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The role of geotechnical engineering during the prefeasibility studies and early works of Cadia East panel caving project, New South Wales, Australia

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Abstract

Cadia East underground project will be one of the largest and deepest panel caves in Australia. The first extraction level will be situated about 1,200 m below surface and a second lift located about 1,400 m which shall be brought into operation simultaneously with the first lift. It has dimensions over 250 m wide with a strike length in excess of 1,200 m and a vertical extent of 800 m this could classify this ore body as a world-class mining project.

The underground operation will be implemented at depth, in a hard and massive rock mass, in high stress environments which will produce important zones of induced stresses due to the mining operation and also geotechnical challenges such as cave propagation, fragmentation, stability issues and cave management will be presented. To make this project economically viable requires application of a large scale and low cost underground mass mining method in order to achieve the required high production rates. The prefeasibility study (PFS) has identified the viability of panel caving.

Several geotechnical studies that include geotechnical characterisation, cavability and fragmentation assessment have been conducted. Subsidence analyses which have allowed definition of subsidence limits have been developed. Additionally, mine scale 3D stress models and stability assessments have been developed to understand the induced stress condition below 1,200 m depth. These previous studies have allowed the definition of geotechnical underground design guidelines which include the more relevant geotechnical parameters for mine design and planning at prefeasibility stage.

Additionally, necessary early works have been started which include parts of the main infrastructure such as ventilation raise bores, some excavations of the material handling system, main exploration decline, portals and tunnels in poor geotechnical conditions. In the last case, it has been necessary to consider ground support and reinforcement of tunnels in weak rock mass.

This paper describes the early works that have been carried out, some necessary geotechnical studies and geotechnical mine design guidelines for Cadia East project during the prefeasibility stage.

1 Introduction

1.1 The Cadia East underground project

The Cadia East ore body is located in Cadia Valley approximately 25 km south east of Orange in New South Wales, Australia (Figure 1) which is wholly owned by leading Australian gold producer, Newcrest Mining Limited. The Cadia East ore body was discovered in 1985 and studies into the viability of the Cadia East resource commenced in the early 1990s. The Cadia East resource, located adjacent to the Cadia Hill open pit, is a massive low grade gold-copper porphyry deposit covered by up to 200 m of overburden. The system is up to 600 m wide and extends to 1.9 km below the surface.

The Cadia East underground project involves the development of the massive Cadia East deposit into Australia's first panel cave. The mine will be the deepest panel cave in the world and Australia's largest underground mine. Mining studies have identified panel caving as the mining method which will deliver the optimum technical and economic outcomes for development of this ore body.

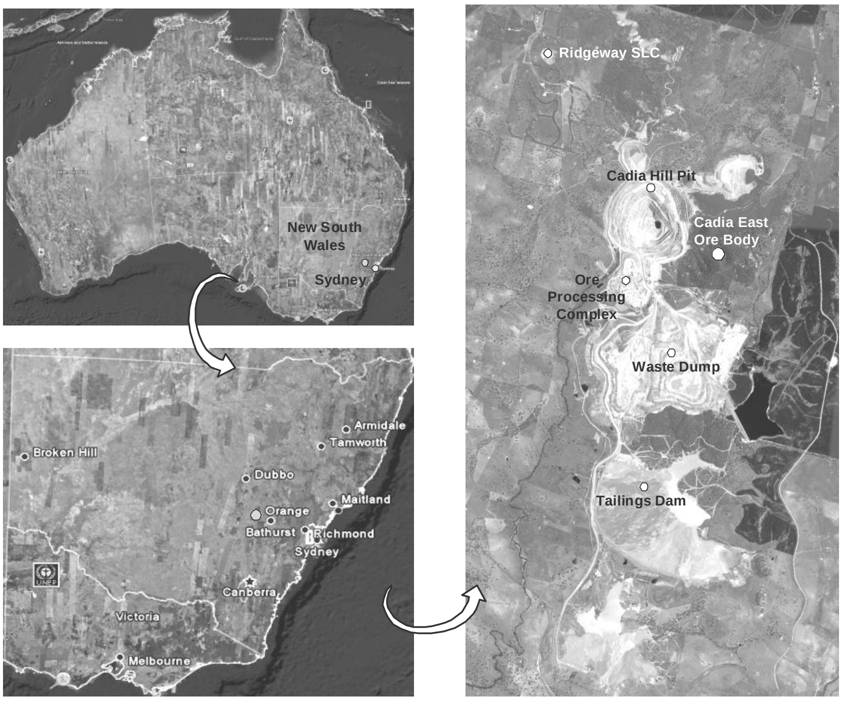


Figure 1 Cadia East project location

The PFS has indicated the viability of a panel caving as the preferred mining option as it delivers the greatest conversion of resource to reserve very high production rates required to meet at Cadia Valley Operation (CVO) processing capacity and the lowest operating mining cost. The Cadia East deposit will be mined as a three lift panel caving operation.

2 Ground support design in weak rock

2.1 General description

Cadia East project has considered some tunnels which will be located in very weak rocks. These excavations presents some special challenges to the geotechnical and mining engineers since misjudgements in the design of support systems can lead to very costly failures. In this context, a case excavated in weak rock is shown. Design and construction details have been simplified to show this methodology. This analysis involves the construction of a tunnel 6.5 m wide and 6 m high. In general the tunnel is excavated with a full face advance thus enabling the ground support system to be installed as close to the face as possible. In this case the excavation has been made with a mechanical excavator and roadheader equipment in order to reduce the damage of the rock mass surrounding the tunnel.

Figure 2 is a longitudinal profile along tunnel alignment which shows the main lithologies present in the tunnel axis.

Geotechnical drill holes were performed in order to assess the geotechnical conditions and rock mass characterisation for this tunnel. Figure 3 shows the same longitudinal profile with the rock mass classification of this excavation. Additionally, ground support recommendations were estimated according to these classification systems.

The rock mass strength was assessed using the geological strength index (GSI) system (Marinos and Hoek, 2001) and rock mass strength parameters, according to Hoek et al. (2002). The correspondents Hoek–Brown and Mohr–Coulomb failure envelopes are plotted in Figure 4.

