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Ground support for large chambers at Didipio mine

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Abstract

The Didipio porphyry gold–copper mine, Philippines, will be constructing two large underground chambers for the storage of water with a required service life of at least 15 years. The chambers will be located at a depth of 500 m below the surface in a blocky dark diorite rock mass having an in situ uniaxial compressive strength of 200–250 MPa and Q ratings ranging from 4–6. This paper summarises the results of analyses based on mining industry design methodologies to recommend optimal crown support and sidewall reinforcement.

The assessment and methodology satisfy industry design approaches which involved analytical and empirical methods. Analytical systems, such as the boundary element method numerical stress modelling, addressed the potential stress-induced failure analysis and calculated the size of overstress zones around the stopes. Wedge analysis examined structurally controlled failure due to mean joint sets. Empirical methods provided estimates of Q-rating and empirical design charts. Cable bolt patterns and specifications were based on a combination of wedge analyses, numerical modelling, and empirical design charts, using parabolic dome theory.

Keywords: *ground support, large chambers, water stope, long-term excavations, support capacity, support design, cable bolt, pressure arch*

1 Introduction

The Didipio mine is in the northern part of the Philippines. The porphyry–copper mineralisation is predominantly hosted by the Tunja monzonite, which intrudes the dark diorite (DKD). Capital development including the decline, accesses, vent shafts and pump chambers are developed in the DKD.

Capital Pump Station 1 (CPS1) with two water storage chambers will be constructed for desilting and pumping purposes. The vertical distance from existing Capital Pump Station 2 (CPS2), as shown in Figure 1, is approximately 120 m. Figure 2 indicates the section view of the two crowns of proposed water storage stopes.

A mixed design workflow was applied in the CPS1 ground support study since variation in the design approaches imply that there is no standard safety factor, nor absolute rules for design acceptability to guarantee a failure-proof rock structure (Hoek 2023). Hence, a combination of analytical and empirical analyses from different methodologies were considered in this paper.

Four design methods were applied to this study: (1) three-dimensional boundary element method (BEM) numerical stress modelling, (2) kinematic wedge analysis, (3) parabolic dome theory, and (4) empirical design charts.

The purpose of the paper is to outline the design workflow utilised and to recommend a crown ground support design that is expected to perform for the life of mine (LOM).

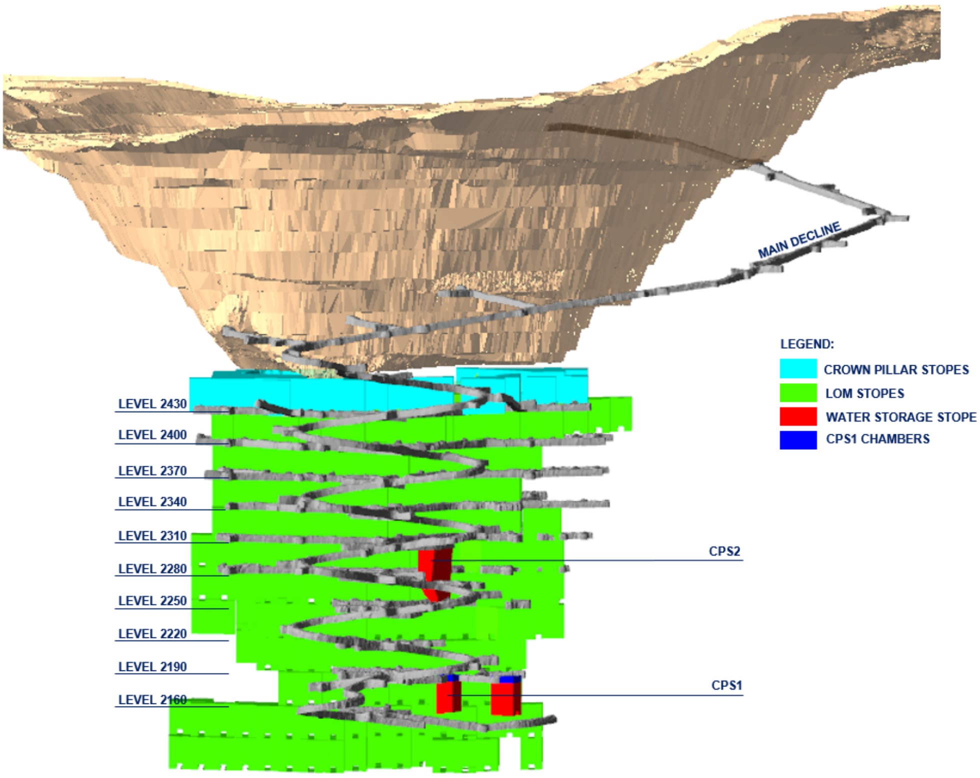

Figure 1 Auxiliary view showing the location of the proposed water storage chambers of CPS1

