

# Analysis and design of slopes for Rajo Sur, an open pit mine next to the subsidence crater of El Teniente mine in Chile

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## Abstract

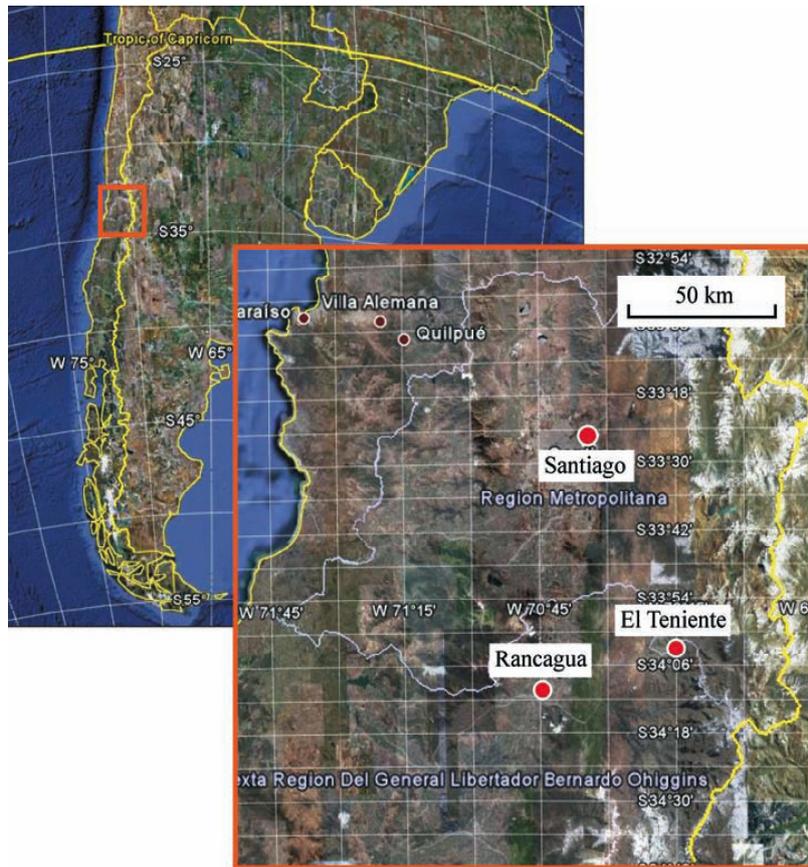
*El Teniente Division of CODELCO is conducting a pre-feasibility engineering evaluation of Rajo Sur project in Chile. This project involves construction of an open pit to mine a copper deposit near the southern border of the caving subsidence crater high in the mountains above El Teniente mine. A geotechnical assessment is being carried out to evaluate the stability of the walls of the pit, particularly in sectors next to the subsidence crater, where the rock mass is of very poor mechanical quality. To evaluate the mechanical stability of the open pit, a series of geotechnical studies have been conducted. These studies have involved, among others, site investigations and slope stability analyses based on limit equilibrium and elastoplastic continuum approaches; they have also involved development of a complex three-dimensional mechanical model to assess the influence of subsidence cracks and abrupt topography on the stability of the planned open pit walls (particularly in the southern boundary of the crater). This paper provides a general description of the geotechnical study carried out for Rajo Sur project, including geological and geotechnical characterizations of the site, slope stability analyses and proposed design angles for the pit walls.*

## INTRODUCTION

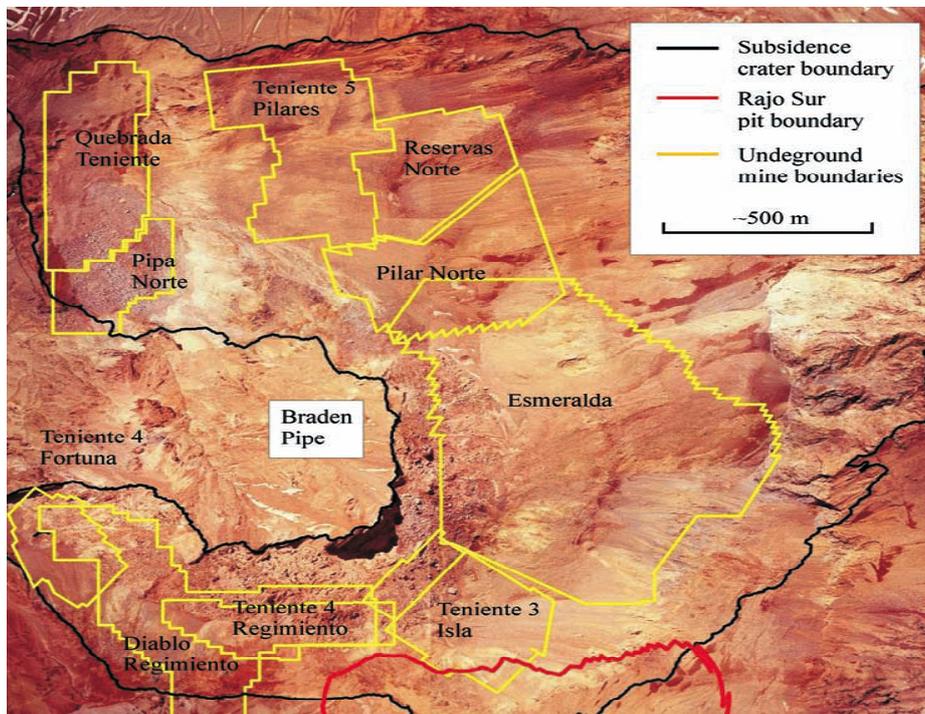
El Teniente mine complex, owned and operated by CODELCO (National Copper Corporation of Chile), is located about 80 km south of Santiago, Chile, at an elevation of 2,500 masl (meters above sea level) in the Andes mountains (see Figure 1). The mine complex includes a series of underground mining operations that use the method of blockcaving to extract copper mineral deposits. Figure 2 shows an aerial photograph of the mountain where El Teniente complex is located, indicating also the layout of the different mines underground. From this photograph it can be seen that the underground mines are emplaced surrounding an intrusive pipe (the Braden pipe) —it is in the rock mass that surrounds the pipe where copper rich deposits exist [1]. The photograph also shows the extent of the subsidence crater, a region of highly (mechanically) unstable ground, resulting from the underground block-caving operations.

