Case Study: Stability Assessment in Underground Excavations at Vazante Mine - Brazil

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Abstract. Currently, most studies on stability of underground excavations include two separate analyses: the elastoplastic behavior of rock masses and/or kinematic analysis of possible wedges and blocks formed in the excavation walls. This paper presents a case study carried out at the Vazante Zinc Mine in Minas Gerais, Brazil, where studies on stability of underground excavations in discontinuous media included survey reports, laboratory tests and in-situ collected data. In this context, where galleries and mining stopes are excavated in discontinuous media, collapse events caused by the presence of discontinuities are common. First, the spatial orientation, geometric arrangement and mechanical characteristics of the discontinuities intercepted by the core samples were collected. The spatial orientation was based on guide layers, which are discontinuities with known dip direction and variable and dip. The geotechnical characteristics of the discontinuities were obtained by correlation with the roughness degree and the nature and weathering degree of the filling material. From there, the geological-geotechnical models were developed, which were the basis for the finite element analysis in discontinuous media of the designed excavations in the sections 13225 and 13300, between levels 210 and 345 of the mine. For comparison and complementation, wedge kinematic analysis and finite element analysis in equivalent continuous media were performed and, later, an arrangement for the reinforcement system was suggested. The results of these studies show that, in general, continuous models tend to be more conservative and have wider deformation zones, while discontinuous models are able to show in more detail where the displacements occur, and how the families of discontinuities affect the stability of excavations.

Keywords: discontinuities, discontinuous media, displacements, mining stope, reinforcement systems, underground excavations.

1. Introduction

Stability studies of underground excavations are mostly conducted using numerical methods that consider rock masses as continuous media, such as the finite element method, and/or wedge kinematic analysis. Basically, numerical methods use elastic and plastic geotechnical parameters of the material, and the discontinuous media (with joints, beddings and geological faults) are commonly correlated to continuous media through equivalent geotechnical parameters. The kinematic analysis is used to evaluate the equilibrium of the wedges and blocks formed by the intersection of the discontinuities in the walls of the excavations, based only on the geotechnical parameters of the joint families. Both methods have intrinsic limitations.

The numerical analysis by continuous media does not reproduce the mechanical behavior of the discontinuities present in the rock mass, while in the kinematic analysis only the blocks and wedges located in the excavation surface are evaluated. Over the last decade, many codes with mixed concepts were created, making it possible to reproduce a scenario closer to reality, where the geotechnical behavior of rock and discontinuities are simultaneously evaluated.

This study deals with the most common methodologies for characterization and geological-geotechnical modeling of discontinuous rock masses, and proposes a methodology for stability studies in such environments. For that purpose, a case study was carried out in a certain region of the Vazante Underground Mine, in Brazil, where collapse events related to the formation of slabs and blocks are recurring.

The city of Vazante is located in the northwest region of the State of Minas Gerais, approximately 504 km from Belo Horizonte and 354 km from Brasilia, federal district of Brazil. The Vazante Mine is located in the geological context of a shear zone, through which mineralized fluids have percolated and zinc minerals and other associated minerals have crystallized. The residual rocks of the Vazante Zinciferous Reservoir correspond to a dolomite, marl and phylcite sequence, susceptible to an accelerated weathering process that has generated cavities, often carrying mud and water under pressure. This association of factors is negative from the geotechnical point of view since it generates risk for the excavations. As shown in Fig. 1, the ore-bearing rocks are pink dolomites interspersed with metapelitic rocks, usually marls and slates, in addition to the dolomitic and Fe-carbonate breccias (siderite and ankerite).
The relationship between the preferential planes of bedding, faults and fractures, as well as the definition of water flow in the rock mass, can also represent risks associated with displacement and collapses during the underground excavation of galleries and mining stopes. Therefore, the purpose of this study is to identify and model such areas, and to indicate a long-term stabilization system to be adopted for each particular case. For that end, data from drillings in the studied area, located between the profiles 13.200 and 13.350 and the elevations 210.00 to 345.00 m, as well as survey data, previous laboratory test data and field data were used in this study. The drilling campaign was made from the gallery 345 GP-Shaft (Research Gallery), totaling 23 holes in 4 profiles.

Based on the drillhole core samples, the mean spatial orientations and spacing of the discontinuities were described, and also the roughness characteristics and the weathering degree of the filling material.

