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Snowy 2.0 Down Stream Surge Shaft Chamber Geological/Geotechnical Hazards and Design Challenges

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ABSTRACT: The Snowy 2.0 Down Stream Surge Shaft Chamber (DSSC), with approximately 37 m in width, is one of the world's widest chambers ever built in hydropower projects, if not the widest. Once construction is completed, the chamber will provide sufficient underground space to accommodate the equipment for excavating the downstream surge shaft with a 28 m excavation diameter and 200 m depth using raise boring and shaft sinking methods. Considering the size of the DSS chamber, the geological and geotechnical conditions become even more prominent and critical to ensure the safe design and work environment during the shaft excavation considering the 150 years of project life during the operational phase. The detailed design of the DSS chamber was completed in 2024, followed by construction, which will be completed in the first quarter of 2025. This paper presents the geological and geotechnical hazards and associated risks identified during the ground investigation stage, the foreseen ground behaviours and the challenges the designers faced during the design stage, involving solutions and design decisions made by the Design Joint Venture (DJV). This paper also provides information on adopted construction sequences and methodologies, limitations, and preliminary design verification by the use of the observational method, which assesses geological and geomechanical conditions, geotechnical monitoring, and instrumentation data acquired during the construction phase.

1 INTRODUCTION

1.1 *General Layout*

The Snowy 2.0 Pumped Storage Power Plant (PSPP - Snowy 2.0) is a major pumped-hydro expansion of the existing Snowy Scheme, located in the Kosciuszko National Park (New South Wales, Australia). The project aims to increase the scheme's generation capacity by 2,200 MW and will connect the existing reservoirs impounded by the Tantangara and Talbingo dams, via approximately 27 km of power waterway tunnels and an 800 m underground power station.

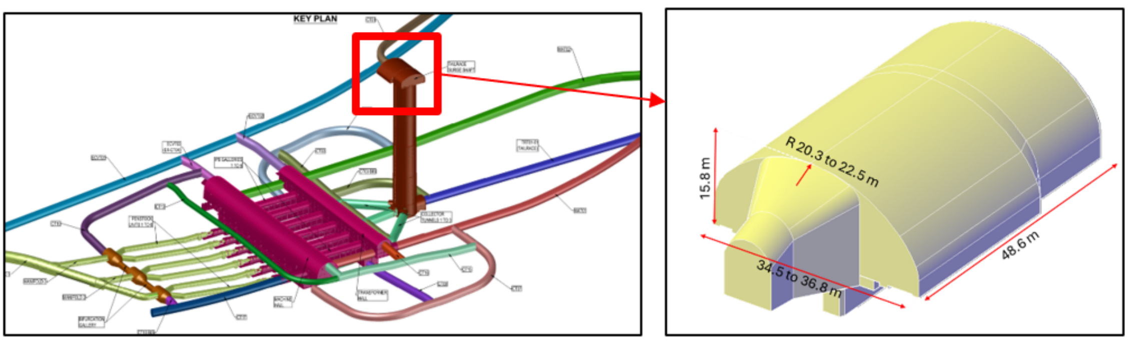


Figure 1. General layout of the Power Station Complex (PSC, left): Geometry of the upper chamber (right).

As part of the powerhouse complex, the downstream surge shaft is an equilibrium chimney with a chamber located at the top. The chamber is connected to the CT01connection tunnel via a transition zone and extends 48.6 meters in length. The cross-section is asymmetrical for the first 33.1 meters due to a lateral egress path, totalling 36.8 m in width. Beyond the egress path, the cross-section becomes symmetrical, with an overall width of 34.5 meters to accommodate the downstream surge shaft collar, excavation equipment and a gantry crane. The maximum height of the cavern is 15.8 meters for both asymmetrical and symmetrical sections (Fig.1). With its significant dimensions, the upper surge shaft chamber is the widest underground structure in the Snowy 2.0 project and one of the widest in the world.

1.2 *Geomechanical Inputs*

The DSS is entirely placed within the interbedded and interlayered siltstones and sandstones of the Ravine Beds West Formation (RBW). The rock mass is of good quality, with generally high RQD and GSI values. The initial geological and geotechnical assessment outcomes revealed the unlikelihood of certain ground conditions and, subsequently, ground behaviours and ground response to the drill and blast excavation associated with soft ground, groundwater, and high induced stress conditions. Therefore, the focus for the detailed design analyses remained on the discontinuity-driven and stress-related behaviour of the rock mass. Analyses revealed that there are no significant risks associated with the in-situ and induced stress behaviours, or with the influence of water.

