Geomechanical evaluation of large excavations at the New Level Mine - El Teniente

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Abstract

The New Level Mine is a 130.000 tpd panel caving project set to start in 2017 at the El Teniente mine. VP-NNM CODELCO (Vice-President Office of the New Level Mine) is currently finishing a detailed engineering design of the underground mine. The evaluation considers, the design of the crusher cavern Nº1 located in the Braden Pipe, which is a waste rock chimney located in the central part of the ore body. A geo-mechanical study has been carried out to evaluate the stability of the planned infrastructure and to provide recommendations about the design of underground caverns and galleries, including support. As part of this study, empirical methods, two-dimensional and three-dimensional continuum models have been developed and applied to evaluate the influence of the high stresses and different geotechnical units, on the mechanical response of the excavation. This paper introduces general aspects of the New Mine Level underground project and discusses in particular geo-mechanical analyses and design carried out to evaluate stability and support of some of the large excavations involved in the project.

1 Introduction

El Teniente copper mine is located in the central part of Chile, Cachapoal Province, VI Region, about 50 km NE from Rancagua City and about 70 km S-SE from Santiago City (Figure 1).

At the El Teniente mine, the copper and molybdenum mineralization occurs in andesites, diorites and hydrothermal breccias surrounding a pipe of hydrothermal breccias called Braden Pipe and located in the central part of the ore body. The Braden Pipe has the shape of an inverted cone, with a diameter of 1,200 m at surface and a vertical extent of more than 3000 m. The Braden breccias are waste rock. Therefore, the different productive sectors of El Teniente mine are surrounds the Braden Pipe, and the main infrastructure and access shafts are located inside the pipe (Pereira et al. 2003).

The New Mine Level is a 130,000 tpd panel caving project set to start in 2017 at the El Teniente mine. The mining project considers using the panel caving method to mine copper ore. The Vice-President Office of the New Level Mine (VP NNM) has finished a detailed engineering evaluation of the project, which considers the construction and operation of several mining units to be operated independently from each other.

Among the most important elements of the permanent mining infrastructure to be designed and constructed first are large crusher caverns, designated as SCh Nº 1, SCh Nº 2 and SCh Nº 3 caverns. These caverns are required to reduce the ore size from the operation mining sectors that will guarantee the continued operation for a period of 50 years or more.

The objective of this paper is to present general aspects of the design of one of the crusher chambers (SCh Nº1 cavern), including the interpretation of geotechnical site investigation data and use of empirical, analytical and numerical methods to determine the appropriate permanent support to be considered for this cavern.

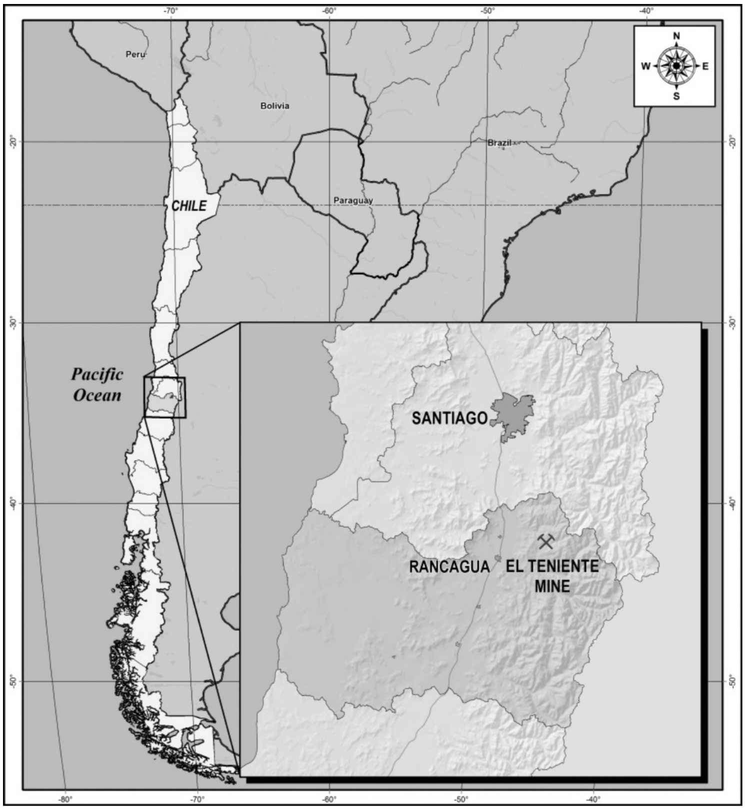


Figure 1 El Teniente mine location in relation to Santiago and Rancagua cities in the central part of Chile

2 Geotechnical characterization

Until the early 90's the Braden Pipe was considered an almost homogeneous body, composed by a concrete-like rock called Braden breccia and, in its perimeter, by a breccia containing coarser rock blocks, called Marginal Breccia (Pereira et al. 2003). However, the behavior observed at different sectors of the Braden Pipe indicated differences that could only be explained by the presence of different breccia types. Therefore, a comprehensive geological characterization of the Braden Breccia was developed in the past, which allowed a much more detailed zonation of the Braden Pipe and the definition of several breccia types (Floody 2000 & Karzulovic 2000). The main breccia types are the following:

a) Sericite Breccia – this breccia constitutes a majority of the pipe.

b) Chlorite Breccia – found primarily in the southern portion of the pipe.

c) Tourmaline Breccia – characterized by large clasts and vein-like occurrence.

d) Marginal Breccia – hard breccia at the boundary of the pipe.