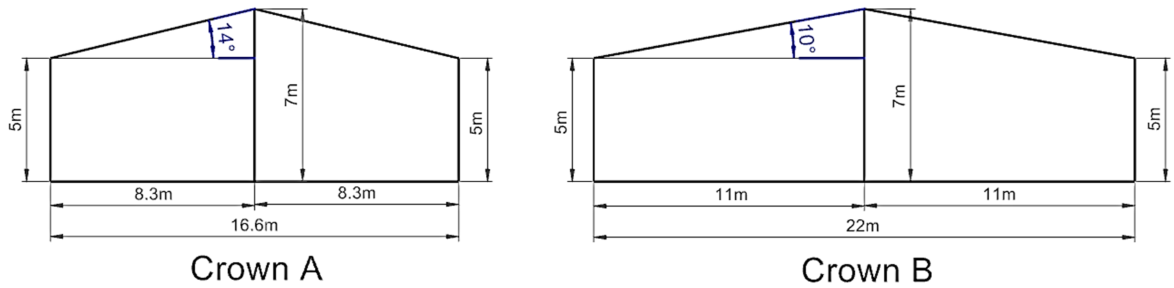

Figure 2 Section view through the crown of the proposed water storage stopes

2 Methodology

The design methodology presented in this paper involved analytical and empirical methods of geotechnical design. The following steps satisfy industry design guidelines such as the approaches of Olsson & Palmström (2014), Hoek et al. (1995), and Hoek (2023):

- Rock mass characterisation through an empirical rock mass classification system.
- Determination of stress-induced damage through the BEM numerical stress modelling.
- Evaluation of structurally controlled failure due to mean joint sets through wedge analysis.
- Ground support recommendation such as cable bolt pattern and specifications through hand calculation using parabolic dome theory.
- Empirical design charts.

Based on the design methodology chart of Olsson & Palmström (2014), the condition of the chambers falls under discontinuities in a competent rock which could induce block fall, loosening or cave-in as shown in

Figure 3. This system entails analytical methods (e.g. numerical modelling and wedge analysis) and empirical systems such as the Q-system and Barton design chart.

As shown in Figure 4 (Hoek 2023), the CPS1 chambers can be classified as large caverns in jointed rock. Stereographic projection and kinematic wedge techniques are used for the determination and visualisation of all potential wedges in a rock mass.

