Stress Calibration at Cuiabá Mine, Brazil

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ABSTRACT: The construction of any numerical model requires several input parameters such as rock mass properties or geometry of the mine, to be representative. One of these inputs is the in-situ stress condition obtained from stress measurements, which in some cases are rather difficult to locate far enough of the excavation to obtain reliable in situ data. This study used the methodology proposed by McKinnon (2001), assuming that in every location of the model the stress tensor was the result of the superposition of gravitational, tectonic and induced stresses. Thus, applying a linear regression using least squares method the in-situ stress condition was determined, obtaining an acceptable average error and validating that the location of the stress measurements was acceptably located. Posteriorly, two mining sectors of interest were calibrated using more detailed 3DEC models, replicating consistently the behavior observed by AngloGold Ashanti.

KEYWORDS: In Situ Stress, Stress Calibration, 3DEC.

1 INTRODUCTION

1.1 Background

Cuiabá Mine is located at Sabará, at the northwest portion of Quadrilátero Ferrífero - Central region of Minas Gerais, Brazil. The underground gold mine belongs to AngloGold Ashanti (AGA), and is currently operating down to 1200 m depth, adopting sublevel stoping as the main mining method. Understanding the rock mass behavior and the in-situ stress condition at Cuiabá Mine is essential to optimize the mine layout, increase the operation sustainability and understand effects of the deepening of the mine.

The mine is situated in the Nova Lima Group, inferior unit of Rio das Velhas Supergroup, which is essentially constituted of metavolcanic, metavolcanoclastic and metasedimentary rocks. The gold mineralization is presented in massive, banded or disseminated sulphide minerals such as pyrite, pyrrhotite and arsenopyrite, that are associated with the Banded Iron Formation (BIF) that occurs in the inferior unit of Nova Lima Group. The host rock mass in the intermediate and superior units of Nova Lima Group, is composed mainly of pelitic and volcanic rocks, showing a prominent metamorphic foliated structure.

Geomechanical characteristics of the orebody and the host rock mass were obtained through drill core logging, laboratory tests and cell mapping. In simplified terms, there are notably different behaviors between the orebody and the host rock. The first lithotype (BIF) is a hard and fragile rock, meanwhile the second lithotype is composed by different types of schists (MANX, X1, XG, MBAX) as shown in Figure 1.

Figure 1. Simplified Geological Model – Elevation 270 m

The schists are also highly anisotropic due to their mineralogical variability and the presence of foliation, a major structural feature that leads to possible instabilities in the excavations.

Besides the rock mass strength and structural characteristics, the magnitude and direction of in
situ stresses is one of the most important input parameters in numerical analysis of mine excavations. Small changes in these stresses can lead to great changes in the mine design and consequently in the financial result of a project. At Cuiabá Mine, the in-situ stresses at the current production levels are high enough to fail the rock mass, and stress induced failures such as spalling, buckling and breakouts are observed in development drifts and ventilation raises excavated in schists. Also, BIF drillholes samples have been presenting various intervals with disking, which is an indication of high deviatoric stresses.

This work presents the results of an in-situ stress calibration for Cuiabá Mine, using overcoring stress measurements and a mine scale three-dimensional numerical model. To calibrate the rock mass strength parameters, the resulting stress condition obtained was introduced in small and detailed models used to back analyze the behavior observed in two instability cases:
- hanging wall failure in Serrotinho orebody (SER) on level 16;
- sill pillar failure in Fonte Grande Sul orebody (FGS) between levels 15.1 and 14.

2 PROPERTIES

2.1 Intact Rock Strength

The geology consists in two main units: the BIF orebody characterized as a hard rock with an average UCS of 180 MPa and a coefficient of variation of 32%, and the moderately hard host composed by schistose rocks with an average UCS of 60 MPa and a coefficient of variation of 55%. In both units there is considerable variability.

According to mine staff observations, the BIF orebody strength appears to increase with greater depth. The average UCS was calculated for several depth ranges making it possible to estimate the relation shown in Equation 1 below.

\[
UCS \text{ (MPa)} = 0.15 \times \text{Depth (m)} + 105 \quad (1)
\]

In case of the schistose rocks, their variability is linked to the direction of the applied load with respect to the anisotropy planes. Using the lab tests in schist material where the load direction was measured, and the relation proposed by Jaeger (1960), resulted in a reasonable fit to the data when 2 MPa of cohesion, 35° friction angle for joints and 65 MPa of UCS for intact rock.