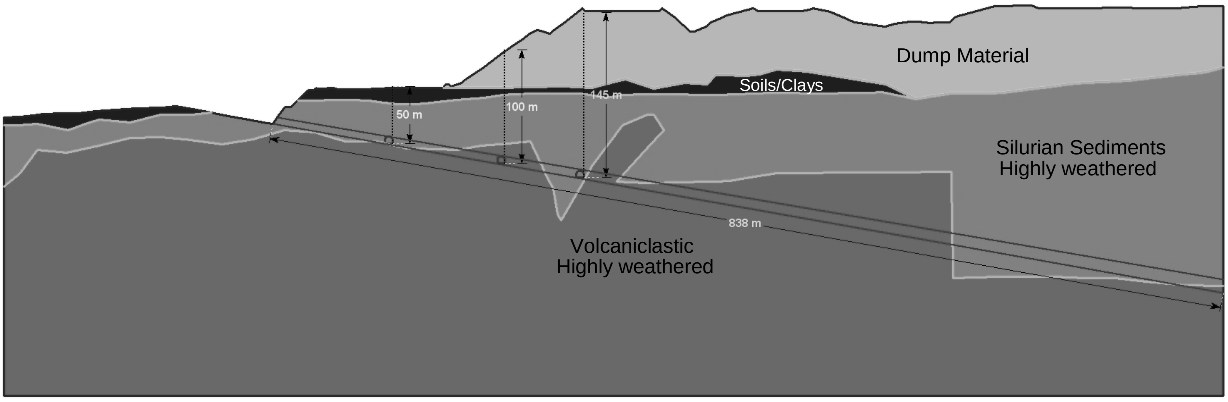


Figure 2 Longitudinal profile along tunnel axis with the main lithologies

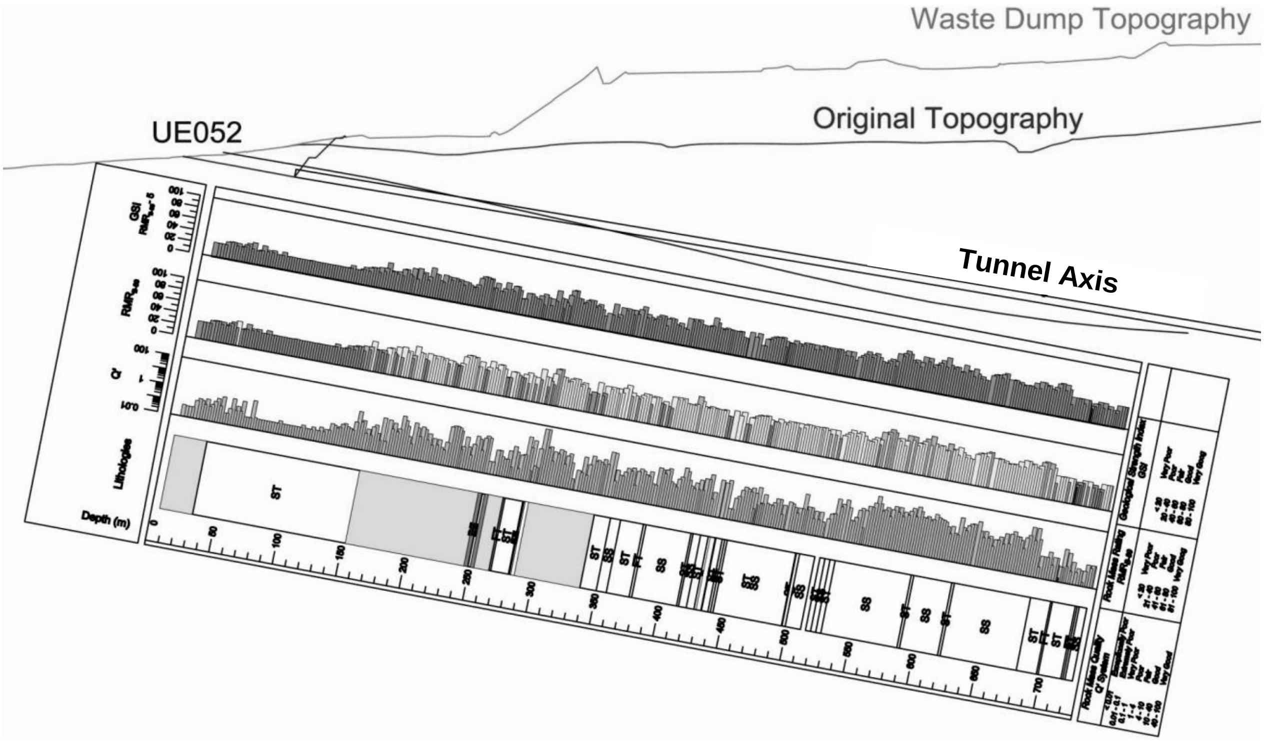


Figure 3 Geotechnical rock mass classification in a longitudinal profile along tunnel axis

2.2 In situ stress conditions

There are no in situ stress measurements available in the vicinity of the tunnel. In these analyses it has been assumed that the ratio of vertical to horizontal stresses parallel to the tunnel axis is 2:1 and that the ratio normal to the tunnel axis is 1.5:1.

2.3 Ground support elements

The stability of the tunnel is controlled by a combination of reinforcement and support systems. The reinforcement consists of rock bolts and friction bolts (Split Set or Swellex) which modify the properties of the rock mass surrounding the tunnel. The support systems involve steel sets fully embedded in fibre reinforced shotcrete (FRS) and these provide resistance to control the convergence of the tunnel. Table 1 describes these support elements.

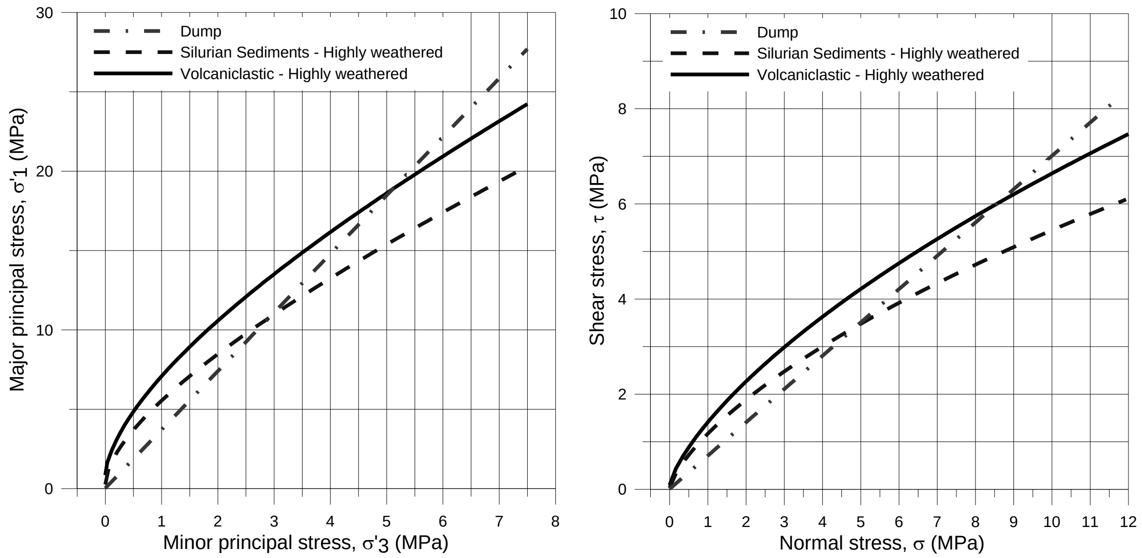


Figure 4 Failure envelopes for individual rock units

Table 1 Ground support elements used

Parameter	Split Set SS-39	Swellex Pm24	Rockbolt Secura Bolts R27	Steel Set Designation
Bolt diameter	39 mm	43–52 mm	23 mm	W 200 × 59 (mm × kg/m)
Tensile capacity	0.12 MN	0.2 MN	0.24 MN	
Young’s modulus	200 GPa	200 GPa	200 GPa	
Bond strength	0.17 MN/m	0.2 MN/m	0.2 MN/m	
Residual tensile capacity	0 MN	0 MN	0.0024 MN	
Spacing	1.0 m	1.25 m	1.3 m	

The split set, rock bolts and steel set respond to the deformation of the rock mass surrounding the tunnel as soon as the tunnel advances, but the shotcrete is only 1 day old at this stage and it has not yet developed its full capacity. While it does not carry its full share of the load, because its stiffness is low, this load may be sufficient to induce failure in the shotcrete. An important issue that has to be considered in the design of this support system is the time dependent properties of the shotcrete layer. Table 2 indicates the shotcrete properties used in these analyses.

Table 2 Shotcrete properties

Parameter	Shotcrete 1 Day	Shotcrete 7 Day	Shotcrete 28 Day	Shotcrete FRS 28 Day
Young’s modulus	24,000 MPa	30,000 MPa	36,000 MPa	20,000 MPa
Poisson’s ratio	0.15	0.15	0.15	0.20
Compressive strength peak	21 MPa	30 MPa	40 MPa	50 MPa
Tensile strength peak	2.0 MPa	3.0 MPa	5.0 MPa	7.0 MPa
Thickness	100 mm	100 mm	100 mm	100 mm

2.4 Excavation sequence

The current excavation cycle and ground support follow the sequence outlined:

- Excavate 2 m in advance of last installed steel set. The distance to face from last installed steel set is 2.5 m which allows installation of a steel set closest to face for next advance.
- Apply 100 mm FRS (floor to floor) to exposed ground and top two thirds of face.
- Cure time to 3 MPa at 1 hour before re-entry. A final compressive strength of FRS is 40–45 MPa at 28 days which increase the temporal ground support capacity.
- Install two rings (2 × 15 bolts) of 2.4 m split set at 1 m ring spacing as a temporary support.
- Install two by steel support sets with 1 m spacing.
- Complete infill spray 200 mm FRS between sets leaving set closest to face exposed for continuation of next cycle. The final compressive strength of FRS is 35 MPa at 28 days.