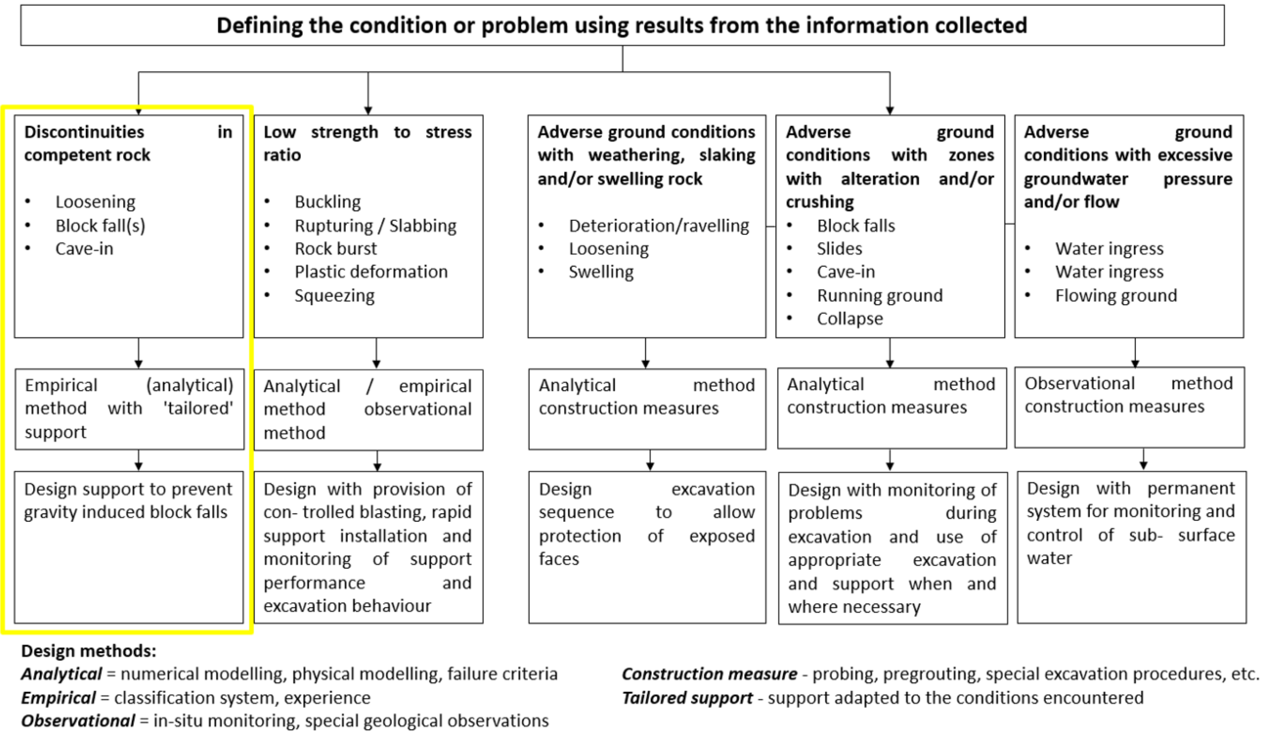

Figure 3 The plot of the study in 'Defining the condition or problem using results from the information collected' (Olsson & Palmström 2014)

Plot of the condition of the project in the "Typical problems, methods of analysis, critical parameters and acceptability criteria for underground excavations" (Hoek 2023)				
Large caverns jointed rock	Gravity driven falling or sliding wedges or tensile and shear failure of rock mass, depending upon spacing of structural features and magnitude of in situ stress	• Shape and orientation of cavern in relation to orientation, inclination and shear strength of structural features in the rock mass. • In situ stresses in the rock mass. • Excavation and support sequence and quality of drilling and blasting.	Spherical projection techniques or analytical methods are used for the determination and visualization of all potential wedges in the rock mass. Stresses and displacements induced by each stage of cavern excavation are determined by numerical analyses and are used to estimate support requirements for the cavern roof and walls.	An acceptable design is achieved when numerical models indicate that the extent of failure has been controlled by installed support, that the support is not overstressed and that the displacements in the rock mass stabilise. Monitoring of displacements is essential to confirm design predictions.

Figure 4 The plot of the study in 'Typical problems, methods of analysis, critical parameters and acceptability criteria for underground excavations' (Hoek 2023)

The workflow of this study is based on the systems of Olsson & Palmström (2014) and Hoek (2023), as mentioned. The methodology of *Support of Underground Excavations in Hard Rock* (Hoek et al. 1995), as shown in Figure 5, is arguably the more rigorous design process for underground geotechnical design. The latter covers data collection and rock mass characterisation which were not captured in the previously mentioned approaches. Further, the Hoek et al. (1995) methodology includes both stress-induced and structurally controlled failures with detailed processes such as the assignment of rock mass properties and the assignment of shear strength to potential failure surfaces. The assessment of blast-induced damage, excavation monitoring, and installation quality control were also captured by this method, although will not form part of the scope of the study.

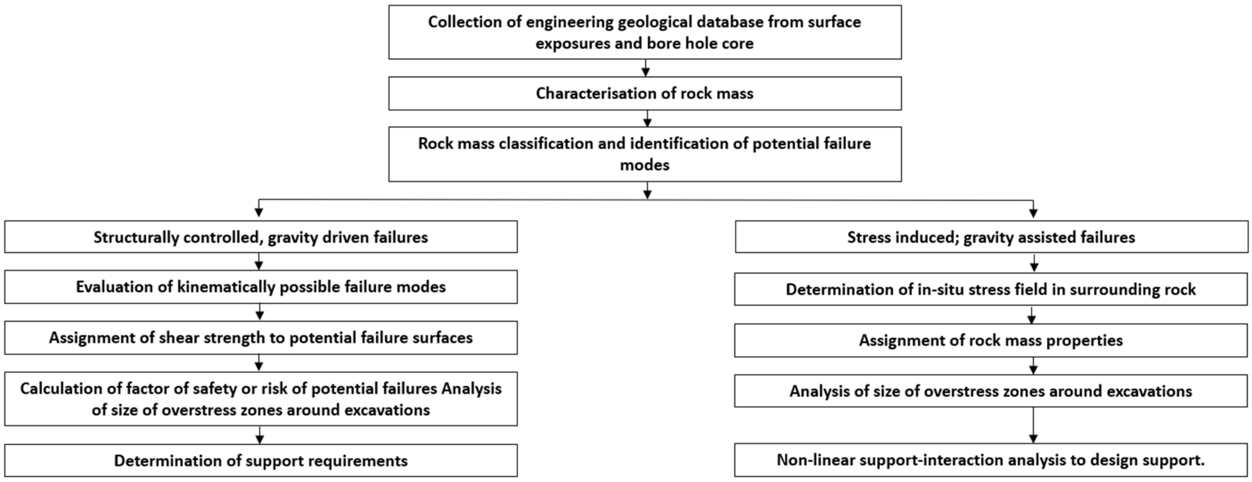

Figure 5 Design of underground excavations in hard rock (Hoek et al. 1995)

The compliance of the workflow to the design methodology (Hoek 2023) are as follows: geotechnical data collection was acquired through face inspections, scanline mapping, Didipio mine Q Block model and drillhole database. The rock mass characterisation was carried out using the Q-system (Norwegian Geotechnical Institute [NGI] 2022) which is an empirical rock mass classification system.

