Failure Analysis of an Intersection under Squeezing Rock Conditions

H Ozturk¹ and B Foo²
¹Department of Mining Engineering, Middle East Technical University, Dumlupinar Bulvari, Ankara, Turkey
²Mining Rock Mechanics Consultant, Vancouver, British Columbia, Canada

ozhasan@metu.edu.tr

Abstract. The development of Turkey’s deepest coal mine is currently underway to the proposed depth of 1200 meters (m) below surface at the Soma Coal Basin. An 11 m intersection forming an access way from one of the main declines was excavated at approximately 900 m below surface collapse within hours of completion even after being heavily supported by TH34 and cable bolts. It was discovered during the excavation failure investigation and rehabilitation work that the intersection was excavated below a water bearing stratification and faulted zone of the coal seam. The failure analyses concluded that the failure was contributed by a number of factors, mainly the overburden pressure of the overlying material along with the inrush of high-pressured groundwater with the very poor rock mass surrounding the excavation. This study investigates the possible reasons of the failure, numerical modelling of the intersection and alternative support suggestions for such squeezing rock conditions.

1 Introduction

The case study area of the subject matter is located at Soma Coal Basin, west of Turkey in the region of the largest economic lignite coal bearing basin of the country. To date, the underground mine development work has been continuing for the deepest coal mine in the country to the proposed depth of approximately 1200 m below the surface. A summary of the stratification of the area is shown in figure 1. The coal-bearing stratum or Soma Formation starts with a basement grey conglomerate with fine-medium sized grains cemented with sand and silt. A hard, massive lignite zone having a thickness ranging between 15 m and 22 m overlays the basement formation with a bluish grey hard, massive marl covering the lignite zone [1].

A rockmass classification was carried out at the intersection of the case study along with the review of exploration drilling cores within the area of the incident to determine its quality (figures 2-3). The rock mass quality of the area was classified based on the Rock Mass Rating (RMR₀) and Tunnelling Quality Index (Q) systems proposed by Bieniawski [2], Barton et al., [3] along with the determination of the Geological Strength Index (GSI) for very poor jointed rock masses [4]. The uniaxial compressive strength of the rock mass was provided by the company of the case study based on the information from their database.
Figure 1. General Stratigraphic Section for Soma Coal Basin [1].

Figure 2. Intersection Cross-Section Area (left), Excavation after the Collapse (right).

Figure 3. Intersection rock (clayey sandstone/shale units).

The collapsed intersection was discovered to be situated in a clayey sandstone/shale, highly schist and fractured geological unit of the Soma Basin during the inspection of the excavation and the examination of the cores from the exploratory drilling. This unit is very heterogeneous, metamorphosed heavily jointed, anisotropic, and faulted with discontinuous surfaces which are highly weathered well-polished surfaces as depicted from the core photographs in figure 3.

Apart for being situated in a poor to very poor rock mass, it is also anticipated that the collapse of the excavation at the intersection is contributed by the overbearing pressure of the ground approximately 900m above the excavation and the presence of pressurized ground water within the vicinity and surrounding the excavation. It was observed that the inflow of the groundwater into the excavation causes the clayey sandstone/shale to swell, disintegrate and being flushed into the excavation contributing to a drastic drop in material cohesion, fiction and an overall reduction of the effective strength of the material. The overburden stress along with the ground pressure is predicted to be a major
contributor to the collapse of the excavation after the installation of the ground support as illustrated in figure 2 (Right).

According to the prediction of tunnel squeezing problems in weak heterogeneous rock mass proposed by Hoek and Marinos [5] (figure 4) the failure of the intersection falls in the “E” category of the tunnel convergence, based on preliminary analyses by determining the amount of tunnel convergence information observed in the field and estimation of the rock mass strength and overburden stress. The elaborated geotechnical issues listed by Hoek and Marinos [5] for this category of strain acting of the tunnel was precisely what occurred at the interaction following its excavation.

Figure 4. Tunnel Convergence vs. Rock mass strength/In-situ stress [5].

1.1. Intersection Support Elements and Design before Failure

The support installed at the intersection, as presented in figure 5 (left) just before the collapse, showed a heavily supported excavation with 6 m long pre-tension resin grouted cable bolts at a 0.75 m spacing, with mire mesh and TH34 grade Steel Sets installed at the similar spacing as the cable bolts. The cable bolts were reported to be grouted with resin only 2 m from it anchoring point instead of being fully grouted.