2.5 Ground support definition

The methodology utilised in these analyses involves the use of support capacity diagrams (Hoek et al., 2008; Carranza-Torres and Diederichs, 2008) using a combination of reinforcement and support methods. The support systems involve steel sets fully embedded in shotcrete (FRS) which provide resistance to control the convergence of the tunnel. The analysis of the support capacity was carried out using numerical analysis in Phase2D Version 7 software (Rocscience Inc., 2008). Six nodes triangular elements (lineal deformation) and plain strain analysis were chosen for these models. Furthermore, in order to consider the tridimensional effects of the advancing face and to estimate the proper deformation of the rock mass prior to support installation, the material softening method was applied. This methodology allows activation and reaction of the installed support system as deformation occurs during the tunnel advance.

The calculation process results in a set of moment versus axial thrust and shear force versus axial thrust diagrams for the steel set and the shotcrete. In the case of the shotcrete, the diagrams were calculated for 1, 7 and 28 day strengths as defined in Table 2. Also these analyses were calculated to 28 days for FRS. The support capacity plots in Figure 5 show the results to 28 days. As is shown in the diagrams (Figure 5), the support system and excavation sequence are appropriate with regard to the in situ stresses and rock mass properties encountered in the area (factor of safety over 1.6).

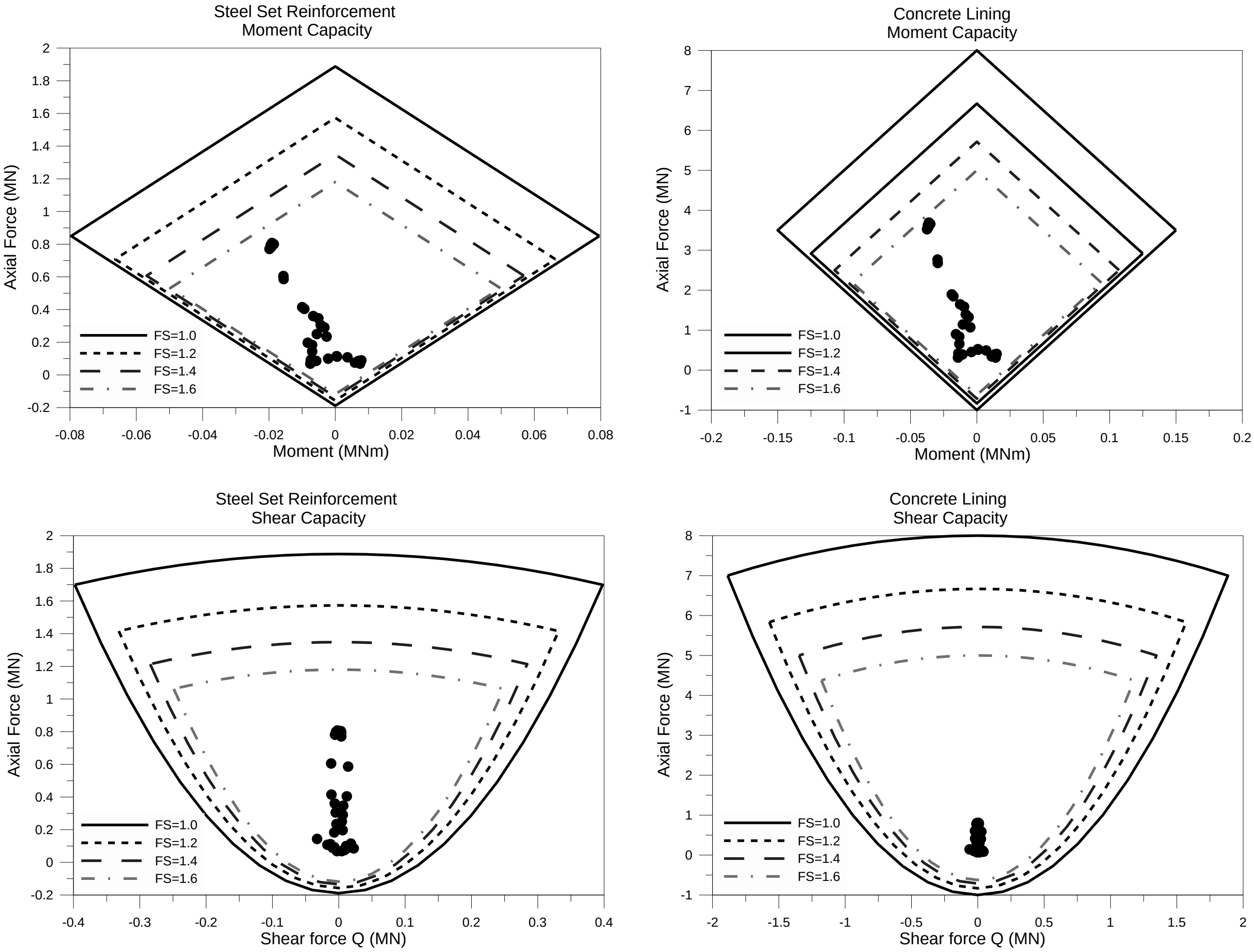


Figure 5 Support capacity diagrams for a 20 cm FRS shotcrete lining, reinforced with steel set W 200 × 59

3 Geotechnical assessment ventilation raisebored

Cadia East underground project will require a main ventilation system based on several ventilation shafts excavated by raise bore rigs. These vent raises are required for the development and preparation of the exploration declines, main infrastructure, extraction and undercut layout and finally for production.

Up to now, three raise bores been excavated and they are in operations. The next step is to excavate VR5 and VR6 system. This VR5 system will be divided into two raises named VR5A and VR5B. The VR5A depth is about 270 m, where the collar is located at the surface and the bottom of this raise would be at 5460 RL. The VR5B depth is about 390 m, where the collar is located at 5371 RL and the bottom of this raise would be at 4987 RL.

Three geotechnical holes have been drilled from the proposed location of VR5A and VR5B in order to have geological and geotechnical information. The first drillhole has been drilled for surface to investigate VR5A (250 m depth), and other two have been drilled from underground (420 and 150 m depth).

3.1 Stability assessment

3.1.1 Critical raisebored class parameters

The quotient (RQD/Jn), representing the structure of the rock mass, is a crude measure of the block or particle size, with the two extreme values (100/0.5 and 10/20) differing by a factor of 400. If the quotient is interpreted in units of centimetres, the extreme particle sizes of 200 to 0.5 cm are seen to be crude but fairly realistic approximations. Probably the largest blocks should be several times this size and the smallest fragments less than half the size.

The quotient (Jr/Ja) represents the roughness and frictional characteristics of the joint walls or filling materials. This quotient is weighted in favour of rough, unaltered joints in direct contact. It is to be expected that such surfaces will be close to peak strength, that they will dilate strongly when sheared, and they will therefore be especially favourable to raise-bore stability.

Decreased stability can be expected in rock masses with small to medium block size and low interblock strength. A raise-boring classification is used for the two parameters described above. Critical raisebored class parameters by using one of these drillholes are detailed in Figure 6(a).

