Ground support modelling involving large ground deformation: Simulation of field observations – Part 1

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Abstract

The Kristineberg mine has a long history of large ground deformation which consequently incites ground control problems for the mine. Over the years the mine has developed various mining techniques, backfilling and ground support procedures to manage this problem. In general the ground control problems at the mine are highly influenced by the wall rock geology. The wall rock, that is the footwall and hanging wall, comprise of highly altered chlorite schist, which are internally referred to as talc-schist. They very often occur as seams with thickness barely ranging from 0.1 m to as wide as 3.0 m. Coupled with high ground stresses the talc squeezes and slides into the stope if undercut by the excavation, or either bends or bulges inwards when exposed but not undercut depending on the loading direction. The deformation magnitudes have often been reported to be in the order of 0.2 to 0.5 m and seldom up to 1.0 m. Conventional rock support system, consisting of fibre re-enforced shotcrete and rebar rock bolts, has regularly failed under these conditions. As part of Ground Support Research Initiative at Luleå University of Technology a monitoring program was designed to measure ground deformation and the response of the ground support system. Numerical modelling was conducted to capture the responses as observed during monitoring. The numerical models revealed all the typical mechanisms of instability that have been conceptualized through observations and earlier studies. Talc obviously was the most influential lithology that controlled the deformation characteristics of the stope and ultimately on the rock support system. Combinations of bending, bulging, shearing and tensile mechanisms induced a complex loading pattern on the rock support system. Often the rock bolts, for example, would experience all of these mechanisms at once or during different stages of the excavation rounds as a cut is developed.

1 Introduction

The Kristineberg mine, owned by Boliden Mineral AB, has a long history of large ground deformation which causes stability problems for the mine. Over the years the mine has developed various mining techniques, backfilling and ground support procedures to manage the problems associated with those problems. In general the ground control problems at the mine are highly influenced by the wall rock geology. The wall rock, that is the footwall and hanging wall, comprise of highly altered chlorite schist, which are internally referred to as talc-schist. They often occur as seams with finite thickness barely ranging from 0.1 m to as wide as 3.0 m. Coupled with high ground stresses the talc squeezes and slides into the stope if undercut or exposed depending on the loading direction. The deformation magnitudes have often been reported to be in the order of 0.2 m to 0.5 m and seldom up to 1.0 m. The conventional rock support system, consisting of fibre re-inforced shotcrete and rebar rock bolts, has regularly failed under these conditions.

To study the rock support response to large ground deformation the Kristineberg mine therefore provided an ideal test site. Hence, a field investigation was conducted at the mine in the J-orebody in a stope located at a depth of 1200 m. Monitoring was conducted when Cut #4 of stope J10-3 was mined. The monitoring involved; convergence measurements, total station surveys, borehole photogrammetry, damage mapping, and instrumented bolt measurements. Two systems of rock support were utilized in the investigation with the objective of evaluating their performances. The rock support systems were: (i) shotcrete + rebar and (ii)
shotcrete + D-bolt. They were installed in alternating 5 m long rounds as the cut advanced. Bolting was regularly spaced in a 1.0 m by 1.0 m pattern.

To construct the numerical models the geology of the stope was extracted from the drill-hole data and stope face mapping. First, the numerical models were simulated without ground support to study the deformation and failure characteristics of the rock mass around the stope. Next, the rock supports were installed and the response of the rock support to the observed ground deformations was analysed.

The numerical models revealed all the typical mechanisms of instability that have been conceptualized through observations and earlier studies (e.g. Krauland et al, 2001; Board et al, 1991). The talc schist obviously was the most influential lithology that controlled the deformation characteristics of the stope and ultimately the rock support system performance. Combinations of bending, bulging, shearing and tensile mechanisms induced a complex loading pattern on the rock support system. Often the rock bolts for example, would experience all of these mechanisms simultaneously or during different stages of the excavation rounds as the cut is developed.

It is difficult to make precise conclusions regarding the performance of the two rock bolt types utilized in the test stope. This is due largely to two reasons: (i) the instrumented bolt measurements were quite unsuccessful due to many of them failing prematurely and (ii) the deformation magnitudes experienced in the test stope were much smaller than expected and thus inducing less strain on the rock bolts. Nevertheless, the convergence measurements on the rock bolt heads did indicate to some extent that, the D-bolt being ductile, appears to relax with increasing deformation, while the rebar bolt being stiff does not. This observation is made at least after the stope has advanced more than 30 m from the point of measurement, by which time the stope convergence has settled.