The extent of potential modes of failure were investigated through Map3D (Map3D International Ltd 2022) which is a BEM three-dimensional stress modelling software. This calculates the size of the overstress regions around the chambers which is in accordance with the design process to cover stress-induced and gravity-assisted failure. Meanwhile, the stereographic projection in Dips 8.0 (Rocscience 2023a) forms preliminary data to perform a wedge stability analysis through Unwedge (Rocscience 2023c). This step follows the structurally controlled and gravity-driven failure evaluation proposed by the workflow (Hoek et al. 1995). The software also satisfies the assignment of shear strength to potential failure surfaces and the Factor of Safety (FoS) computation as a design step.

The cable bolt patterns and specifications were further assessed through the parabolic dome theory hand calculation. This complies with the support design step suggested by the design process (Hoek et al. 1995).

3 Rock mass characterisation

3.1 Rock mass quality

The host rock mass of the chambers is DKD which is a grey-black, medium-grained, equigranular diorite to weakly plagioclase and clinopyroxene-phyric clinopyroxene-diorite (Wolfe & Cooke 2011). A considerable amount of testing of intact rock was undertaken to increase confidence in the results. DKD, as shown in Figure 6, has a median and average uniaxial compressive strength of 238 MPa and a first quartile UCS value of 211 MPa. Table 1 presents the average intact rock properties of DKD in the Didipio mine.

Table 1 Average intact rock properties of dark diorite in Didipio mine

Rock type	UCS (MPa)	Young's modulus (GPa)	Density (kg/m^3)	Poisson's ratio
Dark diorite	238	108	2,800	0.32

(a) (b)

Figure 6 Figures showing competent dark diorite in Didipio mine. (a) Photo of a drillcore demonstrating excellent rock quality designation; (b) Photo of active heading exposing dark diorite lithology

3.2 Rock mass characterisation

The rock quality tunnelling index, Q-system (NGI 2022), was applied to characterise the rock mass in compliance with the methodology (Hoek et al. 1995). Values for the parameters were acquired from the Q block model, face inspections, scanline mapping, and drillhole database. The data and Q calculations are shown in Table 2 which yielded a range and median Q ratings of 4.7–5.6 and 5.3, respectively.

Table 2 Q parameters of the water storage stopes based on geotechnical data

Wall	Lithology	RQD	Jn	Jr	Ja	Q’	Jw	SRF	Q	Ground type
Backs	DKD	90	12	1.5	2	2.8	1	1	5.6	Type 1
North	DKD	90	12	1.5	3	3.9	1	1	5.6	Type 1
East	DKD	85	12	1.5	3	3.8	1	1	5.3	Type 1
West	DKD	75	12	1.5	2	2.5	1	1.25	4.7	Type 1
South	DKD	85	12	1.5	2	4.7	1	1	5.3	Type 1

The initial Q calculations are consistent with the most recent geotechnical inspection as crown development progresses. Didipio mine uses three rock mass categories, Types 1–3, with Type 1 as the most favourable ground condition. This is described as a moderately strong rock mass with two to three well-developed joint sets. These structures are usually tight, and the ground generally remains intact; however, scats could occur.

4 Cavern stability assessment

4.1 Empirical rock mass quality and rock support chart

The assigned excavation support ratio (ESR) (NGI 2022) is 1.3 since the chambers are permanent mine openings. The shorter spans of the excavations, which are 16 m and 20 m, are divided by the ESR to attain the equivalent dimension (De). The median Q-rating and the De were plotted to the rock mass quality and rock support chart (NGI 2022) for empirical ground support recommendation.

Figure 7 shows the plot on the chart which recommends a ground support combination of systematic rockbolting, and at least 50 mm of fibre-reinforced sprayed concrete. A 3.4–3.8 m rockbolt length is computed through the following equation (Lang 1961):

$$L = (2 + 0.15B)/ESR \tag{1}$$

where:

L = required length of the rockbolt.

B = excavation width.

ESR = excavation support ratio as explained in the Q-system.