For each of these breccias, there is variability in the size of the fragments or clasts and in the mineral constituents and alteration of the matrix cement. In the Braden Sericite Breccia, there appears to be an effect of the ratio of Sericite/Quartz content in the cement to the compressive strength of rock samples. Figure 2 represents a plan view containing the location of crusher cavern Nº1 and showing the different geotechnical units as interpreted from the available geological and geotechnical information from the site. The main geotechnical units are the Sericite Braden Breccia unit (BBS), Chlorite Braden Breccia unit (BBC), Tourmaline Braden Breccia unit (BBT) and the Dacitic Porphyry unit (PDAC).

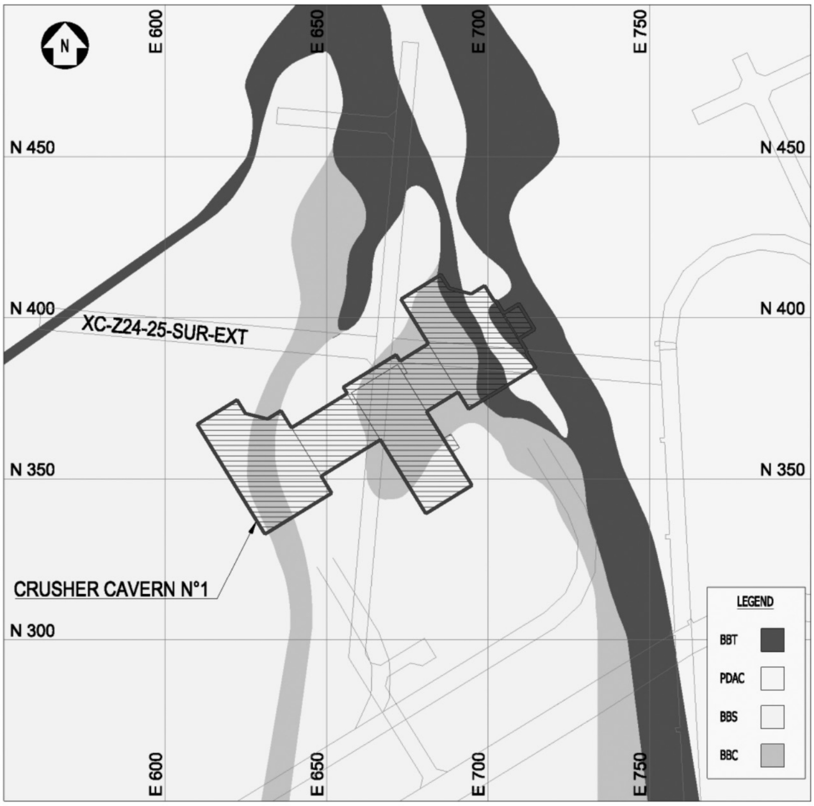


Figure 2 Plan view at mine level 1790 of the Crusher Chamber SCh Nº1 location, indicating the main geotechnical units as interpreted from available geotechnical information (taken from SRK, 2014)

In general, the BBS, BBC and BBT units are rock masses of good quality with a Bieniawski's RMR value larger than 70; for details about the Bieniaswki's classification system see Bieniaswki (1989). For example, Figure 3 shows a photograph of some representative cores of the main geotechnical units at the site location of SCh Nº1; solid and intact cores, few joints, low fracturing, a common characteristic of the BBS, BBC and BBT units which translates into good quality rock mass, can be observed in the photograph.

As part of the geotechnical characterization, a database with geotechnical information from site investigations (geotechnical boreholes) at El Teniente Mine was analyzed; this database was created and is maintained by VP-NNM (VCP 2010a and VCP 2010b). In particular, values of geotechnical parameters describing the quality of the rock mass, including Fracture Frequency (FF), Rock Quality Designation (RQD), Intact Rock Strength (IRS) and Bieniawski's Rock Mass Rating (RMRB).

Based on geotechnical window mapping of drifts and galleries close to the site location of the SCh Nº1, a characterization of the rock mass quality in terms of the Geological Strength Index (GSI) and Barton's Q-system values were revised (for details about these systems see, Hoek, 1994, Hoek & Brown 1997, Hoek et al. 2002; Barton et al. 1974; Grimstan and Barton 1993; Barton, 2002). The resulting range of these values, expected to be encountered during excavation of the SCh Nº1, is shown in Table 1.

a)

b)

c)

d)

Figure 3 Cores of the main geotechnical units at the site location of the SCh Nº1.a) BBS. b) BBC. c) BBT and d) PDAC

From a structural geology point of view, the site where the crusher cavern will be emplaced has been referred to as 'Brecha Braden Marginal' (or 'Braden Breccia Marginal Structural Domain'). Analysis of the available geological information has revealed the existence of three systems of minor faults and two joints sets. Table 2 summarizes the orientation of these structural systems.

The in-situ stress state considered for the design of the crusher cavern SCh Nº 1 was obtained from overcoring tests performed at XC-01-AS site Nº 5 (undercutting level 1880). Table 3 summarizes the in-situ stress field at crusher cavern location.