3.1.2 Raisebored quality, Q_R

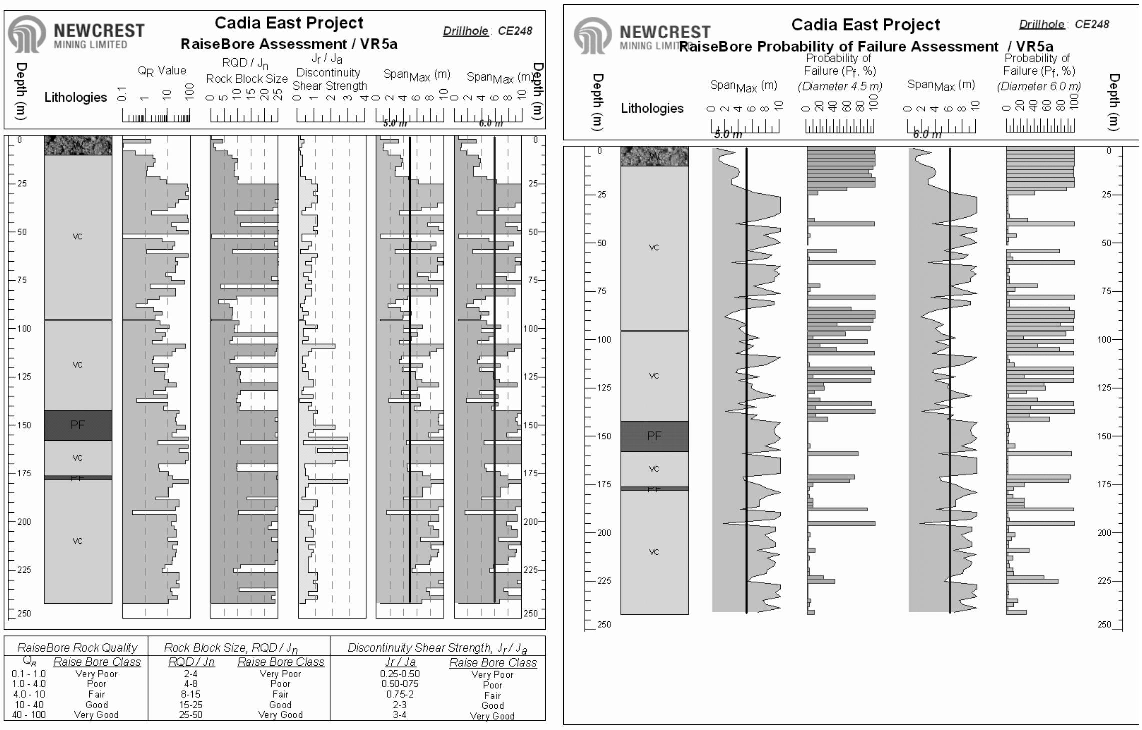
Data from the geotechnical drillholes logging have been processed to provide an estimate of stable raisebored diameters using the modified Q_R-System of McCracken and Stacey (1989). The critical raisebored rock mass qualities are detailed in Figure 6(a). The overall proportion of poor quality ground is an important consideration of raise bore stability; however, the thickness and distance between zones of poor quality ground is critical in considering the overall stability of the raise.

3.1.3 Raise dimensions and stability

The maximum unsupported span for the face and sidewall of the raise has been assessed using the relationship:

$$Span_{Max} = 2 * RSR * Q_R^{0.4} \quad (1)$$

Where RSR is the raise-bore stability ratio. McCracken and Stacey (1989) recommend an RSR of 1.3 for a ventilation shaft with a service life of approximately 10 years and a 5% probability of failure. For a short-term excavation and a 25% probability of failure, an RSR of 2.5 can be used. An example of stability assessment for a raise — by describing the different critical parameters — is shown in Figure 6(a).



(a) (b)

Figure 6 Long-term geotechnical assessment, maximum unsupported span and probability of failure assessment of ventilation raisebored VR5 system Cadia East project

3.1.4 Probability of failure, P_F

In order to assess the uncertainty associated with raise-boring condition, probabilities of failure (P_F) along the raisebored length have been assessed. The method uses the rock quality data and raisebored size to assess potential probability of failure.

Calculating the P_F of a design is equivalent to evaluating the probability that a design with $Span_{Max} < Span_{Design}$ will fail, which is the probability that the maximum unsupported span will be less than 5 m, i.e. $P_F = P(Span_{Max} < 5.0)$. Similarly, the probability that the maximum unsupported span will be 6 m diameter have been assessed as $P_F = P(Span_{Max} < 6.0)$.

In terms of standard deviation, geotechnical stability analyses consider uncertainty in the raise-bore quality, Q_R, which was assessed considering coefficients of variations (C_V) of 20%. The advantage of this approach is that it allows the effect of geotechnical uncertainty on the value of Q_R to be taken into account. The assessment of probability of failure is shown in Figure 6(b).

These results are summarised in Figures 6(a) and 6(b) where the wall stability based on conditions intersected by drillhole has been assessed in terms of rock mass condition, raisebored critical parameters and probabilities of failure for long term stability condition.

4 Subsidence assessment analysis Cadia East project PFS

The prediction of this phenomenon in cave mining is particularly difficult due to the size and depth of most operations, and the complex and discontinuous displacements generated at the surface. The type of subsidence expected is to be even more conspicuous for the deeper caves of the future.

Empirical, analytical and numerical methods are used to assess subsidence during the prefeasibility study.

The combination of these methods — any of those with its own strengths and weakness — in conjunction with engineering criteria had enabled us to propose design values of subsidence’s geometry suitable for this stage of the project. The methods considered have been: analytical method, Brown and Ferguson method (1979); analytical method, limit equilibrium method; and numerical method, finite element method.

4.1 Analytical method, Brown and Ferguson method (1979)

The method of Brown and Ferguson is based on the following assumptions (Brady and Brown, 2004):

- Mining and caving occur for a large distance along strike compared with the cross-sectional dimensions shown in Figure 7. As a consequence, the problem may be reduced to one of two dimensions. Calculations are carried out for each domain thickness perpendicular to the plane of the cross section.
- The initial position of the hangingwall face is defined by known values of the geometrical parameters $H1$, Hc, $Z1$ (Figure 7).
- The extent of caving at the new mining depth, $H2$, is defined by a tension crack which forms to a critical depth and strikes parallel to the orebody.
- Failure of the hangingwall rock mass occurs along a critical, planar, shear surface whose location is determined by the strength properties of the rock mass and the imposed effective stresses.
- The hangingwall rock mass has homogeneous and isotropic mechanical properties.
- Its shear strength can be defined by an effective stress form of Coulomb’s criterion.
- Water may enter the tension crack and seep along the potential failure surface into the underground excavations, producing a triangular distribution of excess water pressure along the shear surface.
- Simplified distributions of stress within the caved and caving masses are used.

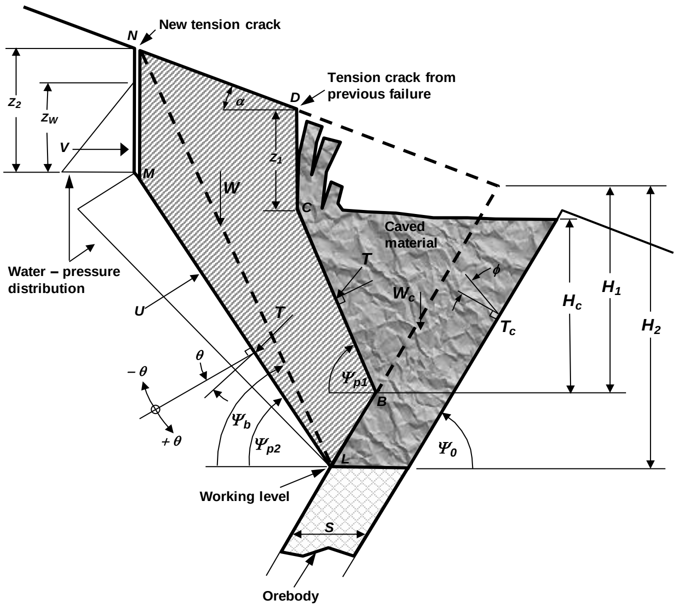
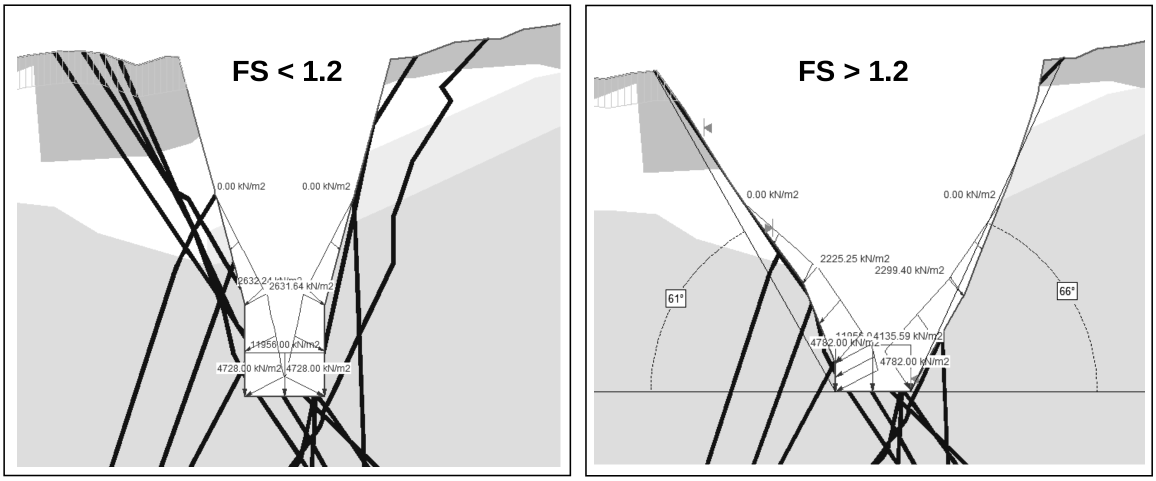


