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Estimation of support requirement for large diameter ventilation shaft at Chuquicamata underground mine in Chile

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ABSTRACT: The Chuquicamata Underground Mine project in the Atacama Desert in northern Chile is one of the largest planned mining projects in the world to use the method of block caving with macro-blocks option to mine copper ore.

VP-CODELCO (Vice-President Projects Office of the National Copper Corporation of Chile) is currently developing a feasibility engineering evaluation of excavated infrastructure for the project. The evaluation considers, among others, the construction of two ventilation shafts of internal diameter 11 meters and approximate depth of 970 meters. A geo-mechanical study has been carried out to evaluate the stability of one of these shafts and to provide recommendations about support requirement. As part of this study, empirical methods, confinement-convergence analytical models, and two-dimensional and three-dimensional continuum models have been developed and applied to evaluate the influence of the stresses and existing geological features, such as the presence of two major shear zones and different lithological units, on the mechanical response of the excavation. This paper introduces general aspects of the Chuquicamata Underground Mine project and discusses in particular geo-mechanical analyses carried out to evaluate stability and support requirement for the large diameter ventilation shaft.

Subject: Analysis techniques and design methods

Keywords: mine design, rock support, stability analysis, numerical modeling

1 INTRODUCTION

The Chuquicamata Underground Mine project is located in the Atacama Desert in northern Chile (see Figure 1).

The mining project contemplates using the method of block caving with macro-blocks caving option to mine copper ore; the project is to become one of the largest underground caving mining operations in the world. VP CODELCO

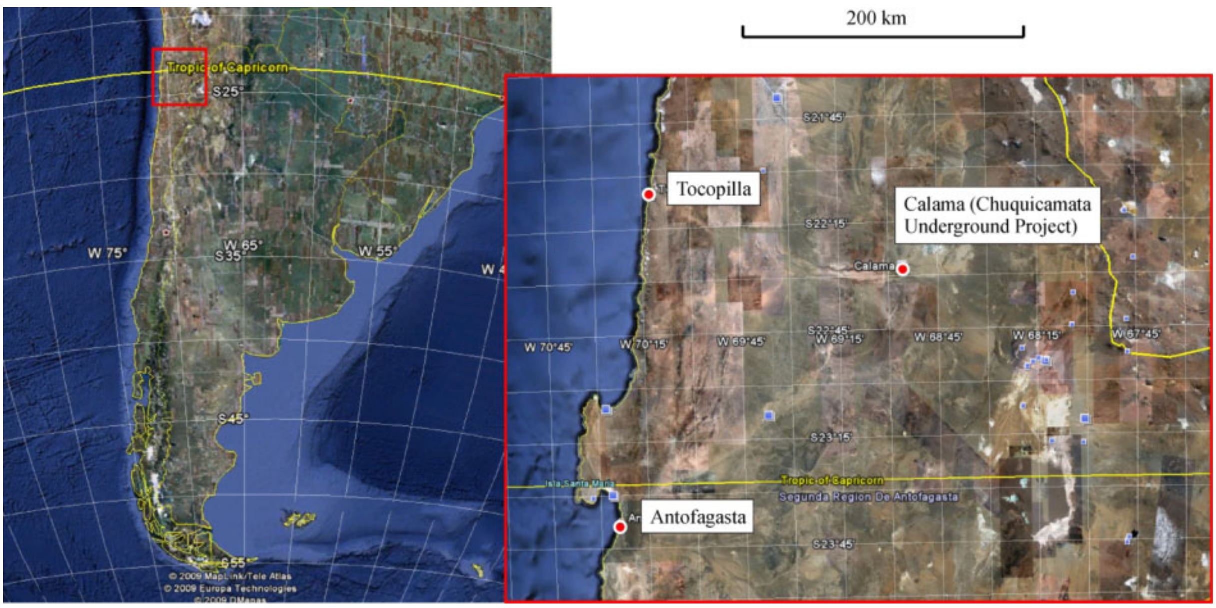

Figure 1. Chuquicamata mine location in relation to Antofagasta and Calama cities in northern Chile.

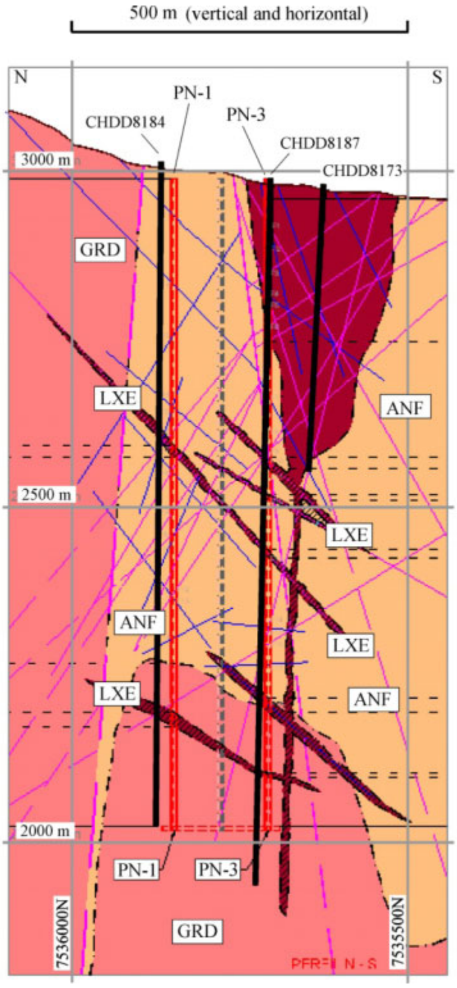

Figure 2. Vertical profile (with orientation North-South) containing the axis of shaft PN-1, indicating existing geotechnical units as interpreted from available geotechnical information.

