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Back Analysis and Closure Considerations for the North-East Pit Wall at the Globe Progress Gold Mine, Reefton, New Zealand

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ABSTRACT

In April 2015 a kinematic slope failure occurred on the north-east (NE) pit wall at the Globe Progress gold mine in Reefton, New Zealand. The failure involved planar sliding, toppling and translational slumping through the entire height (220 m) of the wall. A probabilistic back analysis using Rocscience's 2D limit equilibrium software, Slide, was undertaken to determine rock mass strength parameters that led to failure. Back analysis results were then carried into a post closure model, also using Slide. The post closure model examines how the stability of the NE pit wall will respond to increasing pit water levels as the mine is transitioned into a pit lake. The back analysis found that to replicate the April failure, the greywacke – argillite bedding planes required an apparent cohesion of 32 kPa and a friction angle of 28°. It is interpreted that these strength parameters are a result of a period of heavy rainfall prior to failure. Post closure modelling concluded an increase in pit water level is not expected to induce instability of the NE wall, given no other external conditions occur in conjunction with pit water rise.

Keywords: Slope stability, Kinematic failure, Back analysis

1 INTRODUCTION

The Globe Progress gold mine (mine) operated by OceanaGold is 6 km south of the Reefton Township on the South Islands West Coast (Figure 1). The mine area receives an average precipitation of 2,000 mm per year, which plays a role in decreasing stability of the pit walls (Kennedy 2009). Mining in the pit was completed in September 2015. At this time the pit was put into a care and maintenance (C&M) phase for two years, during this period the water in the pit will be kept approximately 30 m deep. Following this phase the pit will fill with 124 m of water.

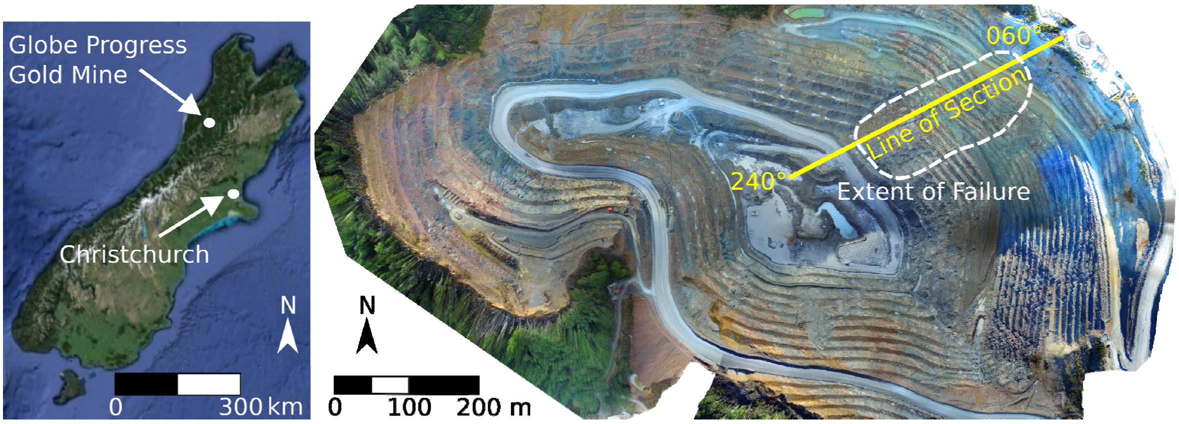

Figure 1. Mine location (adapted from Google Earth 2013) and mine pit. The Line of Section is used for stability modelling throughout this research (adapted from OceanaGold 2015)

This paper aims to describe a back analysis of the stability of the NE pit wall to determine the internal rock mass strength parameters that led to the April 2015 failure. The other aim of this paper is to analyse how the NE pit wall will respond to increased saturation as pit water levels rise following closure of the mine. Both the back analysis and the post closure analysis are important due to the potential for loss of life and damage to infrastructure in the event of a collapse of the pit wall.

1.1 Geological Setting

The geology of the Globe Progress pit is interbedded greywacke and argillite Greenland Group sediments. The argillite beds may be as thin as 30 mm (Clark 1996) with the greywacke beds ranging from 5 m - 10 m thick (Wells 2010). Two Shear Zones are also present; The Globe Progress Shear Zone runs north-west to south-east and the Oriental Shear Zone traces along the base of the NE wall from north to south. The NE pit wall is dominated by an anticline – syncline fold sequence (Figure 2). Descriptions of the geological units on the NE pit wall characterised using New Zealand Geotechnical Society (NZGS, 2005) guidelines are given in Table 1.