![Figure 2. UCS lab tests and numerical results for ubiquitous joint model using 2 MPa of cohesion, 35° friction angle for joints and 65 MPa of UCS for intact rock.](image)

The numerical model considered schistocity oriented parallel to the orebody/host rock contact.

2.2 Rock Mass Quality

The 3D rock mass model provided for this study consisted on solids for different ranges of RMR$_{89}$. This model had differences between the observed behavior mentioned on the previous section and the RMR ranges. Considering the low discretization of the RMR model (composed only by 5 ranges) and hence the reduced variability of the data for calibration purposes, the raw data from core logging and cell mapping was analyzed to better determine a distribution of rock mass quality index per unit. This determination was possible using the methodology presented by Hoek et. al. (2013), which uses parameters usually obtained from core logging process as expressed in Equation 2.

\[
GSI_{2013} = 1.5 \times J_{cond89} + RQD/2 \quad (2)
\]

The estimation using the previous relation proved to be a conservative approach to the rock mass quality, making GSI$_{2013}$ a more suitable index for this study.

2.3 Spatial variability and properties of materials

Knowing that in most cases the rock mass is not homogeneous, it becomes imperative to quantify and incorporate variability into the
stability analysis. There are several works in numerical modeling area that show as a good practice the use of specific values lower than the average, taken from the statistical distribution of GSI and/or UCS to define the rock mass strength for the analysis (e.g. Jefferies et. al., 2008). A recent work by Renani et. al. (2018) for numerical modeling suggests the use of the relation in Equation 3, where COV($\sigma_{ci}$) is the Coefficient of Variation of UCS, $\sigma_{ci}$ mean the average UCS and $\sigma_{ci}$ det is the reduced value of UCS used in a deterministic analysis to give the same Factor of Safety (FoS) similar to mean FoS calculated from a full probabilistic analysis.

$$\sigma_{ci}^{\text{det}} = \sigma_{ci}^{\text{mean}} [1 - 0.36 \text{COV}(\sigma_{ci})] \tag{3}$$

Table 1. Summary of Intact Rock Properties

<table>
<thead>
<tr>
<th>Unit</th>
<th>Density (kg/m$^3$)</th>
<th>UCS (MPa)</th>
<th>GSI$^{2013}$</th>
<th>Poisson E ($\text{GPa}$)</th>
<th>$m_i$</th>
</tr>
</thead>
<tbody>
<tr>
<td>BIF</td>
<td>3100</td>
<td>182</td>
<td>58</td>
<td>161</td>
<td></td>
</tr>
<tr>
<td>X1_ext</td>
<td>2600</td>
<td>62</td>
<td>34</td>
<td>50</td>
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</tr>
<tr>
<td>X1_01</td>
<td>2600</td>
<td>63</td>
<td>29</td>
<td>53</td>
<td></td>
</tr>
<tr>
<td>XS_01</td>
<td>2600</td>
<td>63</td>
<td>35</td>
<td>51</td>
<td></td>
</tr>
<tr>
<td>XG_estructura</td>
<td>2600</td>
<td>58</td>
<td>34</td>
<td>46</td>
<td></td>
</tr>
<tr>
<td>MANX_int</td>
<td>2600</td>
<td>61</td>
<td>36</td>
<td>48</td>
<td></td>
</tr>
<tr>
<td>MANX_ext</td>
<td>2600</td>
<td>65</td>
<td>37</td>
<td>52</td>
<td></td>
</tr>
</tbody>
</table>

Table 1 shows a summary of intact rock properties and Table 2 a summary of rock mass quality. Differences between the elastic modulus (Ei) of the BIF orebody and the schist host rocks are noted, from which can be defined in a qualitative approach that BIF has a more brittle behavior than the other rocks.

Table 2. Summary of Rock Mass Properties

<table>
<thead>
<tr>
<th>Unit</th>
<th>GSI$^{2013}$</th>
<th>Rock Mass</th>
<th>Ei ($\text{GPa}$)</th>
<th>$m_i$</th>
</tr>
</thead>
<tbody>
<tr>
<td>BIF</td>
<td>69</td>
<td>14</td>
<td>0.22</td>
<td>97</td>
</tr>
<tr>
<td>X1_ext</td>
<td>50</td>
<td>17</td>
<td>0.25</td>
<td>55</td>
</tr>
<tr>
<td>X1_01</td>
<td>55</td>
<td>16</td>
<td>0.25</td>
<td>64</td>
</tr>
<tr>
<td>XS_01</td>
<td>51</td>
<td>21</td>
<td>0.26</td>
<td>55</td>
</tr>
<tr>
<td>XG_estructura</td>
<td>52</td>
<td>15</td>
<td>0.25</td>
<td>43</td>
</tr>
<tr>
<td>MANX_int</td>
<td>68</td>
<td>16</td>
<td>0.23</td>
<td>60</td>
</tr>
<tr>
<td>MANX_ext</td>
<td>63</td>
<td>18</td>
<td>0.23</td>
<td>55</td>
</tr>
</tbody>
</table>