Figure 5. Intersection support (before failure) (left), plan view of intersection and Span (right).
There is no surface retention support such as shotcrete installed on the back, sidewall or at the face of the excavation including no ground support system installed on the floor of the excavation. The cable bolt plates of intact and undamaged cables were inspected after the collapse of the excavation and it was observed that these plates were not even deformed, indicating that the cable bolts installed were not carrying any load during the failure the of the excavation. A simplified plan view of the support design for the case study area is presented in figure 5 (left). Since the excavated area was composed of two openings: one at 5.5 m and the other at 6.7 m width at an oblique angle to each other, the span of the intersection was estimated to be approximately 11 m, as highlighted.

There was further evidence during the examination of the collapsed excavated area, that the inrush of the watering bearing faulted zone approximately 7 m above the crown or back of the excavation caused the failure of the TH34 Steel Set material along with material flushing in from the face of the excavation.

1.2. Rock Mass Characterization and Geo-mechanical Properties

The GSI value of the rock mass at the intersection was estimated to be approximately 25 as a result of the field investigation. Similarly, RMR_
0 was used for the classification and the estimated value is approximately 30 or rated as a “Poor Rock”. The Q values were also estimated, during which consideration for both the cores and underground observations were taken into account with the estimated RQD value around 40%.

The Q values were estimated to be as 0.11 or classified as “Very Poor” to “Extremely Poor” rock mass when considering that the excavation was dry during the time of the excavation (figure 5) and revised to have a rating of 0.033 as “Exceptionally Poor” rock mass when the inrush of the groundwater was considered after the collapse of the intersection.

Table 1 summarizes the estimated geo-mechanical properties assumed from the rock mass observed for the case study and based on the information provided during the preparation of this article where the UCS is the uniaxial compressive strength; \( \sigma_{ci} \) is the intact rock uniaxial compressive strength; \( \sigma_{cm} \) is the rock mass uniaxial compressive strength; \( E_i \) is the deformation modulus of intact rock, \( E_m \) is the deformation modulus of the rock mass modulus, \( \phi \) is the internal friction angle, \( C \) is the cohesion, \( D \) is blasting factor, \( m_i \) is Hoek-Brown’s constant.

| GSI | RMR | Q | UCS \((\sigma_{ci})\) (MPa) | Ei (GPa) | Tensile Strength (MPa) | Poison’s ratio | UCS \(\sigma_{cm}\) (MPa) | mi | D (kPa) | \( \phi \) (deg) | \( E_m \) (MPa) |
|-----|-----|---|--------------------------|---------|----------------------|---------------|------------------------|-----|--------|-------------|-------------|
| 25  | 30  | 0.033-0.11 | 19 | 3.7 | - | 0.25 | 1.24 | 12 | 0.5 | 685 | 14 | 131 |

2 Influence of Groundwater

Ongoing groundwater inflow-pressure tests at various wells were analyzed to evaluate the groundwater conditions within the vicinity of the excavation and the intersection. There are five (5) dewatering wells located on site with pressure readings between 10 to 17 bars (145.0 to 246.6 Psi) with water inflow rates recorded to be approximately 4.0 liters per sec (l/s) for well 1; 1.0 l/s; 0.5 l/s; 5.5 l/s; 3.0 l/s for wells 2, 3, 4 and 5, respectively. The wells were not dewatered during the time leading to the failure of the excavation due to inadequate pumping capacity at the time.

An interesting finding was observed after reviewing the manometer and flow readings from the wells following the failure of the excavation at the intersection. Two of the wells located north of the intersection and at the fault zone recorded readings up to 1.7 MPa with flow rates of 20 l/s.

There is evidence suggesting that the hydrostatic pressure of the groundwater finds itself back into the weakly faulted excavated rock mass and played a role in the failure of the area. The dewatering
Efforts were improved after the incident to manage the hydrostatic pressure acting on the excavation. It is noteworthy to state that despite these measures, there is still a well ahead and east of the intersection that still records high pressure readings. Geotechnical challenges are likely be encountered if adequate measures are not taken to manage groundwater or hydrostatic pressure within the fault area.

Preliminary observations made from the groundwater inflow and pressure readings from the current dewatering wells have been useful as preliminary indicators in predicting potential geotechnical issues. However, it recommended to request additional hydrogeological data collections, surveys and studies along with detailed geotechnical analyzes to further assist in the design of and development of both the decline and the mine.

The effect of water pressure (1.7 MPa) on ground reaction curve [6] based on the geo-mechanical properties given in table 1 can be seen in figure 6. If there were no water issues in the intersection, the application of cable bolts and steel sets would appear to have just enough support pressure at the right time (not too soon, not too early) at the right support stiffness (figure 6). However, the effect of water pressure shows that the amount of support provided by the bolts and steel set is just enough. Therefore, it should be noted that the additional 20 cm shotcrete application does not only generate an extra support pressure (figure 6), but provides additional structural integrity to the overall stability as well.