Table 1. Engineering geological descriptions of the rock units on the NE pit wall

Rock Unit	Engineering Geological Description
Greywacke	Slightly weathered, blueish grey, fine – medium sand, massive, weakly metamorphosed, GREYWACKE, very strong, moderately thick, narrow, inclined, undulating smooth bedding, two joint sets (Greenland Group)
Argillite	Slightly weathered, dark grey, fine grained, massive, ARGILLITE, strong, thin, very narrow, inclined, undulating smooth bedding (Greenland Group)
Oriental Shear Zone	Sandy SILT with minor fine gravel and trace clay, dark blueish grey, massive, soft, moist, non-plastic, (sheared and hydrothermally altered Greenland Group)

1.2 Kinematic Failure

On 24 April 2015 a significant slope failure occurred on the NE wall of the pit, following two months of heavy rain. A schematic engineering geological model of the wall was developed (based on scanline structural data and mapping), that illustrated the April 2015 failure was structurally controlled (Figure 2). Beds dipping into the pit on the western limb of the anticline resulted in planar sliding from the lower parts of the slope. This induced toppling on the eastern limb of the anticline where the beds above the sliding failure are dipping out of the pit. Translational slumping was also initiated at the top of the pit wall due to relaxation of the beds below. During the failure OceanaGold staff reported hearing cable bolts snapping, indicating the failure plane was within the cable bolts 15 m deep operational zone (PSM 2015). The 90 m wide failure is bound on the northern side by the Oriental Shear Zone and on the southern side by an unknown structural feature.

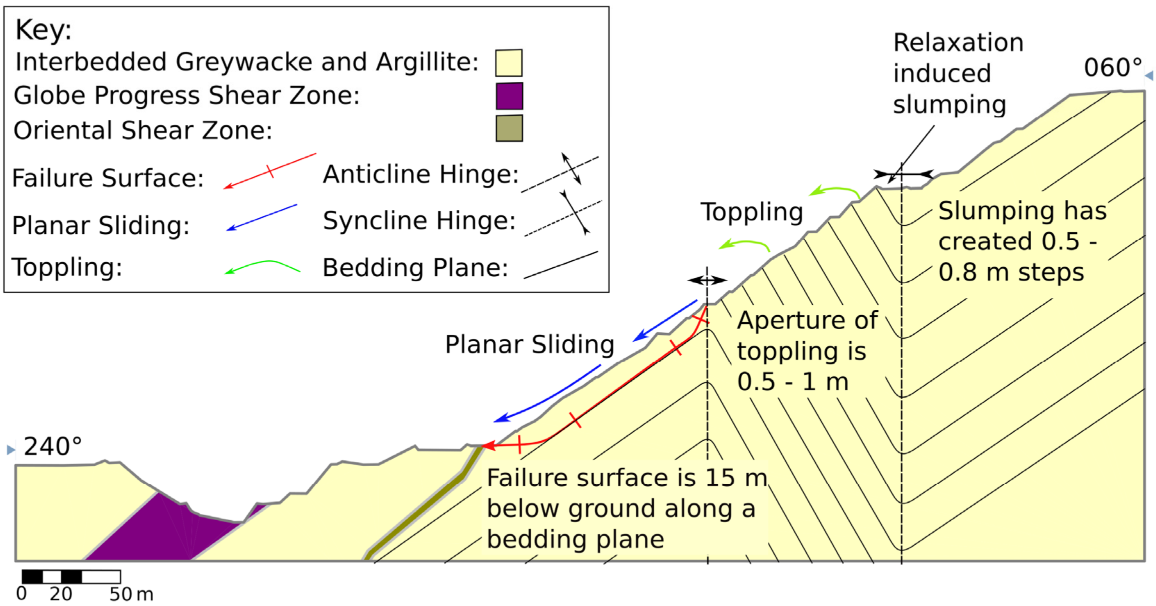

Figure 2. Schematic engineering geological model through the NE pit wall

2 STABILITY ANALYSIS

2.1 Rock Strength Quantification

Laboratory testing was used to identify rock strength parameters of the different geological units within the pit. The units were analysed with various failure criteria based on the best testing methods available for the rock type. Tables 2 to 4 summarise the strengths derived from the laboratory testing and the failure criterion used.