2. Spatial Orientation of Discontinuities

The orientation of the discontinuities (joints, fault and beddings) intercepted by the drillholes during the Vazante Mine exploration campaign was obtained by a preset dip direction value for the dolomite and marl bedding (S0) family, in addition to the Vazante Fault (FVZ). Several authors, as Rostirolla et al. (2002), Bhering (2009) and Charbel (2015), have found that these two structural features show little variation in the dip direction, based on measurements obtained on the surface and underground, at various depths, both in the north (Sucuri Mine) and in the south (Lumiaideira Mine).

Rostirolla et al. (2002) summarized the mean orientation values of the discontinuities in the open pit area, correlating them with the deformation phases, established by the same authors, and the mean values of S0 and Vazante Fault, with very close dip directions, of 317 and 316, respectively.

Bhering (2009) obtained measurements for the S0 family and the FVZ in mappings of transverse galleries at sites near the study area, at levels 388 and 345, between profiles 11.000 and 13.000. The mean orientation measured at these locations was 332/16 for beddings, and the average for the Vazante Fault was 326/73.

Charbel (2015) mapped several transverse galleries in regions next to the area of studies, at levels 345 and 388, between profiles 12.500 and 13.100. The dip direction obtained for S0 varied between 280 and 345, with dip between 10° and 50°. The dip directions of the Vazante Fault varied between 300 and 335, with dip between 50° and 80°. Figure 2 shows one of the mappings performed on a transverse gallery that passes through the hanging wall and the foot wall of the fault, showing the strong dip direction trend to NW of the S0 and Vazante Fault.

Based on the premise that guide layers (with little variation in the dip direction) are frequent in the rock mass of Vazante Mine, the orientation of the discontinuities around them was obtained from the α and β angles, introduced in the literature by R.E. Goodman in 1976. The smaller angle between the axis of the hole and the straight line of maximum gradient of the plane is called α angle. The β angle is determined between the bottom line of the hole, formed by the points of minimum elevation in the orthogonal sections with respect to the axis of the core sample and the apical trace, which is the extension of the major axis of the discontinuity in the section, in the side of the core sample. Figure 3 schematically shows the angles required for orientation of the discontinuities in core samples.
In the study area, all the drillholes were planned at azimuth 136, arranged in sections with several dips, every 25.00 m, depending on the average dip direction of the Vazante Fault (azimuth 316), recorded in the study of Rostirolla et al. (2002). Thus, the drillholes intercepted the Vazante Fault frontally, as well as the dolomite and marl bedding (S₀) with average dip direction of 317. Therefore, all bedding planes and Vazante Fault (FVZ) were recorded with dip direction to the azimuth 316, and with the angle β being 0° or 180°, as a function of the inclination of the hole. The dip of S₀ and Vazante Fault varied according to angle α found in the core sample. Figure 4 relates stereonets and typical features of the main families of discontinuities in the study area.

### 3. Geotechnical Parameters

In the study area, the rock mass is geotechnically classified as class II and class III (RMR System). In other words, it is controlled by the set of discontinuities and their characteristics. The material between the discontinuities is correlated to the intact rock. The geotechnical parameters of the intact rock are related to the study of Charbel (2015).
who compiled the data of uniaxial compression strength ($\sigma_c$) and elastic modulus ($E_i$) of intact rock collected by IPT (1994b) and Bhering (2009), making the statistical treatment of these values for dolomites, dolomitic breccia and willemite breccia. Then, the Hoek-Brown strength criterion for intact rocks was applied, using the elastic constant of the material, $m_i$, equal to 9, the rock quality parameter $s$ and the constant $a$, equal to 1.0 and 0.5, respectively. The equation of the Hoek-Brown criterion is shown in Table 1.

The Poisson coefficients ($\nu$) adopted for dolomites and breccias are between 0.2 and 0.4, while the unit weights ($\gamma$) are provided by density tests in core samples.

For the stability analysis of rock masses in equivalent continuous media, the equivalent deformation modulus $E_{mr}$ was determined from the elastic deformation modulus of the intact rock ($E_i$) and the Hoek-Brown parameters for equivalent rock masses $m_b$, $s$ and $a$. Therefore, the determination of the GSI (Geological Strength Index) is essential.

The mass disturbance factor “D” was estimated to be 0.8, considering that the excavation method disturbs the rock mass, mainly due to the lack of control in the quantity and sequencing of explosives. Table 2 summarizes the Hoek-Brown parameters for equivalent continuous media analyses.