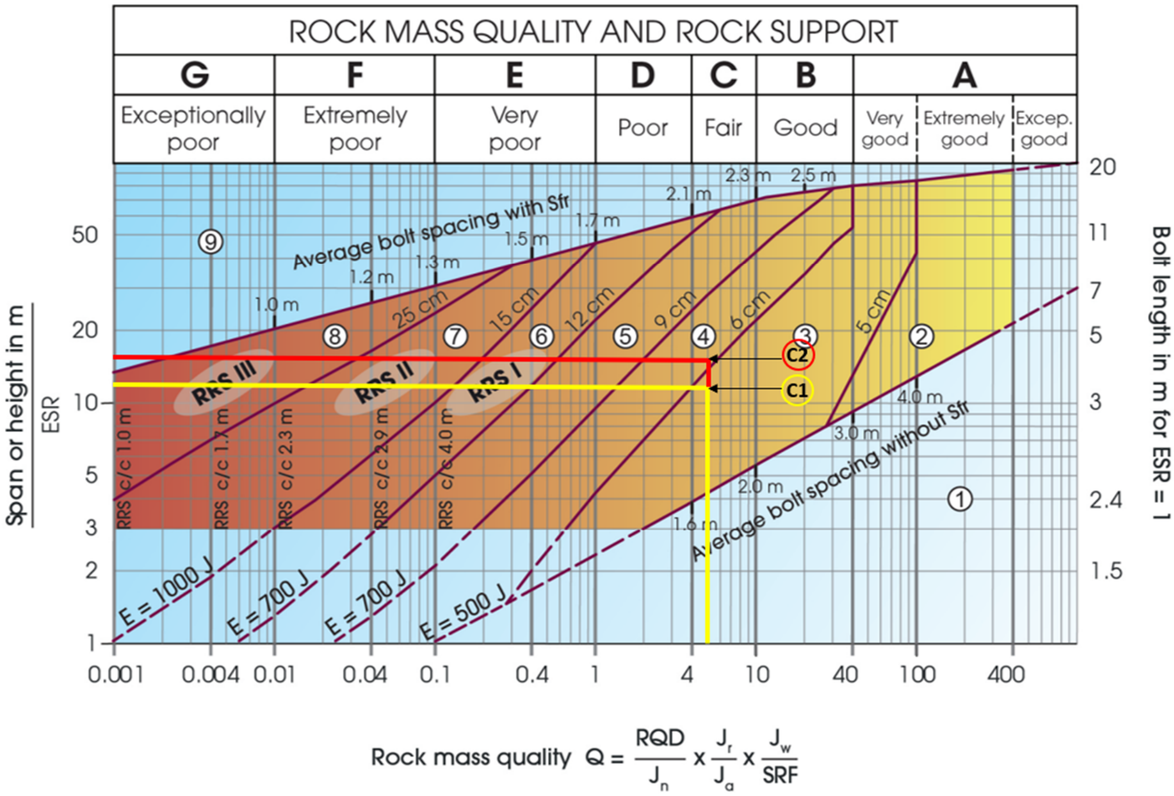

Figure 7 Figure showing the plot of the Q-rating and the equivalent dimension (De) of the chambers into the rock mass quality and rock support chart (NGI 2022)

The required length of the grouted cablebolts was 6.9–7.7 m, determined by the equation below and as shown in Figure 8 (Hutchinson & Diederichs 1996).

$$L = (0.7)\ \text{Span}^{(0.7)} + 2\ \text{m} \tag{2}$$

where:

L = required length to be grouted for grouted cablebolts.

Span = excavation span (shorter dimension) in metres.

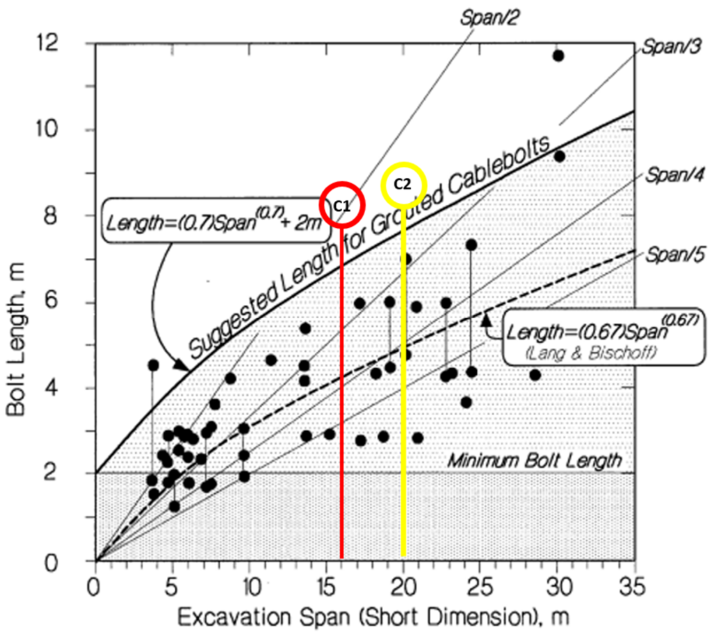

Figure 8 Figure showing the plot of the chambers in the bolt lengths in current practice chart (Hutchinson & Diederichs 1996)

4.2 Stereographic projection and kinematic analysis

The kinematic analysis and stereographic projection are a preliminary approximation of the kinematic potential of wedges susceptible to sliding. The data from scanline mapping and face inspections was used in the Dips software to visualise Stereonet plots. As shown in Figure 9, there is a probability of gravity-induced movement and wedge failure due to mean joint sets derived from the plot.