Table 2. Rock strength parameters using Mohr-Coulomb failure criterion

Sample	Testing Method	Apparent Cohesion (kPa)	Friction Angle (°)
Bedding plane	Direct shear box	0	25
Oriental Shear Zone pug breccia	Direct shear box	6	30

Table 3. Rock strength parameters using the Generalized Hoek-Brown (Hoek, Carranza-Torres and Corkum 2002) failure criterion

Sample	Testing Method	UCS (MPa)	GSI	Mi	D
Greywacke	UCS	150	55	19	0
Argillite	Point load	31	55	6	0

Table 4. Rock strength parameters using the Barton-Bandis failure (Barton and Bandis 1990) criterion

Sample	Testing Method	JRC	JCS (MPa)	Residual Friction Angle (°)
Bedding plane	Schmidt hammer	7	38	25

The friction angle of the bedding plane (Table 2) was inputted into the Barton-Bandis failure criterion along with Joint Roughness Coefficient (JRC) derived in the field and Joint Compressive Strength (JCS) taken from Schmidt Hammer testing (Table 4). This calculated the apparent cohesion and friction angle of the bedding plane to be 90 kPa (standard deviation = 35 kPa) and 30° (standard deviation = 2°), respectively.

2.2 Back Analysis

The back analysis model (Figure 3) was established in slope stability software using topography obtained from a Digital Terrain Model (DTM) of the NE pit wall taken in March 2015. The argillite bedding failure surface (enlarged section of Figure 3) was set as a 200 mm thick zone 15 m below ground, using structural data (Figure 2). The water table was allowed to vary between 2 m and 7 m below ground (OceanaGold 2013). The rock strength parameters used in the model were taken from Tables 2 – 4 apart from the bedding plane data. The bedding plane parameters were reduced from the laboratory obtained results to the lower bound of one standard deviation. This was due to the model not corresponding to the observed failure with the laboratory tested rock strength parameters. The reduced bedding plane mean cohesion and friction angle used were 50 kPa and 25°, respectively.

The back analysis focuses on the failure plane along the argillite bedding. It is inferred that following heavy rainfall, water would have infiltrated the bedding planes along joints through the rock mass. The model ran the analysis 1000 times within the range of cohesion (40 kPa – 110 kPa) and friction angle (15° – 36°) bedding plane parameters, determining which combinations of parameters would lead to failure of the pit wall within the defined constraints. The failure toe exit angle was chosen as 180° (from horizontal) to replicate what was observed in the pit.