The shear strength of the discontinuities in the rock mass of the Vazante Mine was obtained from five fundamental parameters: basic friction angle ($\phi_b$) and residual friction angle of the filling material ($\phi_r$), roughness (JRC and Jr), weathering degree of the infill material (Ja), uniaxial compressive strength of the discontinuity wall (JCS) and fill height (Hp).

Based on the data mentioned above, it was possible to determine the shear strength ($\tau$) by the Barton-Bandis and Mohr-Coulomb methods for each geotechnical group of discontinuities.

The discontinuities with the same characteristics of shear strength ($\tau$) were separated into “Geotechnical Groups”. These groups are basically defined by the dominant factor of resistance (L-filling, R-Rock and M-Mix), filling material (CA-Carbonates, MG-Marl, A-Clay), weathering degree (J1, J2, J3 and J4), and, only for the mixed discontinuities, the roughness degree (R1, R2 and R3), as shown in Fig. 5.

The values of the weathering degree of the fillings are correlated with the values of Ja of Barton, in other words, for Ja equal to 1, the degree is J1, if Ja is equal to 2, the degree is J2, and so on until J4.

The roughness degree (R1, R2 and R3) varies according to the JRC value, with R1 having JRC values between 4 and 6, R2 for JRC between 8 and 10, and R3 for JRC values above 12.

Before the presentation of the shear strength equations for each geotechnical group of discontinuities, the basic friction angle ($\phi_b$), unconfined wall compression strength of the discontinuity, and the stiffness coefficients Kn and Kh, will be related. These parameters are required for the calculations to be developed.

![Figure 5](image-url)

Figure 5 - Table of comparison between discontinuities with shear strength controlled by rock, fillings and mix, where Hp/Hr is the relation between fill height and roughness height and $\phi_r$ is the residual friction angle of the filling material.

### Table 1 - Values of $m_i$, $\sigma_c$ and Hoek-Brown equation for each intact rock.

| Rock              | $m_i$ | $\sigma_c$ (MPa) | $\phi' = \phi_r + [m_i(\sigma_c/\gamma) + s] \exp(\gamma s)$ | Poisson ($\nu$) | $\gamma$ (kN/m$^3$) |
|-------------------|-------|------------------|-----------------------------------------------------------------|-----------------|---------------------|
| Dolomite          | 9     | 124.00           | $\phi' = \phi_r + 124 [9(\sigma_c/124)+1]\exp(0.2 s)$           | 0.2             | 27.00               |
| Dolomitic Breccia (BXD) | 9   | 101.00           | $\phi' = \phi_r + 101 [9(\sigma_c/101)+1]\exp(0.2 s)$          | 0.2             | 30.0                |
| Willemite Breccia (BXW) | 9  | 108.00           | $\phi' = \phi_r + 108 [9(\sigma_c/108)+1]\exp(0.2 s)$          | 0.2             | 35.0                |

### Table 2 - Hoek-Brown geotechnical parameters of the Vazante Mine rock masses.

| Rock mass        | $m_i$ | $E_i$ (GPa) | $\sigma_c$ (MPa) | GSI | D     | $m_t$ | $s$     | $a$     | $E_{mr}$ (GPa) |
|------------------|-------|------------|------------------|-----|-------|-------|---------|---------|----------------|
| Dolomites        | 9     | 58         | 124              | 70  | 0.8   | 1.509 | 0.0106  | 0.5     | 18.97          |
| D. Breccia (BXD) | 9     | 55         | 101              | 50  | 0.8   | 0.5   | 0.0005  | 0.5     | 5.0            |
| W. Breccia (BXW) | 9     | 63         | 108              | 50  | 0.8   | 0.5   | 0.0005  | 0.5     | 5.8            |
For the evaluation of the basic friction angles ($\phi$), tilt-tests were performed with BQ core samples of dolomites, breccias and marls.

Schmidt hammer tests were performed on discontinuities of carbonate rocks (dolomites and breccias), with weathering degrees varying from J1 to J4. According to Barton & Choubey (1977), who formulated the first version of the Barton-Bandis shear strength criterion, the method is applicable to unfilled discontinuities; it means rock-to-rock contact in the walls of the discontinuities. Therefore, rebound values of the Schmidt hammer ($r$) were collected for discontinuities of carbonate rocks, without filling, in several galleries of the Vazante Mine. The frequency histograms of the $r$ measurements for discontinuities with intact wall and weathered wall are shown in Fig. 6.