2.4 CaveHoek Constitutive Model

CaveHoek is a numerical constitutive model developed by Itasca for cave mining. However, it allows the representation of rock mass behaviors where the following phenomena are present:
- progressive reduction in strength from peak to residual levels (strain-softening);
- dilation, bulking, fragmentation and mobilization of the failed material;
- both Hoek-Brown and ubiquitous joint behavior.

The use of this numerical constitutive model allows a much more realistic characterization of the rock mass, considering both elastic and plastic behavior of the material, controlled by the critical plastic shear strain.

The ratio of the horizontal to vertical stress ($K_0$) calculated from data obtained from the 3DEC model responds according to common estimations found in literature (Hoek, 1978) as seen in Figure 5. In addition, in-situ stress measurements agree with the estimated behavior for Cuiabá Mine following the relation $K_0=200/z+0.5$, where $z =$ depth (m). It can be noted that in shallow areas the tectonic stress (horizontal) is greater than the vertical stress which is translated in $K_0>1.0$; whereas at deeper levels $K_0<1.0$ due to a greater vertical stress.

3 IN-SITU STRESSES

3.1 Field Measurements

Table 3 shows the in-situ stress measurements available, incorporated and used as basis for the calibration model built in 3DEC.

3.2 Methodology

According to McKinnon (2011), the premining stress field at any location is assumed to be comprised of gravitational and tectonic components. Considering that gravitational stress can be calculated knowing the depth at which the measurement was taken and assuming tectonic stress acting entirely in the horizontal plane, it is possible to calculate the measured stress as a linear combination of unitary stresses applied in different cartesian directions. Thus, the error between the stress measurement and tensor of the model in the same location can be minimized using least squares fit, obtaining finally the coefficient of each unitary stress.
According to the data in Table 3, σ₁ at Level 14 shows a remarkable difference with the rest of the data. This can be explained by the location near a fold in the orebody of this measurement. Due to this, it was decided to discard the data corresponding to Level 14.

3.3 Results

Several models ran separately using different directions for the unitary stresses until reaching elastic equilibrium. The stress tensor was obtained from each model in the locations where the measurements were performed. Applying Equation 4 for each location the error was calculated, and an average global error was estimated considering every measurement, obtaining a value of 34% which is typical for such studies. Table 4 shows the percentage of error obtained from the numerical model in each location.

\[
\varepsilon = \sqrt{\frac{\sum (\Delta \sigma)^2}{\sum (\Delta \sigma_{measured})^2}}
\]  

(4)

Once equilibrium of the numerical model has been reached using a plastic constitutive model, it was possible to estimate the in-situ stress condition as the following:

- the vertical stress (σzz) is similar to σ2;
- at the surface (zero depth) σ1 is approximately 15 MPa;
- minimum principal stress (σ3) is approximately one third of σ2.

To corroborate the validity of the relation proposed, in-situ stress measurements were evaluated in comparison with the relation determined in a previous study by Itasca (2009). This new estimation appears to be better than the previous relation proposed, as it can be seen in Figure 3 and Figure 4.

Figure 3. In-situ stress measurements evaluated in stress relationship developed in 2009.

The ratio of the horizontal to vertical stress (K₀) calculated from data obtained from the 3DEC model responds according to common estimations found in literature (Hoek, 1978) as seen in Figure 5. In addition, in-situ stress measurements agree with the estimated behavior for Cuiabá Mine following the relation K₀=200/z+0.5, where z = depth (m). It can be noted that in shallow areas the tectonic stress (horizontal) is greater than the vertical stress which is translated in K₀>1.0; whereas at deeper levels K₀<1.0 due to a greater vertical stress.

Figure 4. In-situ stress measurements evaluated in stress relationship developed in 2017.

