material zones) in the models were defined by the distribution of the major lithology types and the five geotechnical domains (A to E) identified according to rock mass quality (see Table 2). The Hoek, Carranza-Torres and Corkum (2002) rock shear strength model was applied for the majority of the rock mass, except for the large fault and thrust zones of very poor conditions for which the Mohr-Coulomb model was considered more appropriate. Properties were defined for each material based on investigation data (core logging, mapping and laboratory testing data) and engineering judgement.

**Table 2: Indicative geotechnical conditions associated with each domain**

<table>
<thead>
<tr>
<th>Domain</th>
<th>Descriptor</th>
<th>Rock types</th>
<th>Rock mass conditions</th>
<th>Rock mass classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Very good</td>
<td>MP, magnetite skarn, MD in south-east area of pit</td>
<td>Strong to moderately strong, massive to large blocky rock. Occasional small fault zones.</td>
<td>RMR 50  MMR 33  Q' 7.8  Q 2.1</td>
</tr>
<tr>
<td>B</td>
<td>Good</td>
<td>PMD, east wall endoskarn, unaltered MD, limestone</td>
<td>Strong to moderately strong, large and medium blocky rock (locally small blocky). Occasional small fault zones.</td>
<td>RMR 47  MMR 29  Q' 4.0  Q 1.1</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>
<td>RMR 44  MMR 29  Q' 2.9  Q 0.8</td>
</tr>
<tr>
<td>D</td>
<td>Poor</td>
<td>Altered MD, 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>
<td>RMR 31  MMR 17  Q' 1.0  Q 0.3</td>
</tr>
<tr>
<td>E</td>
<td>Very poor</td>
<td>Thrusts, fault zones</td>
<td>Highly brecciated, granular and/or highly plastic fault breccia and gouge. Significant fault zones of very poor quality.</td>
<td>RMR 15  MMR 8  Q' 0.5  Q 0.07</td>
</tr>
</tbody>
</table>

**Rock stress input for Drainage Tunnel Modelling**

A drainage tunnel is situated beneath the open pit. Completed in 2010, the tunnel passes beneath the centre of the pit and is accessed from the pit floor by two vertical drainage shafts (see Figure 1). It was originally intended to provide drainage of the pit as it is progressively deepened, however it is also now intended as an exit way for water from the large-scale drainage measures designed to depressurise the West Wall.

**Stress field analysis**

A preliminary first-pass suite of Examine^2D^ elastic numerical modelling analyses based on the WASM in situ stress model (the only relevant data available at the time) was completed by Ok Tedi Mine staff in 2008; well before the drainage tunnel workings had been advanced beneath the open pit floor footprint. In 2008, the pit floor was 400 m+ above the crown of the tunnel. A summary of the results of this modelling is presented in Table 3. This modelling confirmed that the zone of concentrated stresses beneath the pit floor was significant and would progressively migrate downwards as the pit workings were mined down by another 300 m+ to their final design depth. Ultimately, this pit-related stress concentration zone would increase several-fold the stresses around the drainage tunnel itself; years after the tunnel had been finished and supported. Long-term tunnel stability concerns were immediately flagged. These concerns provided the incentive for better in situ stress measurements (i.e. from the drainage tunnel itself in December 2010), and for more detailed examination of in situ stress issues at Ok Tedi Mine and the expected stress changes resulting from open pit mining and their potential impact on already installed ground support in the drainage tunnel.
To address the concerns, a modelling analysis of the drainage tunnel performance during the planned mining was carried out, consisting of two phases. The first phase involved 3D elastic modelling for determination of the changes in the principal stress states at the tunnel location during the planned ongoing mining, whilst the second phase involved 2D elasto-plastic analyses of the tunnel excavation and support system.