With the purpose of providing flexibility in production rates and goals, and also with strategic planning purposes (e.g., to have alternative mineral stock piles available, shall the underground operations at El Teniente become disrupted), El Teniente Division of CODELCO is currently evaluating at a pre-feasibility engineering level the project Rajo Sur [2, 3, 4]. This project involves the development and operation of an open pit near the southern border of the subsidence crater (see red line in the lower part of the photograph in Figure 2). Figure 3 shows a photograph of the Rajo Sur site together with the various elements that comprise the planned mining development; this includes the open pit, the access roads, waste dumps and stock piles.

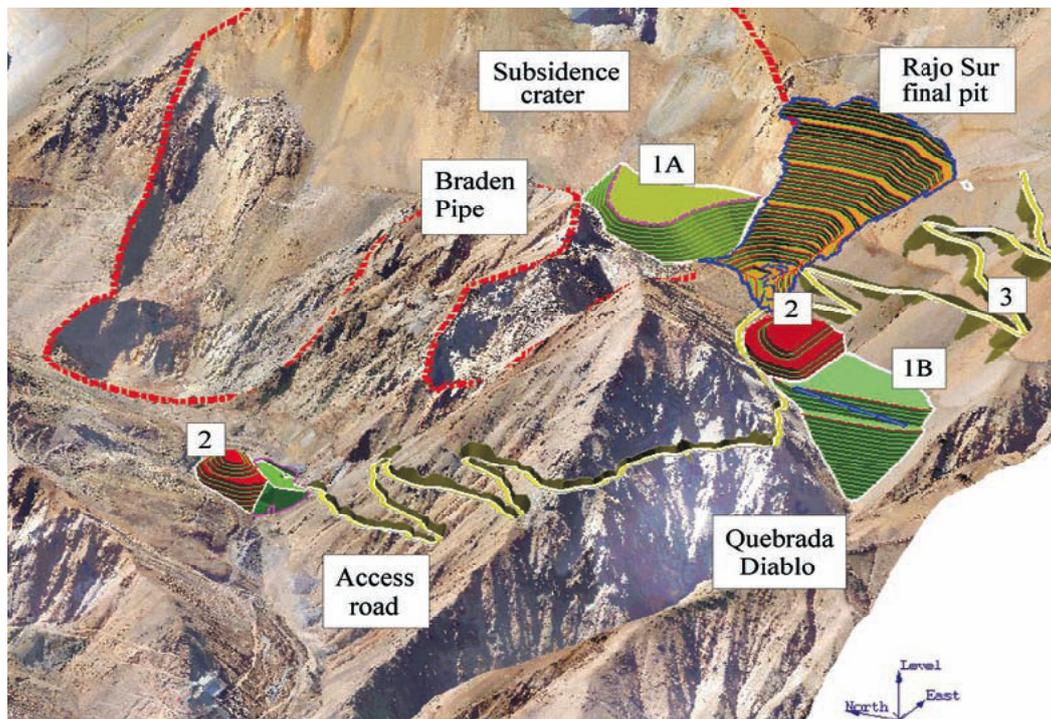
The close proximity of the open pit mine to the subsidence crater (see Figures 2 and 3), makes Rajo Sur a challenging project, and a clear example of open pit mining in difficult ground conditions. The following sections in this paper describe main aspects of the various studies carried out for evaluating the feasibility of constructing the Rajo Sur open pit.



**Figure 1** - El Teniente mine location, in relation to Santiago and Rancagua cities in Chile.



**Figure 2** - Aerial photograph of subsidence crater at El Teniente —the photograph indicates the boundaries of the crater, underground mining works, and northern border of the planned Rajo Sur open pit.



**Figure 3** - Aerial photograph of Rajo Sur project site —dotted red lines indicate current subsidence crater boundary (see Figure 2); also indicated are waste dump alternatives (1A and 1B), stockpiles (2) and secondary road (3).

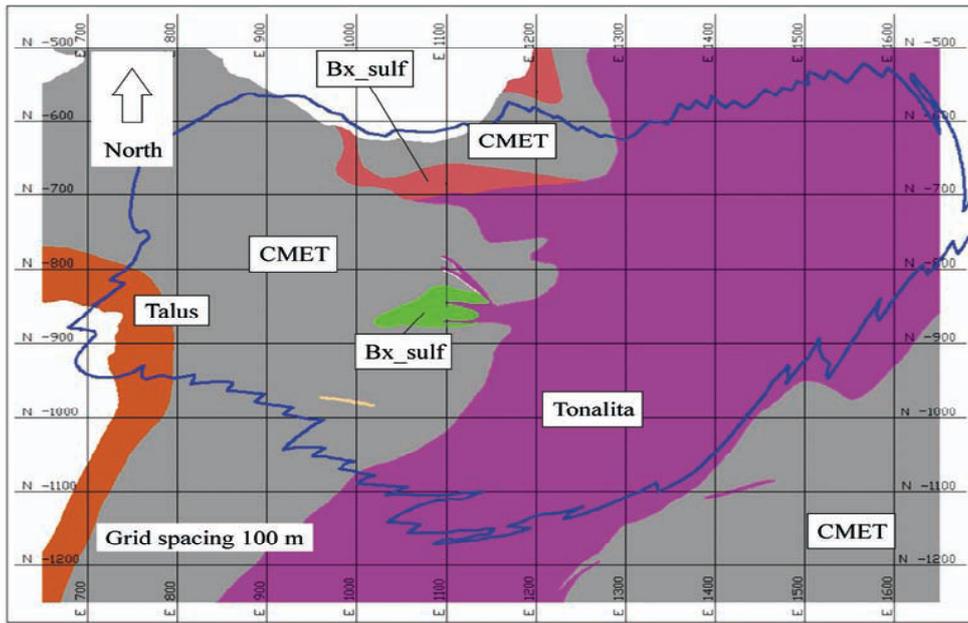
## GEOLOGICAL CHARACTERIZATION

The main rock types at Rajo Sur site include micro-porphyric to porphyric rocks of andesite-basalt type, grey colored and of mafic composition, corresponding to CMET ('Complejo Máfico El Teniente') type; these CMET rocks present moderate to intense argillic and/or quartz-sericitic alteration, and light to moderate chloritization; other rocks at Rajo Sur site include Tonalite to Diorite and Tonalite to Diorite porphyry rocks, with moderate to intense argillic and/or quartz-sericitic alteration of difficult classification and characterization. In some sectors a ferrocrete stratum can also be found overlying these rocks. Figure 4 shows a schematic representation of the lithology described above; the figure also indicates the boundaries of the planned pit in the final excavation stage.