2 Background of test site

2.1 Kristineberg mine

Boliden Minerals’ Kristineberg Mine is one of the deepest mines in Sweden and it is located in the municipality of Lycksele, Västerbotten county, in northern Sweden. Mining at Kristineberg presently takes place between 850 and 1320 m below the ground surface. The ore bodies are polymetallic and contain zinc, copper, lead, gold and silver. They are mined using a mechanized overhand cut-and-fill mining method for narrow orebodies and drift-and-fill for wide orebodies (greater than 8 m). Stopes are typically 150 m along the strike and are developed from the center and mined in each direction by breasting. The cuts are typically 5 to 6 m high and 6 m wide and are excavated in 5 m rounds by drilling and blasting. One stope is approximately 50 to 60 m high.

2.2 Ground control problems

The primary ground control problems at the Kristineberg mine are closely related to the geological conditions around the stopes. The presence of the weak chlorite quartzite and talc-schist in both sidewalls and the weak interfaces between the rock units along the hangingwall and footwall contacts results in failures in the sidewalls in a pattern which is fairly common in the mine. The failure initiates within the weak talc-schist, followed by slip along the contact if it is undercut by stope opening.

As the stope is excavated, the relatively hard ore in the roof is subjected to increased stresses, thus forcing the ore to punch into the weak sidewalls and then drag downwards. In the process it induces shearing in the footwall, roof-parallel fractures and typically bending failure in the hangingwall. The behaviour is illustrated by Figure 1.
Figure 1  Typical failure mechanisms in the Kristineberg mine (Krauland et al. 2001)

3  Field monitoring

3.1  Investigation site description

Figure 2 shows the plan of the site of field investigation. Stope J10-3 shown in the figure is located at a depth of 1195 m in the J-ore zone. The field investigations were conducted in Cut #4 of Stope J10-3. Investigations in Cut #5 of J10-3 and Cut #1 in Stope J10-4 were part of a different project and therefore are not reported in this paper.

The vertical sections through Y825, Y850 and Y875 are shown in Figure 3. R1 to R10 represent the breasting rounds. The rebar bolt and D-bolt were alternated within these rounds in order to study their respective performances. To conduct numerical simulations, vertical sections of the stope were also extracted as shown in Figure 3 and local geology as observed in the bore holes through sections Y825, Y850 and Y875 were added to the sections to complete the models for simulations.

Figure 2  Stope J10-3 Cut #4 where field investigations were performed.
3.2 Monitoring

The monitoring involved; convergence measurements, total station surveys, bore-hole photogrammetry, damage mapping, and instrumented bolt measurements. Convergence measurements involved both tape and bore extensometers. The instrumentation pattern and relevant monitoring methods are illustrated in Figure 4. Sections S3:4 and S3:6 are high density monitoring sections where all monitoring systems stated above were utilized and were necessary to study the behaviour of the ground support.

3.2 Field monitoring results

A summary of the field results are presented in this section to facilitate the numerical simulations presented later. Figure 5 shows the maximum convergences observed from tape extensometer readings. These convergences depend on the local geology in these sections as well as the distance from the stope face. There is some evidence from S1 to S4 where the D-bolt appears to relax, allowing more deformation as the cut is reaching its maximum convergence. Figure 6 shows the borehole images over the 42 days of monitoring and Figure 6 and 7 show the convergence of the footwall between sections S5 and S6 where damage mapping showed cracking of shotcrete and corresponds to Figure 5c where the footwall shearing is quite significant.
Figure 4  Monitoring plan. S1 to S10 represent the instrumented sections. Sections S3:4 and S6:7 are high density monitoring sections (after Perez & Nordlund, 2014).

Figure 5  Maximum HW-FW convergence with alternate rebar and D-bolt supported rounds.
Figure 6  (a) HW convergence in section S1 and (b) HW borehole closure in section 1. The colour dots represent days when observations were made.

Figure 6  (a) FW convergence in section S2 and (b) FW borehole closure in section 2. The colour dots represent days when observations were made.