![Ground reaction curve of the intersection](image)

**Figure 6.** Ground reaction curve of the intersection [6].

3 Empirical Support Design
Published empirical underground excavation stability and support design tools proposed by Bieniawski [2] and Grimstad and Barton [7] including those suggested by Hoek and Marinos [5] were employed into the failure analyse along with numerical modelling to determine its compatibility as early identification tool to assist in predicting, mitigating and preventing future occurrences of such event.

The collapse of the intersection immediately after the construction coincides with the predicted stand-up time suggested by Bieniawski [2] (figure 7).

The ground support design and implemented support system in the area of the case study met all the recommendations proposed by all the empirical design tools (figure 8) with the exception of the application of Shotcrete.
The application of shotcrete along with the reduction of hydrostatic pressure surrounding the area with proper dewatering has the likelihood to improve and provide additional structural integrity to the overall stability to the area as suggested by empirical design tools and identified by numerical modelling performed on the case study area, which will be further elaborated in the following section.

4 Numerical Modelling

Underground opening geotechnical problems such as the one presented in this case study are best evaluated with a three dimensional (3D) numerical modelling software. However, as a “worst-case scenario”, the intersection of this case study was evaluated in two dimensional (2D) by considering the widest section of the intersection (i.e. 11 m) as a preliminary analysis to determine the induced displacement on its stability.

The overburden stress was estimated to be 21.0 MPa and the horizontal stresses of 6.5 MPa at the depth of 900m below the surface. Iterative support scenarios were modelled with RS2 [8] 2D finite element software capable of modelling a complex multi-stage tunnel in weak or jointed rock masses.

Results from the analyses suggest a support design with 20 cm thick shotcrete having an UCS of 20 MPa in addition to the TH34 steel supports at 75cm spacing around the excavation, including an inverted concrete floor at 30 MPa UCS together with the cable bolts of 8.0 m in length instead of 6.0 m (17.8 mm diameter, 0.2 MN bolt strength, fully grouted at 8 m long with 0.35 water to cement ratio) with similar spacing to the steel sets that will be needed to satisfy the stability of the intersection.
Figure 9 presents the total displacement of the modelled excavation: 11 mm from the roof and an additional 14 mm from the floor. Based on the models that were evaluated, the prescribed ground support with the addition of shotcrete should provide the necessary radial support pressure for the given displacement.

It was from the results generated from the numerical analyses that the axial load and bending moments of the shotcrete, TH34 steel sets as well as the axial stress and axial load acting on the cable bolts were within acceptable limits as presented in figures 10 and 11.

**Figure 9.** Total Displacement Contours of Excavated Area.

**Figure 10.** Axial Load and Bending Moments on Shotcrete (Left) and TH34 Steel Set (Right).

**Figure 11.** Axial Stress (Left) and Axial Load (Right) on the Cable bolts.

5 Conclusions
The following summarizes the enhanced support design system recommendations for squeezing rock conditions which was implement since the collapse of the excavation:

1. Immediate installation of ground support as the excavation advances and extending the application of the support system into additional areas within the close proximity to the excavated area, to reduce the extent of the unsupported span.
2. Application of a 20 cm thick steel fiber reinforced shotcrete extending beyond the width of the excavation to all exposed rock surfaces including the face of the excavation as illustrated in figure 12 (left). The areas should be cleaned from all debris and washed to improve the adhesion of the shotcrete. Wet shotcrete is recommended for quality and dust control, and safety purposes. Drainage pipes will be installed in areas with excessive water inflow prior to application of shotcrete.

![Figure 12](image-url)  
Figure 12. Intersection shotcrete and bolt design (plan view) (left) cross-section support design (right).

3. Fully grouted 8.0 m length, 17.8 mm diameter cable bolts will be installed with welded wire mesh on top the shotcrete at 0.75 m spacing down to 0.25 m from the floor of the excavation, providing the sufficient surface retention and anchoring of the rock mass prior to the installation of the steel sets.

4. Installation of TH34 steel sets at 75 cm apart to the excavation face and past any excavated area of an intersection. Spaces in between the steel sets should be minimized by in filling the voids between the steel ribs with wooden wedges and planks to provide additional confinement to the overall ground support element and controlling displacement limits.

5. Finally, the ring is closed with the floor invert concrete and TH34 (figure 12, right).

After the rehab work of the failed intersection with forepoling, the suggested support system was applied and the intersection has shown no stability problems since then. Therefore, passive support (steel set), active support system cable bolts, surface retention bolt plates, shotcrete and mesh, are all needed in such squeezing conditions [9]. It should also be noted that the water drainage is necessary for such squeezing rock conditions. Effective bonding of cable bolts is achieved by coating or inserting a flexible hose on to the cable and fully grouting it. Quality control is needed for grout to hole diameter and void ratio. Pull out tests are required as well.

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