(Vice-President Office of the National Copper Corporation of Chile) is finishing a pre-feasibility engineering evaluation of the project, which considers the construction and operation of at least two macro-block 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, there are two large air ventilation shafts, designated as PN-1 and PN-3 shafts. These ventilation shafts are required to have an internal diameter of 11 meters and a depth of approximately 970 meters. The ventilation shafts are also required to have a permanent liner that will guarantee the safe and continued operation of the shafts 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 shafts (PN-1 shaft), including the interpretation of geotechnical site investigation data and use of empirical, analytical and numerical methods to determine the appropriate temporary and permanent support to be considered for the construction and operation of the shaft.

2 GEOTECHNICAL CHARACTERIZATION

Figure 2 represents a North-South cross-section containing the axis of shaft PN-1 and showing the different geotechnical units as interpreted from the available geological and

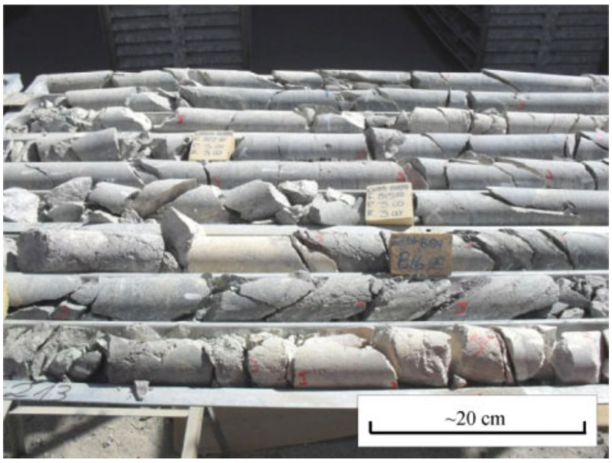

Figure 3. Core showing the Structurally Lixiviated unit (LXE) at depths 814-to-818 meters as recovered from perforation CHDD8184 – see Figure 2.

geotechnical information for the site. The main geotechnical units are the Granodiorite unit (GRD), Amphibolite unit (ANF) and Structurally Lixiviated unit (LXE) – the designations CHDD8184, CHDD8187 and CHDD8173 in Figure 2 correspond to boreholes used to generate the cross section.

In general, the GRD and ANF units are rock masses of good to very good quality with a Laubscher's RMRL value larger than 60, while the LXE unit, which is associated to the presence of structural faulting, is a rock mass of poor to very poor quality – for details about the Laubscher's classification system see Laubscher (1977) and Laubscher (1984). For example, Figure 3 shows a photograph of core of the LXE unit at a depth of approximately 800 meters; intense fracturing, a common characteristic of the LXE unit which translates into core disintegration, can be observed in the photograph.

Based on core recovered from borehole CHDD8184 (which coincides approximately with the axis of shaft PN-1 – see Figure 2), a characterization of the rock mass quality in terms of the Geological Strength Index (GSI) was carried out (for details about the GSI system see, for example, Hoek & Brown 1997 and Hoek et al. 2002). The resulting distribution of GSI values with depth, expected to be encountered during excavation of the shaft PN-1, is shown in the diagram of Figure 4. The different points in the diagram represent estimated values of GSI for the three different geotechnical units identified in the borehole CHDD8184. Average values of GSI in the order of 50 are characteristic of GRD and ANF units, while values of GSI in the order of 30 are characteristic of the two LXE units expected to be encountered by the shaft at approximate depths of 400 and 800 meters – see Figure 2.

As part of the geotechnical characterization, a database with geotechnical information from site investigations at Chuquicamata Mine was analyzed – this database was created and is maintained by VP-CODELCO. 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), Laubscher's Rock Mass Rating (RMRL), and Barton's Q-system values were revised – for information about these systems see Barton et al. (1974); Bieniawski (1989); Hoek et al. (1995).

Figure 5 shows the distribution of RQD values plotted against the corresponding values of FF obtained from analysis of borehole CHDD8184 as interpreted from borehole televiewers and recovered core. The distribution of measured

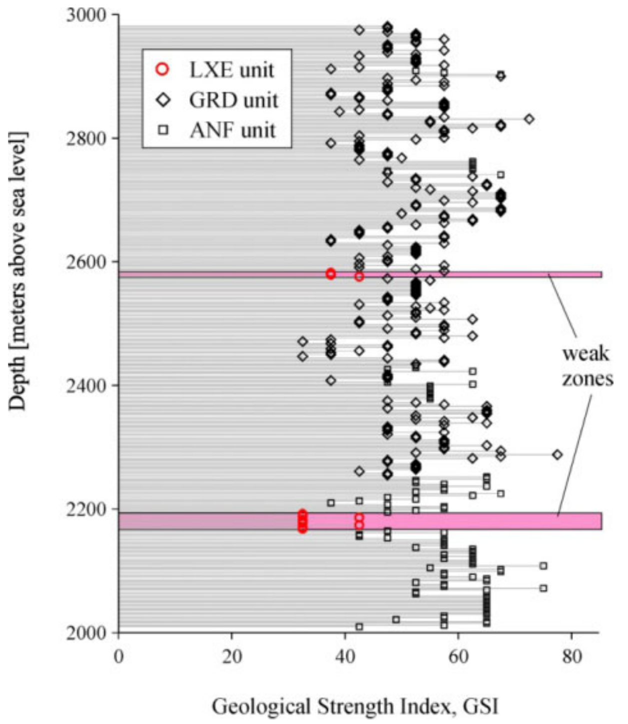

Figure 4. Distribution of the Geological Strength Index (GSI) as a function of depth, as interpreted from borehole CHDD8184 which coincides approximately with the axis of shaft PN-1.