Values of strength and deformability for all the geotechnical units were computed according to the generalized Hoek-Brown failure criterion (Hoek et al. 2002; Hoek & Diederichs, 2006) and following some specific recommendations to the El Teniente mine by Diederichs (2013). The mechanical parameters were derived from laboratory unconfined, triaxial and tensile testing of rock samples and estimations of values of Geological Strength Index from geotechnical window mapping in the main access tunnel (TAP), drifts and galleries next to the SCh Nº1 location.

Table 1 Classification systems values of the rock mass at the SCh Nº1 location

UGTB	*RQD (%)*	*RMR_{B89}*	*Q'*	*GSI*
BBS	70 – 100 (80)	60 – 92 (72)	1.2 – 250 (14)	56 – 90 (69)
BBC	94 – 100 (98)	70 – 85 (77)	40 – 100 (70)	63 – 82 (72)
BBT	80 – 100 (90)	72 – 82 (75)	5 – 71 (23)	61 – 80 (73)
PDAC	79 – 100 (89)	N/I	N/I	65 – 86 (72)

( ): Mean values.
Q': modified Barton's Q-system (Jw/SRF = 1).
RMR_{B89}: Rock Mass Classification system (Bieniawski ,1989).
RQD: Rock Quality Designation (Deere, 1963).
GSI: Geological Strength Index (Hoek ,1994).
N/I: No available information.

Table 2 Structures at the site location of the SCh Nº1 (VCP, 2010b)

SETS	*Minor Faults*		*Joints*	
	Dip / DipDir	Nº data	Dip / DipDir	Nº data
S1	84° / 125°	12	75° / 324°	34
S2	83° / 035°	7	35° / 010°	21
S3	76° / 172°	6		

Table 3 In situ stress field representative of the site location of the SCh Nº1

Principal Stresses	*Magnitud (MPa)*	*Bearing (°)*	*Plunge (°)*
σ1	50.73	344.0	-7.8
σ2	33.11	75.5	-10.7
σ3	26.50	218.6	-76.7

Table 3 summarizes the mechanical parameters for the rock mass, for the three geotechnical units analyzed with the Hoek-Brown method. [In Table 4, m_i is the Hoek-Brown intact rock parameter; σ_{ci} is unconfined compressive strength of the intact rock; γ is the specific gravity of the intact rock; E_i is the modulus of deformation of the intact rock; GSI is the Geological Strength Index; m_b, s and a are Hoek-Brown rock mass parameters; and E_{RM} and ν are the deformation modulus and Poisson's ratio of the rock mass, respectively.

To calibrate and validate the stress field and rock mass properties some back-analyses were done to check if the behavior predicted using these properties agrees with the observed behavior. Two-dimensional plane-strain models were constructed for different sections with different geotechnical units and orientations, involving sections for which overbreak were measured. The models were developed using the finite element software Phase2 (Rocscience 2009), which allows analysis of excavations in plane-strain conditions.

Figure 5 shows the results from a finite element back-analysis of one of the sectors considered for the TAP tunnel in Chlorite Braden Breccia unit. The light gray zone in the roof indicates failure by tension and/or yielding, and the black curve shows the measured overbreak each 5 m along the tunnel axis in this particular sector. Different tunnel orientations within the same geotechnical unit were considered for this analysis.

These results indicate that the geomechanical properties of the different type of breccias presented in Table 3 are a good estimate of the rock mass properties for these types of massive rock.

Table 4. Summary of rock mass strength and deformability parameters for the different geotechnical units according to the generalized Hoek-Brown method —see Hoek et al., 2002; Hoek & Diederichs, 2006.

<table>
<tr><th rowspan="2">UGTB</th><th>γ</th><th>GSI</th><th>σ_{ci}</th><th rowspan="2">m_i</th><th>σ_t</th><th>Em</th><th rowspan="2">ν</th><th>c</th><th>φ</th></tr>
<tr><th>(KN/m³)</th><th>Mean value</th><th>(MPa)</th><th>(MPa)</th><th>(GPa)</th><th>(kPa)</th><th>(°)</th></tr>
<tr><td rowspan="2">BBS</td><td rowspan="2">25.9</td><td rowspan="2">70</td><td rowspan="2">81.1</td><td rowspan="2">11.0</td><td>0,768</td><td rowspan="2">29.31</td><td rowspan="2">0,20</td><td>7,336</td><td>34</td></tr>
<tr><td>0,384*</td><td>5,180*</td><td>33*</td></tr>
<tr><td rowspan="2">BBC</td><td rowspan="2">26.6</td><td rowspan="2">72</td><td rowspan="2">77.4</td><td rowspan="2">12.0</td><td>0,782</td><td rowspan="2">25.60</td><td rowspan="2">0,20</td><td>7,578</td><td>35</td></tr>
<tr><td>0,391*</td><td>5,350*</td><td>34*</td></tr>
<tr><td rowspan="2">BBT</td><td rowspan="2">25.4</td><td rowspan="2">70</td><td rowspan="2">100.0</td><td rowspan="2">8.0</td><td>1,302</td><td rowspan="2">23.01</td><td rowspan="2">0,20</td><td>7,448</td><td>33</td></tr>
<tr><td>0,651*</td><td>5,260</td><td>32*</td></tr>
<tr><td rowspan="2">PDAC</td><td rowspan="2">25.8</td><td rowspan="2">73</td><td rowspan="2">144.5</td><td rowspan="2">28.5</td><td>0,662</td><td rowspan="2">34.55</td><td rowspan="2">0,20</td><td>12,078</td><td>48</td></tr>
<tr><td>0,331*</td><td>8,500</td><td>43*</td></tr>
</table>

(*) Ubiquitous properties considers Jennings (1970) criterion with a k = 0.3.