The mean value of “R” for discontinuities with intact walls (J1) is 60, and for weathered discontinuities (J4), “r” is equal to 24. The “r/R” expression value (Barton & Choubey, 1977) for intact walls is equal to 1. The values of “r/R” on the walls with weathering degree equal to J4 is 0.4, and thus the “r/R” values were obtained by linear regression, for the intermediate weathering degrees (J2 and J3), as seen in Fig. 7. This procedure is based on the study of Karzulovic, published in Hoek & Karzulovic (2000), which showed, through rock tests under different weathering degrees, that the loss of resistance to uniaxial compression has a near linear relation with the weathering degrees of the minerals in joint walls.

With the data of “r”, the values of $\sigma$ and $E$ for the unfilled discontinuities from carbonated rocks of the Vazante Mine were obtained through the empirical equations. These values were compared to $\sigma$ and $E$ data obtained in laboratory tests in order to define which equations are more appropriate to the rocks analyzed in this study.

For obtaining $\sigma$, from “r”, the Kahraman equation (1996) reproduced values closer to the data of laboratory tests performed in the Vazante Mine, while the equation of Yagiz (2008), for the calculation of $E$, has shown the best results.

In the literature, the shear strengths of the discontinuities filled with marl and clay were determined by the residual friction angle ($\phi$) and cohesion ($c$) values for application of the Mohr-Coulomb criterion, since there is no data of Schmidt hammer tests for these materials.

The geotechnical parameters of the clays, which mainly fill the faults and fractures of the Vazante Shear Zone, were defined from the values reported by Barton (1970, 1974) and Wyllie & Mah (2004).

The normal stiffness ($K_n$) and shear stiffness ($K_s$) are coefficients that represent the stress required for a unit displacement of the discontinuities, both in the normal direction and tangentially to the plane, adopted from the equations proposed by Barton (1972), Goodman et al. (1968) and data obtained by Kulhawy (1975).

The rough discontinuities are mainly present in breccia and have their shear strength controlled by the material before the joint wall due to the smoothing tendency of the surface when subjected to high stresses. On the other hand, the shear strength of the smooth discontinuities is mechanically controlled by the material of the joint wall and it is typical of the bedding planes of dolomites and marls, as well as fault planes and joints in the Vazante Shear Zone and low angle faults. These features can be met or not, and the shear strength depends directly on this factor. The Barton-Bandis criteria will be applied to unfilled discontinuities, with contact between walls, in dolomites and breccias. Table 3 presents the Barton-Bandis equations for the geotechnical groups in which the shear strength depends on the degree of discontinuity of the walls.

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**Figure 6** - Frequency histogram of the Schmidt hammer rebounds ($r$) measured in breccias and dolomites. Intact rock, on the left, and highly weathered rock, on the right.

**Figure 7** - Linear regression with values of intact rock (Ja = 1) and highly weathered rock (Ja = 4).
Table 3 - Geotechnical parameters and Barton-Bandis equations for shear strength for rough (material before the joint wall) and smooth (material of the joint wall) discontinuities.

| Geotechnical group | Filling material | \( \phi_i \) (°) | \( c \) (kPa) | \( \tau = c + \sigma_v \tan \phi_i \) | \( \phi_i = -20 + 20 (r/R) \) - Barton & Choubey (1977); \( \phi_i = \tan (Jr/Ja) \) - Barton (2002). |
|-------------------|-----------------|---------------|--------------|------------------------------|---------------------|
| RCAJ1 / LCAJ1     | Clay             | 27            | 60           | 1                           | 1                   | 35                 |
| RCAJ2 / LCAJ2     | Clay             | 27            | 48           | 0.8                         | 1                   | 35                 |
| RCAJ3 / LCAJ3     | Clay             | 27            | 36           | 0.6                         | 1                   | 35                 |

\( * \phi_i = (\phi_i - 20) + 20 (r/R) \) - Barton & Choubey (1977).

Table 4 - Geotechnical parameters and Mohr-Coulomb for the smooth discontinuities filled with marl and clay.

| Geotechnical group | Fill material | \( \phi_i \) (°) | \( c \) (kPa) | \( \tau = c + \sigma_v \tan \phi_i \) |
|-------------------|--------------|---------------|--------------|------------------------------|
| LA                | Clay         | 12            | 0            | \( \tau = \sigma_v \tan 12 \) |
| LMGJ3             | Marl         | 20            | 0            | \( \tau = \sigma_v \tan 20 \) |
| LMGJ4             | Marl         | 12            | 0            | \( \tau = \sigma_v \tan 12 \) |

In the smooth and filled discontinuities, the Mohr-Coulomb criterion will be used, with the geotechnical parameters of the filling material according to the Table 4.