Figure 7 Idealised model used in limiting equilibrium analysis of progressive hanging wall caving (after Brown and Ferguson, 1979)

4.2 Analytical method, limit equilibrium method

For the application of the slice limit equilibrium method in the analysis of subsidence, a progressive failure analysis was performed utilising the following methodology (Figure 8):

- As a standard application of the slice limit equilibrium method, a geotechnical section of analysis is made. The geometry of the section is constructed: topographic surface, distribution of the geotechnical domains, main structures and the position of the mining blocks. Finally, the geotechnical properties of the rock mass and structures are assigned.
- Initial conditions of the geometry of connection to surface are assumed. Generally was considered 90° slopes in the lower half of the block and then 75° up to surface.
- Broken material in the crater is modelled as a vertical stress on the bottom of the carter equal to its overburden pressure, and an active pressure on the walls of the crater, considering an active pressure coefficient equal to 0.4.
- A tension crack is defined as 10% of the crater wall height.
- For each wall of the crater of subsidence, a slice limit equilibrium analysis of stability is carried out. A factor of safety (FS) is gotten as output.
- According to the simplification made in the analysis, a FS < 1.2 can be associated to wall instability and potential slope failure.
- In that situation (FS <1.2) the volume within the surface with the lowest FS is removed (unstable volume) and that wall is reanalysed, if the new FS < 1.2 the wall is excavated again and in an iterative process a progressive failure mechanism is modelled until the factor of safety become higher than 1.2.
- If FS > 1.2, the wall is considered that have reached a stable condition and the angle of the wall (angle of break is measured).



Initial geometry is assumed.
Broken material inside of the crater is modeled

Final "stable" geometry, the angle of break is measured.

Figure 8 Subsidence analyses using an analytical method (limit equilibrium analysis)

4.3 Numerical method, finite element method

In order to identify the plastic behaviour of the rock mass and others failure mechanisms, numerical analysis has been performed. In the development of this analysis was considered that:

- Two-dimensional models were developed using the finite element method. PHASE^2 v 7.0 software was utilised (Rocscience Inc. 2008), and different geotechnical cross sections were defined.
- Six nodes triangular elements (lineal deformation) were chosen for these models. The rock mass is homogeneous continuum with elastic–plastic behaviour and was modelled using the Hoek and Brown failure criterion. The main structures were modelled explicitly as interface elements, using the failure criterion of Mohr–Coulomb. Fully drained conditions were assumed.
- The in situ stress was modelled as gravitational, where the vertical stress varies linearly with depth and stress ratios of 2.0 and 1.0 in the EW and NS orientations were respectively considered.
- The rock mass strength was assessed using the Geological Strength Index (*GSI*) system and rock mass strength parameters, according to Hoek et al. (2002).
- The excavation sequence was defined yearly according to the mining plan. An example of the excavation sequence is presented in Figure 9.

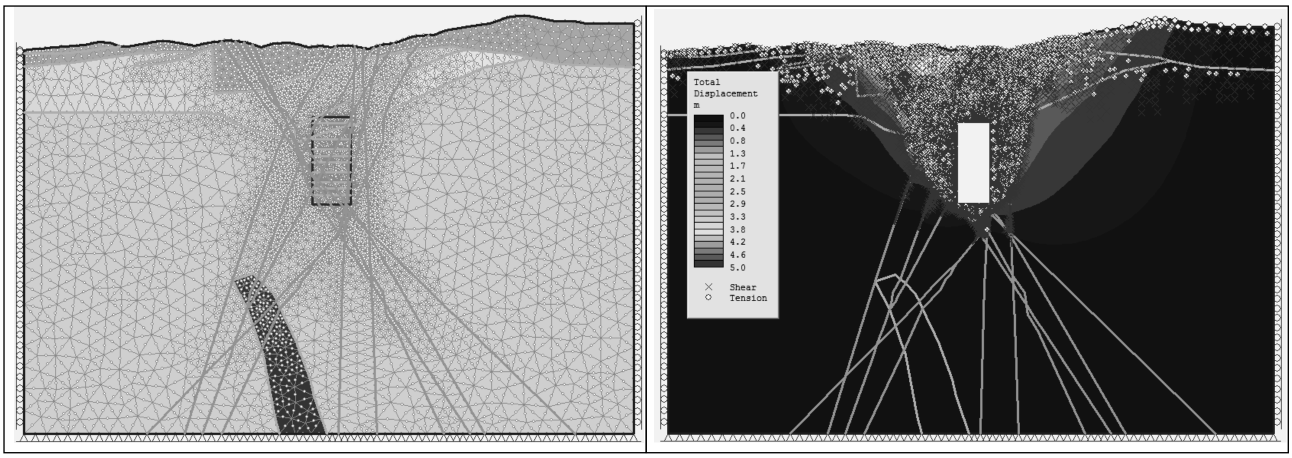


Figure 9 Finite element mesh used in the subsidence analysis and total displacement distribution, active faults (left: dark shading and lines; right: lighter shading and lines) and yielding elements (×) shear, (o) tension as indicator of the extension of the subsidence crater

4.4 Subsidence design charts

Subsidence design charts are based on analytical and numerical methods. These results are shown in Figure 10. This graph shows crater wall height (mining depth) vs. angle of break and the results of these assessment methods are plotted. In addition, in this graph is presented real geometries of crater height and angles of break from others panel caving operations as well.

These curves define the upper limit, lower limit and the best estimation of the subsidence angle or the recommendations of break angles of Cadia East (CE) PFS project. As was mentioned previously each method has its own advantages and disadvantages, but together, they give us a better comprehension of the subsidence phenomena applied to this project.