The rock mass parameters and discontinuities used in the design were based on the evaluation of geotechnical investigations conducted in the area, including geological face mapping data obtained during the excavation of the CT01 inclined connection tunnel (Tables 1 and 3).

Table 1. Geomechanical properties assumed in the design

H [m]	K [-]	g [kN/m^3]	σ_{Ci} [MPa]	m_i [-]	D [-]	E_i [MPa]	GSI [-]	ν [-]	E_{rm} [MPa]	c [MPa]	φ [°]
540	1.7	27	55	7	0	45000	67	0.1	30327	2.288	39.0

2 DESIGN

2.1 *Excavation Sequence and Support*

Validated through the use of numerical models, the designed support system of the upper chamber primarily caters to discontinuity-driven behaviour and consists of primary and secondary arrangements. The design approach prioritises rock bolts as the primary element of the support system, providing sufficient and safe resistance loads against the possible formation of wedges and blocks. The chamber shotcrete lining was designed to serve two main functions: providing continuity and uniformity for the support system and securing minor wedges and blocks that may occur between rock bolts [10], thereby enhancing the overall shear resistance of the support system.

The chamber primary support system was designed to provide adequate and safe resistance to the likely probable wedges formed during multi-stage excavation. It includes 25 cm thick reinforced shotcrete (SFR40) with wire mesh (SL81) and 8 m long rock bolts (CT26WR) in a 1.90 x 1.90 m pattern.

The secondary support system was designed to provide long-term and safe conditions by supporting the probable largest wedges formed, considering the full span of the chamber. This includes an additional 15 cm thick reinforced shotcrete layer and wire mesh (SL81), as well as 11.8 m long rock bolts (CT26WR) in a 1.90 x 1.90 m pattern.

Due to the span of the chamber, the method designed for the surge shaft chamber excavation follows the multiple stages drill & blast approach. After each advance of excavation within several stages, the installation of the support system is to be completed as per the following sequence (Figure 2):

- Stage 1: CENTRAL TUNNEL: Excavation of the central top heading pilot tunnel and installation of the primary support;

- Stage 2: CENTRAL TUNNEL: Bench excavation of the central top heading floor by 2.5 m; installation of the secondary support, 11.80 m long CT-Bolts;
- Stage 3: EAST: Side slashing in 3.80 m advances and installation of the primary support, followed by secondary support. When completing 12 m of fully supported excavation at the east side, the west side slash excavation can be started (See figure 2.b);
- Stage 4: WEST: Excavation of the side slashing in 3.80 m advances and installation of primary support, followed by secondary support; and
- Stage 5: Excavation of the final bench and installation of primary and secondary support.

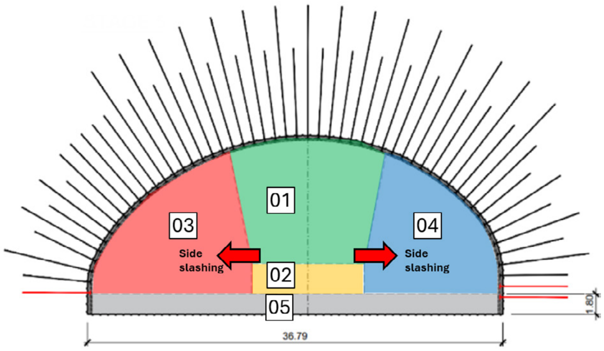


Figure 2. Cross section excavation stages and side slashing for stages 3 and 4.

2.2 *Numerical modelling*

Considering the chamber's favourable ground conditions and based on the outcomes of the initial geotechnical assessment, the ground behaviours associated with the in-situ and induced stresses are not foreseen. The numerical analysis results still provided valuable information and helped validate the earlier findings.

Both 2D and 3D-FDM FLAC numerical models were used to analyse the behaviour of the rock mass related to the stresses during the excavation stages. The main purpose of the models is to verify and confirm the feasibility of the chosen chamber geometry and excavation sequences in terms of total displacements and distribution of plastic zones; and to determine the behaviour of the rock bolts.

A strain-softening behaviour method is shown in Table 2, utilising friction angle and cohesion parameter reductions as functions of plastic shear strain, was employed. Each element of the model, until reaching the first plasticisation, the strength parameters considered are the peak values. Once this limit is exceeded, plastic shear strain begins to occur, initiating a linear reduction in the strength parameters until reaching post-peak values at a plastic shear strain of 2%, which remains constant.