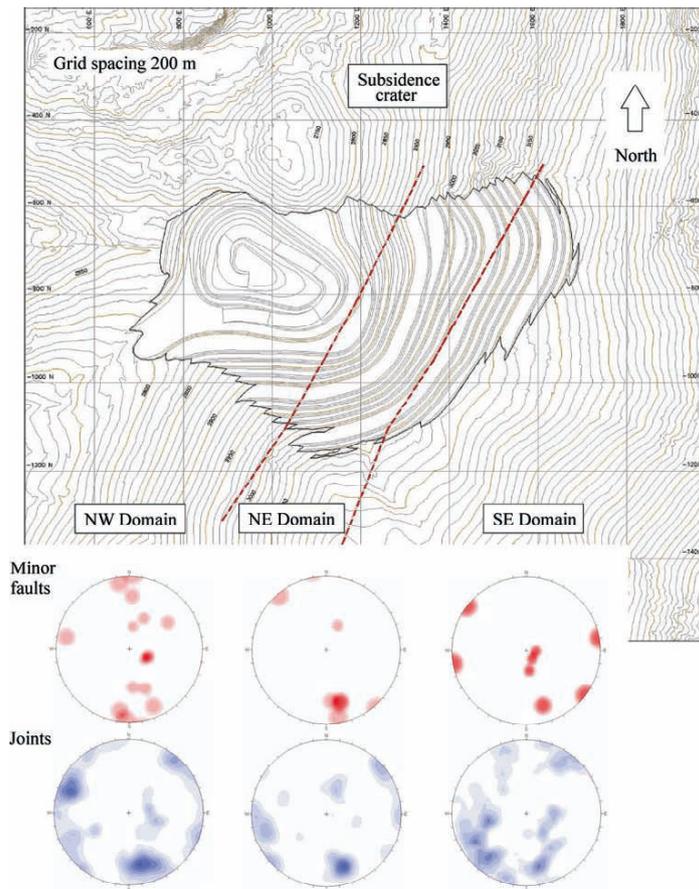
According to the geotechnical information gathered from mapping and logging of several campaigns in the years 2006, 2007 and 2008 in various sectors at Rajo Sur [5, 6], three structural domains can be distinguished based on their predominant location: i) NW domain, located in the lower (altitude) zone of the planned pit; ii) NE (central) domain, located in the medium (altitude) part of the planned pit, and iii) SE domain, located in the higher (altitude) zone of the planned open pit —the boundaries of the different domains, in relation to the boundaries of the planned open pit (for the final excavation stage) are shown in a topographical map in Figure 5. In agreement with Figure 4, the main lithological units in the NW and SE domains correspond to CMET rock type, while in the NE (or central) domain, correspond to Tonalite and Diorite rock types.

Surface mapping and core log analyses carried out by SRK and El Teniente division geologists and engineers, has led to conclude there are not first order (or mayor) faults in the area [5, 6]. The studies have also revealed that there are structural features of 2nd and 3rd order mainly (i.e., minor faults and joints). The occurrence of these structural features are summarized in the stereonet in Figure 5 —these represent distribution of poles of minor faults and joints, respectively, in a lower hemisphere projection.

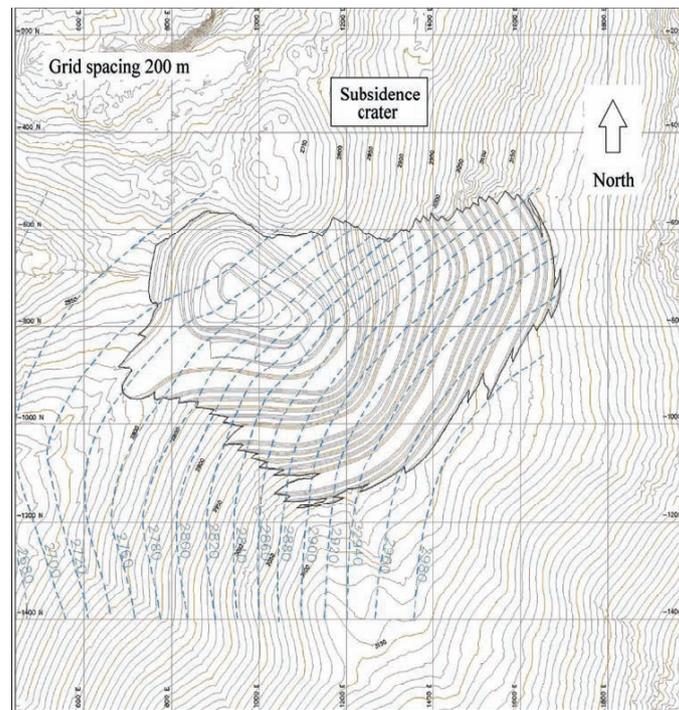
As part of the geological characterization, a hydrogeological conceptual model has also been developed based on piezometers, borehole loggings and outcrop information at the site. Three main hydrogeological units have been identified, namely: i) an alluvial layer with a thickness of 5 to 10 m; ii) a permeable unit related to a secondary geotechnical unit with a thickness of 60 to 170 m; and iii) an impermeable unit. The annual fluctuation of (underground)water levels has shown a difference of 30mbetween winter (snow-icing) and spring (snow melting) seasons. In the developed hydrogeological model, the equipotential curves show a higher hydraulic gradient towards the subsidence crater and parallel to the topographic surface as indicated in Figure 6.



**Figure 4** - Lithological units from geological interpretations for Rajo Sur site (level 2,800 masl represented). Blue line indicates projection of preliminary final pit outer boundaries at the level represented.



**Figure 5** - Structural domains from geological interpretations for Rajo Sur project. In the main plot, red lines indicate boundaries of structural domains. In the stereonets, red dots represent corresponding pole distributions of surveyed minor faults, while blue dots represent poles distributions of surveyed joints — both corresponding to a lower hemisphere projection.



**Figure 6** - Hydrological model developed for Rajo Sur according to site investigations. Blue lines represent equipotential (same water head) curves.