3 Support requirements for the crusher cavern according to empirical methods

Figure 6 shows an isometric view for the crusher cavern that considers mainly the dumping chamber, apron feeder, crusher chamber, main silo, main feeder and lift.

Based on the large experience of excavation of tunnels and caverns in different rock units at El Teniente mine, using the traditional method of full face blasting an appropriate (temporary) support consisting in rockbolts, steel wire mesh and shotcrete were proposed for the cavern (SGM-I-011/2006, VCP, 2010c, among others).

A preliminary estimation of the quantity of permanent support to use during excavation was done using empirical methods. The methods considered were those described by Barton (1974), Palmström & Nilsen (2000), Unal (1983), Hoek (2007) and Hönish (1985), among others. These methods give guidelines for permanent support requirement based on several of the geotechnical indexes discussed earlier on, such as values of RQD, Q and RMR. Table 5 summarizes the characteristics of the recommended support for SCh Nº1 according to the above mentioned methods.

Due to the intrinsic limitations of the empirical methods (particularly in regard to the assumption of isotropy of stresses and rock mass continuity), these methods were used as a first step in selecting a support type for the SCH Nº1; the actual verification of the proposed support was carried out using tri-dimensional numerical models as described in the next sections, which among others, allowed incorporation of several geotechnical units existing in the rock mass and in situ stress field showed in Table 3.

The acceptability criterion for permanent support was established based on factors of safety with respect to failure (in compression) of the support. Based on types of supports used and suggested length spans from empirical methods, factor of safety of 2.0 for permanent support (for static loading and dry ground) were judged appropriate. In this regard, a literature survey did not reveal the existence of established rules for factors of safety to consider for cavern of large dimensions (as the case of the SCh Nº1). For example, Hoek

(2007), suggest 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 stabilize. Pariseau (2007) suggests that the load acting on the support for large excavation should not exceed half the value of the strength of the support material of (shotcrete or concrete) —i.e., this would mean considering a factor of safety of at least 2. For wedge and blocks failures in a large cavern design a factor of safety of 1.5 to 2.0 is commonly used as acceptability criteria (Hoek, 2007).

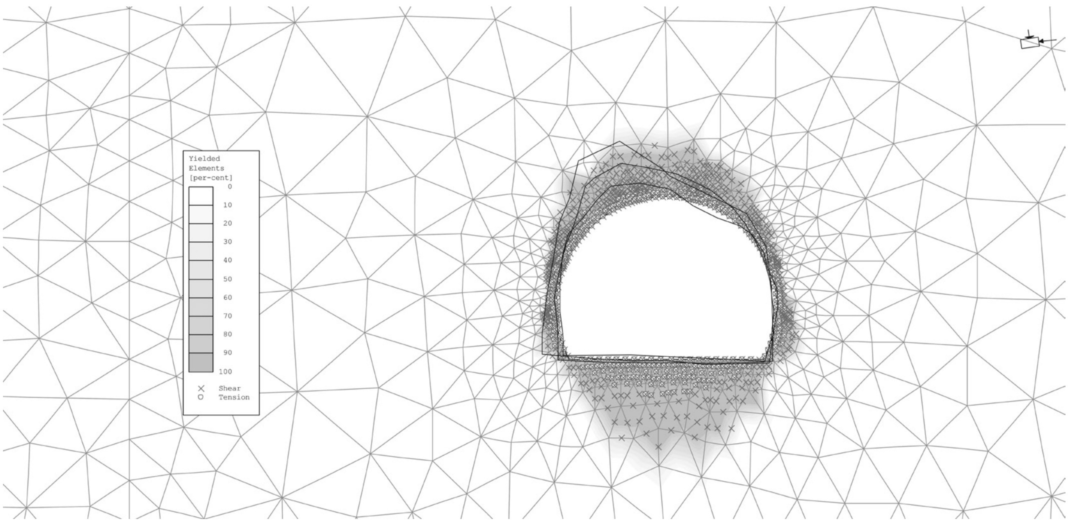


Figure 4 Results from a finite element back-analysis of one of the sectors considered for the TAP tunnel in BBT unit. The light gray zone surrounding the tunnel section indicates failure by tension and/or shear, and the blue curves show the measured overbreak each 5 m along the tunnel axis in this particular sector