Table 2. Parameters of the strain-softening model used

TYPE	OVERBURDEN	γ	ν	E_{rm}	c	φ	$K_{0,H}$	$K_{0,h}$	ψ
	[m]	[kN/m^3]	[-]	[MPa]	[MPa]	[°]	[-]	[-]	[°]
Peak	540	27	0.1	20200	2.29	39.0	2.1	1.3	5
Post-peak	540	27	0.1	20200	1.64	29.4	2.1	1.3	5

For the analysis of the three-dimensional behaviour of the cavern, and in particular, the excavation sequence effect and the interaction with the shaft's excavation, a 3D model has been performed. The analysis of the induced stress distribution and ground deformation in response to the excavation around the DSS was done without including the support system (Figure 3).

The results provided by the 2D and 3D continuous models are consistent and completely aligned. Limited by the modelling of the shaft excavation, the 2D model results adequately present the behaviour of the ground in response to the chamber excavation.

Figure 3 shows the analysis outcomes indicated a maximum total displacement of 3.5 cm, an extension of the plastic zone of 11 m in the crown, and a plastic shear strain of less than 1.5%.

Rock bolts showed good behaviour with limited plastic zones and axial displacement within the acceptable range.

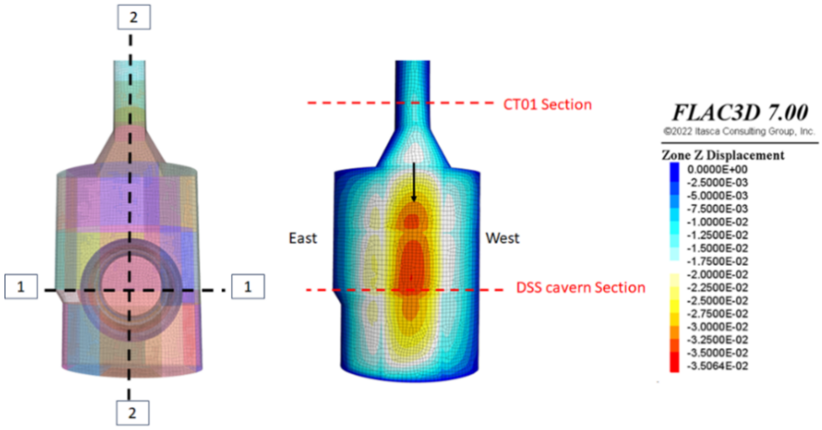


Figure 3. 3D numerical model for the DSS chamber showing stress distribution and estimated displacement.

The maximum representative rock wedge is defined by combining both the limit equilibrium method analysis (purely discontinuum rock mass model) with stress-strain modelling (purely equivalent-continuum rock mass model). In fact, within the cavern's plastic radius and according to plastic shear strain concentration, the development of new fractures contributes to free rock volumes whose shapes and dimensions cannot be predicted solely by the geo-structural conditions of the rock mass.

2.3 *Discontinuous analysis (Wedges)*

Unwedge analyses have been carried out using the discontinuities defined by the geomechanical model, applying load factors to the acting forces, reduction factors to the resisting forces, and imposing a global safety factor of 1.
Different load combinations are considered in the calculations, where load and reduction factors have been applied according to AS-1170 [12], the reduction factor for shotcrete shear strength according to AS-3600 [13], and the reduction factor for the resistance of rock bolts according to AS-5100.3 [14].

2.4 *Maximum wedge over the crown*

The dimensioning rock volume weight is assessed by considering all the key controlling geological and geomechanical features, which include geo-structural discontinuities and the geotechnical properties, as well as the plastic behaviour of the rock mass around the excavation. For the latter, different rock volume shapes and dimensions are considered. The proposed analytical approach considers the maximum triangular wedge geometrically possible at the cavern roof.

The resulting wedge shape is not strictly related to the orientation of existing natural discontinuities in the rock mass. The orientation of the planes delimiting the wedge is defined by considering the most unfavourable conditions. Figure 4 presents that the dimensioning wedge does not depend on actual rock mass quality or discontinuities orientation, but on the shear strength of the rock joints and external acting forces.

When the opening of the wedge is more than double the friction angle (φ), the wedge is unstable and requires support pressure to avoid failure. On the other hand, the wedge is considered stable if the opening is less than two times the friction angle because the sub-horizontal forces acting on the lateral faces tend to lock the wedge in place. Therefore, the friction angle of the discontinuities defines the maximum opening and depth of the potentially unstable wedge.