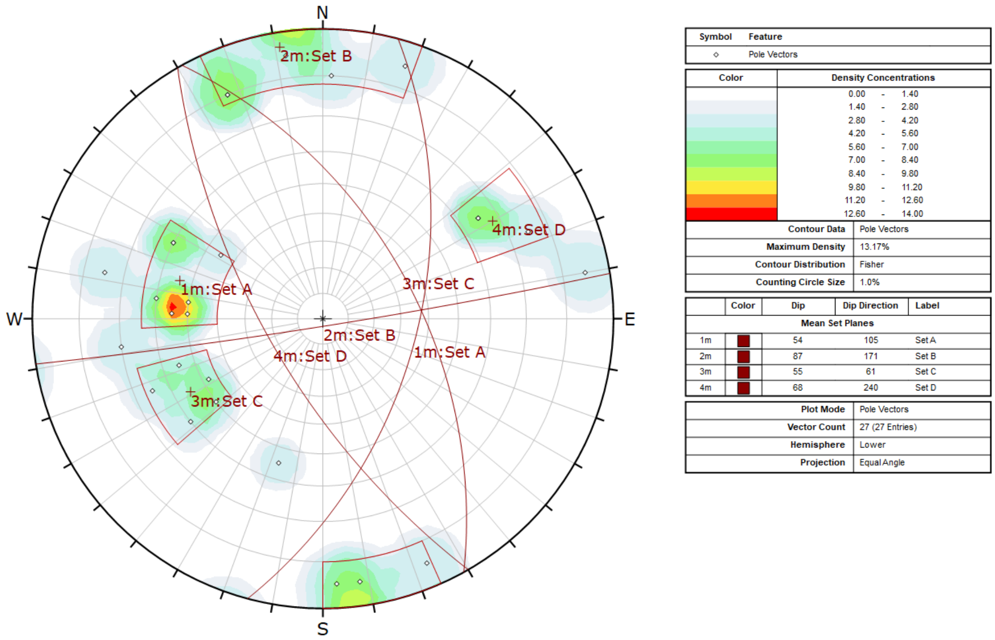

Figure 9 Stereographic projection of the mean joint sets obtained from the recent scanline mapping

4.3 Wedge stability analysis

The main instability problem in the underground mine that required attention is that of potential unstable blocks and wedges with interconnecting geotechnical structures (Hammett & Hoek 1981). This factor was considered in all steps except for the design step mentioned in the methodology. Table 3 shows the results of the wedge analysis carried out through Unwedge. The potential wedges that could slide were scaled following the span divided by 3 rule of thumb based on historical wedge data onsite. The software will always produce the maximum size of wedge; however, in practice, these wedges do not occur because the joint length of 3 plus random sets are not all infinitely long. In DKD, the infinitely long joint sets are the only ones corresponding to the major shears shown in the Didipio fault model. Moreover, in a 6 m wide DKD decline, wedge failures greater than 1.4 m deep in the unsupported area of a 4.3 m cut never occurred, whilst the 20 m (l) × 20 m (w) monzonite stopes have never had wedge failures greater than 4 m deep in the backs.

Structural measurements from geotechnical mapping and inspections near the area were performed and the data was placed into stereographic projection. As crown development commenced, the stereographic projection was updated to incorporate the structural measurements on the excavated ground. Four joint sets were established to update the wedge analysis. The friction angle based on back-analysis and laboratory test is 40^{o} and cohesion is at 23 kPa for joint properties which were simulated in supported and unsupported conditions. The supported condition was simulated through 2 × 2 m spacing cablebolts and 100 mm of shotcrete. The fibre-reinforced sprayed concrete was applied at 100 mm thickness which is more conservative than the 50 mm recommended by the Q-system since the excavation is considered non-entry where rehab is not possible.

Table 3 Wedge prediction with Factor of Safety

Chamber	Wedge (t)	Apex height (m)	Factor of Safety (unsupported)	Factor of Safety (supported)
C1	226	5.3	0	3.6
C2	358	6.7	0	3.2

4.4 Boundary element method numerical stress modelling

Numerical modelling was carried out to simulate stress-induced damages within the rock mass and the excavations surrounding the stopes. This is based on the design methodology of Hoek et al. (1995). This BEM model assumes that DKD is the host rock and monzonite is an intrusion with a major fault structure taken into consideration.