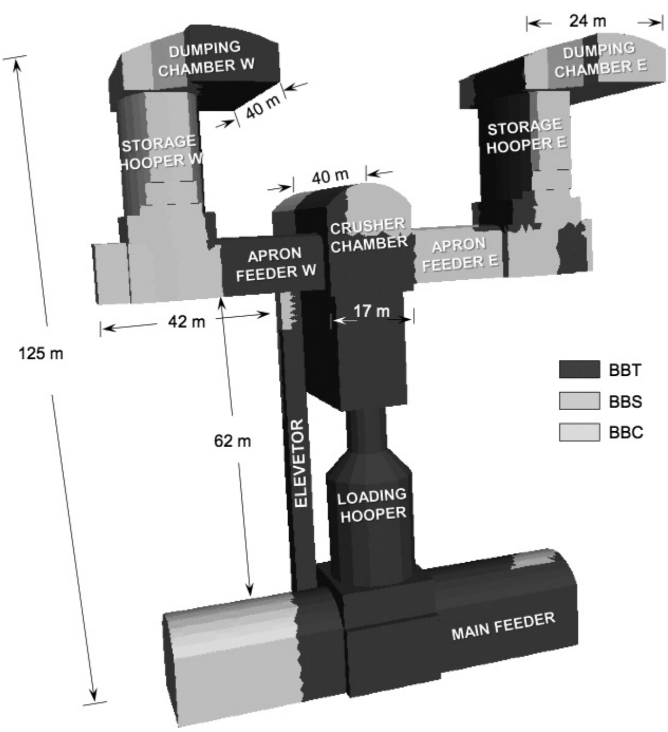


Figure 5 Infrastructure considered for the geomechanical analysis in relation with the main geotechnical units

Table 5. Summary of preliminary permanent support recommended for the SCh Nº1 as derived from application of empirical methods.

Excavation	B × H (m)	Sector	Barton (1974)			Palmstrom & Nilsen (2000)	Hoek (2007)	Unal (1983)		Hönisch (1985)	
			Pattern		Lc (m)			Lc (m)		Shotcrete Thickness (mm)	
			BBS	BBC		Lb (m)	Lb / Lc (m)	BBS	BBC	BBS	BBC
Dumping Chamber	24,3×8,8	Roof	1,3 x 1,3 to 1,7 x 1,7 m; Shotcrete 120 - 150 mm	1,7 x 1,7 to 2,1 x 2,1 m; Shotcrete 50 - 120 mm	7.5 – 8.1	5.8	5.6 / 9.7	4.1 – 14.2	6.3 – 11.2	100 - 150	100 a 150
		Walls			2.4 – 2.6	4.4	N/A			50 (min)	50 (min)
Storage Hooper	14,3×21,2	Walls	1,3 x 1,3 to 1,7 x 1,7 m; Shotcrete 120 - 150 mm	1,7 x 1,7 to 2,1 x 2,1 m; Shotcrete 50 - 90 mm	5.7 – 6.2	4.0	5.2 / 7.4	3.8 – 12.5	6.0 – 9.8	50 - 150	50 - 100
Apron Feeder	9,2×10,8	Roof	1,3 x 1,3 to 1,7 x 1,7 m; Shotcrete 90 - 120 mm	1,7 x 1,7 to 2,1 x 2,1 m; Shotcrete 40 - 90 mm	2.8 – 3.1	3.2	N/A	2.1 – 6.2	2.8 – 5.0	50 (min)	50 (min)
		Walls			2.9 – 3.2	3.0	3.6 / 3.8			50 - 100	50 (min)
Crusher Chamber	16,8×43,6	Roof	1,3 x 1,3 to 1,7 x 1,7 m; Shotcrete 150 - 250 mm	1,7 x 1,7 to 2,1 x 2,1 m; Shotcrete 90 - 120 mm	5.2 – 5.6	4.4	4.5 / 6.7	N/A	N/A	50 - 150	50 - 100
		Walls			11.7 – 12.7	5.3	8.5 / 15.3			150 - 200	150 - 200
Loading Hooper	17,0	Walls	1,3 x 1,3 to 1,7 x 1,7 m; Shotcrete 90 - 150 mm	1,7 x 1,7 to 2,1 x 2,1 m; Shotcrete 50 - 90 mm	5.2 – 5.7	4.6	4.6 / 6.8	3.1 – 10.0	4.5 – 7.9	50 - 150	50 - 100

B: Section Length. H: Section Height. Lb: Bolt Length. Lc: Cable Length.

4 Three-dimensional numerical analysis of the crusher cavern excavation

Three-dimensional models implemented in the finite difference software FLAC3D (Itasca 2007) were constructed for the main infrastructure of the SCh Nº1 (see Figure 6). The three-dimensional models incorporated only the permanent support (with characteristics described in the next section) and the proposed excavation advance, coinciding with the mining design excavation.

The purpose of this model was to account for the actual three-dimensional nature of the excavation problem; the model allowed wall displacements on the large excavation, extent of the plastic-failure zone around the walls of the large excavations, and the performance of the permanent support to be quantified —i.e., the verification of the acceptability criteria in terms of factor of safety described in Section 3. In general, major principal stress (s1) reaches 60 to 80 MPa in the upper part of crusher chamber and apron feeder (see Figure 7a). Unconfined stress (s3 < 4.0 MPa) are observed below of the floor of the dumping chamber (see Figure 7b). Also, a maximum displacement of 4 cm is observed in the floor dumping chamber after the excavation of the crusher chamber (see Figure 7c). Maximum displacements of 5 cm are observed in the intersection of the crusher chamber walls and apron feeder and intersection of loading hooper and main feeder (see Figure 7d).