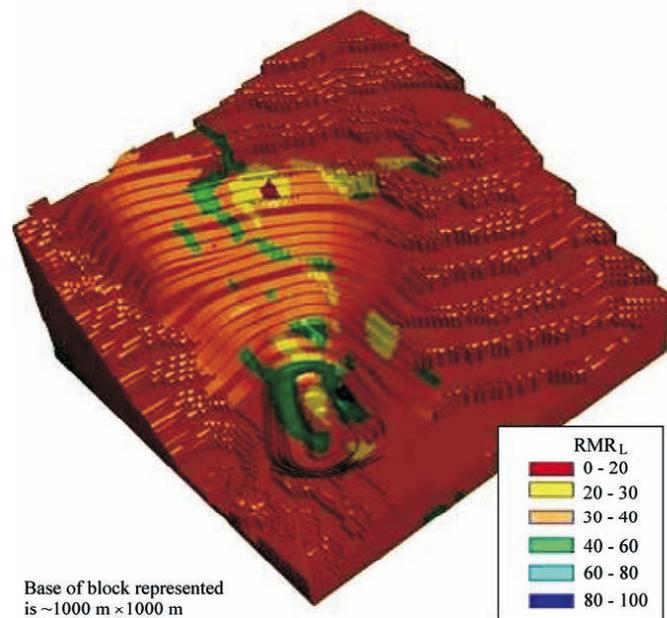
## GEOTECHNICAL CHARACTERIZATION

The geotechnical characterization of the Rajo Sur site has been carried out based on geological-geotechnical borehole logging and surface mapping information [5, 6, 7]. The quality of the rock mass has been rated using the Rock Mass Rating (RMRL) system by Laubscher [8]. The rating has been done based on old drill cores (twenty seven drill cores of 1,155 m in total) and two drill core campaigns performed in 2006 (seven drill cores of 1,260 m in total) and 2007 (twenty four drill cores of 3,306 m in total). With this information, and considering also interpreted information on alteration and geological features such as geological structures, a geotechnical three-dimensional block model has been developed using the software GEMCOM (available from [www.gemcomsoftware.com](http://www.gemcomsoftware.com)). The geotechnical three-dimensional block model has been used to store all gathered and derived geotechnical information for the project, starting with values of RMRL, RQD, and FF (frequency fractures per m), as shown in Figure 7—this figure represents values of RMRL in the area of the planned open pit. The rock mass mechanical properties for the geotechnical units have been evaluated using the Hoek-Brown system [9] as implemented in the software ROCLAB (available from [www.rocscience.com](http://www.rocscience.com)), using also results of laboratory testing (unconfined and triaxial compression tests of intact rock). The effect of uncertainties on the geotechnical data has been accounted for by carrying out Monte Carlo simulations of mechanical parameters, based on recommendations by various authors [10, 11]. The Monte Carlo simulations have been implemented using the software CRYSTAL BALL (available from [www.oracle.com](http://www.oracle.com)). Table 1 summarizes mean geotechnical properties parameters for the various rock mass units at the site, as interpreted with the aid of the three-dimensional block model developed to store and manage the geotechnical information for the project.

Geotechnical unit	$\gamma$ [kN/m <sup>3</sup> ]	$\sigma_{ci}$ [MPa]	$m_i$ [Å]	GSI [Å]	$\nu$ [GPa]	$c$ [Å] kPa	$\phi$ [deg]
CMET - Secondary	26	30	19	30 - 40	0.92	0.28 640	25
				25 - 35	0.74	0.29 550	23
Tonalita - Secondary	25	46	25	30 - 40	0.76	0.28 820	31
				25 - 35	0.61	0.29 710	28
CMET - Primary	28	95	19	50 - 60	7.57	0.23 1510	38
Tonalita - Primary	27	126	25	50 - 60	6.57	0.23 1860	44
Talus	18	-	-	-	0.16	0.30 0	39
Broken material	20	-	-	-	0.50	0.29 150	38

Notation:  $\gamma$  is the bulk unit weight of the rock mass;  $\sigma_{ci}$  is the unconfined compression strength of intact rock;  $m_i$  is the Hoek-Brown parameter; GSI is the Geological Strength Index;  $E$  the rock mass Young's Modulus;  $\nu$  is the Poisson's ratio;  $c$  is the equivalent rock mass cohesion;  $\phi$  is the equivalent rock mass internal friction angle—parameters  $E$ ,  $\nu$ ,  $c$  and  $\phi$  computed with software ROCLAB.

**Table 1** - Summary of geotechnical properties for various geotechnical units—mean values indicated.



**Figure 7** - Geotechnical three-dimensional block model developed for Rajo Sur project —the model shown represents distribution of Rock Mass Rating values (according to Laubscher [8]) in relation to the preliminary design of the pit, for the final excavation phase.

## STABILITY ANALYSES AND DESIGN OF SLOPES

To evaluate the stability of the open pit walls, approaches involving bench-berm design, and slope stability analyses, at bench, inter-ramp and global scales have been applied, following classical bibliography [12, 13, 14, 15, 16]. In particular, the analyses of stability of the pit walls have been performed using limit equilibrium models and finite difference elasto-plastic continuum models, as explained later in the paper. For the design of the open pit walls, the mechanical stability has been quantified in terms of a factor of safety (FOS) and a probability of failure (POF) factor. The acceptability criterion for stability of the various elements conforming the excavation have been chosen to be as follows:

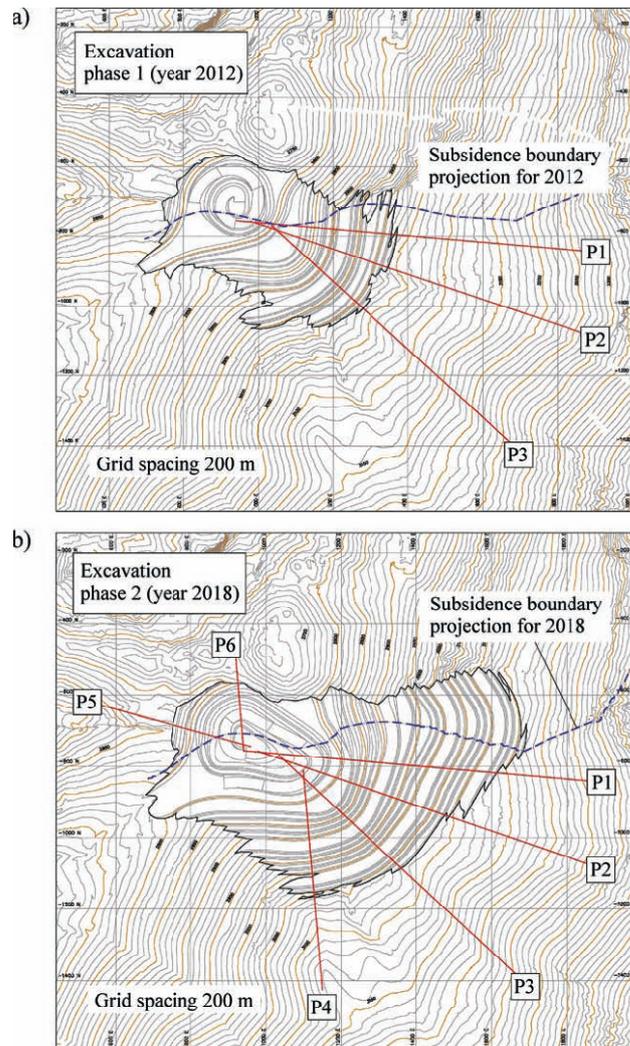
- Bench Scale: FOS > 1.1 and POF < 30%
- Inter-ramp Scale: FOS > 1.2 and POF < 10%
- Global Scale: FOS > 1.3 and POF < 5%

The ranges of values indicated above have been established based on experience in other open pit mining projects, and accounting for various factors, among others, an acceptable degree of safety for personnel and equipment, the characteristics of the monitoring systems used, the availability of double access/exit from the open pit, etc., as described in Steffen and Contreras [11].

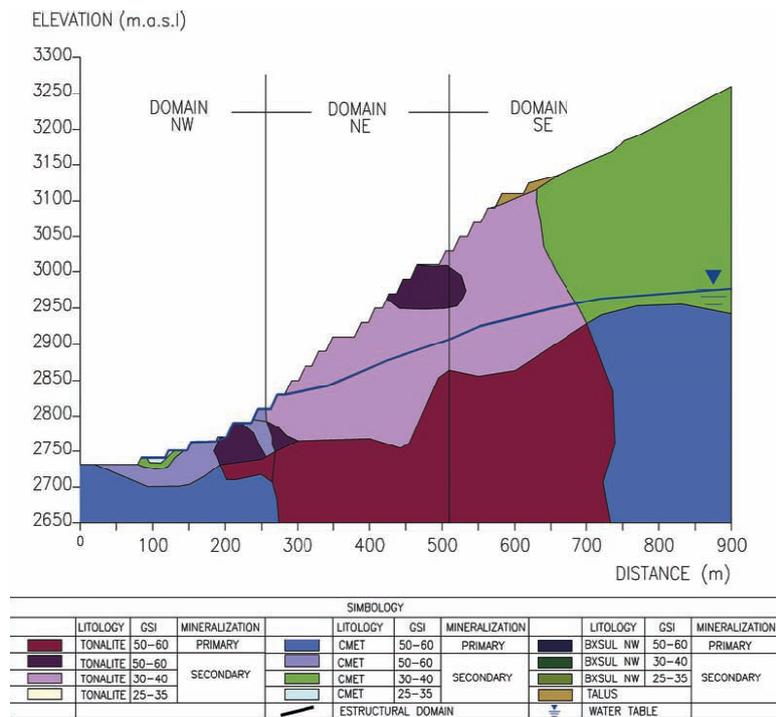
The bench-berm design, that has been selected based on operational aspects, considers a bench height of 10 m in a double bench configuration (20 m total height). Based on stability analyses, the bench face inclination has been determined to be 65°.

For all orientations and domains, stability analyses of structurally controlled bench failures (i.e., planar and wedge types of instabilities) have been conducted. The analyses have been done using the computer codes DIPS, ROCPLANE and SWEDGE (available from [www.rocscience.com](http://www.rocscience.com)). The berm width required for stability has been computed assuming recommendations by Gibson et al. [17], and acceptability criteria determined for this particular project [6]. For the planned open pit at Rajo Sur, the minimum berm width for the double benches results to be in the range of 6.5 to 11.5 m.

For the inter-ramp and global pit wall design, limit equilibrium models have been used to analyze stability conditions. In doing so, two open pit configurations corresponding to an excavation phase 1 (expected to be reached in the year 2012) and an excavation phase 2 (expected in the year 2018) have been considered —see Figure 8. The stability analysis of inter-ramp and global slopes has been carried out for a total of nine cross sections (three for the excavation phase 1 and six for excavation phase 2). The location of these cross-sections are shown in the plan view representation of Figure 8. Also, as an example, Figure 9 shows one of the nine geotechnical cross sections on which stability analyses have been carried out.



**Figure 8** - Plan view of open pit site showing boundaries (in blue) of a) excavation phase 1, corresponding to year 2012; and of b) excavation phase 2, corresponding to year 2018. The figure indicates the position of the various cross-sections considered for the stability analyses.



**Figure 9** - Typical geotechnical cross-section used for the slope stability analysis —the case represented corresponds to section P1 in Figure 8b.

## Two-dimensional limit equilibrium analyses

Each of the nine cross-sections has been analyzed with the option Generalized Limit Equilibrium (GLE) in the software SLIDE (available from [www.rocsience.com](http://www.rocsience.com)).

The following is a summary of the general features of the (limit equilibrium) models and a discussion of some results —the whole analysis is reported in [6]: -To include the effect of rock bridges, probabilistic analyses have been performed to define a directional material strength. These analyses have been carried out using the program STPSIM [18] considering mean and standard deviation values for the strength of rock and structures of the different geotechnical units. Also, length, dip and spacing of the structures in the rock mass have been accounted for in agreement with Baczynski [18].