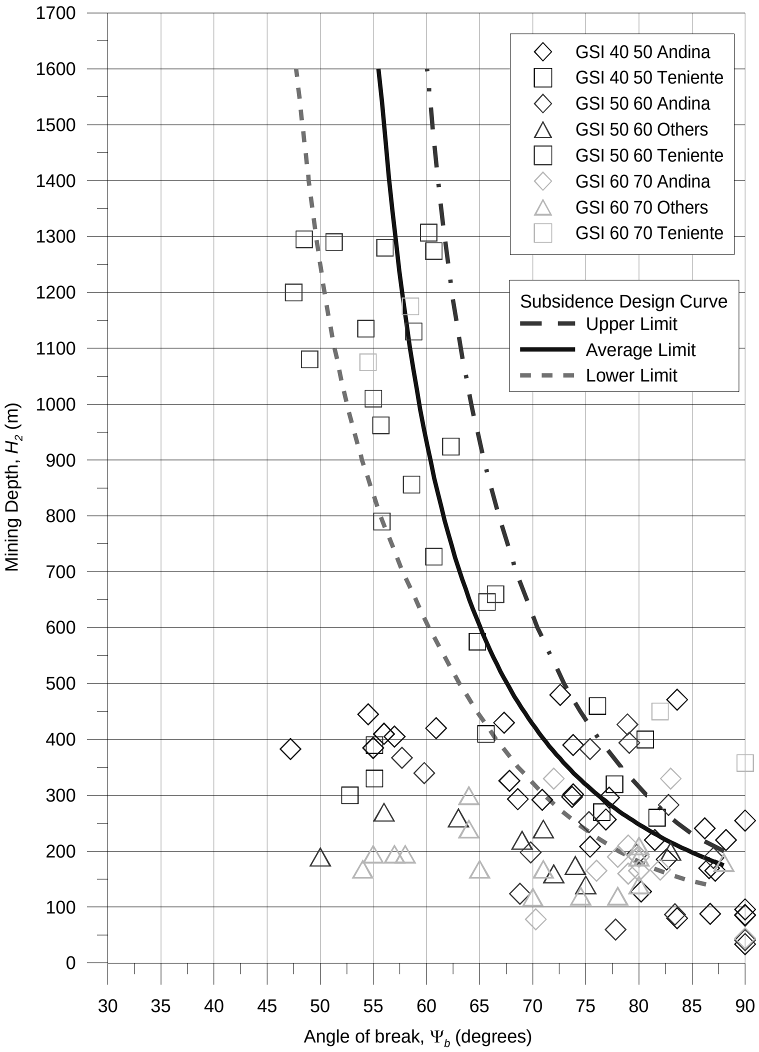


Figure 10 Subsidence design curves CE PFS

5 Mine scale 3D stress model, induced stress assessment

In order to have a better understanding about induced stresses generated by the panel caving and its extraction sequence, numerical analysis was carried out which allows estimation of the stresses state for the various mining geometries.

The mine scale models have been developed in Map3D software package which is based on the boundary element numerical method (BEM). The model considers an initial condition (without caving) and different mining stages (years). These models consider for each year all the cavities created by mining which are defined by the extraction rates and their respective subsidence (break) angles. The distance between undercutting and extraction fronts (undercutting strategy) was also considered into the model.

Additionally, it must be indicated that extraction and undercut layout have not been included in the detail models because the main target is the understanding of the effect of the caving front (face) on the stress distribution. However, some infrastructure has been included such as the main decline, extraction level accesses, conveyor decline and some of the ventilation raise designs.

5.1 Mine scale model

Map3D software has been used in order to build this model. The rock mass was considered as an elastic, continuous and isotropic medium. Also, a surface excavation was generated to include the effect of topography in the model.

5.1.1 Geotechnical domains

The main lithologies that have been included in the mine scale model are shown in Figure 11.

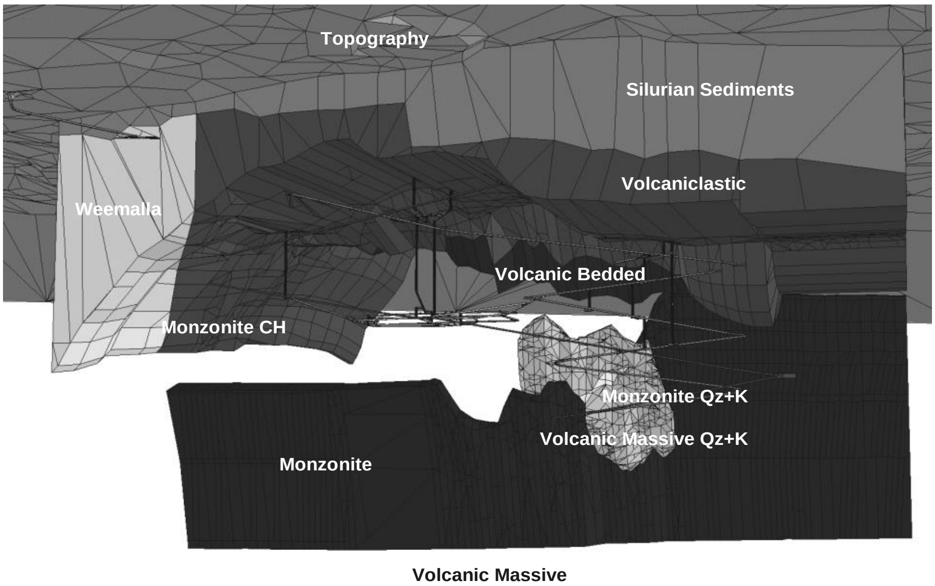


Figure 11 Geotechnical domains at mine scale model

5.1.2 Rock mass strength

The rock mass strengths properties used for these lithologies are expressed as function of the Hoek and Brown's parameters and their elastic modulus these are shown in Table 3.

Table 3 Geotechnical domains Hoek and Brown's parameters

Geotechnical Domain	Hoek and Brown Parameters					
	m_b	s	A	σ_{ci} (MPa)	E_m (GPa)	ν_m
Silurian sediments	0.394	0.0002	0.508	120	4.31	0.26
Volcaniclastic	1.153	0.0023	0.503	170	10.49	0.23
Volcanic bedded	1.292	0.0031	0.503	133	11.77	0.22
Volcaniclastic massive	2.456	0.0063	0.502	170	14.96	0.22
Volcaniclastic massive Qz+K	2.833	0.0094	0.502	150	17.79	0.21
Monzonite	2.947	0.0063	0.502	145	14.96	0.22
Monzonite Qz+K	2.555	0.0042	0.503	120	12.59	0.23

5.1.3 Geotechnical stress state in initial condition (pre-mining)

The in situ stress field of CE PFS project has been defined according to the results of in situ stress measurements carried out by Hulls et al. (2008). Table 4 summarises the pre-mining stress states which have been incorporated in the mine scale model in Map3D as initial stress field (pre-mining).

Table 4 Stress state mine scale model

	σ_1	σ_2	σ_3
σ_i (MPa)	36.9	21.0	17.3
Plunge (°)	5	12	78
Trend (°)	085	354	203
$\Delta\sigma_i$ (MPa)	0.053	0.031	0.025

5.1.4 Constitutive model

For the mine-scale model carried out, elastic constitutive models have been used, because it only solves the equilibrium equations, continuity and elasticity which are expressed in terms of the stress σ_{ij} and strain ε_{ij} (Map3D course, Wiles 2008).

As the model is developed under elastic conditions, the only properties that influence the stress–strain distribution correspond to the Young's Modulus *E*, Poisson's ratio *v* and the initial stress condition (pre-mining) which are the main geotechnical input parameters for Map3D software package. The remaining rock strength parameters are used once the model's runs are finished in order to make comparisons between strength and load (i.e. safety factor based on a failure criterion).

5.1.5 Failure criterion used

The over-stress criterion consists on defining the rock mass strength through a failure envelope (in this case Hoek–Brown criteria has been used) to subsequently obtain the loads from a numerical or algorithmic model. Those loads are included in the strength chart (envelope), and it allows differentiating if the points (σ_1, σ_3), are in failure or not. So in order to define failure or damage zone near the caving front, the over-stress failure criterion has been used. Figure 12 describes these concepts.

This criterion was used considering the stress field provided by the Map3D model and the strength properties for the Volcanic Massive unit which is the host or more predominant domain in CE PFS.

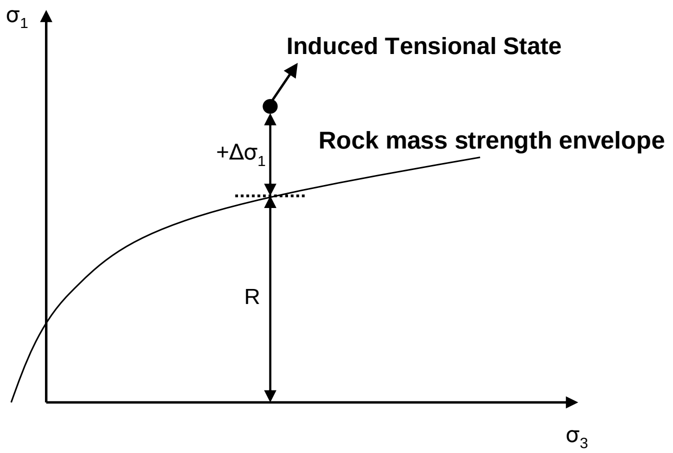


Figure 12 Graphic representation of the over-stress criterion where magnitude Δσ1 corresponds to the excess stress applied to the rock mass above its strength (R)

In order to assess the failure criterion, control points have been defined in the mine scale model. Map3D tool is being conditioned to solve the BEM equations in these points and the stress tensor for these locations is obtained. Finally, these control points are compared with rock mass strength envelope of the Volcanic Massive geotechnical domain to assess stability as per mining stages.