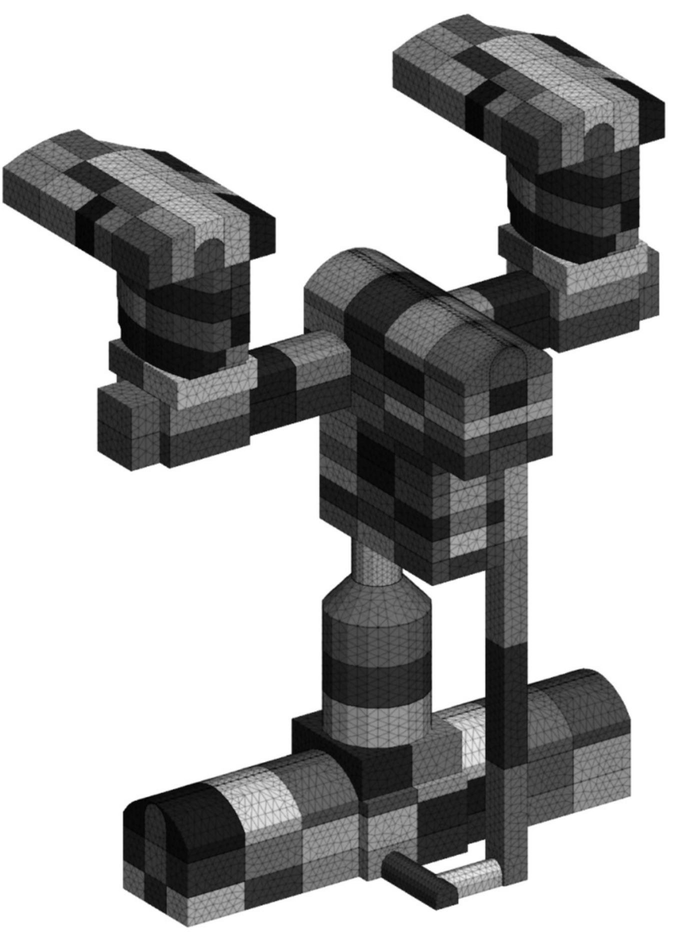
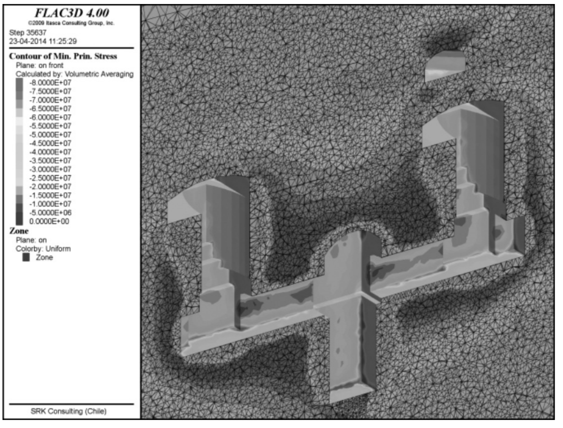


Figure 6 Three-dimensional numerical model of the crusher cavern. The figure shows the 93 advance intervals considered for the excavation in different colors. The model, which incorporates only permanent support, was constructed using the finite difference code FLAC3D —see Itasca (2007)

Analysis of results from these three-dimensional models allowed to conclude that the support (with characteristics described in the next section) satisfies the acceptability criterion —i.e., a factor of safety of 2.0 for permanent support. Figure 8a and 8b shown the results for the double cables installed in the roof of the crusher chamber and the final excavation of the model.

The values of loads resulting in permanent liners (i.e., the values of thrust, bending moment and shear force) were recorded for each of the large excavations analyzed. The values of support loading were plotted in capacity diagrams to verify that the factor of safety values were below admissible limits —for a discussion on the methodology involving verification of support using capacity diagrams, see Hoek et al. (2008); Carranza-Torres & Diederichs (2009). For example, Figure 8c represents capacity diagrams for a permanent support of thickness 0.3 m in the apron feeder roof for the final excavation of the model. In basically all the large excavations, loading in the proposed support analyzed with the capacity diagram approach was found to be within the admissible limits of factor of safety mentioned earlier on.

Finally, to verify the support recommended, a wedge/block analysis was performed based on the structural information provided in Table 2 using keyblock teory (Goodman & Shi, 1985) and the software Unwegde (Rocscience 2009). Figure 9 shows the application of key block theory to the dumping chamber roof. All the keyblocks in the roofs and walls for all the large excavations were verified.



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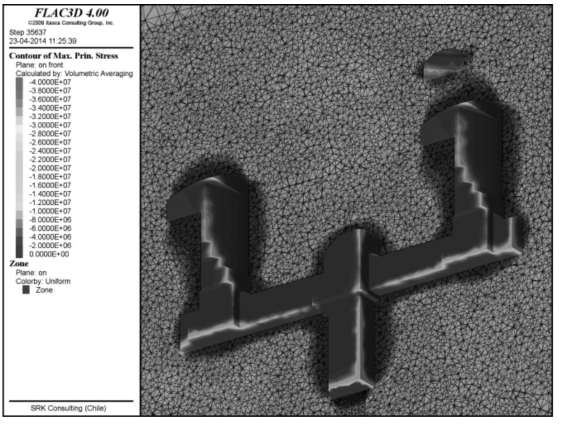