The mixed discontinuities are those in which the principal directions of the stresses acting in the Vazante Group were estimated by some authors such as Pinho (1990), Rostirolla et al. (2002), Dardenne (2000), Dardenne & Schobenhans (2001) and Charbel (2015), from the geotectonic evolution of the Tocantins Province. In general, the arrangement of the vectors of in situ stresses suggested by these authors is the same, but with divergences in relation to which directions correspond to \( \sigma_1 \), \( \sigma_2 \) and \( \sigma_3 \).

According to Charbel (2015), as to what regards to the magnitude of the in-situ stresses in the Vazante Mine, there are geological indicators of achievement of a low rate.

In the present work, the guidelines for calculating the stresses will be those adopted by Charbel (2015) as the stress indicators in the Vazante Underground Mine suggested by him are in agreement with the observations made by this authors.

The vertical stress (\( \sigma_v \)), corresponding to \( \sigma_i \), can be calculated from the following equation.

\[
\sigma_i = \sigma_v = \gamma H
\]

where \( \gamma \) is the unit weight of the rock, assuming 0.027 MN/m³, and \( H \) is the thickness of the column of rock mass above the point where it is desired to measure \( \sigma_v \).

Table 5 - Geotechnical parameters and Barton-Bandis equations for shear strength for mix discontinuities.

| Geotechnical group | Fill material | \( \phi_i \) (°) | \( c \) (kPa) | \( \tau = c + \sigma_v \tan \phi_i \) | \( \phi_i = \tan (Jr/Ja) \) - Barton (2002). |
|-------------------|--------------|---------------|--------------|------------------------------|---------------------|
| MCAJ2R1           | Clay         | 27            | 48(0.8)      | 6(1.5)                       | 2                   | 35                 |
| MCAJ2R2           | Clay         | 27            | 48(0.8)      | 8(2)                         | 2                   | 35                 |
| MCAJ3R1           | Marl         | 27            | 36(0.6)      | 6(1.5)                       | 3                   | 35                 |
| MCAJ3R2           | Marl         | 27            | 36(0.6)      | 8(2)                         | 3                   | 35                 |
| MCAJ3R3           | Marl         | 27            | 36(0.6)      | 10(3)                        | 3                   | 35                 |
| MCAJ4R1           | Marl         | 27            | 24(0.4)      | 6(1.5)                       | 4                   | 35                 |
| MCAJ4R2           | Marl         | 27            | 24(0.4)      | 8(2)                         | 4                   | 35                 |
| MCAJ4R3           | Marl         | 27            | 24(0.4)      | 10(3)                        | 4                   | 35                 |

\( * \phi_i = (\phi_i - 20) + 20 (r/R) \) - Barton & Choubey (1976); \( \phi_i = \tan (Jr/Ja) \) - Barton (2002).
The horizontal stresses $\sigma_x$ and $\sigma_y$, which correlate to $\sigma_2$ and $\sigma_3$, respectively, have a correlation rate of $\sigma_x/\sigma_y = 1.4$.

### 5. Stability Analysis

The sections 13225 and 13300 were selected for the stability analysis, and the drillholes 1, 3, 5, 7 and 9 of each section were logged in order to obtain the geological and geotechnical classifications.

After modeling of the discontinuities, shown in Fig. 8, 2D sections were drawn for the finite element analysis.

Wedge kinematic analyses were made to verify the stability of the excavations walls. The studies were performed using the software Unwedge®, version 4.0, developed by Rocscience. In order to effectively evaluate the results, the Safety Factor (FS), regarding the resisting forces vs. the destabilizing forces, was used, with the adopted critical value of 1.5, following a tendency observed in failed wedges in the Vazante Mine as well as in tutorials and case studies of the software used.

The kinematic analyses of the ore galleries (GM) were used to assist in the adoption of a reinforcement system aimed to guarantee the safety of operations, besides retaining the waste material, causing a greater blasting efficiency.

The tiebacks used to retain the wedges have characteristics similar to those routinely used in the Vazante Mine. The bars have tensile strength of 0.27 MN/m, with an cement-rock adhesion of 0.34 MN/m, with 3.00 m length (1.00 m anchored), and a distance of the lines of 1.50 m. Several geometrical arrangements of the lines were tested, and the most efficient one was used, being very similar to the one used in the Mine of Vazante today.