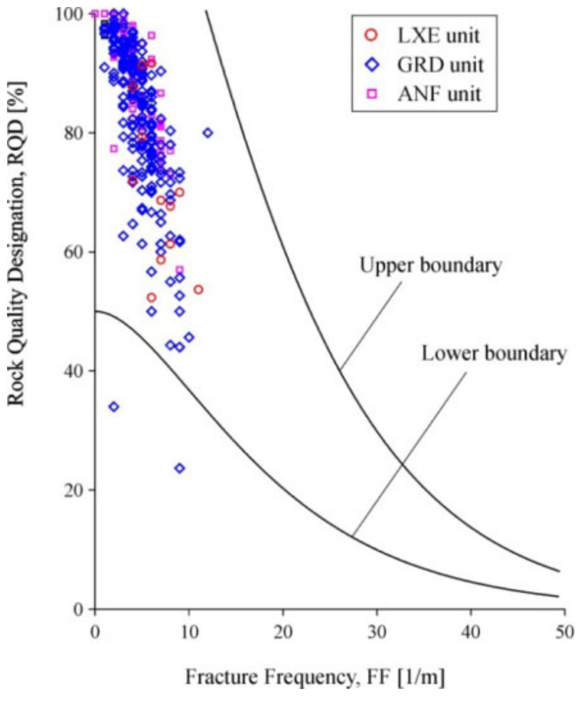

Figure 5. Relationship between Rock Quality Designation (RQD) and Fracture Frequency (FF) values for borehole CHDD8184.

values appear to be within the limits indicated in Figure 5 (inferred from the analysis by Priest & Hudson 1976) and regarded as 'normal' for a fractured rock mass.

From a structural geology point of view, the site where the shafts will be emplaced has been referred to as 'Dominio Strutural Piques' (or 'Shaft Structural Domain'). Analysis of the available geological information has revealed the existence of four systems of faults (referred to as VIF and FT, meaning 'Very Important Fault' and 'Fault Traces', respectively) and two family of joints. Table 1 summarizes the orientation of these structural systems – also Figure 2 indicates the traces of these structural faults on the cross section.

Table 1. Summary of fault and joint orientations for the 'Dominio Strutural Piques' (or 'Shaft Structural Domain') expected to be encountered during construction of shaft PN-1.

System	Faults		Joints	
	Dip angle*	Dip direction*	Dip angle*	Dip direction*
S1	80° ± 4°	151° ± 5°	62° ± 12°	334° ± 43°
S2	61° ± 8°	314° ± 11°	68° ± 14°	149° ± 15°
S3	66° ± 6°	358° ± 19°		
S4	59° ± 4°	193° ± 5°		

* Mean value and standard deviation

The in-situ stress state considered for the design of the shaft was obtained from a stress field model developed by Board & Poeck (2009) using three-dimensional numerical modeling techniques and results from thirteen over-coring tests performed at level 1841 m (Figure 2). The stress model by Board & Poeck (2009) considers evolution of in-situ stresses with development of the caving operations at the site; so for the design of the shafts, the stress state at two different stages in the mining development were considered; the first stage corresponded to construction of the shafts, expected to be at year 2013, while the second stage corresponded to shaft post construction and caving extraction approximately 20 years later, at roughly half the design life of the shafts, at year 2031. This last stage accounted also for the ground subsidence that is expected during the first 20 years of caving operations. Table 2 summarizes the in-situ stress field at different depths corresponding to the two conditions discussed above.

Table 2. In-situ stress components derived from the existing stress-field model, which were considered in the mechanical analysis of excavation of shaft PN-1.

Phase	Depth [m]	Stress component [MPa]					
		σ_{xx}	σ_{yy}	σ_{zz}	σ_{xy}	σ_{yz}	σ_{xz}
Initial (during construction; ~year 2013)	100	10.60	10.80	2.60	0.00	0.00	0.00
	200	11.40	11.70	5.20	0.00	0.00	0.00
	500	14.00	14.30	13.00	0.00	0.00	0.00
	750	16.10	16.40	19.50	0.00	0.00	0.00
	850	17.00	17.30	22.10	0.00	0.00	0.00
Subsidence (post-construction; ~year 2031)	100	1.94	7.81	1.96	-3.17	-0.64	1.24
	200	3.19	9.57	5.23	-3.21	-0.59	0.81
	500	7.67	12.44	11.81	-2.14	-0.55	0.41
	750	12.64	15.39	18.28	-1.03	-0.41	0.24
	850	15.25	16.90	21.50	-0.45	-0.32	0.19

Values of strength and deformability for the three geotechnical units were computed according to the generalized Hoek-Brown failure criterion (Hoek et al. 2002; Hoek & Diederichs 2006). The mechanical parameters were derived from laboratory unconfined and triaxial testing of rock samples and estimations of values of Geological Strength Index for borehole CHDD8184, i.e., for a scan line coinciding approximately with the axis of the shaft. Figure 6 represents minor and major principal stresses at failure from compression tests results (unconfined and triaxial) of intact rock specimens, together with the derived failure envelopes. Table 3 summarizes the mechanical parameters for the rock mass, for the three geotechnical units as analyzed with the Hoek-Brown method. [In Table 3, m_i is the Hoek-Brown intact rock parameter; σ_{ci} is unconfined compressive strength of the intact rock; G_s 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.]

3 SUPPORT REQUIREMENTS FOR THE SHAFT ACCORDING TO EMPIRICAL METHODS

Based on experience of excavation of tunnels and caverns in rock units at Chuquicamata mine, excavation using traditional method of full face blasting and temporary support consisting in rock bolts and cables, and use of shotcrete when encountering the low quality rock mass (LXE unit) were judged appropriate for the shaft.