Figure 5. Ratio of average horizontal to vertical stress at different depth. In black typical values and in red for Cuiabá.
Table 3: In situ Stress Measurements – Principal Stresses (compressive stresses are negative)

<table>
<thead>
<tr>
<th>Measurement</th>
<th>Magnitude (MPa)</th>
<th>Dip (°)</th>
<th>Dip Dir (°)</th>
<th>Magnitude (MPa)</th>
<th>Dip (°)</th>
<th>Dip Dir (°)</th>
<th>Magnitude (MPa)</th>
<th>Dip (°)</th>
<th>Dip Dir (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level 11**</td>
<td>-37</td>
<td>38</td>
<td>127</td>
<td>-23</td>
<td>51</td>
<td>321</td>
<td>-7</td>
<td>7</td>
<td>223</td>
</tr>
<tr>
<td>Level 12*</td>
<td>-24</td>
<td>60</td>
<td>188</td>
<td>-23</td>
<td>26</td>
<td>40</td>
<td>-11</td>
<td>-14</td>
<td>304</td>
</tr>
<tr>
<td>Level 13***</td>
<td>-32</td>
<td>56</td>
<td>303</td>
<td>-23</td>
<td>34</td>
<td>132</td>
<td>-5</td>
<td>4</td>
<td>39</td>
</tr>
<tr>
<td>Level 14*</td>
<td>-81</td>
<td>4</td>
<td>312</td>
<td>-38</td>
<td>9</td>
<td>42</td>
<td>-26</td>
<td>82</td>
<td>199</td>
</tr>
</tbody>
</table>

*CISR (2004), "MEASUREMENT OF ROCK STRESS AT CUIABÁ GOLD MINE BRAZIL"
**SBMR (2016), "Uso de células tipo STT-Furnas na determinação de tensões in situ na Mina Cuiabá – Sabará/MG"
***AGA (2017), "Reinterpretação Das Medicações De Tensões In Situ Por Sobreforação (Overcoring) Nos Níveis 11, 13 E 17"

Table 4. Results of errors obtained from the stress calibration numerical model

<table>
<thead>
<tr>
<th>Measurement</th>
<th>Case</th>
<th>σxx (MPa)</th>
<th>σyy (MPa)</th>
<th>σzz (MPa)</th>
<th>σxy (MPa)</th>
<th>σxz (MPa)</th>
<th>σyz (MPa)</th>
<th>Error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level 11</td>
<td>Measured</td>
<td>-21</td>
<td>-17</td>
<td>-28</td>
<td>12</td>
<td>7</td>
<td>-3</td>
<td>34</td>
</tr>
<tr>
<td></td>
<td>Model</td>
<td>-20</td>
<td>-16</td>
<td>-19</td>
<td>6</td>
<td>-1</td>
<td>0</td>
<td>34</td>
</tr>
<tr>
<td>Level 12</td>
<td>Measured</td>
<td>-15</td>
<td>-20</td>
<td>-23</td>
<td>-5</td>
<td>6</td>
<td>-5</td>
<td>34</td>
</tr>
<tr>
<td></td>
<td>Model</td>
<td>-17</td>
<td>-16</td>
<td>-21</td>
<td>7</td>
<td>-1</td>
<td>0</td>
<td>44</td>
</tr>
<tr>
<td>Level 13</td>
<td>Measured</td>
<td>-18</td>
<td>-13</td>
<td>-30</td>
<td>10</td>
<td>-4</td>
<td>1</td>
<td>24</td>
</tr>
<tr>
<td></td>
<td>Model</td>
<td>-17</td>
<td>-17</td>
<td>-23</td>
<td>6</td>
<td>-1</td>
<td>0</td>
<td>24</td>
</tr>
</tbody>
</table>

4 CALIBRATION

4.1 Mine scale model

Considering the in-situ stress condition obtained from previous process, a mine scale 3DEC model was built, following the mining stages listed below.
- End 2010, containing all previous excavations to this date;
- end 2012;
- end 2014;
- end 2015;
- end 2016.

The sequence adopted for this model was to excavate all stopes in the stage at the same time and to fill them during the next stage of excavation.

The stress condition during each excavation stage at the location of the previous mentioned stress measurements was recorded and its respective error from the original measurement was calculated. The results showed no appreciable deviation from the initial error determined without excavations, proving that the locations chosen to take the measurements comply with the requirement of not being significantly influenced by the mining.

4.2 Detailed models and stress transfer

Two specific sites were selected for calibrating the properties of the materials in the numerical model. The first site is located in Serrotinho Level 16 (SER L16) where a hanging wall failure occurred, and the second site is located in Fonte Grande Sul Level 15.1 and 14 (FGS L15.1-14) where the sill pillar between the mentioned levels presented instability.