Table 3: Results of Examine3D modelling

<table>
<thead>
<tr>
<th>Mining stage</th>
<th>Premining stress (WASM, 2007) (Horizontal:vertical)</th>
<th>Tunnel depth below pit floor (m)</th>
<th>Approximate pit mining-induced stresses at tunnelling depth as a percentage (%) of prepit mining stresses</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stage</td>
<td>Pit floor</td>
<td>H:V (N-S)</td>
<td>H:V (E-W)</td>
</tr>
<tr>
<td>1: Premining</td>
<td>970 m depth of rock cover</td>
<td>2.5</td>
<td>1.5</td>
</tr>
<tr>
<td>2: Current pit</td>
<td>440 m E-W wide pit floor at RL 1470 m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3: Final current pit</td>
<td>100 m E-W wide pit floor at RL 1300 m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4: Plan D west wall cutback pit</td>
<td>300 m E-W wide pit floor at RL 1300 m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5: Final plan D west wall cutback pit</td>
<td>220 m E-W wide pit floor at RL 1260 m</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 2: Isometric view of the 3D elastic model including the mining stages considered for the stress analysis and four key observation points within the decline (4, 5, 6 and 7)

Modelling using FLAC3D software (Itasca) was conducted assuming an elastic behaviour of the rock mass and without the incorporation of the tunnel excavation effects. The main purpose of this modelling was to assess the stress field around the tunnel location as the pit is mined down from 350 m above the tunnel to just 75 m above the tunnel, taking into account the rugged topography of the mine. Figure 2 presents an isometric view of a cross-section of the model developed, showing the mining stages considered. Four key observation points (4, 5, 6 and 7) were defined in the model to assess the stress changes during the mining stages.

The 3D modelling allowed for evaluation of the current conditions in the tunnel and provided a basis for the selection of possible locations for a new bypass tunnel, should this alternative be selected as the most advantageous strategy to ensure the continued drainage of the pit.
Figure 3 illustrates the principal stresses at point 4 (as an example) and their changes with the analysed mining stages for the three *in situ* stress regimes initially assessed. It is important to note the rotation of the principal stresses during mining stages and at the last stage, where $\sigma^3$ tends to have a vertical orientation.

The behaviour of the rock mass can be predicted from the comparison of elastic stresses and the Hoek-Brown failure criteria curve for each field stress regime at the points 4, 5, 6 and 7 for each of the mining stages, as plotted in Figure 4. As the field stress changes through the mining sequence, the failure curves for the main material types within the tunnel are more closely approached or exceeded (predicting yielding of the rock mass).

It is apparent that the WASM stress field may be overestimating the stress conditions at the time of tunnel construction. As the elastic analysis did not include a physical tunnel in the model, the incorporation of this will significantly reduce the minor principal stress from that shown in Figure 4, causing yielding around the tunnel. This has not been observed in the tunnel to date, so the stress regime reported by Mills and Lee were considered to be more representative of the actual current conditions and were used as the input stress regimes for the 2D elasto-plastic analyses of tunnel excavation and support system. These analyses were carried out using Phase2 software (Rocscience).
Tunnel stability and support system assessment

The extra ground support required due to stress changes imposed by the open pit workings as they are mined down to final pit depth have been estimated on the basis of the changes in the stress reduction factor (SRF) in the Q Index. In some tunnel areas, the applicable SRF increases from five when initially constructed to about 50 after stress changes due to mining in the open pit. This means that the mapping-derived Q indices will be divided by ten. However, the final design must consider the geometry of potentially unstable wedges that can be isolated by the geological defects in the side walls of tunnels. Stress changes around the tunnel due to open pit floor mined down to within 75 m of tunnel crown resulted in the Q-index SRF increasing from five to >50 (ten-fold) because of the in situ stress versus UCS relationship, impacting on the category of ground support required.