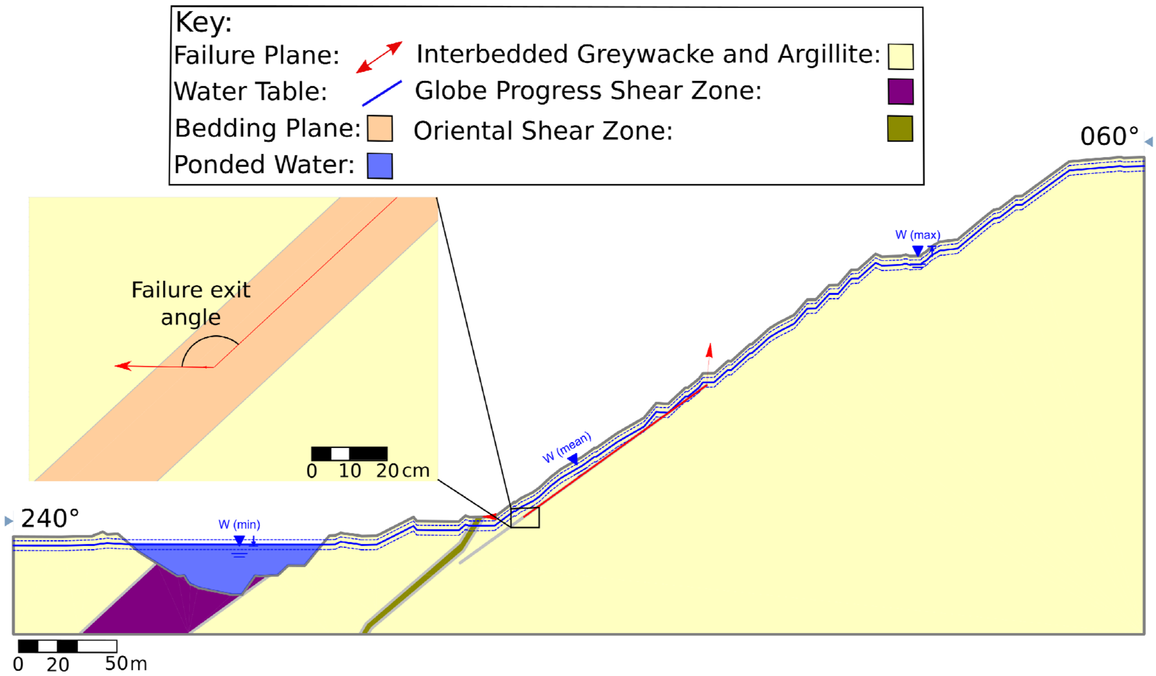

Figure 3. Stability model used to perform a back analysis of the kinematic failure on the NE wall. Enlarged section shows failure surface along the bedding plane interpreted to have failed

2.3 Post Closure Analysis

The post closure model (Figure 4) was produced from a June 2015 DTM. The area of planar sliding was inserted using a failure surface block search allowing the software to choose the most likely place of failure within the block. Failure was constrained to a dip of 36° along the bedding planes. The exit angle of the failure was set to choose the most likely angle between 170° and 250° from horizontal.

An increasing water depth to a maximum of 124 m was modelled. The models began with a water depth of 30 m, as this is the water depth during the C&M period. The rock strength parameters used in the post closure model were calculated from back analysis modelling (Table 5).

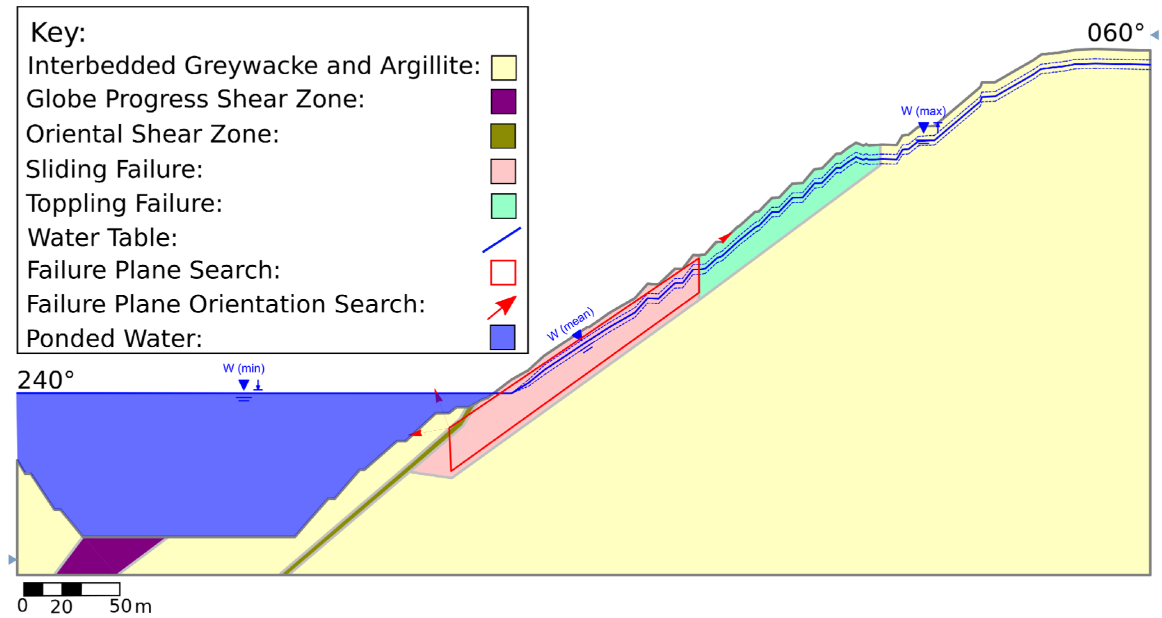

Figure 4. Representative model (75 m water depth) to analyse the post closure water table

3 BACK ANALYSIS

3.1 Results

From 1000 model iterations, 54 models resulted in a factor of safety of 1.0 and thus gave parameters for the April pit wall failure to occur. These results averaged to an apparent cohesion of 32 kPa and a friction angle of 28° along the bedding plane. The results plus the strength of the Oriental Shear Zone and the greywacke at failure are shown in Table 5.