The determination of the in situ stress field of the host rock, which was measured from previous stress measurements, and the assignment of rock mass properties were performed as a requirement of the design workflow (Hoek et al. 1995). These are presented in Tables 4 and 5. The UCS used is the same as the lab value; however, the tensile strength, elastic modulus and Poisson's ratio are based on the overall rock mass value, downgraded using RocData (Rocscience 2023b). The downgrading is a function of the Q' or geological strength index value.

Table 4 Pre-mining stress state for BEM numerical stress modelling

Stress field	Principal stress	Stress gradient (MPa/m)	Dip (°)	Dip direction (°, UG grid)
S01 In situ stress field	σ_1	-0.0600	00	233
	σ_2	-0.0442	00	143
	σ_3	-0.0284	90	360

Table 5 Material properties of dark diorite for BEM numerical stress modelling

Material	UCS (MPa)	Young's modulus (MPa)	Poisson's ratio	Hoek–Brown, mb	Hoek–Brown, s	Tension cut-off (MPa)
Diorite type 1	238	20,551	0.30	6.02	0.01191	-0.471

The strength/stress (SF-A) plot shown in Figure 10 was used to determine the FoS of the rock mass. The red zones indicate potential stress-induced damage which could lead to failure once excavated. The yellow zones demonstrate regions where stress cracks could develop, whilst green zones show areas where stress-induced damage is improbable. This interpretation is based on back-analysis of stopes and development drives at Didipio.

No stress damage is expected at the backs when the stopes are opened without considering the adjacent excavations, as shown in Figure 10. However, the numerical stress modelling demonstrates up to 7 m of stress damage at the backs if LOM stopes are not paste filled.

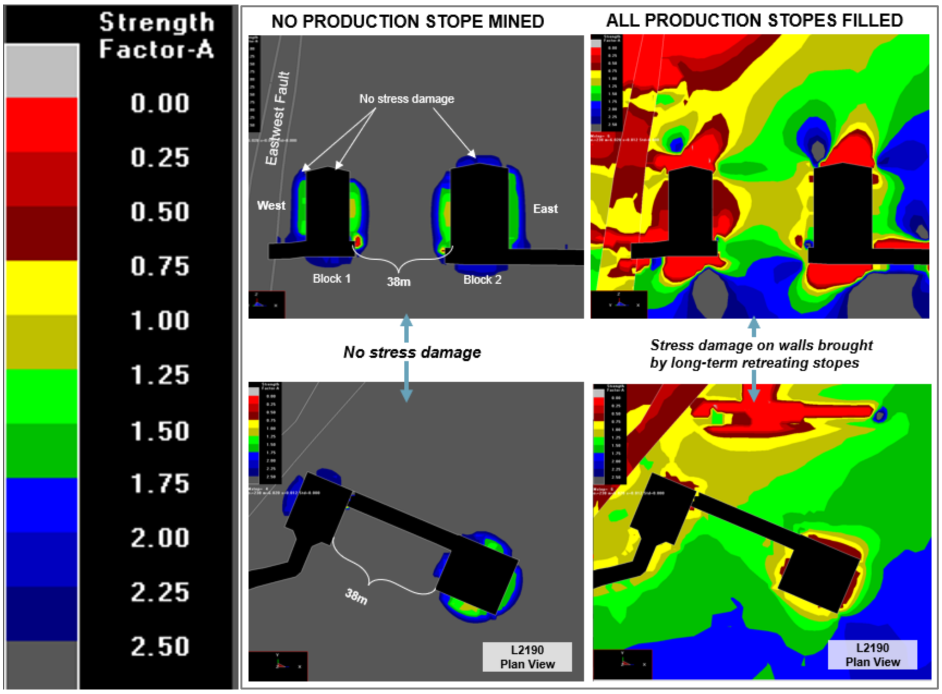

Figure 10 Result of BEM numerical modelling

4.5 Parabolic dome theory hand calculations

The cable bolt estimation was obtained by calculating the mass of the ellipsoid or dome (Hills et al. 2015) above the excavations. The mass of the rock above the chambers must be supported. The following equations further explain the concept whilst the calculation is summarised in Table 6.

$$V = \frac{2}{3}\pi abc \tag{3}$$

$$m = V\rho \tag{4}$$

where:

V = volume of the ellipsoid (hemisphere volume) expressed in m^3.

a = excavation width divided by 2.

b = excavation length divided by 2.

c = excavation span (shorter dimension) divided by 3.

m = mass expressed in tonnes.

ρ = density of DKD expressed in t/m^3.

Table 6 Mass calculation using the parabolic dome theory

Block	a	b	c	Volume (m^3)	Density (t/m^3)	Mass (t)
1	10	8	5.3	894	2.8	2,502
2	10	11	6.7	1,536	2.8	4,300

The capacity of the twin- and triple-strand cable bolts was assumed at 40 and 60 t, respectively. The number of cable bolts was estimated considering the calculated mass of the deadweight of the overlying rock mass derived from the parabolic dome theory, and the capacity of the rock mass to achieve an industry-standard 1.2 FoS. Additional cable bolts were added since it is essential to follow a systematic array to attain a natural pressure arch mechanism (Li 2017). Hence, this yields an FoS of 1.5 which exceeds the standard. The FoS calculation is presented in Table 7.