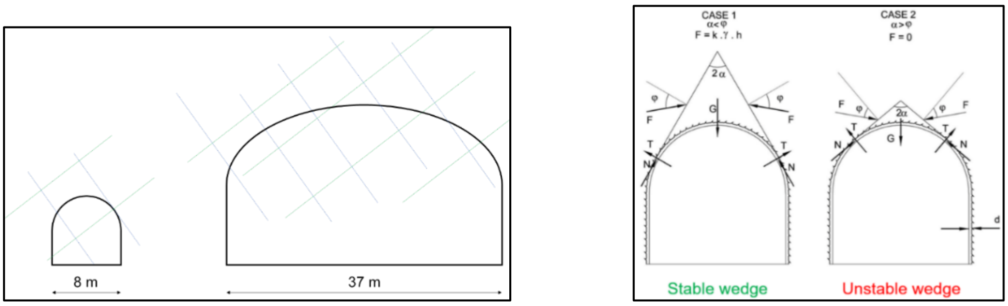


Figure 4. Maximum wedge dimensioning by shear strength of the joints and external acting forces (left) and wedge stability according to shear strength of the joints and external acting forces (right).

3 DESIGN VERIFICATION USING THE OBSERVATIONAL METHOD

Throughout the excavation and after the completion of Stage 1 and the initial advances of Stage 3, the verification of the geomechanical model, the likelihood of unfavourable wedges forming, evaluation of the design support, confirmation, and characterisation of the encountered geology and geological structures were carried out using the observational method. Further assessments are to be carried out using the same principle for the subsequent excavation phases of the chamber, based on data collection from probe holes, monitoring, and site information, among other sources.

3.1 *Geological Conditions of the Central Tunnel*

Excavations of the Central Tunnel confirmed the presence of well-defined interbedded siltstone and sandstone of the Ravine Beds West, identified as being in fair to good quality, dry conditions, with few joint sets. No signs of overstressing were observed on Stage 1 to initial advances of Stage 3.

Consistent with a GSI value of 67 assumed in the design, the rock mass is characterised by an average RMR of 66 GSI, ranging from 65 to 75, with an average of 66 as shown in figure 5 (based on a 10-point rating assigned during face mapping).

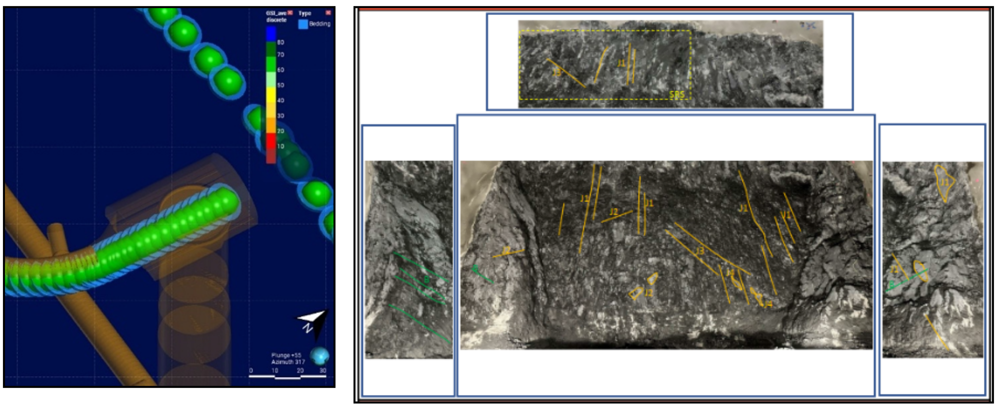
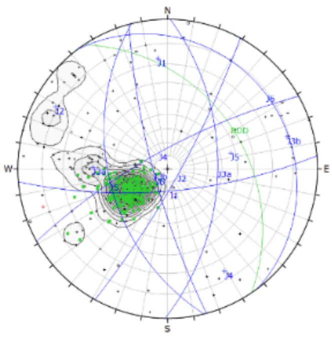


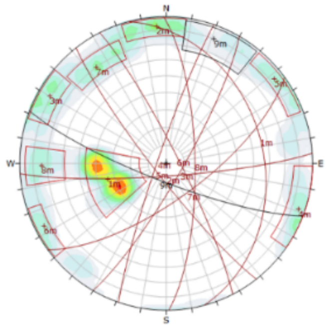
Figure 5. Distribution of RMR and GSI for the DSSC Stage 1 to initial advances of Stage 3.

