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# Stability analysis and support system timing for mine block 3 of Parvadeh 4 coal mine using numerical modelling

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ABSTRACT

Ensuring the stability of the main inclined shaft in coal mining operations is of particular importance. The instability of inclined shaft seriously restricts the safe and efficient mining of coal resources. In this paper, stability of main inclined shaft of mine block 3 of Parvadeh 4 coal mine in Tabas is modeled to design support system and to determine its optimal installation time by using the finite difference numerical method. To assess the stability conditions of the excavation, we employed the empirical method introduced by Barton, which facilitated the proposal of an appropriate support system. Subsequently, to evaluate the proposed support system and ascertain its optimal installation time, numerical simulations were conducted using Fast Lagrangian Analysis of Continua in 3 Dimensions (FLAC 3D) software. This involved analyzing the rock mass properties surrounding the main inclined shaft, understanding the mechanics of the surrounding rocks, and applying in-situ stresses and boundary conditions within the model. The results indicated that the optimal timing for installing the combined support system—comprising shotcrete with a thickness of 10 centimeters, a mesh frame with a diameter of 6 millimeters, and a spacing of 15 centimeters, along with designed rock bolts—occurs during the initial stage of drilling and bolt installation, accompanied by stress release. In the final phase, to evaluate the proposed support system, the interaction diagram (bending moment-axial force diagram) obtained from the SAP2000 software was analyzed. The findings from this section demonstrated that the proposed support system exhibits satisfactory performance against axial force and bending moment in the given conditions.

KEYWORDS

Numerical modeling, finite difference, inclined shaft, support system, Tabas coal mine

## I. INTRODUCTION

With the increasing depletion of shallow mineral deposits worldwide, deep underground mining is being extensively developed (Zhang et al., 2021). The depth of coal extraction globally shows a rising trend each year, with the current average extraction depth of coal seams reaching 700 meters (Wu et al., 2022). Consequently, deep mining faces more complex geological and engineering conditions .

In particular, when drilling inclined shafts, the surrounding rock easily causes stress redistribution or interference, leading to excessive swelling of the floor and bidirectional displacement, resulting in significant deformation and instability of the inclined shafts. The risks associated with excessive deformation and instability of inclined shafts can lead to severe losses and economic damages, severely limiting the safe and efficient extraction of coal resources (Wang et al., 2020). Studying the deformation characteristics, failure mechanisms, and behavior of surrounding rocks in a deep inclined shaft is fundamental for assessing stability and proposing appropriate support systems, which hold both theoretical importance and practical value in engineering to ensure the safe and efficient extraction of a deep mine (Wu et al., 2022).

The stability of inclined shafts in underground coal mines has been the subject of extensive research, with numerous studies conducted in recent years (Yuan et al., 2018; Zhang et al., 2018; Song et al., 2019; Yuan et al., 2019; Wu et al., 2019; Renshu et al., 2020; Wu et al., 2022; Zhao et al., 2023; Ren et al., 2023; Si et al., 2024). These studies have primarily employed a variety of methodologies, including empirical methods, analytical approaches, physical modeling, and numerical modeling (Zhao et al., 2023; Bruneau and Hudyma, 2003) .

Empirical methods necessitate extensive data collection through field observations and measurements, leading to the development of empirical equations or charts for analyzing and predicting the stability of inclined shafts. However, these methods are inherently site-dependent and are limited to applications within similar geological contexts (Zhao et al., 2023). Analytical methods, on the other hand, often rely on simplifications that treat rock layers as beams or

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frames. This approach is constrained by its limited applicability, as the complexity of geological structures can result in inaccurate stability estimates.

Physical modeling involves constructing isometric models using various materials, such as gypsum, to simulate three-dimensional geological bodies. However, this approach is often challenging and time-consuming due to the intricacies involved in replicating surfaces, slopes, and complex geological formations (Sartkaeo et al., 2019) .

In contrast, numerical methods offer a more comprehensive framework for analyzing stability by considering factors such as in-situ stress conditions, complex geological structures, and the mechanical properties of rock masses. Techniques such as the Finite Element Method (FEM), Discrete Element Method (DEM), Boundary Element Method (BEM), Finite Difference Method (FDM), and hybrid methods have been developed for this purpose. The validation and reliability of these numerical models are typically established through rock mechanics testing, field investigations, rock mass classification, and preliminary numerical simulations (Chen and Teng, 2024; Huang et al., 2023; Ha et al., 2021; Abbasi et al., 2021).

Rock masses are frequently segmented into discontinuous bodies due to geological features such as fractures, faults, and joints. The Boundary Element Method effectively describes model characteristics and computes nominal values for engineering problem-solving. However, this method is primarily suited for simpler geological contexts, making large-scale stability analysis of inclined wells more challenging (Hamdi et al., 2018) .

The Discrete Element Method excels in modeling geological structures like joints and faults but requires advanced computational resources for large-scale three-dimensional modeling of entire mining areas (Cheng et al., 2018). Furthermore, the Finite Difference Method and Finite Element Method can be integrated with two- or three-dimensional interfaces to simulate discontinuous geological bodies (Vyazmensky et al., 2010). Notably, the Finite Difference Method employs an explicit solution approach that does not necessitate matrix formation, resulting in lower equipment requirements. Additionally, this method can effectively simulate large deformations and complex structural models without significant additional effort (UCIST, 1997).