An elasto-plastic model was considered necessary for the tunnel stability and support system assessment. For this purpose, a 2D plain strain model was developed incorporating the mining stages by reproducing the corresponding stress fields obtained from the elastic model (see Figure 3) by means of applying boundary displacements, which were back-calculated from the stresses using Hooke’s Law for plain strain conditions. The observed conditions within the tunnel after construction and up to the current pit stage seem possible within both the Mills and Lee field stress regime models; although the former results in a more developed yielded zone around the tunnel and a larger demand on the support system, with some bolts predicted to have failed in tension (see Figure 5). A detailed comparison of these results with the actual observed conditions of bolts and rock mass in the tunnel would be required in order to judge which field stress model would be more appropriate to simulate the geomechanical behaviour.
Kinematic failure analysis was carried out in order to assess the likelihood of occurrence of structurally-controlled failures and the effectiveness of the support system installed to prevent this type of failure. The analysis was completed with Unwedge software (Rocscience) using the available database of rock discontinuities obtained from previous mapping in the drainage tunnel. Two analyses were performed; one with and one without the in situ stress effect.

The results of the kinematic analysis indicate a low likelihood of occurrence of wedge failures when the confining effect of field stress is included (Figure 6a). It was assumed that very small wedges of depth of failure less than 1 m will fail/be removed during tunnel construction. It was also considered that wedges of failure depth greater than 4 m will be very unlikely to fail as they will have greater confinement, and greater friction and/or cohesion along joint planes. With the confining effect of field stresses, it is predicted that no wedges within this size range will have factors of safety (FoS) less than one.

An increase in the likelihood of wedge failure is apparent when the effect of the field stresses are not considered (Figure 6b). The Mills model results in a more developed yielded zone around the tunnel, reflecting the condition of no field stress effect in the kinematic analysis. A larger confining stress effect and therefore a reduced likelihood of wedge failures would be expected under the Lee field stress model. Therefore, a detailed observation of actual conditions of the tunnel in terms of occurrence of structurally-
controlled failures would provide an additional criterion to judge the validity of the field stress models used in the analysis.

Figure 6 – Kinematic analysis: (A) with field stresses; (B) without the effect of field stresses.

Rock stress input for West Wall Stability Modelling

Distinct element analyses using UDEC software (Itasca) were carried for two key sections (the central and south sections) within the West Wall to assess the stability of the proposed cutback design. These analyses were detailed and comprehensive, with investigation of the effects of: structural fabric; a blast and unloading disturbance zone; a target groundwater pushback (depressurisation) to 250 m behind the proposed cutback face; and appropriate seismic loading (pseudo-static analysis). The sets of analyses were carried out incorporating both the Mills and Lee alternative interpretations for in situ stress conditions, as it was decided that only these two regimes were considered representative for the West Wall rock mass.

Within the UDEC sections, the excavation from premining topography to the final cutback slope was simulated using four excavation stages with appropriate groundwater levels assumed for each excavation stage. These stages were defined to better simulate the stress path, whilst balancing the efficiency of the model run. The performance of the final proposed mining excavations were analysed. The stability of the West Wall was assessed in terms of FoS for slope failure, as interpreted using the strength reduction factor (SRF) technique. In this technique, the shear strengths of each material type were uniformly reduced by the same factor, which was progressively increased until initiation of failure.

The summarised results for the initial UDEC stability analyses are listed in Table 4. The interpreted FoS values have been based on the critical SRF at failure, as interpreted from displacement history plots, displacement magnitude and velocity plots, and plasticity indicator plots (showing zones that are yielding/have yielded in shear and in tension). The overall slope failure mechanism is illustrated in Figure 7. Several distinct zones of shallow (inter-ramp) failure become evident in the sections with progressive strength reduction, which are especially pronounced under seismic loading. FoS for failure of the overall slope, and the lowest FoS (earliest development of failure) for the many inter-ramp failure mechanisms identified, are listed in Table 3. Plots for the South section comparing the major principal stress (at left) and minor principal stress (at right) at the critical SRF of 1.35 for the Mills and Lee stress regimes are presented in Figure 8.