The identified dominant joint during excavations aligned with those defined in the design. Additionally, the sets (table 3) indicate minor rotations in dip and the direction of dip. The rock mass proved to be of good (favourable) quality, with only one healed sheared feature was identified during the key cuts of the initial advances of the Stage 3 excavation. In general, the sets had a persistence of less than 3 m, undulated, slightly rough, and with apertures less than 1 mm and hard infill

Table 3. Discontinuity sets identified in the Central Tunnel and considered in the design.

Set	DSS STAGE 1 Dip/Direction of Dip	DESIGN Dip/Direction of Dip
Bedding	35/056	34/075
J1	71/163	83/176 (J1)
J2	80/111	81/120 (J2)
J3A	53/090	83/063 (J5)
J3B	79/255	83/232 (J4)
J4	77/335	85/289 (J3)
J5	44/256	-
J6	11/053	70/144 (J6)
J7	-	74/089 (J7)
J8	-	80/201 (J8)





3.2 *Geotechnical Monitoring and Ground-Support System Evaluation*

In accordance with the design, the ground-support system and the identification of potential large-dimension wedges are evaluated through geotechnical monitoring of multiple sections along the excavation. Geotechnical monitoring presented in figure 6, included total displacement monitoring points, multiple rod extensometers, anchor load cells, laser scans, systematic and non-systematic shear displacement monitoring, and transversal probe hole investigation.

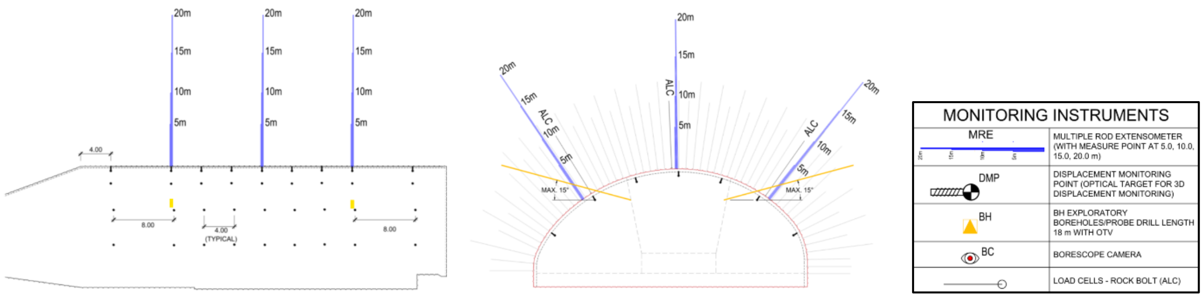


Figure 6. Geotechnical monitoring of the DSS chamber with the location of DMP's, MRE's. ALC's.

The total displacements recorded at the monitored sections during Stage 1 excavations remained stable, with values of less than 13 mm. Rock bolt load cells recorded less than 5 kN, and extensometers recorded less than 1.7 mm (Table 4).

The systematic shear displacement monitoring carried out on the designed 89 mm and 10 m long drainage holes, using an endoscope, showed a dry, very homogeneous rock mass where sheared features were not detected. OTV inspections of the four executed transversal probe holes, drilled towards Stage 3 and 4, also did not confirm the presence of sheared surfaces.

Table 4. Geotechnical monitoring results of the DSS during Stage 1 excavations

SECT. TYPE	INTSTR., QUANT.[2]	DIST. OF APPL.	CHAINAGE, MAXIMUM VALUE[1] 0+004	0+012	0+016	0+020	0+024	0+028	0+032	0+036	0+044
SU1	DMP, 7	Ea. 4 m	< 9.3		< 10.1	< 10.4	< 10.0	< 9.8	< 10.9		< 10.7
SU2	DMP, 7 MRE, 3 ALC, 3	Ea. 20 m		< 8.0 -0.6 < 5			1.7 < 5			< 12.5 -0.6 < 5	
SU3	DMP, 5	At Sheared Plane	No sheared features were identified during the excavations of Stage 1.								

[1] DMP (displacement monitoring point, total in mm), MRE (multi rod extensometers, mm), ALC (anchor load cells, kN).

[2] Quantity of instruments is detailed for full excavation span.

Non-systematic shear displacement monitoring was carried out for the single shear identified on the face mappings of Stage 3 initial cuts via endoscopic inspection of the drainage hole in proximity to the identified feature. The presence of sheared features was not confirmed.