5.1.6 Caveback geometry

In order to build the geometry of the caveback, it is necessary to calculate the caveback height and length. These two geometrical aspects are determined considering the following as input data:

- the block height
- subsidence break angle (Volcanic Massive is consider 70°) this value is required both for the cavity projection and for the calculation of the caveback height and length
- in terms of break/extraction ratio, a ratio of 3:1 has been adopted, which corresponds to about 30% extraction (empirical rule of El Teniente mine)
- extraction angle, assuming a value equal to 40° that considers equilibrium between the undercutting and drawbell incorporating rates. Using this angle and the break/extraction ratio it is possible to determine the break angle.

5.1.7 Caved zone

The broken material generated by caving was considered as fill material which acts as an internal force on the wall of the caveback. These internal forces are acting normal to surfaces with a magnitude defined in terms of depth and gravitational weight. The magnitude is defined in terms of depth and gravitational weight on caveback. The gravitational weight was considered as 100% on caveback (conservative condition). The broke material properties used in these analyses were Young's Modulus E = 250 MPa and Poisson's ratio ν = 0.25 (The International Caving Study II, 2005).

The 3D models about caveback geometries caused by caving mining were built based on the parameters defined previously. Figure 13 shows this mining configuration.

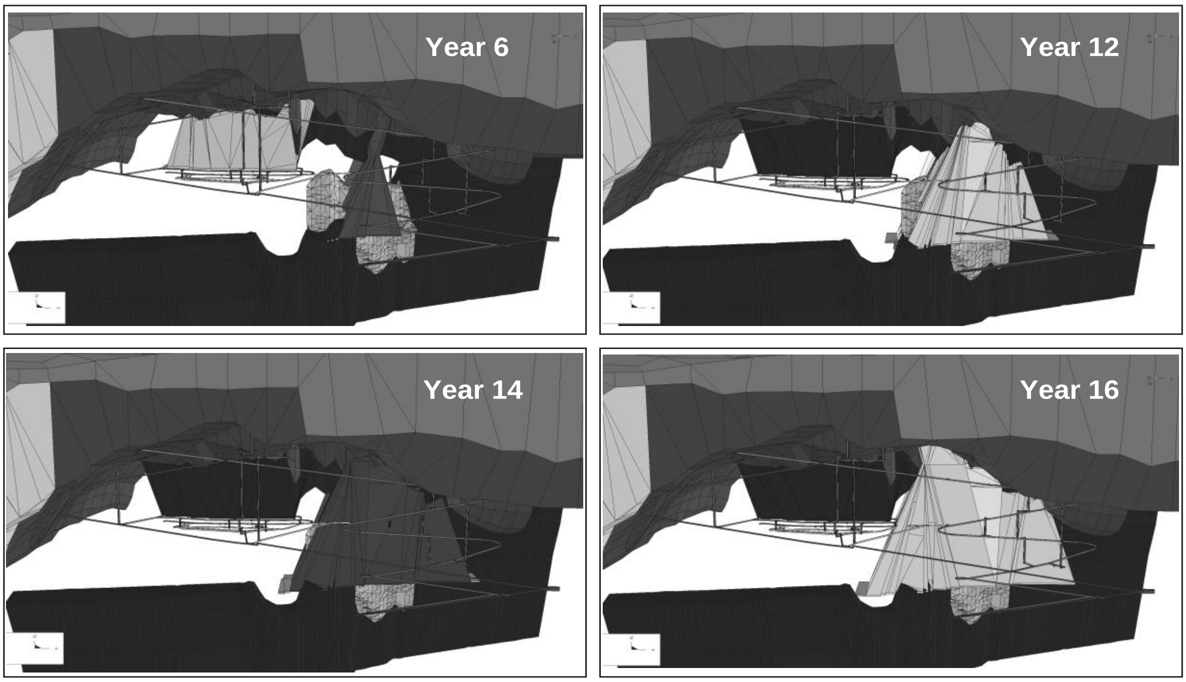


Figure 13 Mine scale model according to mining growth of CE PFS

5.2 Induced stress assessment

As it was previously indicated, rock mass strength is assessed in terms of over-stress criterions, which consists on defining the rock mass strength through Hoek and Brown's failure envelope and subsequently obtain values of σ_1 and σ_3 in order to define control points with over-stress condition. The results of rock mass strength envelope for Lift 1 are shown in Figure 14(a).

Additionally, the caveback over-stress conditions ($\Delta\sigma_1$) corresponds the excess stress based on Hoek and Brown criterion which were assessed year by year as well as the position and extension of caveback and its relationship with the induced stresses. An application example for Lift 1 is shown in Figure 14(b) (year 14). This figure compares the in situ and induced stress conditions on a specific year of Lift 1 (4631RL). Blue dots are control points before caving (in situ stress condition) whilst black dots corresponds to induced stresses associated to the geometry and position of the cave-back for this particular year.

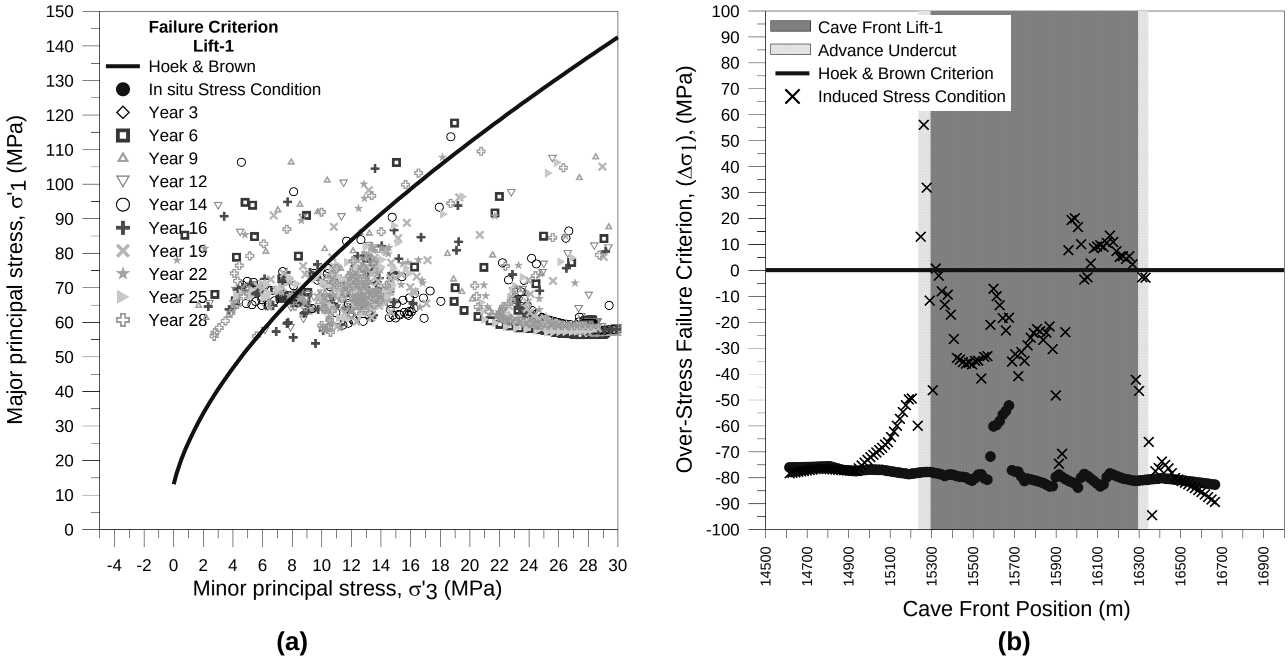


Figure 14 Rock mass strength envelope and caveback over-stress assessment across Lift-1

6 Geotechnical guidelines

Geotechnical guidelines for CE PFS have been developed using relevant geological and geotechnical information. The geotechnical characterisation model is based on a very powerful and robust geotechnical platform (Catalan et al., 2008), based on: geological and geotechnical data gathered mainly from both surface and underground drillholes (more than 400 km), acoustic televiewer logging, in situ stress measurements (10) as well as geotechnical underground exploration galleries (1,200 m).