Table 7 Cable bolt requirements to support the overlying deadweight of the rock mass

Chamber	Mass (t)	FoS = 1 (twin)	FoS = 1.2 (twin)	FoS = 1 (triple)	FoS = 1.2 (triple)
1	2,502	63	76	42	51
2	4,300	108	130	72	87

Since there is a plan to install SMART cable bolts, which are only applicable to twin-strand, these will be installed in Chamber 1. Chamber 2 will have triple-strand cable bolts installed to reduce the number of cable bolts. A total of 163 pieces of galvanised cable bolt will be installed in the two chambers considering the deadweight of the rock mass.

4.6 Comparative evaluation of support recommendations

The length of the support should pass 1 m beyond the failure zone (Li 2017). Further, the critical embedment length testing in DKD shows that 1 m of embedment length will provide 25 t of pull resistance in a single-strand cable bolt. Compliance with this rule is presented in Table 8.

Table 8 Summary of support recommendation from analytical and empirical analyses and compliance to 1 m anchorage beyond the failure zone

Parameter	Analysis	Value	1 m anchorage beyond the failure zone
Apex height (C1)	Wedge stability	5.3 m	Actual design length = 7.3 m cable bolt
Apex height (C2)	Wedge stability	6.7 m	Actual design length = 8.3 m cable bolt
Stress damage (C1 and C2)	Stress modelling	0 m	Stable even without support
Stress damage (long-term), C1 and C2	Stress modelling	7 m	Actual design length = 7.3–8.3 m cable bolt
Length for grouted cable bolts (C1)	Empirical	6.9 m	Attained; More conservative approach
Length for grouted cable bolts (C2)	Empirical	7.7 m	Attained; More conservative approach
Ellipsoid height (C1)	Deterministic	5.3 m	Actual design length = 7.3 m cable bolt
Ellipsoid height (C2)	Deterministic	6.7 m	Actual design length = 8.3 m cable bolt

C1 = Chamber 1; C2 = Chamber 2

The 3.4–3.8 m long rockbolt recommendation of the Q-system would stabilise the crown development cuts; however, this does not compensate for the maximum wedge height and stress damage. The cable bolts could address this.

The fibrecrete and bolts recommended by the Q-system as a support system are efficient in minimising displacement and energy absorption capabilities. However, rock mass classification systems do not predict potential wedge sliding and fall-out. The wedge analysis addressed this concern.

The 7.3–8.3 m triple-strand cable bolts will anchor potential wedges. Further, the cable bolts yielded favourable FoS based on the wedge analysis. The excavations, without considering the stress flow from other mine openings, will not expect stress damage due to competent rock mass; however, the long-term stress model indicates up to 7 m stress damage at the backs. This will be compensated by the cable bolts.

The empirical chart developed by Hutchinson & Diederichs (1996) is a more conservative approach than the concept of Li (2017); however, this is only limited to the anchorage of wedges.

4.7 Sidewall reinforcement

The sidewalls of the upper chamber will be supported with galvanised twin-strand cablebolts. Meanwhile, the remaining portion of the sidewalls will not be supported with cable bolts due to lack of access and is considered non-entry. Blast controls such as low-density emulsion and rock mass grouting will be employed to limit damage and stabilise the chamber sidewalls.

5 Conclusion

The results of the analytical and empirical analyses show that the final support recommendation for the water storage chambers is the combination of 7.3 to 8.3 m long galvanised cable bolts, and 100 mm fibre-reinforced sprayed concrete yielding an overall FoS of 1.5 using parabolic dome theory and FoS of 3.2–3.6 derived from wedge analysis.

Ground support shall exceed at least 1 m beyond the thickness of the plastic zone to promote anchorage (Li 2017). Therefore, the 8.3 m long cable bolts are sufficient to compensate for the 7 m stress damage yielded by the numerical stress modelling. On the other hand, the 5.3–6.7 m maximum theoretical wedge and 2,502–4,300 t overlying deadweight of the rock mass will be supported by 163 pieces of twin- and triple-strand galvanised, plated and tensioned cable bolts. This ground support mobilises the rock mass residual strength by providing confinement and controls the inward displacement of the walls.

The 2 × 2 m cable bolt spacing derived from the Q-system will guarantee that the bolts will interact with each other and promote natural pressure arch mechanism (Li 2017). The 100 mm fibre-reinforced sprayed concrete will limit the displacement by serving as areal coverage and retention of blocks.

Acknowledgement

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