In the transportation galleries (GT), the transit of people and equipment, often for an extended period of time, makes the retainment system very important for the safety of mine operations in general.

Figure 9 shows the stability analysis for the 300-GM2 gallery and for the 210-GT gallery, in the section 13300, where the wedges with the most critical safety factors were detected.

The mining stopes at the Vazante Mine are generally retained through the hanging wall of the ore body with cables in order to avoid operational dilution from the blasting of the waste material. The safety of people and machinery are also a motivation for the stabilization of unstable zones of mining stopes.

The modelled sections show that the ore bodies assume different angulations in different zones of the mineralization and, consequently, mining stopes with different angles and stress concentration. The locations where the lower angulations are verified, between 40° and 60°, are related to the Low Angle Fault Zone, where there is a fault that displaces the ore body between the levels 326 and 270. The other stopes above the level 326 and below the level 270 have in general an inclination between 60° and 80°.

The different inclinations of the mining stopes lead to different instability conditions, where in general the less inclined stopes showed unstable wedges on the hanging wall and the footwall of the ore body, whereas in stopes with higher inclinations, unstable wedges are limited by the hanging wall zone. Therefore, two retention patterns were initially adopted, one with stabilization only of the hanging wall, and the other with stabilization of the hanging wall and the footwall. The cables used for the studies are the same as those currently adopted at Vazante Mine, with a diameter of 15.00 mm, a young modulus of 200 GPa and a tensile strength of 0.27 MN, as well as variable lengths between 6.00 and 20.00 m, and line spacing of 1.50 m.

Figure 10 shows the kinematic analyses of wedges, with cable arrangement and FS, for the stopes 270-300, and 240-270 in section 13300, where the most critical wedges were detected.

The finite element analyses were performed using Rocscience’s Phase2® software, version 9.0, and allowed...
the analysis of the stress concentrations induced in the excavations designed for the study area. Discontinuous models were developed, with the modeling of the discontinuities present in each region, and for the purpose of comparison, continuous models were also produced and analyzed with the adoption of equivalent parameters for the rock masses.

According to Curran et al. (2008) and Azami et al. (2013), the geotechnical parameters of the rock mass in the discontinuous models should be close to the values for the intact rock. Thus, the parameters of Hoek-Brown for a GSI of 90 for the waste rocks (Pink and Gray Dolomites), and an GSI of 80 for the Dolomitic and Willemite Breccias were assumed. The assumption of GSI of 80 for the breccias comes from the difficulty of representation of localized discontinuities, without persistence in the Vazante Fault Zone. The Hoek-Brown parameters, for continuous media and discontinuous media, are summarized in Table 6.

The sections 13225 and 13300 were split into upper and lower sections, in other words, into four sections of analysis, two sections between levels 345 and 300, and two between levels 210 and 300. All sections have an orientation from NW to SE, approximately. The stress-strain analysis induced in the designed excavations was simulated in stages, according to the order of execution. In this way, it was possible to analyze the stability of the excavations un-
der stress conditions closer to reality, taking into account the interferences generated by neighboring excavations. In addition, it was possible to evaluate basic assumptions for the design of the mining stopes, such as the possibility of application of the VRM method, with direct connection between the top and base galleries of the stopes, and subsequent filling, or if a sill pillar is required, and which thickness is required.

Next, in Fig. 11, the geological model and excavations for the upper 13300 section, chosen to illustrate the methodology, are shown with the final solutions for the design of the mining stopes. The analysis steps presented are from the excavation of the galleries (step 2), and the first stage of analysis consists in the stress distribution in the rock mass without the excavations.

From relations between vertical and horizontal in-situ stresses shown before ($\sigma_v = 1.4 \sigma_h$), the values adopted for the top section of profile 13300 were $\sigma_v = 9.5$ MPa and $\sigma_h = 7$ MPa.

An elastoplastic constitutive model was adopted, with residual geotechnical parameters of the rock masses estimated, based on values reported by Crowder & Bawden (2006), obtained through the GSI of the material. For materials with GSI above 70, the residual Hoek-Brown parameters are $m_r$ equal to 1, $s$ equal to 0, and dilation of 0. For materials with GSI less than 50, the $m_r$ value assumed is equal to $0.5 \times m_b$, $s$ equal to 0, and dilation also 0. It is worth remembering that to obtain the residual or post-peak geotechnical parameters of rock masses is extremely difficult and the best way to acquire these data would be through instrumentation, which does not exist in the Vazante Mine, and subsequent back analysis.