Additionally, different geotechnical studies and analyses have been carried out in the prefeasibility study of Cadia East project allowing the generation of reliable models capable of representing the anticipated geotechnical conditions of caving processes and for prediction of probable behaviour at distinct depths and geometries of the project.

All this information and these studies have allowed the generation of a set of geotechnical underground design guidelines for the CE PFS which are summarised in Tables 5 and 6.

Table 5 Geotechnical guideline CE project PFS

<table>
<tr><th colspan="2">Mine Design Parameter</th><th colspan="2">Benchmarking Values</th><th colspan="4">Design Guidelines CE PFS Project</th></tr>
<tr><td colspan="2">Rock mass rating
(Laubscher, 1990)</td><td colspan="2">$50 \leq RMR_{L\text{-}90} < 60$</td><td colspan="2">Monzonite
Monzonite Qz+K
Volcanic massive
Volcanic massive Qz+K</td><td colspan="2">$60 \leq RMR_{L\text{-}90} < 77$ (μ= 69)
$51 \leq RMR_{L\text{-}90} < 71$ (μ= 61)
$53 \leq RMR_{L\text{-}90} < 70$ (μ= 62)
$49 \leq RMR_{L\text{-}90} < 70$ (μ= 59)</td></tr>
<tr><td colspan="2">In situ stress field</td><td colspan="2">σ_1: 38 Mpa ±10.5 MPa
σ_3: 20 MPa ± 7.8 MPa
σ_v: 23 MPa ± 8.0 MPa</td><td colspan="2">Magnitudes
σ_1: 0.053 × depth (MPa)
σ_2: 0.031 × depth (MPa)
σ_3: 0.025 × depth (MPa)</td><td colspan="2">Orientations
σ_1: 05/085 (plunge/trend)
σ_2: 12/354 (plunge/trend)
σ_3: 78/203 (plunge/trend)</td></tr>
<tr><td colspan="2">Access</td><td colspan="2">Decline</td><td colspan="4">Decline</td></tr>
<tr><td colspan="2" rowspan="2">Block height</td><td colspan="2" rowspan="2">210 m</td><td colspan="2">Lift 0</td><td colspan="2">Lift 1</td></tr>
<tr><td colspan="2">350 m</td><td colspan="2">400 – 590 m</td></tr>
<tr><td rowspan="3">Footprint area</td><td>< 50,000 m^2</td><td colspan="2">30,000 m^2</td><td rowspan="3">Lift 0</td><td></td><td rowspan="3">Lift 1</td><td></td></tr>
<tr><td>50,000 a 100,000 m^2</td><td colspan="2">75,000 m^2</td><td></td><td></td></tr>
<tr><td>> 100,000 m^2</td><td colspan="2">170,000 m^2</td><td>123,750 m^2</td><td>435,625 m^2</td></tr>
<tr><td rowspan="4">Caving initiation</td><td>Area</td><td colspan="2">10,000 m^2</td><td rowspan="4">Lift 0</td><td>26,900 m^2</td><td rowspan="4">Lift 1</td><td>26,900 m^2</td></tr>
<tr><td>Shape</td><td colspan="2">Square</td><td>Square</td><td>Square</td></tr>
<tr><td>Measures to facilitate</td><td colspan="2">Slot</td><td></td><td></td></tr>
<tr><td>Hydraulic radius</td><td colspan="2">20 to 30 m</td><td>33 to 49 m</td><td>33 to 49 m</td></tr>
<tr><td colspan="2" rowspan="4">Fragmentation</td><td colspan="2" rowspan="4">1.8 to 2.0 m^3</td><td rowspan="4">Lift 0 / Lift 1</td><td>Primary</td><td colspan="2">64% to 78% ≤ 2.0 m^3</td></tr>
<tr><td rowspan="3">Secondary</td><td colspan="2">25 m draw = 74% to 85% ≤ 2.0 m^3</td></tr>
<tr><td colspan="2">50 m draw = 83% to 92% ≤ 2.0 m^3</td></tr>
<tr><td colspan="2">100 m draw = 93% to 97% ≤ 2.0 m^3</td></tr>
<tr><td colspan="2" rowspan="2">Subsidence</td><td>RMR < 70</td><td>α > 45°</td><td>Lift 0</td><td colspan="2" rowspan="2">Overall break angle (α_B□)</td><td>63°</td></tr>
<tr><td>RMR > 70</td><td>α > 60°</td><td>Lift 1</td><td>58°</td></tr>
</table>

Table 6 Geotechnical mine design guideline CE project PFS

Mine Design Parameter			Benchmarking Values	Design Guidelines CE PFS Project		
Undercut level	Drifts	Spacing	15 m	Lift 0 and Lift 1	15 m	
		Height	4 m		4.5 m	
		Width	4 m		4.5 m	
	Undercutting strategies		Post-			
			Pre-			
			Advance-		Advanced-undercutting	
	Undercut rate	Post-	1,797 m²/month			
		Pre-	3,229 m²/month			
		Advance-	2,350 m²/month		2,500 m²/month	
	Undercut shape	Fan-cut				
		Flat-cut	2,125 m²/month			
		Crinkle-cut	2,187 m²/month		High undercut	
	Undercut height		8 m		8 m	
	Undercut orientation				N 15° W	
Extraction level	Drifts	Crown-pillar thickness	17 m	Lift 0 and Lift 1	18 m	
		Spacing	30 m		30 m	
		Height	4 m		4.5 m	
		Width	4 m		4.5 m	
	Layout	El Teniente			30 × 20 m	30 × 18 m
		Herringbone			30 × 20 m	
		Offset Herringbone			30 × 20 m	
		Orientation			Extraction drives Lift 0 N – S ± 20°	Extraction drives Lift 1 N 10° W ± 20°
	Draw points	Spacing	15 m		18 to 20 m	
		Influence area	225 m²		270 to 300 m²	
		Height	4 m		4.5 m	
		Width	4 m		4.5 m	
	Perimeter drive and access level	Height			5.0 m	
		Width			5.0 m	
	Draw rates		100 to 200 m/day		Height Block (%)	Draw Rate (mm/day)
					0 to 5	95
					5 to 10	115
					10 to 15	130
					15 to 20	150
					20 to 25	170
					25 to 30	195
					> 30	225

7 Conclusions

This paper describes different methodologies that have been used in Cadia East project prefeasibility stage which have allowed to increase the geotechnical knowledge as well as to have a better understanding of some critical tasks of the largest and deepest panel caving in Australia.

A methodology based on the support capacity diagrams for the design of tunnel linings in weak rock has been used. The most important geotechnical factors as well as the sequence of installation and activation of the different support elements have been taken into account.

A method of quantifying the geotechnical risk to a raisebored has been utilised. This methodology has been calibrated with other raisebored that have been built in Cadia East project. The approach that has been outlined provides a good indication of the overall geotechnical constructability of these vertical excavations.

Three methods have been presented to predict the subsidence angles in cave mining. Their results in terms of crater wall height (mining depth) vs. angle of break have been plotted on a design chart. These curves define the limits of the subsidence angle or the recommendations of break angles of CE PFS project.

A mine scale 3D stress model based on a boundary element numerical method has been carried out in order to assess the induced stresses generated by the panel caving and its extraction sequence. These results have been assessed in terms of the rock mass strength and the caveback over-stress conditions every year. These results have allowed a preliminary understanding of the induced stresses during the life of the caving.

Geotechnical parameters for mine design of Cadia East panel caving project, supported by different geotechnical studies as well as a detailed benchmarking process, have been generated. These guidelines include the most relevant geotechnical parameters for mine design and planning at the prefeasibility stage.

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