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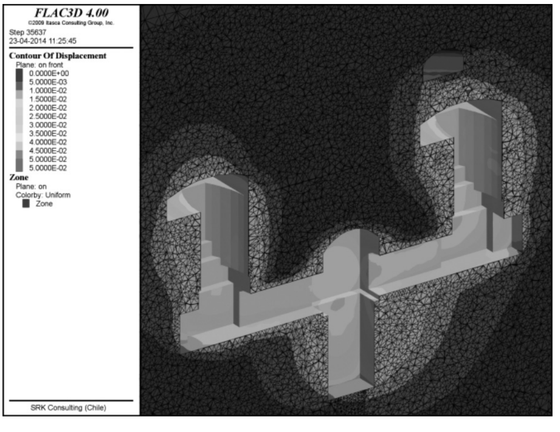

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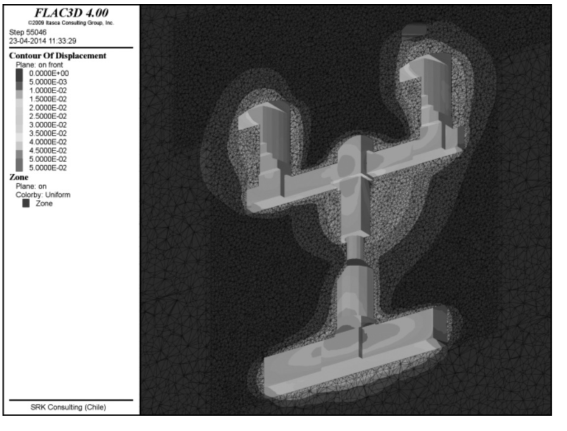

d)

Figure 7 Representation of the results in the model sliced by a cross section plane located at the midpoint of the apron feeder. Represented are: a) major principal stresses after crusher chamber excavation, b) minor principal stresses after crusher chamber excavation. c) displacements after crusher chamber excavation and d) displacements for the final excavation model

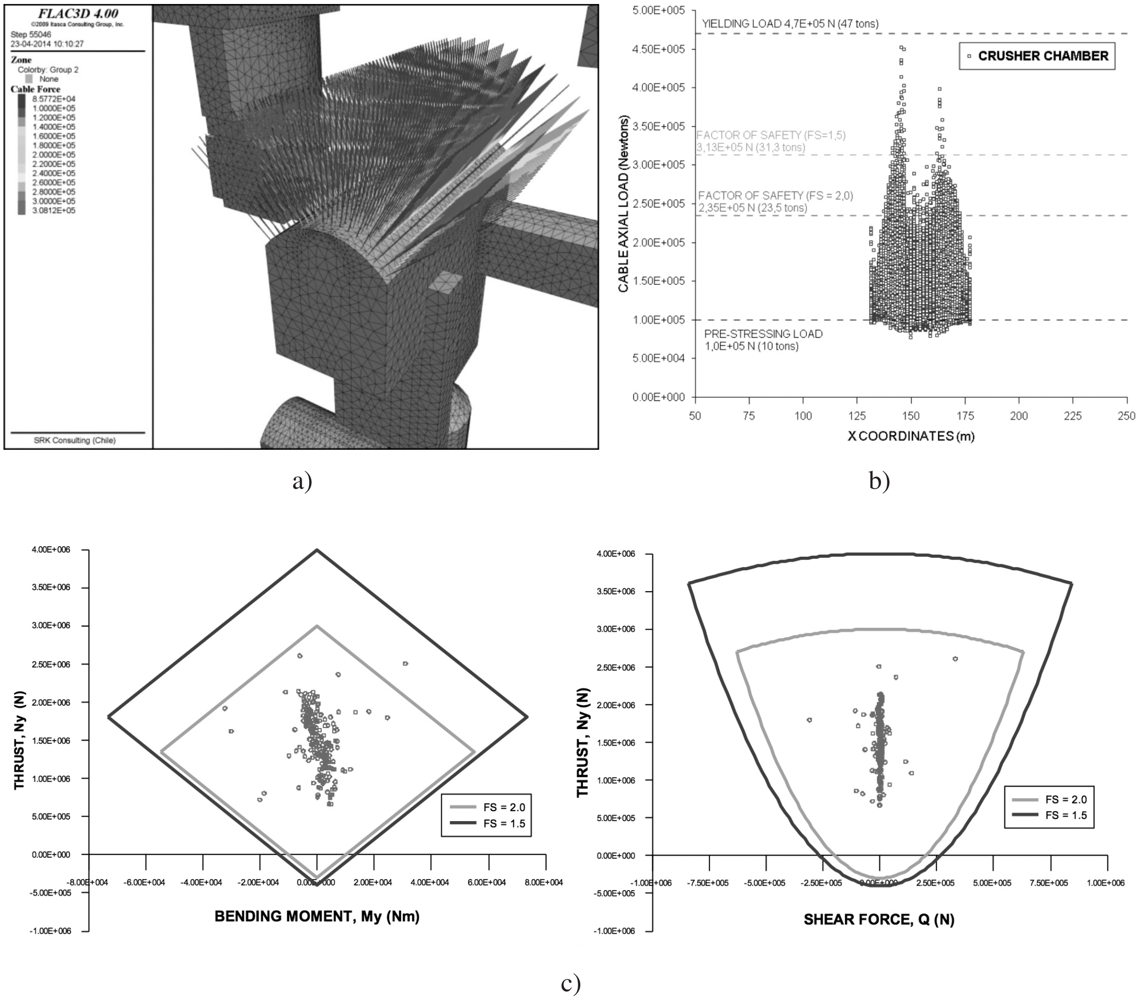

Figure 8 Support performance for some of the main large excavations. a) Axial force for cables in the crusher chamber roof at the end of excavation. b) Resulting axial force for cables installed in the crusher chamber at the end of excavation (yielding load, pre-stressing load and factors of safety of 1.5 and 2.0 also are shown). c) Capacity diagrams for shotcrete liner in apron feeder at the end of excavation