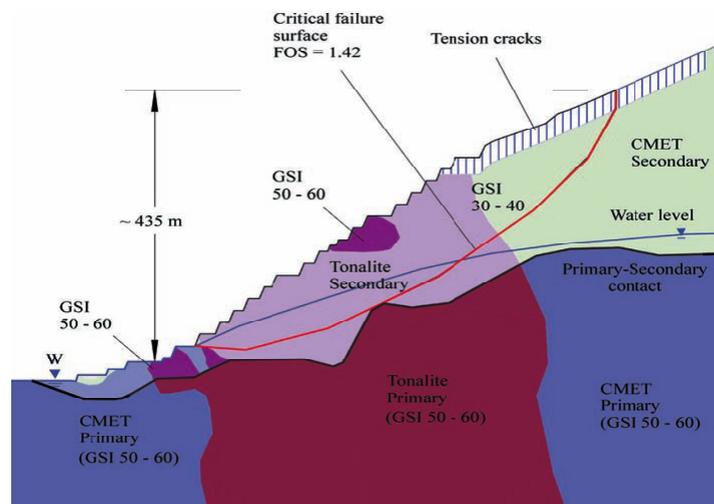
- For each section analyzed, various possible modes of failure have been considered, including failure of lower and upper inter-ramps, failure of intermediate inter-ramps, and failure of the overall slope, accounting again for the various lithological units.
- Tension cracks have been considered to be totally filled with water; these are assumed to simulate the effect of snow/rain infiltrating into (cracks in) the ground.
- A 'path search' technique has been used to find the most critical failure surface in each of the sections analyzed with the limit equilibrium model. For the most critical failure surface, the probability of failure has been computed using the methodology suggested by Duncan [19], that in the case described in this paper, and as mentioned earlier, accounts for the uncertainty associated to values of cohesion, internal friction angle, dip angle of structures and phreatic surface position.

The results obtained from limit equilibrium models suggest that all slopes analyzed are stable, with FOS > 1.4 and POF < 5%, for all cases considered. As an example, Table 2 summarizes values of FOS and POF computed for the various sections represented in Figure 8. Figure 10 shows a typical output from the limit equilibrium slope stability analysis corresponding to section P1 in Figure 9.

Excav. phase	Profile (in Fig. 8)	$\acute{\alpha}$ [deg]	FOS [no-units]	POF [%]	H [m]	W [MN/m]
1	P1	37	1.78	2.1	300	767
1	P2	37	1.57	3.9	335	1236
1	P3	37	1.73	2.3	333	111
2	P1	43	1.42	4.9	435	1869
2	P2	44	1.53	3.0	275	1452
2	P3	46	1.55	3.5	355	1006
2	P4	46	1.53	2.9	325	1191
2	P5	34	2.16	< 1	95	84
2	P6	39	2.13	< 1	70	36

Notation:  $\alpha$  is the slope angle; FOS is the factor of safety; POF is the probability of failure; H is the height of potential failure volume (measured from slope toe); W is the weight of potential failure volume, per unit length of failure surface.

**Table 2** - Summary of results of limit equilibrium slope stability analyses for the various cross sections indicated in Figure 9.



**Figure 10** - Limit equilibrium stability analysis (with software SLIDE) of the geotechnical section represented in Figure 9. Results of the analysis indicate that at limit equilibrium, the failure reaches a height of approximately 435 m. In the planned condition, the geotechnical section has a FOS of 1.42 and a POF of less than 5%.

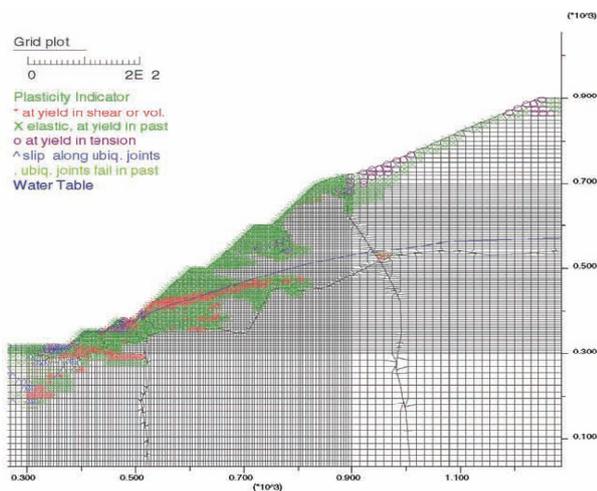
## Two-dimensional elasto-plastic continuum analyses

In addition to the limit equilibrium stability analyses just described, continuum elasto-plastic analyses have been carried out using the software FLAC (available from [www.itasca.com](http://www.itasca.com)). In what follows, results for the section P1 (Figure 9) that showed a relatively low value of FOS and a relatively high value of POF are described. This section (P1) has also been taken as the basis to analyze expected deformational behavior of the rock mass in the vicinity of the crater. The following is a summary of some general features of the (elasto-plastic continuum) models, as fully described in [6]:

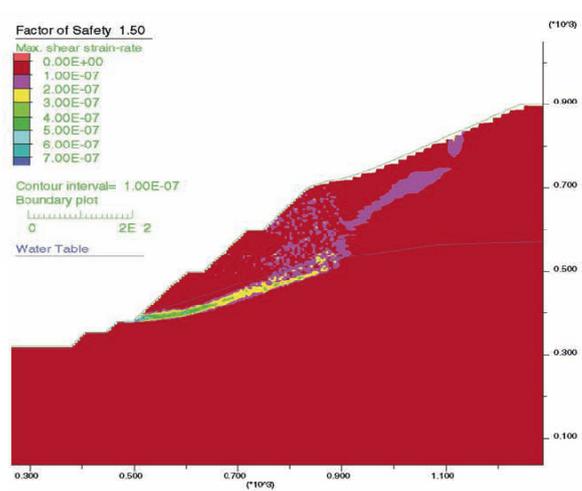
- The in situ stress state has been considered to be consistent with measured regional ground (i.e., virgin) stresses in the area; this involves a lithostatic vertical stress, a ratio of horizontal-to-vertical stress in the EW direction equal to 1.2, and a ratio of horizontal-to-vertical stress in the NS direction equal to 0.8—in the two-dimensional analyses performed with FLAC, the in situ stresses are the resolved components of the three-dimensional stress tensor mentioned above in the plane of analysis.
- Jointing of the rock mass has been accounted for by using a ‘laminated’ or ubiquitous-joint constitutive model in the (FLAC) models.
- The same structural domains (with their corresponding strength and deformability properties) and phreatic surface as considered in the limit equilibrium analyses have been used in the continuum elasto-plastic models.
- The FOS in the continuum models has been computed based on the shear strength reduction technique—see, for example, [20, 21].