In order to simulate the effect of the strain that occurs prior to the installation of the tiebacks in the galleries, an internal pressure of 50% of the stress induced externally to the excavations was assumed against the walls when the tiebacks were installed. In this way, the tiebacks begin to act after half the strain has already occurred, which simulates a situation closer to reality.

The total critical displacement was used to evaluate the strain state of the excavation walls and corresponds to the displacement value in the rock mass from which the dis-
placement process begins; in this study a value of 2.00 cm was assumed. This value is based on the studies developed by Charbel (2015), based on back analysis of mining stopes with great operational dilution, reaching values of up to 60% at level 388 of Vazante Mine.

The characteristics of the tiebacks and cables used to retain the excavations in the finite element analysis are the same as those used in the wedge kinematic analysis.

The finite element meshes used in the analysis for the discontinuous media followed a standard, and the discretization used the intersection points of the discontinuities between them, and with the walls of the excavations and lithological contacts. In addition, other points were created along the lines of the discontinuities, lithological contacts, excavation walls and containment elements, where a distance greater than 0.50 m was observed between the pre-established points. The meshes adopted for the equivalent continuous media models followed a simpler pattern, with pre-definition of the distance between the discretization points, and densification of the points on the walls of the excavations, lithological contacts and along the retaining elements.

All ore galleries (GM) located in the study area were analyzed for their stability by valuation of the total displacements in the rocky masses around the excavations in continuous and discontinuous media.

The 326 GM gallery of section 13300 was chosen to demonstrate the results obtained in the finite element stability studies. The results of the analysis by continuous and discontinuous media showed similarities, with displacements exceeding 2.00 cm in the roof of the gallery.

By observing the strain zones formed in the non-reinforced excavations, it is evident that the interaction between the discontinuities and the rock blocks, reproduced in the discontinuous analysis, details more efficiently where the largest accumulations of stress will occur. In that analysis, the largest displacements are verified within a wedge shape formed in the roof, which resembles the wedges found in the wedge kinematic analysis. In the analysis made after the installation of the tiebacks, the verified displacements are below 2.00 cm, validating the efficiency of the reinforcement system, as can be seen in Fig. 12.

As previously described, the transport galleries (GT) are used for long periods during the life of the mine, leading to the analysis of long-term displacements, caused by suc-
cessive excavations of the mining stopes. Therefore, the
studies to be presented below show the stability situation
after the excavation of all stopes (most critical situation),
and the behavior of the anchorage system in the same anal-
ysis step.

The 300 GT gallery studies, in the section 13300, in-
dicated very close results for the models without reinforce-
ment, with displacements from 2.50 to 3.00 cm. The appli-
cation of the tiebacks limited these displacements to
1.50 cm in the discontinuous model, and it is considered
that it practically did not work in the analysis by continuous
media, with the same 2.50 cm, compared to the analysis
without tiebacks.

In addition to providing safety for persons and equip-
ment during the operating activities of the mine and retain-
ing waste material, reinforcements in the mining stopes
work restricting the strain zone generated by the concentra-
tion of stresses at certain points of the rock mass around the
excavation, so that they have as less influence on the neigh-
bor ing excavations (galleries and other mining stopes) as
possible.

The design of the reinforcement system was deter-
mined through wedge kinematic analysis, and reevaluated
in the study of stress vs. strains.

The analyzed stope, section 13300, is between the
levels 326 and 345, with inclination of approximately 60°,
and height just over 15.00 m. The upper sill pillar is 5.00 m,
below the gallery 345 GM, and the lower one is 7.00 m, be-
low the gallery 326 GM.

The total displacements observed in this stope were
close for the continuous and discontinuous analyses, with-
out the application of the reinforcement system, with values
between 4.50 and 5.50 cm. With the application of the rein-
forcement, the total displacements were reduced to values
between 3.00 and 4.00 cm in the analysis by continuous
media, and to values below 2.00 cm in the analysis by dis-
continuous media.

The total displacements observed in the sillon pillars re-
mained in the same order of value in relation to the analyses
by continuous media, with and without the installation of
the cables, while there was a reduction of values close to
3.00 cm, for values of 1.50 cm with the cables applied.

Figure 13 illustrates the continuous and discontinu-
ous media analyses of the stopes between the GM and GM
galleries 326.