Figure 7  (a) Shear movement in the FW observed between sections S5 and S6 where shotcrete cracking were also observed. (b) FW borehole closure between S5 and S6.
2.3 Discussions on field monitoring results

It is difficult to make precise conclusions regarding the performance of the two rock bolt types utilized in the test stope. This is due to two reasons: (i) the instrumented bolt measurements were quite unsuccessful due to many of them failing prematurely and (ii) the deformation magnitudes experienced in the test stope were much smaller than expected and thus inducing less strain on the rock bolts. Nevertheless, the convergence measurements on the bolt heads does indicate to some extend that, the D-bolt, being ductile, appears to relax with increasing deformation, while the rebar bolt being stiff does not. This observation is made at least after the cut has advanced more than 30 m from the point of measurement, by which time the stope convergence has settled.

Surface failure mapping by visually assessing and recording damages to shotcrete and failure of rock bolts were carried out during the monitoring period. Although, majority of the rockbolts remained intact, the shotcrete showed cracking as the faced advanced. Cracks were immediately evident when the face advanced 5 to 6 rounds ahead.

3 Numerical simulations

3.1 Unsupported models

Numerical analyses were performed for profiles Y825, Y850 and Y875 (Figure 4) with their respective geology obtained from the boreholes that intersected them. As they are unsupported models no ground supports were applied. The objective was to study the ground deformation behaviour without support.

Tables 1 and 2 show the inputs used in the numerical models. The in-situ stresses are those from hydraulic fracturing measurements reported by Stephansson (1993) as:

\[
\sigma_v = 0.027Z, \sigma_H = 2.8 + 0.04Z, \sigma_h = 2.2 + 0.024Z
\]

where; \(\sigma_v\) is the vertical stress, \(z\) is the depth, \(\sigma_H\) is the maximum principal stress and is parallel to the orebody, and \(\sigma_h\) is the intermediate principal stress and is perpendicular to the orebody.

Table 1 Material parameters for numerical analyses (also earlier calibrated by Board et al, 1991)

<table>
<thead>
<tr>
<th>Material</th>
<th>Youngs modulus, E (MPa)</th>
<th>Poissons ratio, (\nu)</th>
<th>Rockmass compressive strength, (c_{pm}) (MPa)</th>
<th>Cohesion, (c) (MPa)</th>
<th>Friction, (\phi) (°)</th>
<th>Tension, (\sigma_t) (MPa)</th>
</tr>
</thead>
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<td>Cordierite-quartzite</td>
<td>39000</td>
<td>0.25</td>
<td>23.2</td>
<td>6.7</td>
<td>30</td>
<td>0.40</td>
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<tr>
<td>Chlorite-quartzite</td>
<td>18000</td>
<td>0.30</td>
<td>17.3</td>
<td>5.0</td>
<td>30</td>
<td>0.20</td>
</tr>
<tr>
<td>Talc-schist</td>
<td>15000</td>
<td>0.30</td>
<td>5.2</td>
<td>2.0</td>
<td>15</td>
<td>0</td>
</tr>
<tr>
<td>Orebody</td>
<td>27000</td>
<td>0.25</td>
<td>15.1</td>
<td>5.0</td>
<td>23</td>
<td>0.40</td>
</tr>
<tr>
<td>Backfill*</td>
<td>81</td>
<td>0.35</td>
<td>0.7</td>
<td>0</td>
<td>36</td>
<td>0.04</td>
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</tbody>
</table>

* Backfill parameters after Knutsson, 1981.

Table 2 Talc-schist interface properties

<table>
<thead>
<tr>
<th>Normal stiffness, (k_n) (GPa/m)</th>
<th>Shear stiffness, (k_s) (GPa/m)</th>
<th>Cohesion, (c) (MPa)</th>
<th>Friction, (\phi) (°)</th>
<th>Tension, (\sigma_t) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6000</td>
<td>60</td>
<td>0</td>
<td>5</td>
<td>0</td>
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3.1 Results from unsupported models

The ground deformation behaviour for the unsupported stopes through profiles Y825, Y850 and Y875 are shown in Figures 8 to 10, respectively. In profiles Y825 and Y850 the talc occurs both in the HW and FW. The undercutting of talc in the HW resulted in significant ground deformation in the HW. In the FW the deformation is small since the talc seams are located further behind the FW surface. Total station measurement in Section 6 (S6), inserted in Figure 8, confirms the deformation behaviour observed from the numerical simulation for profile Y850. No total station measurements were available for profile Y825. Camera images through the borehole located in the FW, between S5 and S6 (i.e. between R5 and R6), show significant shearing of the FW talc (see Figure 7).