Shotcrete deterioration, such as cracks, spalling, or deformation indications on the support, including rock bolts, during the excavation activities of Stage 1 until the initial advances of Stage 3, was not reported.

3.3 *Ocurrence of overbreaks and support verification*

The most critical overbreaks of the Stage 1 excavation were identified at chainage 0+001 and 0+021, with lengths of 0.63 m and 0.77 m, respectively. A trend of overprofiling and construction-related overbreaks towards the crown and right-side walls can be attributed to the presence of bedding planes in combination with sub-vertical sets (J2 and J3b).

The primary support was verified in both the temporary and permanent phases using Unwedge (figure 7), considering the discontinuities identified in the face mappings. The assessment confirmed, aligning to the design, that the potential wedges have a FoS greater than 1.

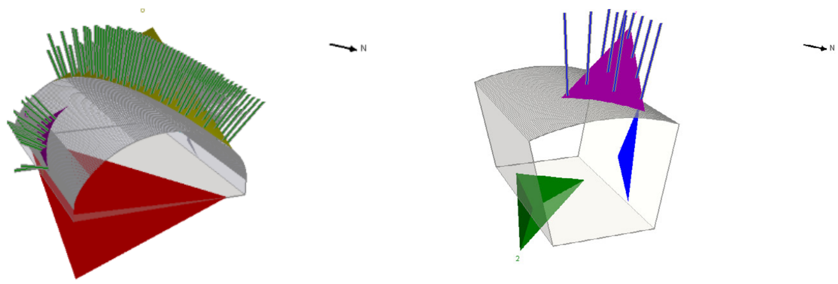


Figure 7. Discontinuous Unwedge analysis for design (left) and stage 1 central tunnel temporary phase (right).

4 GEOTECHNICAL RISK MANAGEMENT AND DESIGN OPTIMISATION

For the mitigation of any geotechnical risks associated with the design of such wide excavations, as is the downstream surge shaft excavation, the potential formation of the largest possible wedges on the full span of the excavation was evaluated and detailed.

This risk was then translated into the execution of the excavation by performing transversal probe holes and shear displacement monitoring to identify any possible shear features. Systematic and non-systematic shear displacement monitoring was also prescribed in the design and later executed during excavation activities.

The multi-staged excavation sequence was halted at defined locations and excavation phases by the execution of different hold points. Hold points consisted of evaluating the design assumptions against the actual conditions before allowing the continuation of new stages of excavation. The confirmation of the design assumptions according to the encountered conditions permitted for design optimisations on the sequence of execution of the chamber.

5 DISCUSSION AND CONCLUSIONS

The downstream surge shaft of the Snowy 2.0 Pumped Storage Power Plant, one of the widest in the world and currently under construction, is an equilibrium chimney with a chamber localized at the top.

The detailed design was undertaken based on the available geotechnical investigations and face maps of the connecting tunnel structure to the chamber. The primary support is designed to hold the possible wedges during the multi-stage excavation, while the secondary support is designed to hold the largest wedges that may form across its full span. The design was evaluated using 2D and 3D numerical modelling for the analysis of rock mass behaviour and rock bolts during excavation, considering strain-softening behaviour. Both the 2D and 3D models aligned, adequately reproducing the behaviour of the excavation, including the displacement, dimension of the plastic zone, and plastic shear strain. The representative wedges formed were defined with limit equilibrium analyses with a discontinuum rock mass model and a stress-strain modelling for an equivalent-continuum model. Discontinuum analysis used different combinations of acting forces, loads, and reduction factors for the resisting forces to achieve a global safety factor of 1.

The verification of the geomechanical model used in the design was conducted using the observational method, which involved assessing the characterised geology, geologic structures, the formation of potential wedges, and stability evaluation. For this purpose, geological face mappings, probe hole data with OTV, laboratory test results, overbreak reports, shear displacement monitoring and geotechnical monitoring data were used.

The analysis confirmed that the rock mass parameters have similar values to those assumed in the design. The very few shears identified during excavation were not confirmed during shear displacement monitoring and transversal probe holes. Therefore, the presence of sheared features that could potentially form a large wedge encompassing the full span of the excavation was not confirmed. The analysis of the reported overbreaks and actual conditions permitted the validation of the support. The installed support has not presented any indication of deformation or deterioration.

6 ACKNOWLEDGEMENTS

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