The results obtained in all the finite element analysis
made on sections 13300 and 13225 for the galleries in con-
tinuous and discontinuous media can be briefly seen in
Fig. 14 and Fig. 15. Figure 16 highlights the results of the
analysis and stopes only for section 13300.

6. Conclusions and Recommendations

In general, it can be concluded that the purposes of the
study were achieved as all the possible information from
the drill core surveys of the studied region was obtained,
which provided data for the analysis of the stability of the
excavations in discontinuous media. Previous knowledge
of the local geological-structural framework, and of the
geotechnical behavior of the rock masses, were also essen-
tial for the geological-geotechnical modeling.

The total displacements of the rock masses on the
walls of the galleries, after the installation of the set of

Figure 13 - Finite element analysis of the stope between the galleries 326 GM and 345 GM, on section 13300. On the left, the analysis by continuous media, and, on the right, the analysis by discontinuous media. Above, the analysis without the cables, while below the analy-
sis with the cables, just after the excavation.
tiebacks, remained close to the limit of 2.00 cm, a value considered critical for rupture processes according to back analysis of overbreaking stopes. In the discontinuous models, all the displacements were below 2.00 cm after the application of the tiebacks. In the continuous model, some displacements reached approximately 2.50 cm. Anyway, this result is considered satisfactory, because of the conservative nature of the method. Therefore, the tieback systems

Figure 14 - Summary of the characteristics of the rock masses, results of the analysis and reinforcement system of the ore galleries.

Figure 15 - Summary of the characteristics of the rock masses, results of the analysis and reinforcement system of the transport galleries.
initially suggested by the wedge kinematic analysis for the GM’s and GT’s can be considered satisfactory for all the analyzed galleries.

The mining stopes had their reinforcement system initially defined by the kinematic analysis of the critical wedges, in the same way as the galleries. These analyses indicated wedges with a safety factor lower than 1.3 on the hanging wall, foot wall and roof of the stopes, which were stabilized with cables, in order to reach a FS of 1.5.

The thickness of the sill pillars varied according to the inclination of the stopes, having 5.00 m in excavations with inclinations greater than 60°, and 7.00 m for stopes with an inclination lower than 60°. In the presence of parallel stopes, as occurs in the lower portion of section 13300, sill pillars of 7.00 m are also suggested.

The results obtained for the finite element analysis, by continuous media and by discontinuous media, were very different for the studies of the mining stopes, contrary to the analysis made in the galleries. The total displacements observed, with and without the installation of the reinforcement system, are much greater in continuous models. In the analysis by discontinuous media, the majority of the results of total displacements, after the application of the cables, showed values close to 2.00 cm, whereas in the analysis by continuous media the displacements exceeded significantly the 2.00 cm in almost all the cases.

Local geological contexts were also of great importance for the evaluation of the efficiency and the need for reinforcement. In the stopes in which the foot wall area of the ore body is composed of gray dolomite, the displacements are much reduced, being below 2.00 cm in all cases, regardless of the inclination and height of the stopes. In the stopes where the foot wall is composed of dolomitic breccia, according to the analysis made, it is required to apply a cable as the total displacements exceeded 2.00 cm, and the largest deformation areas were verified in the stopes with inclination less than 60°. The presence of thick bundles of marl in the hanging wall region also induce high total displacements, as evidenced in the analysis of the stopes between the levels 300 and 326, in section 13300.

Therefore, the cable system applied in the mining stopes was considered satisfactory for the analysis of critical wedges, with safety factors greater than 1.5, and for finite element analysis with discontinuous media, with total displacements close to 2.00 cm. However, in the analysis by finite elements by continuous media, the observed displacements were high, even after the application of the reinforcement system, with values between 7.00 and 10.00 cm.
Finally, complementary studies are recommended, namely for comparison of the results obtained with the actual results of the excavations to be executed in the study region, so that the methodology and the parameters are calibrated for the analysis by discontinuous media. In addition, the post-peak, or plastic, geotechnical parameters of the rock masses involved in the analysis should be better studied through instrumentation and back analysis. Stability studies by distinct elements are also indicated, since the geotechnical characterization of the discontinuities performed in this work provides enough information for that, in order to compare the results obtained in the finite element analysis and wedge kinematic analysis. The shearing and axial stresses in the cables can also be better studied through the discontinuous models as it is possible to evaluate relative movements between blocks delimited by the discontinuities present in the rocky masses.

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