A preliminary estimation of the quantity of temporary support to use during excavation was done using empirical methods. The methods considered were those described by Merritt (1972), Merritt & Baecher (1981), Palmström & Nilsen (2000), Unal (1983) and Bieniawski (1993). These methods give guidelines for temporary support requirement (in the latter case also unsupported span and unsupported time) based on several of the geotechnical indexes discussed earlier on, such as values of RQD, Q and RMR. Table 4 summarizes the characteristics of the recommended support for shaft PN-1 according to the above mentioned methods.

A relevant observation from application of the empirical methods summarized in Table 4 is that according to Bieniawski (1993), the shaft could be excavated with an unsupported span of approximately 3 meters – the 3 meters of unsupported span correspond to the length above the shaft front (or shaft base) and is coincident with the excavation advance in each blasting cycle.

In terms of permanent support, considering the critical importance of continuous operation of the shaft for at least 50 years, a permanent concrete liner of at least 0.5 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).

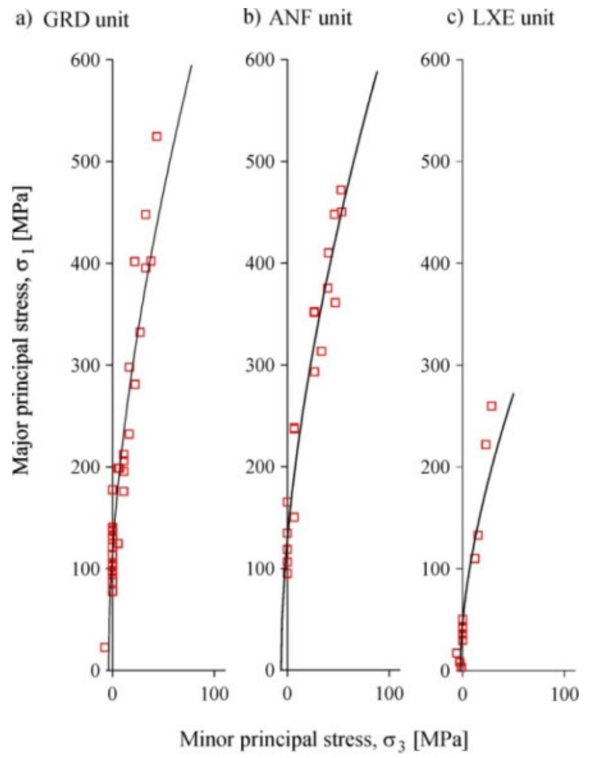

Figure 6. Shear strength failure envelopes for intact rock specimens for a) Granodiorite unit (GRD); b) Amphibolite unit (ANF); and c) Structurally Lixiviated unit (LXE) as obtained from unconfined and triaxial compression testing.

4 SUPPORT REQUIREMENTS FOR THE SHAFT ACCORDING TO ANALYTICAL METHODS

The convergence-confinement method of tunnel design (see for example, AFTES 1978) was applied to analyze the support requirements with particular regard to the supporting effect of the excavation front (i.e., the shaft base). The dimensioning of support according to this method is based on the construction of a longitudinal deformation profile (or LDP), a ground reaction curve (GRC) and a characteristic curve for the support (or SCC). Details of the implementation of the convergence- confinement method, including equations to apply in the case of tunnels excavated in rock masses that satisfy the Hoek-Brown failure criterion are described in Carranza-Torres & Fairhurst (2000) – this reference was taken as a basis for the application of the method in shaft PN-1.

The most relevant aspects of the application of the convergence-confinement method to the shaft are described below.

The geometry of the shaft was assumed circular with a radius of 6 meters. For each of the shaft sections analyzed, an average value of far-field stresses according to Table 2 was considered. Also, for construction of the the LDP and the GRC, the mechanical properties of rock mass listed in Table 3 were considered.

A total of five shaft sections corresponding to levels 2,900 meters (above mean sea level), or depth 100 meters below the ground; 2,800 m (or depth 200 m); 2,500 m (or depth 500 m); 2,250 m (or depth 750 m); and 2,150 m (or depth 850 m) were analyzed. These sections were selected so as to cover the

Table 3. 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.

Abbr.	Geotechnical unit	m_i	σ_{ci} [MPa]	G_s	E_i [GPa]	GSI	m_b	s	a	E_{RM} [GPa]	ν
GRD	Granodiorite	28.2	115.6	2.60	55.4	40-50	2.055	0.00065	0.5081	5.870	0.26
ANF	Amphibolite	20.7	128.1	2.87	68.0	55-65	3.081	0.00480	0.5028	18.500	0.22
LXE	Structural Leach	20.3	46.3	2.65	29.2	30-40	0.919	0.00017	0.5159	1.670	0.28

advance of the shaft through the three geotechnical units at various depths below the ground surface (see Figure 2).

An excavation advance of 3 meters without temporary support, and 20 meters without permanent support were considered in the construction of the LDP, GRC and SCC. For the temporary support, three situations were assumed, namely: a) rock bolts only; b) shotcrete only; and c) rock bolts and shotcrete – within these situations, various sizes and thicknesses of support were evaluated.

The acceptability criterion for temporary and 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 unsupported time and length spans from empirical methods (see Section 3), factors of safety of 1.5 and 3.0 for temporary and 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 shafts of large diameter (as the case of the shaft PN-1). For example, Obert et al. (1960), suggest factor of safety between 2 and 4 in compression and between 4 and 8 in unlined shafts (these factors of safety refer to the ratio of maximum stress to strength in the rock mass). Pariseau (2007) suggest that the load acting on the support 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.