The following are some results obtained from the continuum models, again, as fully reported in [6]:

- The maximum displacements occur locally in the lower benches, in the Tonalite unit, and these do not appear to exceed 1.25 m. Hence, the ratio between the slope crest horizontal displacements and the slope height is smaller than 0.5%. Hoek and Karzulovic [22] suggest that slopes begin to show signs of instability when this ratio is greater than 2%.
- Large deformations are observed at the toe of the slope in areas of contact of secondary and primary lithological units—see Table 1.
- The plasticity indicators (failure through the rock mass by shearing and/or tensioning, and slipping along ubiquitous joints) indicate that tensile failure occur at the slope crest; also, they indicate that shear failure occurs mainly in the secondary/primary contact of the CMET unit (i.e. in rock mass of poor geotechnical quality) located at the slope toe (see Figure 11).
- The factor of safety computed with the strength reduction technique results to be 1.5; according to the computation, the failure surface that grows from the toe of the pit reaches 460 m of height at the time of failure (see Figure 12).



**Figure 11** - Two dimensional elasto-plastic continuum analysis (with software FLAC) for cross section P1, in Figure 9b. The figure represents mesh of elements and plasticity indicators in the final pit configuration, after the various successive excavation stages considered in the analysis.



**Figure 12** - Computation of factor of safety, FOS, for the same model represented in Figure 11, using the strength reduction technique.

## Three-dimensional elasto-plastic continuum analyses

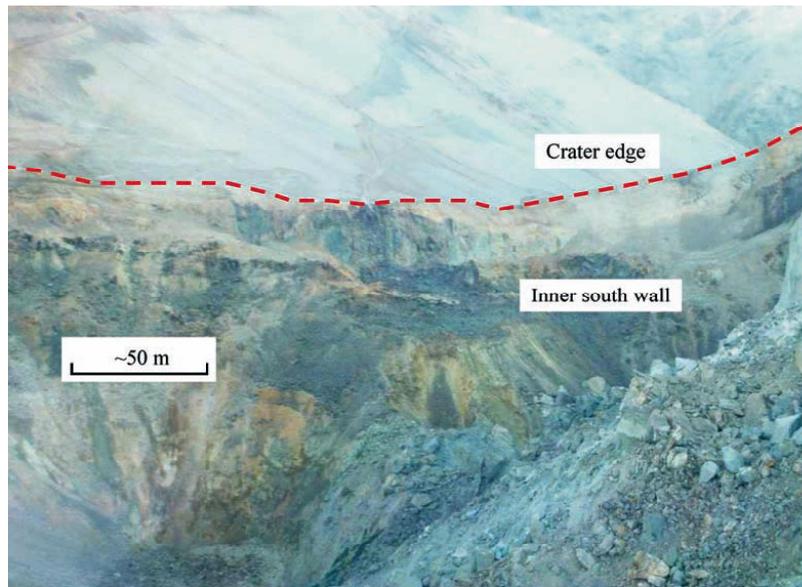
As discussed in the introduction to this paper, the ground in the caving subsidence crater of El Teniente mine complex is of poor mechanical quality, i.e., low strength and high deformability.

To assess the influence of the poor quality of the ground, the abrupt topography and material changes at the boundary of the crater, in relation to the planned emplacement of the pit, a three-dimensional elasto-plastic continuum model that incorporates realistic topographical and lithological features has been developed using the code FLAC3D (available from [www.itasca.com](http://www.itasca.com)). The characteristics of the model (initial stresses, lithological units, etc.) are similar as for the two-dimensional models discussed earlier—in particular, for the three-

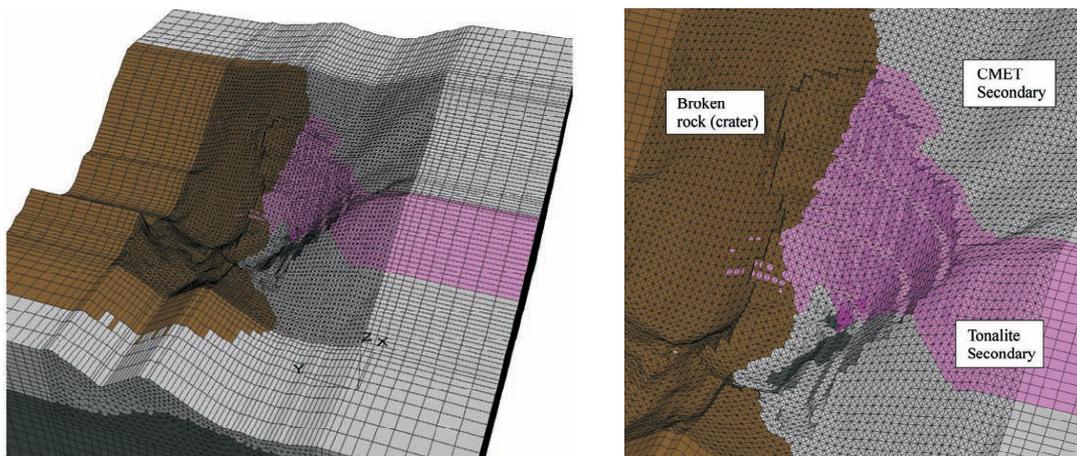
dimensional model, the topographical features have been constructed from existing digital elevation models. Figure 14 shows views of the three-dimensional model.