Table 4: Summary of UDEC analysis results (factors of safety)

<table>
<thead>
<tr>
<th>Section</th>
<th>Stress regime</th>
<th>Seismic loading</th>
<th>Overall slope failure</th>
<th>Earliest inter-ramp failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Central</td>
<td>Lee</td>
<td>No</td>
<td>1.15–1.20</td>
<td>~1.00</td>
</tr>
<tr>
<td></td>
<td>Yes</td>
<td></td>
<td>~1.00</td>
<td>&lt;1.00</td>
</tr>
<tr>
<td></td>
<td>Mills</td>
<td>No</td>
<td>~1.15</td>
<td>~1.00</td>
</tr>
<tr>
<td></td>
<td>Yes</td>
<td></td>
<td>&lt;1.00</td>
<td>&lt;1.00</td>
</tr>
<tr>
<td>South</td>
<td>Lee</td>
<td>No</td>
<td>1.30–1.35</td>
<td>~1.05</td>
</tr>
<tr>
<td></td>
<td>Yes</td>
<td></td>
<td>~1.25</td>
<td>&lt;1.0</td>
</tr>
<tr>
<td></td>
<td>Mills</td>
<td>No</td>
<td>1.35–1.40</td>
<td>1.0–1.05</td>
</tr>
<tr>
<td></td>
<td>Yes</td>
<td></td>
<td>1.20–1.25</td>
<td>&lt;1.0</td>
</tr>
</tbody>
</table>
Figure 7: Plot of plasticity indicators showing the failure mechanism occurring with the Central Section under static conditions (SRF = 1.15)

Figure 8: Comparison of major principal stress (left) and minor principal stress (right) at critical SRF of 1.35 for: (A) Mills stress regime; and (B) Lee stress regime
It can be seen from these results that for this particular large open pit slope the different estimated premining stress regimes have little effect on the modelled stability of the slope, generally differing by only 0.05 or less in terms of interpreted FoS (and with similar patterns of strain development and relaxation) for a range of scenarios at both overall slope and inter-ramp scale. This was further validated by indicative supplementary stability analyses carried out using Phase\textsuperscript{2} software that included the WASM stress regime as well as several other ‘randomly selected’ stress fields with significant variation. The results of these analyses indicated only a 0.03 difference in interpreted FoS values across all assessed stress regimes, providing further support to the conclusion that the in situ stress regime has little effect on the stability of the large open pit slopes at this site. It is acknowledged that such a conclusion would not necessarily be valid for others sites, where a combination of different rock mass conditions, pit geometries and different principal stress orientations relative to the pit walls may present a different outcome.

**Rock stress in Gold Coast Stability Modelling**

The rock mass behaviour of the Gold Coast region during proposed sublevel open stoping activities was modelled to predict its influence on the stability of the open pit wall. FLAC3D software (Itasca) was used to provide appropriate modelling of the detailed topography and complex geology, and to simulate the mining sequence taking place from 2013 through 2023. This involved evaluation of: the declines and access development; the stopes and pillars; the open pit East Wall; and underground and open pit interaction. Due to the complexity of the study, the interpretation of the results was focused first on each element as an independent entity to evaluate its own performance. Once the behaviour of each element was understood, the interaction between them was analysed (underground/open pit interaction).

The approach assessed the stress behaviour of the rock mass using the Hoek-Brown failure criterion, adjusted for estimating the non-linear confinement-strength relationship of each geotechnical unit, including the influence of the blast damage and stress relaxation induced by mining activity simulated in the model. This aspect was important when it comes to including the rock mass response to underground mining, which has been previously affected by induced stress and blasting due to surface mining. The methodology used in the model construction was developed as follows:

- Construct the initial model, and define the material properties based on Hoek-Brown failure criteria
- Estimate the current stress conditions at Gold Coast, considering the initial topography and mining stages, until the current condition is reached
- Include the mining sequence of the Gold Coast pit, considering the open pit and underground activities
- Consider the rock mass response to underground mining of material that has been previously affected by induced stress and blasting due to surface mining (illustrated in Figure 9, which presents a schematic representation of the approach taken in the model); this is taken into account by increasing the factor of disturbance that has been defined using Hoek-Brown from 0.5–1.