The application of the convergence-confinement method to the five shaft sections discussed earlier on and the various support configurations, allowed selection of a typical support configuration to be used as temporary and permanent support for shaft PN-1 – the characteristics of this support will be described later on in Section 7. When the proposed support was analyzed with the convergence-confinement method, the acceptability criterion in terms of factor of safety was satisfied for all sections analyzed. For example, Figure 7 summarizes the rock support interaction analysis for temporary support for one of the five sections analyzed (this corresponds to the shaft section at level 2,500 m, or depth 500 m). In the figure, ratios of resulting support pressure (p_s^D) and allowable pressure on the support (p_s^{max}) are indicated to satisfy the

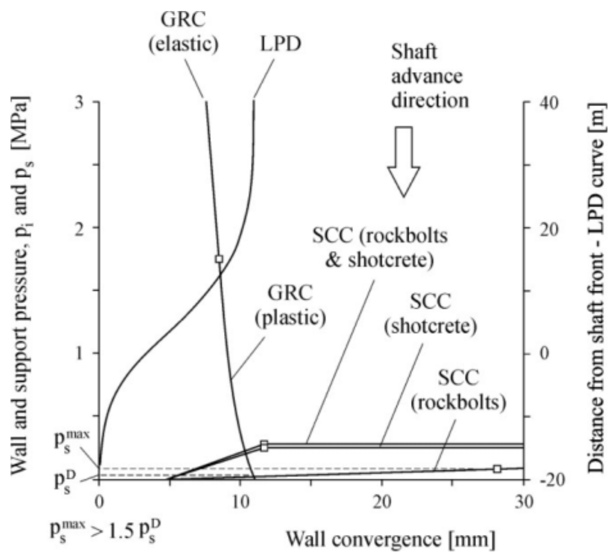

Figure 7. Application of the convergence-confinement method for shaft section PN-1 in the Granodiorite unit (GRD). The diagram represents the Longitudinal Displacement Profile (LDP), the Ground Reaction Curve (GRC) and different Support Characteristic Curves (SCC) considered – see Carranza-Torres & Fairhurst (2000) for equations used and construction of the diagram.

Table 4. Summary of temporary support recommended for shaft PN-1 as derived from application of empirical methods.

Empirical method	Recommended type of support			Length of rockbolt (L_b) or cable (L_c)
Merritt (1972)	Geotechnical unit LXE (RQD=64%)		Rockbolt pattern spacing 1.2 to 1.8 m	
	Geotechnical unit GRD (RQD=86%)		No required	
	Geotechnical unit ANF (RQD=94%)		No required	
Merritt (1981)	LXE	Q_{min} = 2.4 Q_{avg} = 2.8 Q_{max} = 3.2	Rockbolt pattern spacing 1.3 to 1.3 Steel wire mesh	L_b = 3.125 m
	ANF	Q_{min} = 11.6-15.0 Q_{avg} = 10.2-17.3 Q_{max} = 26.9	Rockbolt pattern spacing 2.2 to 2.2 Steel wire mesh	
	GRD	Q_{min} = 6.0 Q_{avg} = 10.2 Q_{max} = 17.3	Rockbolt pattern spacing 1.7 to 1.7 Steel wire mesh	
Palmström & Nielsen (2000)	LXE (highly fractured regions)			L_b = 6 to 7 m
	GRD and ANF LXE (lowly fractured regions)			L_b = 3 to 5 m
Unal (1983)	LXE	RMR: 54-56	Pattern: 1,4 x 1,4 m	L_c = 8 m
Bieniawski (1993)	Geotechnical unit	Unsupported span	Unsupported time	
	LXE (RMR=55)	2.7 to 3 m	1,000 hours	
	GRD (RMR=65)	3.1 m	10,000 hours	
	ANF (RMR=70)	3.5 m	10,000 hours	

prescribed (minimum) factor of safety for all three support types combinations.

Due to the intrinsic limitations of the convergence confinement method (particularly in regard to the assumption of isotropy of stresses and rock mass continuity), the method was used as a first step in selecting a support type for the shaft; the actual verification of the proposed support was carried out using numerical models as described in the next sections, which among others, allowed incorporation of geological structures existing in the rock mass.

5 TWO-DIMENSIONAL NUMERICAL ANALYSIS OF THE SHAFT EXCAVATION

Two-dimensional plane-strain models were constructed for five different sections of shaft at similar positions as those on which the convergence-confinement method was applied. The models were developed using the finite element software Phase2 (Rocscience 2009), which allows analysis of excavations in plane-strain or axi-symmetric conditions.

The purpose of these numerical models was to incorporate the major geological structures (the faults indicated in Table 1) and the non-hydrostatic in-situ stress state prior to excavation (see Table 2), and to evaluate the influence of these conditions on the performance of support (e.g., the development of tension on the support due to bending). In this regard, the numerical models incorporated explicitly only major subvertical structures (structures with dip angle larger than 60 degrees). Table 5 lists the most relevant input parameters for the five sections analyzed with the two dimensional numerical models.

The presence of the excavation front (or shaft base) was accounted for by an scheme of stress relaxation prior to installation of support, following current numerical modeling practice. In order to determine the amount of relaxation to consider prior to support installation, longitudinal displacement profiles were constructed using axi-symmetric (Phase2) models. Figure 8 represents relationships of shaft wall displacement and distance with respect to the shaft base for the five sections analyzed. From the curves in Figure 8, the values of stresses prior to installing the support in the Phase2 models were derived – details of the application of the relaxation scheme using the longitudinal displacement profiles can be found in the Phase2 software documentation (Rocscience 2009).