The following is a brief summary of some results obtained from the model, as reported in [6]:

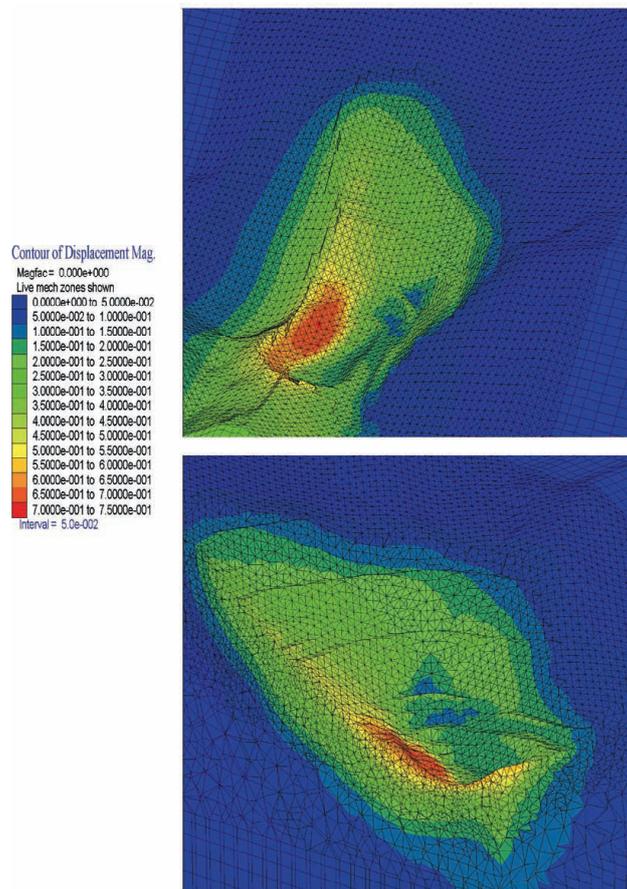
- The largest ground displacements for the planned open pit occur near the subsidence crater boundary, above Diablo Regimiento underground mine (see Figures 15 —also, see Figure 2).
- In general, maximum deformations are localized next to the crater and at secondary/ primary contacts.
- In the north wall, an unconfined zone of 30 to 50 m (measured from the subsidence crater towards the south) can be observed.
- Large displacements and tensile type of plastic failure are observed in the NE wall, in agreement with site observations.



**Figure 13** - Photograph (taken from helicopter) of the scarp wall at the subsidence boundary, as seen from inside the crater and looking towards the south —i.e., the planned open pit is to be located behind the scarp wall.



**Figure 14** - a) General and b) close-up views of three dimensional elasto-plastic continuum model of the open pit for Rajo Sur project. Different colors indicate different geotechnical units, in agreement with Figure 4. The model shown is for the final stage of excavation (the model developed with the software FLAC3D considers progressive excavation of the open pit).

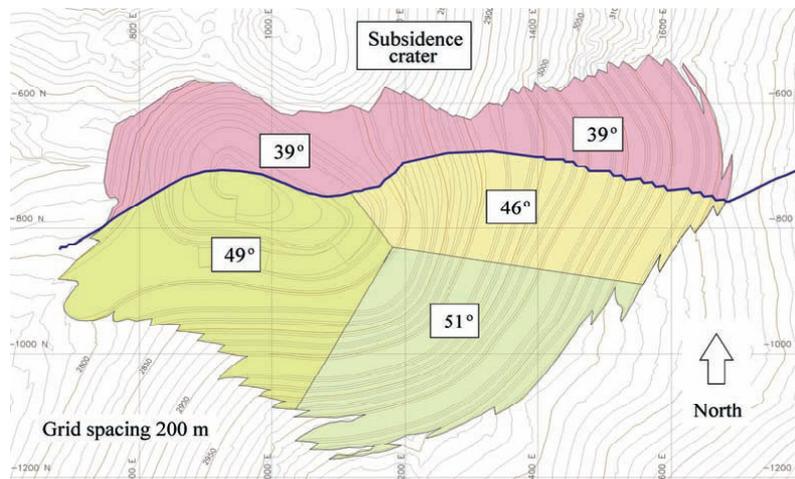


**Figure 15** - Contours of total displacements on a) model surface and b) cut-plane, for the final stage of excavation, represented in Figure 14. Large values of displacements are seen to be concentrated near the crater boundary.

### Slope angles

The different geotechnical analyses discussed earlier on have allowed definition of geometrical characteristics for the open pit walls at Rajo Sur—in particular inclination angles for the pit walls. Below, a summary of various geometrical quantities are provided—a full discussion on recommended values, including their rationale, are provided in [6]:

- The inter-ramp slope angles (toe to toe) vary, depending on the orientation of the walls and structural domain boundaries, between 46° to 51°—this is for all walls in the pit except for those in the broken rock (next to the crater edge) for which the design angle is 38°—see Figure 16.
- The bench face angle is 70° for all rock units, except for the broken rock (next to the crater edge), for which it is 60°.
- The bench height is 20 m for all units, except for the broken rock, where it is 10m.
- The maximum inter-ramp height is 100 m (for most units) and 60 m for broken rock.



**Figure 16** - Summary of slope inclinations (inter-ramp toe-toe angle) for excavation phase 2 (year 2018), resulting from the various stability analyses. Blue line indicates expected location of subsidence boundary in the year 2018.

## FINAL COMMENTS

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This paper has presented a general description of studies conducted at the prefeasibility stage for Rajo Sur project in Chile. The main goal of the study has been to provide a preliminary geotechnical design for an open pit mine planned in existing difficult ground conditions, next to the caving subsidence crater of El Teniente mine.

Based on geological and geotechnical characterization of the site, and by application of limit equilibrium and elasto-plastic continuum models, including a complex three-dimensional model that incorporates topographical and lithological features, the stability of the open pitwalls next to the subsidence crater has been assessed. Based on the results of the different analyses performed, and based on the available information at the pre-feasibility stage, construction of the open results, in principle, technically feasible.

In a next feasibility stage, it is recommended that the geotechnical site characterization is improved. For example, a more elaborated plan of rock laboratory testing for the main geotechnical units identified in this study, particularly for the broken material next to subsidence crater, is recommended. Also, considering that subsidence due to underground caving operations is a dynamic process, and that the location of the crater boundary is seen to have a pronounced effect on the stability of the planned pit walls, studies of propagation of subsidence due to caving are recommended; such studies should consider different scenarios for underground mining development, including contingency plans. Associated to studies on caving propagation, it is recommended that the mechanical behavior of the disturbed zone of loose material (that dislodges into the crater) and the extent of the areas to undergo large movement next to the boundary of the crater are also assessed in more detail.

This shall be of particular relevance for taking necessary measures when excavating and operating the open pit mine, so as to guarantee safety of the personnel working next to the crater boundary. Also, in a next feasibility stage, a detailed plan of movement monitoring should be established for the project. This monitoring plan should be an integral part of management and supervision works—during construction and operation of the open pit mine. In this regard, operational protocols need to be mandated by mine management for each operating environment. These protocols should consider aspects such as backfilling to stabilize voids (caves) within the crater or slope reinforcement to improve stability of bench faces during operation of the planned mine.

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