Table 5. Rock strength parameters that resulted in failure of the NE pit wall derived from back analysis

Sample	Apparent Cohesion (kPa)	Friction Angle (°)
Bedding Plane	32	28
Oriental Shear Zone	6	30
Greywacke	2710	54

3.2 Discussion

The shear strength of the bedding plane established by the back analysis is weaker than was determined from laboratory testing. Intact shear and Schmidt hammer tests on the bedding plane found the apparent cohesion to be 90 kPa and friction angle to be 30°. Results of the back analysis indicate for failure to occur, the apparent cohesion is 32 kPa and friction angle is 28°. These parameters denote the average required strength of the bedding plane for planar failure to occur with the modelled pit wall geometry.

The results of laboratory testing were reduced in the stability model due to ground truthing. It was found that a probabilistic analysis with the initial laboratory tested strength parameters resulted in no failure of the wall. The rock strength parameters were therefore reduced to a mean cohesion and friction angle of 50 kPa and 25°, respectively to allow failure to occur as was observed in the mine. It can be inferred that heavy rainfall caused increases in pore pressure within the bedding planes. This combined with the unfavourable dip into the pit and the mass of the overlying rock, resulted in the planar sliding failure that then migrated up the pit wall inducing toppling and translational slumping.

Shear strength reduction modelling was undertaken using Rocscience's Phase^2 software for comparison; however being a finite element program neither structural data nor a failure surface could be input into the model. This resulted in a large-scale rotational failure occurring through the NE wall. Given the structure of the wall this was considered infeasible and the model was disregarded. This gives confidence in the structurally controlled stability model providing an accurate representation the observed failure in the mine.

The results of the back analysis are important because they can be carried forward to determine how the wall will respond to post closure water rise. Continued work would be to calibrate both laboratory tested and modelled results. This would require further laboratory testing on a range of samples from the NE pit wall as well as stability modelling in other software packages.

4 POST CLOSURE IMPLICATIONS

4.1 Results

The post closure model outputted eight factors of safety for the NE pit wall during the two year C&M period and subsequent filling of the pit (30 m – 124 m water depth). The different models effectively determine how the wall would respond over the filling period (Figure 5).

4.2 Discussions

The lowest factor of safety during filling of the lake is 1.2 at 75 m water depth, the factor of safety then increases as water level increases. This implies rising pit water levels are not expected to induce pit wall failure under the saturated conditions modelled, without other external factors. There are two possible reasons for failure not being induced as compared to the April 2015 failure. The first is the weight of water within the pit will buttress the pit wall. Figure 5 supports this by illustrating the pit wall is least stable when the water depth is 75 m, which corresponds to the pit water saturating the toe of the

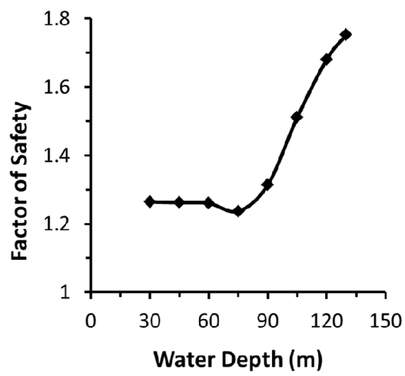

Figure 5. Plot of factor of safety versus water depth

slide only, with very little buttressing support force (Figure 4). As the water fills above 75 m the factor of safety increases due to a corresponding increase in buttressing force. The second possible reason for a stable post closure wall is the altered geometry. Since April 2015, due to both the failure that occurred and further mining within the pit the shape of the wall has changed favourably.

Additional work should examine other external conditions that could induce failure. Given the mines location, the main external condition to be researched is instability of the wall during seismic activity.

5 CONCLUSIONS

Back analysis effectively determined the rock strength parameters of the bedding plane that resulted in failure of the NE pit wall at the mine on 24 April 2015. It is inferred that the bedding planes dipping on limbs of an anticline within the NE pit wall experienced a loss of strength resulting in planar sliding into the pit. The loss of toe support then produced toppling through the mid – upper pit wall and relaxation induced translational slumping along the upper wall as illustrated by the engineering geological model. These results are important for understanding how prone to failure other sections of the pit wall with similar geomorphologies are and how they may react given comparable conditions.

Post closure models carried forward from the back analysis found that with no other external factors, pit water level rising is not expected to induce pit wall instability. This is important because of the potential negative consequences to both life and infrastructure in the event of further failure of the NE pit wall. Further research should be undertaken to better categorise the normal conditions rock strength and assess the effects of seismic loading on the pit wall using the parameters derived from the back analysis.

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