Figure 9 shows the two-dimensional plane-strain model corresponding to the section at level 2,500 m (depth 500 m) excavated in unit ANF – note that the traces of the subvertical major structures are visible in the model. The different colors in the representation of Figure 9 are contours values of resulting displacements after excavation and installation of temporary and permanent support. The values of maximum wall displacements and strain (computed as the ratio of dis placement and shaft radius) obtained with the Phase2 models are summarized in the last two columns in Table 5.

The values of loads resulting in temporary and permanent liners (i.e., the values of thrust, bending moment and shear force) were recorded for each of the five sections analyzed and for the two in-situ stress configurations summarized in Table 2 (these corresponds to the years 2013 and 2031 respectively, as discussed in Section 2). 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

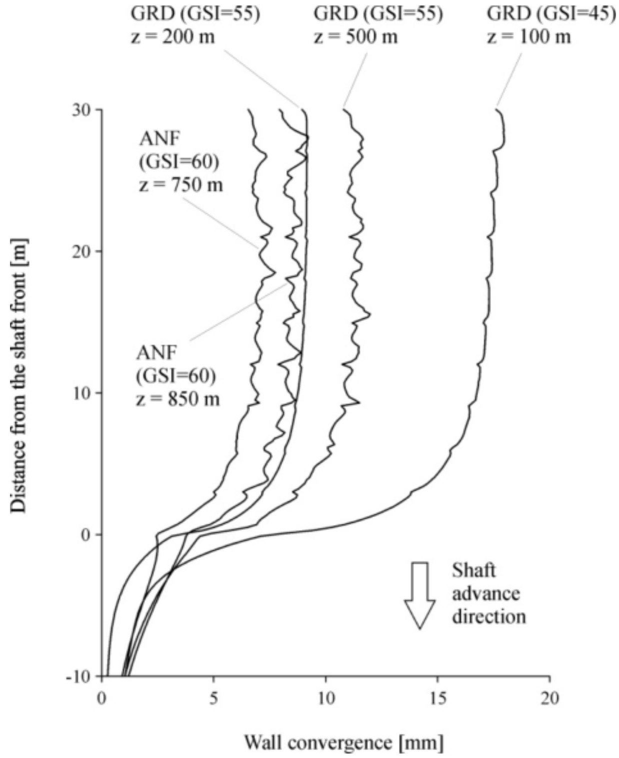

Figure 8. Longitudinal displacement profiles obtained from Phase2 axi-symmetric models for the five different conditions of shaft depth and geotechnical units expected to be encountered during excavation.

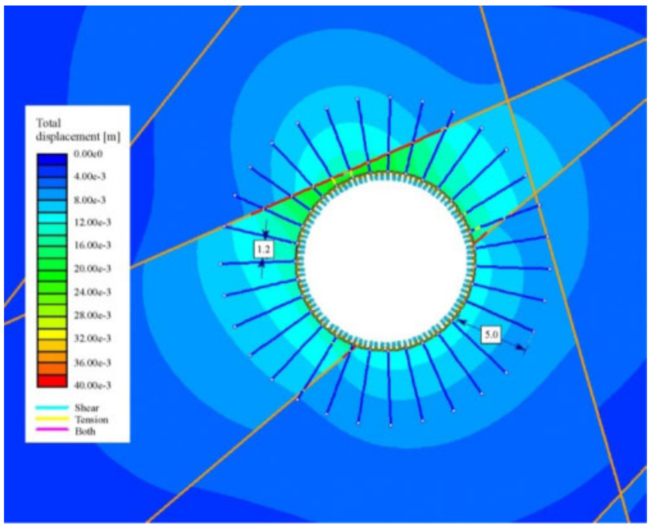

Figure 9. Resulting contours of displacements from a Phase2 model for level 2,500 m (depth 500 m), considering a combination of rock bolts 5 m long, spaced 1.2 meters in the circumferential direction, and 1.0 meter in the shaft axial direction, and with a permanent concrete liner of thickness 0.5 m.

on the methodology involving verification of support using capacity diagrams, see Hoek et al. (2008); Carranza-Torres & Diederichs (2009). For example, Figure 10 represents capacity diagrams for a permanent support of thickness 0.5 m for the initial operation condition at year 2013 (Figures 10a and 10b); and and for a later operation condition at year 2031 (Figures 10c and 10d). In basically all cases, 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.

Table 5. Position and stress state considered for shaft sections analyzed with two-dimensional models – the table also includes maximum values of displacements obtained with the Phase2 models.

Level (meters)	Geotechnical Unit	Major principal stress [MPa]	Minor principal stress [MPa]	Maximum displacements [cm]	Strain (radial displacement / radius) (%)
2900	GRD (GSI=45)	15 - 20	0 - 2	6.0	1.0
2800	GRD (GSI=55)	20 - 25	2 - 5	2.0	0.5
2500	GRD (GSI=55)	25 - 27	4 - 8	2.5	1.2
2250	ANF (GSI=60)	26 - 28	7 - 10	1.0	0.3
2150	ANF (GSI=60)	27 - 30	8 - 12	0.6	0.2