In the profile Y875 the talc occurs in the FW and also through the middle of the orebody. The talc seam in the middle of the orebody is fully undercut by the stope, while the seam in the FW is partially undercut. The deformation of the talc in profile Y875 occurs in the form of shearing, sliding and squeezing. The total station measurement of Section 3 (S3), inserted in Figure 10, is the closet total station measurement to profile Y875, since none was available for Section 1 (S1). The inserted total station measurements for S3 shows similar deformation pattern as observed numerically in profile Y875. In fact, the talc seam that is located on the FW of Y875 appears to extend to S2 and S3. This is clearly evident from the fact that, the borehole camera images in the FW at S2 show significant shearing within the talc (see Figure 7).

The numerical simulations of the unsupported stopes show a number of mechanisms involved in deformation of the talc including; bending, shearing and dilation. These mechanisms are important to know as they will ultimately affect the response of the rock support elements.

Figure 8  Ground displacement behaviour around profile Y825 in Cut #4. Large deformations occur in the HW where the talc-schist seam has been undercut.

Figure 9  Ground displacement behaviour around profile Y850 in Cut #4. Large deformations occur in the HW where the talc seam is closest to the open stope. Inserted on the right is the total station profile through Section 6 located in the vicinity of Y850.
3.2 Supported models

Profiles Y825, Y850 and Y875 were also simulated with rock supports installed. Profile Y875 was supported with D-bolt and shotcrete, while profiles Y850 and Y875 were supported with shotcrete and rebar to conform to the actual installation. The rock bolts are 2.7 m long and regularly spaced 1.0 m apart. Table 3 gives the properties of the rock bolts. The D-bolt, considered a high strength-ductile rock bolt, has two components; the non-deformable anchors and deformable shanks. Like the rebar the D-bolt yields at 0.2% strain, however, while the rebar is assumed to fail at yield (which may not be true in practice), the D-bolt is reported to strain plastically up to 14% before failing (see Li, 2010). In the numerical simulation the D-bolt is modelled in two parts; the non-deformable anchors are assigned very high strength and modulus, while the shanks are assigned the reported strength and modulus of the D-bolt. However, to make the D-bolt strain plastically a residual strength equal to the yield strength is assigned. Table 4 shows the fibre re-enforced shotcrete parameters.

Table 3  Strength and stiffness parameters of the Rebar and D-bolt

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<tr>
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<th>Rebar</th>
<th>D-bolt</th>
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<td></td>
<td>Anchors</td>
<td>Sanks</td>
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<tr>
<td>Young’s Modulus (GPa)</td>
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<td>200 000</td>
</tr>
<tr>
<td>Tensile Strength (kN)</td>
<td>200</td>
<td>100 000</td>
</tr>
<tr>
<td>Residual tensile strength (kN)</td>
<td>0</td>
<td>100 000</td>
</tr>
</tbody>
</table>

Table 4  Fibre re-enforced shotcrete parameters; mechanical parameters Malmgren, 2005.

<table>
<thead>
<tr>
<th>Thickness (cm)</th>
<th>Modulus (GPa)</th>
<th>Peak UCS (MPa)</th>
<th>Residual UCS (MPa)</th>
<th>Peak flexural tensile strength (MPa)</th>
<th>Residual flexural tensile strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>20</td>
<td>40</td>
<td>5</td>
<td>5</td>
<td>2.5</td>
</tr>
</tbody>
</table>

3.2 Ground reaction curve

A ground reaction curve (shown in Figure 11) was created for the stope and calibrated against monitoring data in order to assist in determining when the rock support is to be installed in the models. In the ground reaction curve the critical limit is reached when the rock mass modulus is reduced to about 25% of the original value, resulting in inward displacement of about 60 mm. However, for the numerical modelling the 20% limit for modulus reduction is chosen. With this limit a convergence of 75 mm was obtained, which is when compared to the convergence measurement data, that approximately 50% of the convergence would have occurred before the ground support elements (shotcrete and rock bolts) were installed.
Figure 11  Ground reaction curve for the stope. The critical limit is reached when the modulus is reduced to 25%. For modelling 20% limit for modulus reduction is chosen.