Detailed models were generated for both cases, using a FISH routine developed by Itasca to transfer the stress condition from the mine scale model (less detailed) to the detailed sectors for analysis.

For SER L16, the stress condition captured at the end of the 2012 stage was transferred. The sequence for replicating the observed behavior was according the following:
- mining 2014 stope;
- failure which resulted in overexcavation of the hanging wall of the stope;
- rockfill of 2014 stope;
• mining of 2015 stope;
• rockfill of 2015 stope;
• mining of 2016 stope without filling.

This calibration model considered mainly the MBAX_Externo unit located in the hanging wall area. A default residual envelope for the CaveHoek model considers 30% porosity according to the empirical method developed by Barton and Kjønsli (1981), which is approximately zero cohesion and 42° friction angle; but if the envelope is defined according to a 10% porosity, the result is substantially different, thus it becomes a fundamental property to calibrate in this detailed model. Also, the critical plastic shear strain must be determined to define if the rock will behave in a more elastic or plastic fashion via multiplier ($\text{mult-ecrit}$), which modifies the Equation 4 used to calculate the critical plastic shear strain:

$$d = \text{zone size}.$$ If the value of the multiplier approximates to zero, the model will simulate perfectly brittle behavior, while in contrast a value near infinite will produce a perfectly ductile behavior. Several trial cases were analyzed adjusting variables such as the percentage of porosity, $\text{mult-ecrit}$ and dilation.

4.3 Calibration results

The SER L16 model replicated consistently the observed instability in the hanging wall. A comparison between the DXF delivered by AGA using laser scanner and the unstable blocks determined by the model can be seen in the Figure 6.

This instability was reproduced using a residual porosity = 10%, $\text{mult-ecrit} = 0.1$ and for the anisotropy, peak cohesion = 2 MPa, peak friction = 27°, residual cohesion = 0 MPa and residual friction = 20° for the MBAX_Externo and a residual porosity = 30% and $\text{mult-ecrit} = 0$ for the BIF; both materials used 10° of dilation. The instability is controlled by the anisotropy planes in the hanging wall with a β angle greater than the friction angle, and as seen on Figure 2, which leads to lower values of UCS.

For FGS L15.1-14, a sudden change of inclination was noted near the failure area with concentration of stress magnitudes around 40 MPa before excavation. Figure 7 shows higher levels of plasticity in the sill pillar, indicating behavior near or in residual condition.

AGA also provided information related to microseismic events registered for the same period of this last model. The majority of these events occurs within the BIF orebody, in the same locations where the model shows zones with plasticity indicators as seen in Figure 8, which means the zone has reached the failure envelope in tensile or shear condition.
5 CONCLUSIONS

5.1 Intact rock strength and rock mass behavior

Based on the lab test two separated responses were clearly seen from the materials on Cuiabá Mine. The BIF can be classified as a competent rock with an UCS greater than 180 MPa as it increases with depth and a brittle behavior, whereas the rest of materials have lower UCS and anisotropy produced by the schistocity of metamorphic rocks. The strength of the anisotropy for these schist materials was determined using an analytical approach, resulting in 2 MPa of cohesion and 27°-35° friction.

Due to the low discretization of the RMR model and hence a low variability of data for the calibration, a different approach using the raw data from drill logging and map cells was made by Itasca to obtain the distribution of rock quality index by unit.

5.2 In-situ stress condition

To determinate the in-situ stress condition a methodology widely employed by Itasca was used, obtaining 34% of average error for the estimation. Applying this initial stress condition to the 3DEC model, a new relation for the principal stresses was determined, showing slightly better estimations than the previous study made in 2009. Also, the particular value of Ko for Cuiabá Mine was estimated, matching the expected results according to the literature.

5.3 Calibration models

Two sectors with historical instabilities were chosen to calibrate the properties of the materials. The results showed an acceptable correlation with the site observations. Listed below are the assumptions and parameters derived from these detailed models:
- BIF: Perfectly brittle behavior (critical plastic shear strain ~0)
- Rest of the materials: Ductile behavior (~10% of the critical plastic shear strain calculated by default in CaveHoek)

5.4 Proposed three-dimensional analysis

A complementary work applying the results of this part of the study is in progress, in order to provide guidelines for the sill pillar thickness, sublevel design and/or ground support requirements based on stability charts built as result of hundreds of generic and simplified 3DEC models to represent different and design parameters such as stope height, strike length, orebody width and depth and inclination. This unique approach could provide greater utility and flexibility instead of evaluating specific mine plans.

REFERENCES