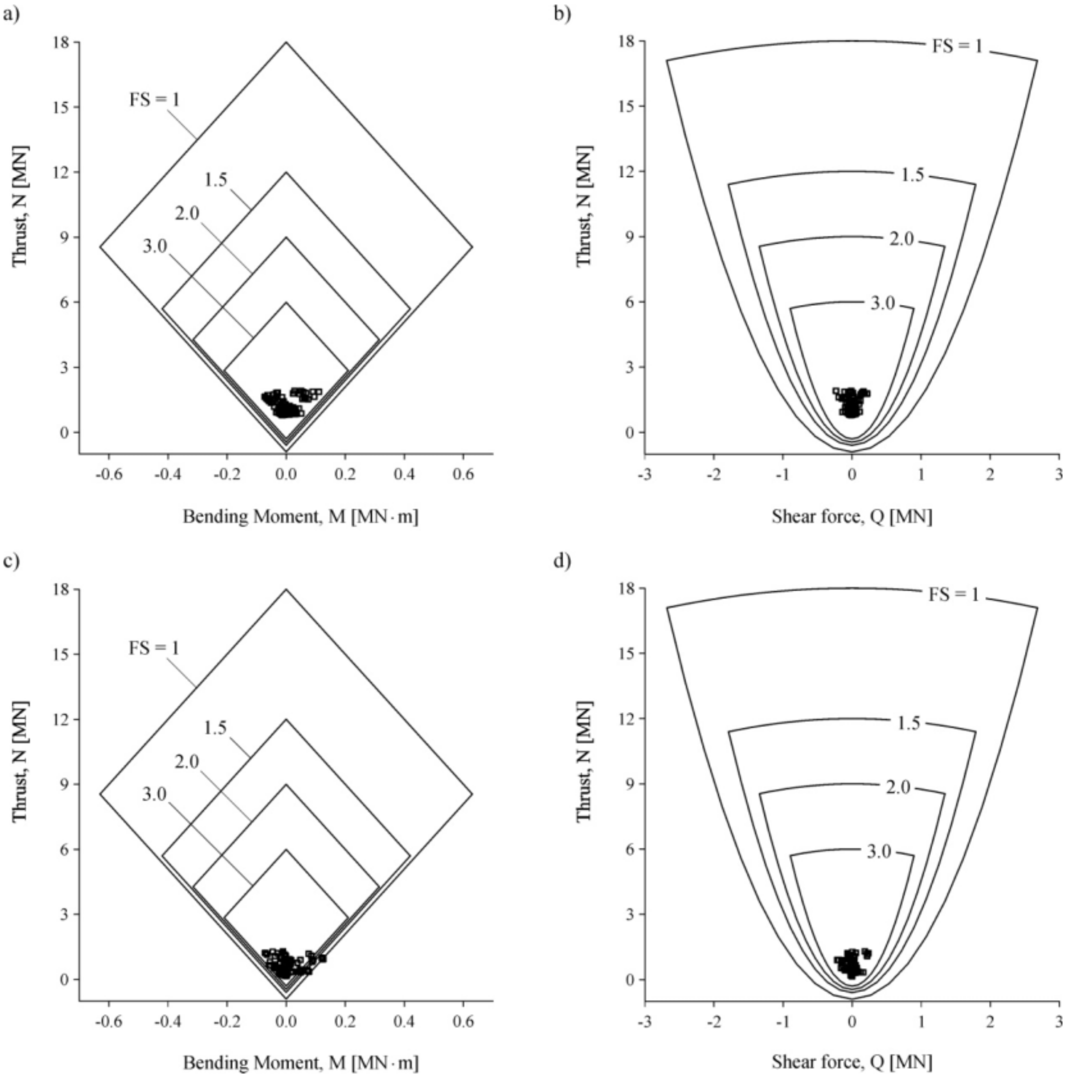

Figure 10. Capacity diagrams and support loading obtained from the Phase2 models corresponding to level 2,800 m (depth 200 m) considering a permanent concrete liner 0.5 m thick. Diagrams (a) and (b) correspond to the initial operation condition of the shaft at year 2013, while diagrams (c) and (d) correspond to the later operation condition at year 2031.

6 THREE-DIMENSIONAL NUMERICAL ANALYSIS OF THE SHAFT EXCAVATION

Three-dimensional models implemented in the finite difference software FLAC3D (Itasca 2007) were constructed for what are considered to be critical sections of the shaft. These are the initial stretch of shaft close to the ground surface with low in-situ stresses, and the expected intersections of the shaft with the Structurally Lixiviated unit (LXE), at depths of approximately 400 and 800 meters – see Figure 2. For example, Figure 11 shows a view of the three-dimensional model corresponding to the shaft encountering the LXE unit at the approximate depth of 800 m.

The three-dimensional models incorporated both temporary and permanent support (with characteristics described in the next section) and the proposed excavation