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![Figure 9: Failure criteria considered for rock mass affected by mining activity. NB: (*) – reduction in material properties due to blasting damage of the rock mass and stress relaxation due to overburden removal, caused by underground and open pit activity](image)
Based on the modelling work undertaken for both the drainage tunnel and the West Wall, in which multiple stress regimes were used during the modelling, the Mills in situ stress regime was considered most appropriate for the model. It generally produced only a marginally lower FoS than the Lee regime during the West Wall slope stability analyses, however the larger ratio of major to minor principal stresses produced a significantly larger region of predicted rock mass yielding around the drainage tunnel. Therefore, in light of the uncertainty as to the most representative stress regime, it was considered expedient that the more conservative (‘worse case’) Mills regime should be used as the input for further analyses. Geotechnical domains B, C and D are present in the Gold Coast area, and Hoek-Brown failure criterion curves were identified for each of these materials.

**Stopes and pillars**

The yielding zones predicted for the stopes and pillars coincided initially with the weak material zones (Domain D). It was expected that approximately 54 per cent of the stopes will be excavated in good quality rock; 26 per cent in fair quality rock; and, 20 per cent in poor rock mass conditions. Figure 10 presents an example cross-section of the rock mass domains in which the underground mining will be situated. Stopes located in weak rock (Domain D) require further detailed evaluation of the stress environment and expected rock mass behaviour before excavation.

Figure 10: An example cross-section through the stopes to illustrate the initial rock mass conditions: (A) isometric view with the example cross-section; (B) vertical cross-section (strike 315°) – planned stopes and rock mass domains

Based on the geotechnical domains, the mining sequence and the predicted stress behaviour, an assessment of developing rock mass conditions for each year of underground mining was carried out. The projected stoping for each year was superimposed on the predicted pattern of rock mass behaviour/conditions (as defined by the FLAC3D plasticity indicators) following the previous year’s activities. The results for each year were presented in a series of sections, an example of which is provided for year 18 (see Figure 11). Figure 12 provides a summary of the proportions of good, fair and poor rock mass conditions in which stope development will occur and distribution of materials with regards to failure criteria.
In year 18 approximately 63 per cent of the stopes are excavated in Domain B; 25 per cent in Domain C and 12 per cent in Domain D (see Figure 12a). From the total, 54 per cent of the rock mass would be under unfavourable stress conditions (yielding in the present); and, 23 per cent would have experienced increased stress levels (yielded) in the past (see Figure 12b). Figure 12c provides a graphical representation of the failure criterion and stress distribution observed for each material, showing that the critical zones are related with the poor rock mass (Domain D) conditions and in pillars between excavated and backfilled stopes, which have to withstand the stress redistribution resulting from the confinement reduction of the rock mass.

Based on what it is observed for each year of underground mining, the behaviour of the rock mass in the early stages is controlled mainly by its original quality, as defined by the geotechnical domains. The planned mining sequence is generally from outside towards the centre, so as the underground mining progresses the sequence of extraction tends to overload the pillars towards the centre, leading to yielding of the rock mass towards the end of mining.
Figure 12: Rock mass at year 2018 before being excavated: (A) original distribution of rock mass quality (DOM); (B) yielded rock mass distribution at time of stopes excavation; (C) failure criteria

Open pit – East Wall
Modelling has not considered open stopes during the underground mining sequence (backfilling is effectively assumed to be immediate). Therefore, only the mining-induced rock mass response was determined and not the potential for caving. Under this model, the East Wall does not change in shape, keeping the same overall slope angle throughout the modelling sequence. The performance of the East Wall rock mass during excavation has been assessed at each year by assessment of plasticity indicators and shear strain increment for a number of selected sections. This analysis indicates the overall slope behaviour to be stable.