3.2 Results of the supported models

Figures 12 to 13 show the results from the supported stope models. The yielding of the rock bolts correspond to the areas where the talc layers are actively deforming. In profile Y825 segments of the rock bolts in the HW have yielded in tension, corresponding to the HW talc that has been undercut and is deforming into the stope. Similarly in profile Y850 the rock bolts also yielded in tension in the HW, where the talc is actively deforming by bending. In Profile Y875 the rock bolts that intersected the talc seams in the roof and FW have yielded in tension. The yielding are consistent with the mechanisms that drive the deformation of talc around the stope. For example, the rock bolts installed in the talc seam in the roof of Cut #4 in profile Y875 are yielding in tension since the sliding of the talc down the middle of the stope induces tensile stress due to the downward drag and high lateral stresses. This phenomenon is known to drive the back parallel cracks at the Kristineberg mine, leading to the church dome shaped roofs.

Profile Y875 particularly shows significant yielding of the shotcrete liner all around the profile. This profile is in fact located in the area where large shear movements were observed in the FW via the borehole camera (see Figure 7). In the HW borehole camera images showed closure of the borehole as the HW converges into the stope (see Figure 6)
4 Discussions

It was not possible to develop geological models that accurately represent the geology around the stopes because of incomplete and missing data. However, by comparing the drillhole geological data and the face and roof mapping that was carried out in cuts #3 and #4, as well combining geological knowledge it was possible to develop geological models of the wall rock through profiles Y825, Y850 and Y875. The borehole camera snapshots and failure mapping of cut #4 also indicated that, the deformation mechanisms observed were consistent with the interpreted geology.

Numerical modelling based on the “real” geology was necessary as it would then be possible to compare field observations against results from numerical simulations, as well as identify geology and mechanism that control ground deformation around stopes. Furthermore, it provided the basis for model calibration. The models with real geology clearly showed the talc to be the most influential lithology that controlled deformation around the stope. Three main mechanisms associate with talc were observed; shearing, dilation and bending. Bending typically occurs if the talc seam is thin. In this case it is forced to separate from the contact by compressive force from the ore in the roof and the lateral stresses. Dilation occurs with thick talc seams, where they squeeze and “belly” into the stope by dilating. Shearing is coupled with dilation.
Since the talc-schist is very weak it is punched by the ore into the roof immediately above the stope. The ore then drags downwards under its own weight and in the process forces the talc-schist to slip along the clay filled contacts.

The segments of rock bolts and shotcrete apparently yielded in areas where the talc was actively deforming. It was not possible to evaluate the difference between the performance of the D-bolt and rebar, since the deformations in the test stope were smaller than anticipated. The majority of the strain gauged rock bolts failed and data was unreliable, making it difficult to assess the performance of the rock bolts.

It is also possible that an entire length of a rock bolt can be located inside the weak talc, if it is sufficiently thicker than the bolt length or parallel to the bolt application as in profile Y875. In this instance the bolt will be straining with the talc, in which case the bolt may not be useful in anchoring the talc to the competent host rock.

The deformation of the stope is also a function of face advance. This can be seen from the fact that deformation continued to increase as the face advanced, with furthest rounds showing the largest deformation compared to rounds closest to face. Furthermore, shotcrete cracking was not seen in the early stages of the excavation until the final rounds were excavated. It is also estimated that at around 50% of the deformation has already occurred prior to installation of monitoring instruments. Displacements tracked from the unsupported numerical models show displacements to be almost twice that observed in the test stope, which of course were measured in supported ground conditions.

5 Conclusion

The interaction between ground support and rock mass during large ground deformations are complex and require an understanding of the mechanisms that drive these deformations. As learnt from the Kristineberg experience these mechanisms are significantly influenced by the characteristics of the wall rocks. At the Kristineberg mine a combination of shearing, bending, bulging and direct tensile and compressive loading were observed, thus subjecting the rock support elements to complex mechanisms often occurring simultaneously. The influence of talc-schist on the overall ground deformation behaviour was clearly evident.

Acknowledgement

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References