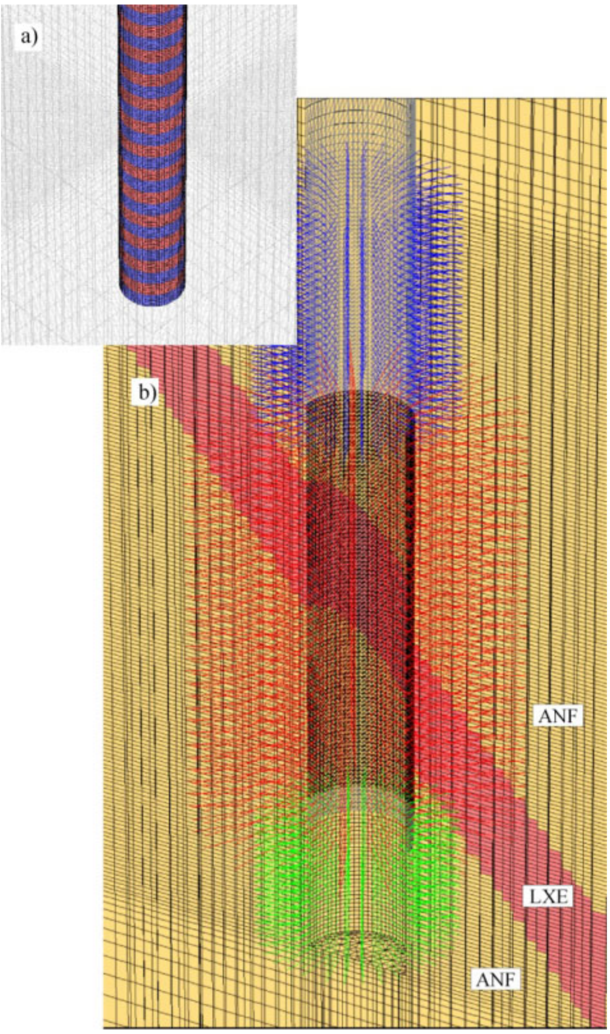

Figure 11. Three-dimensional numerical model of the advance of the shaft excavation through the Structurally Lixiviated unit (LXE) at an approximate depth of 800 meters. The inset in the figure shows the 3 m advance intervals considered for the excavation of the shaft. The model, which incorporates temporary and permanent support, was constructed using the finite difference code FLAC3D – see Itasca (2007).

advance of 3 meter intervals, coinciding with the blasting length.

The purpose of these models was to account for the actual three-dimensional nature of the excavation problem; the models allowed to quantify shaft wall displacements, extent of the plastic-failure zone around the walls of the shaft, and the performance of both temporary and permanent support – i.e., the verification of the acceptability criteria in terms of factor of safety described in Section 4.

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 1.5 for temporary support and a factor of safety of 3.0 for permanent support. Also, the model including the LXE unit at the approximate depth of 400 m showed a maximum displacement of the shaft wall of 6 cm and a maximum extent of the plastic zone (behind the shaft wall) of the order of 2 m. Similarly, the model including the LXE unit at the approximate depth of 800 m showed values of 8 cm (wall displacement) and 4 m (plastic extent), respectively.

7 PROPOSED SHAFT SUPPORT

Based on experience in design of shaft support and on the application of empirical, analytical and numerical models described in previous sections, for the sections of shaft crossing the good quality rock mass units (ANF and GRD units), temporary and permanent support with the characteristics summarized in Table 6 were proposed. As seen in the table, the temporary support consists mainly of rock bolts (and wire mesh) with quite uniform characteristics for most of the length of the shaft; also, the permanent support consists of a concrete liner that varies in thickness between 0.4 and 0.6 meters according to the depth.

For the first 42 meters of shaft excavated, in which low stress confinement in the rock mass could translate into ground instability, as well as for the areas of shaft expected to cross the LXE unit (see Figure 2), heavier temporary and permanent support were proposed. The characteristics of these are summarized in Table 7.

Table 6. Summary of temporary and permanent support proposed for most of the length of the shaft – i.e., in areas others than those described in Table 7.

Shaft depth	Structural elements for stabilization						
	Temporary support					Permanent support	
	Element	Characteristics	Length	Pattern	Additional	Element	Characteristics
42 - 400 m	Anchored rockbolts with resin grouting	Threaded steel bar Type A 630-420H (or similar) of 22 mm diameter	4.0 m	1.2 m x 1.0 m (circ. x axial)	Steel wire mesh #10006	Concrete liner casted in situ (with steel reinforcement)	t = 400 mm concrete H45
400 - 800 m		Threaded steel bar Type A 630-420H (or similar) of 25 mm diameter	5.0 m	1.2 m x 1.0 m (circ. x axial)	Steel wire mesh #10006		t = 500 mm concrete H45
800 - 1000 m				1.5 m x 1.0 m (circ. x axial)	Steel wire mesh #10006		t = 600 mm concrete H45

Table 7. Summary of temporary and permanent support proposed for the shaft at three critical locations – i.e., low stress field region for the initial 42 meters of shaft, and intersection of shaft with LXE units at approximate depths 400 and 800 meters, respectively.

Case	Structural elements for stabilization						
	Temporary support				Permanent support		
	Element	Characteristics	Length	Pattern	Element	Characteristics	
Upper sections (0 a 42 m)	Anchored rockbolts with resin grouting	Threaded steel bar Type A 630-420H (or similar) of 22 mm diameter	4.0 m	1.2 m x 1.0 m (circ. x axial)	Concrete liner casted in situ (with steel reinforcement)	0 - 3 m	t = 2500 mm concrete H60
						3 - 39 m	t = 1500 mm concrete H60
						39 - 42 m	t = 900 mm concrete H60
	Mesh	Wire mesh type #10006				0 - 42 m	
Poor rock mass quality (LXE) (390 a 440 m)	Double cables	Cables (single strand or birdcage) 15.6 mm	12 m	1.5 m x 1.0 m (circ. x axial)		t = 600 mm concrete H60	
	Shotcrete	t = 300 mm H45					
	Mesh	Steel wire mesh #7509					
Poor rock mass quality (LXE) (790 a 840 m)	Double cables	Cables (single strand or birdcage) 15.6 mm	12 m	1.5 m x 1.0 m (circ. x axial)		t = 700 mm concrete H60	
	Shotcrete	t = 300 mm H45					
		Steel wire mesh #7509					

8 FINAL COMMENTS

This paper has described several aspects of the process of determining temporary and permanent support for the large diameter shaft PN-1 at Chuquicamata Underground Mine. The shaft is to be excavated in a rock mass of generally good quality (ANF and GRD units), with the exception of areas along the depth of the shaft that are expected to be in a rock mass of poor quality (LXE unit).

The support recommended for shaft PN-1, 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.

Also, the characteristics of the support recommended for the shaft 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. In particular these restrictive assumptions (which had to be considered due to unavailability of detailed information at this stage of the design) will be evaluated in a future final design phase.

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