Open pit/underground mining interaction
Based on the points discussed above, the planned mining activity has two significant years: year 2016 when an interaction between underground and open pit is first observed; and year 2018, when the central pillars and the surrounding rock mass start to be affected by the combined mining activity (surface and underground). Due to the surface mining and overburden reduction, there is a relaxation of the stresses on the surface of the pit, generating yielding of the rock mass (rebound effect).

At year 2016, the rock mass affected by the pit mining, begins to coalesce with that affected by the underground mining. As expected, a higher open pit/underground interaction is observed during the later
stages of mining. At year 2018 the surrounding rock mass is affected by both mining operations, particularly due to the pillars in the centre of the underground mining being reduced, increasing their loading. This interaction was earlier presented graphically, in Figure 12. This analysis indicates the overall slope behaviour to be stable. It was observed that the underground mining would affect the stack with a berm height at 1468 m in year 2018, which is predicted to start to yield when it is excavated. This is shown in Figure 13, which presents a cross-section showing the yield zones and the materials that are affected by the mining activity in year 2018. This behaviour seems to be related with the poor rock mass (Domain D) conditions at that location, combined with the underground mining activities.

Figure 13 – Cross-section (strike 315°) – open pit and underground mining interaction at year 2018: (A) plasticity indicator; (B) rock mass units.

Summary

In situ stress measurements are by their nature point measurements of the in situ stress at the point of measurement. Some variation across the rock mass is expected. Measurements at different locations, depths, rock types and/or determined by different techniques yield different estimates of the in situ stress under this range of conditions. Ok Tedi Mine is no exception. The hydro-fracturing method inferred a hydrostatic model, and acoustic emissions a highly anisotropic horizontal stress regime, whereas overcoring measurements using ANZI strain cells inferred a rock-type dependent but more benign intermediate stress regime. Principal stress directions are not necessarily horizontal and vertical; perhaps impacted by rugged topography. Lastly, the stress model adopted for mine design evolves with time as new data becomes available. There is always an element of judgement involved with the extrapolation from point measurements of the in situ stress to a representative in situ stress field.

This paper has provided an insight into how various in situ stress regime interpretations were used to model the proposed new open pit and underground excavations at Ok Tedi Mine. Even if the premining in situ stress regime was uniquely consistent, considerable redistribution of stress directions and magnitudes are expected to develop due to interaction between surface open pit and underground excavations. Numerical modelling is a multistage approach; the analyst strives to interpret and input the most appropriate ‘starting’ stress conditions in the specific mine area being investigated. Even then, the outcome is only as good as the model assumptions reflect rock mass reality.

This paper provides comparisons between the effects of various interpretations of the in situ stress field on three specific mine components in the context of the continuing and progressively enlarging and deepening open pit operations. The three components include a below-pit-floor drainage tunnel, a 1000 m high slope cutback of the West Wall and extraction of the ore in the SE Wall by a combination of open pit and underground stoping methods.

Numerical modelling has been used to evaluate (in terms of stress-strain response) the stability and long-term ground support requirements for the drainage tunnel, to assess the stability and sequencing of surface and underground excavations for the sublevel opening stoping operation and to evaluate the progressive dewatering (via drainage gallery and/or horizontal drainage holes) and stability of the very high West Wall at each stage of its proposed cutback.

Numerical modelling results demonstrate that response and stability of some high open pit slopes is not especially sensitive to the adopted premining stress regime. Conversely, stress redistribution and concentration between surface and underground mine workings are very dependent on initial stress assumptions and ground extraction sequence. The underground stability consequences of misinterpreted
initial *in situ* stress regime are significant. From the studies carried out at Ok Tedi, it is generally indicated that the starting stress regime impact on the FoS stability outcome for Ok Tedi pit slopes is in the order of 3–5 per cent, compared with a potential 100 per cent to 150 per cent or more increase in the near excavation concentrated stresses for the underground openings.

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