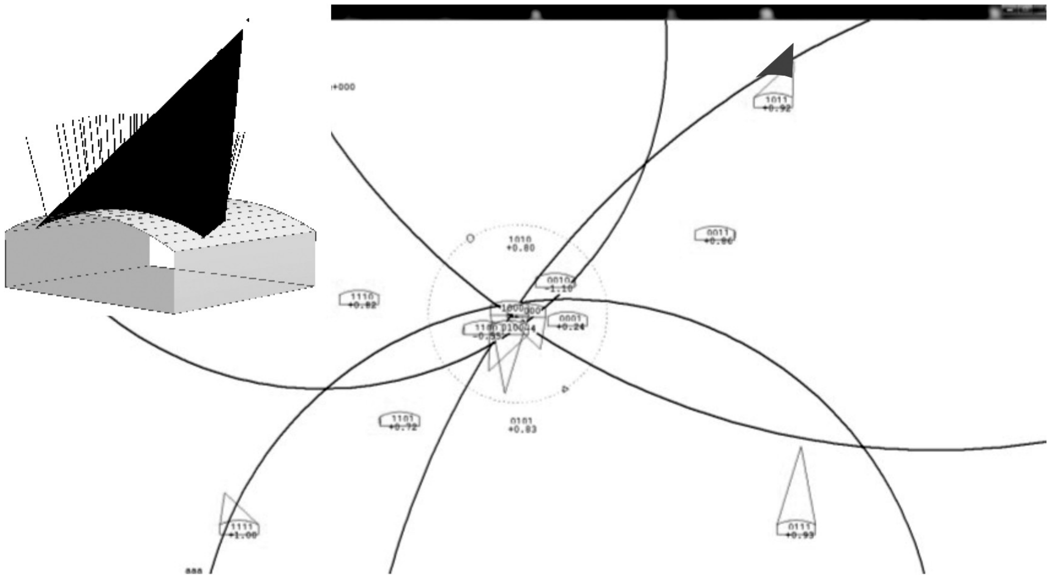

Figure 9 Dumping chamber section showing maximum removable blocks for each JP superimposed on the stereographic projection of the JPs. To the upper left, the analysis for the roof with Unwedge program to verify the support recommendations for the JP 1011 block (shaded in red)

5 Proposed crusher cavern support

Based on experience in design of large excavations support and on the application of empirical, analytical and numerical models described in previous sections, for the large excavations crossing the good quality rock mass units (BBS, BBC and BBT units), permanent support with the characteristics summarized in Table 6 were proposed. The temporary support consists mainly of rock bolts (and wire mesh) with quite uniform characteristics for most of the large excavations.

For the large excavations (dumping chamber, storage hooper, crusher chamber and apron feeder), in which high stress confinement in the rock mass could translate into ground instability, heavier permanent support proposed.

Table 6 Summary of permanent support proposed for the Crusher Cavern SCh Nº1

Excavation	B (m)	H (m)	Sector	Cables*		Shotcrete
				Pattern	Length (m)	
Dumping Chamber	24,3	8,8	Roof	1,0 x 1,0	10	H30 t = 300 mm
			Walls	2,0 x 2,0	8	
Storage Hooper	14,3	21,2	Walls	1,5 x 1,5	14	H30 t = 150 mm
Apron Feeder	9,2	10,8	Roof	1,0 x 1,0	14	H30 t = 200 mm
			Walls	1,5 x 1,5	12	
Crusher Chamber	16,8	43,6	Roof	1,0 x 1,0	15	H30 t = 300 mm
			Walls	1,5 x 1,5	15	
Loading Hooper	17	-	Walls	1,5 x 1,5	12	H30 t = 200 mm

B: Section Length. H: Section Height.

(*) All the cables are doubles single strand of f = 15.6 mm, additionally a steel wire mesh C443 was recommended.

6 Conclusions

This paper has described several aspects of the process of determining the permanent support for the large crusher cavern SCh Nº1 at the New Mine Level project at El Teniente mine. The crusher cavern is to be excavated in a rock mass of generally good quality (BBS, BBC and BBT units), in a medium to high stress environment.

The support recommended for crusher cavern, as described in this paper is not definitive and will have to be optimized once construction techniques are selected in a future phase of design of the underground infrastructure.

The characteristics of the support recommended for the crusher cavern are based on the assumption of the rock mass is dry and that dynamic loading on permanent liner (e.g., due to blasting during future caving operations) is neglected. Also, a sensitivity analysis for Hoek-Browm parameters, ubiquitous model and an increment of the in situ stress was considered and the proposed support was found to be within the admissible limits of factor of safety mentioned earlier on.

In terms of permanent support, considering the critical importance of continuous operation of the crusher cavern for at least 50 years, a permanent concrete liner of at least 0.3 meters thickness was judged appropriate. This permanent support thickness was established based on current practice used in civil engineering tunnel projects, and not based on the empirical methods described above.

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