While the existing literature provides a robust foundation for understanding stability analysis and support estimation in underground coal mines, this paper aims to contribute novel insights by focusing specifically on the unique geological conditions and support systems of Mine Block 3 in the Parvadeh 4 Coal Mine. Given that Block 3 has recently completed its exploratory works and is now in the design phase for access tunnels to the mineral deposit, there have been no prior rock mechanics studies conducted in this area. Considering the geological conditions, existing stresses, and other relevant factors, there is a critical need to design an appropriate support system.

This study analyzes the stability of the main inclined shaft of Block 3 in Parvadeh 4 using both empirical and numerical methods, specifically employing the finite difference method within FLAC3D software. The results from these methods demonstrate significant consistency with one another, reinforcing the validity of the findings. Furthermore, this research proposes a required support system and examines the installation time, which is a crucial factor in stability analysis .

The timing of support system installation is vital; if the system is installed too early, it must bear all rock loads, necessitating a stronger support system and leading to increased costs. Conversely, if installation occurs too late, the stability of the inclined shaft may be compromised, potentially resulting in roof collapses. This paper identifies the optimal timing for installing the support system, thereby enhancing the understanding of stability dynamics in inclined shafts under complex geological conditions. By integrating recent advancements in numerical modeling with case-specific data, this study seeks to fill a critical gap in the current literature and provide valuable insights for future engineering practices in similar contexts.

## II. The study area

The Parvadeh coal mines are located 85 kilometers southwest of Tabas in South Khorasan Province, Iran, covering an area of approximately 1200 square kilometers. Block 3 is situated in the northern part of the Parvadeh 4 area, characterized by an irregular shape and an area of 8.9 square kilometers (Fig. 1). Based on exploratory data, the geological reserves of this mine have been estimated at 13.25 million tons, with layers C1 and B2 being targeted for extraction via the mechanized longwall mining method.

Phase one of this mine involves extracting layer C1 up to a level of +710 meters, which is located at a depth of less than 200 meters below the surface. The geological conditions of the Parvadeh 4 mining area are complicated and extensive, with coal seams exhibiting outcrops and reserves at shallow depths and thin thicknesses, low dip angles, and relatively simple hydrogeological conditions. By integrating these conditions with inclined shafts in the adjacent mine, the design for the inclined shafts in this mine has been developed. This mine will be accessed through three inclined shafts, comprising a main inclined shaft, a secondary inclined shaft, and a ventilation inclined shaft.

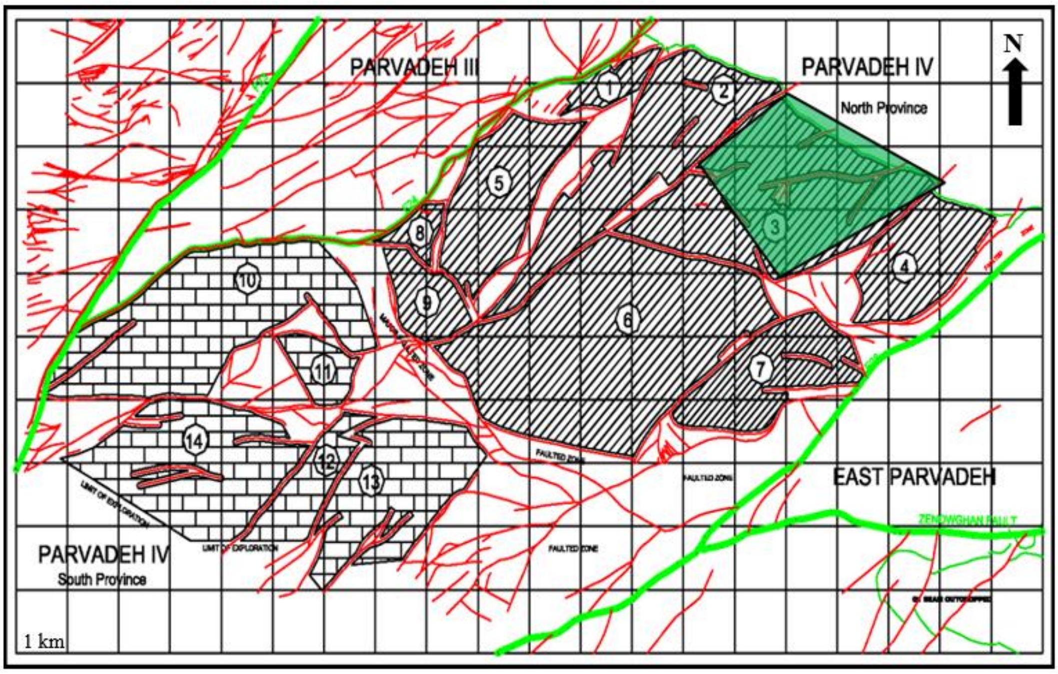


**Fig 1.** Location of Block 3 in the Parvadeh 4 mine area (CMC, 2018)

The main inclined shaft is responsible for coal transport and the entry of clean air. Up to a depth of 20 meters from the main inclined shaft's opening, it has been reinforced with U-shaped metal supports to prevent collapse, serving as a safe exit. In the initial design phase, the slope of the inclined shaft is set between 5°, 7°, and 10°, reaching a horizontal alignment at level 710. The inclined shaft will be situated 4 to 6 meters above layer B2. The wall arch profile has a useful width of 4.6 meters, a height of 3.3 meters, a functional cross-sectional area of 12.9 square meters, and a drilling cross-section ranging from 14.3 to 15.3 square meters (Fig. 2).

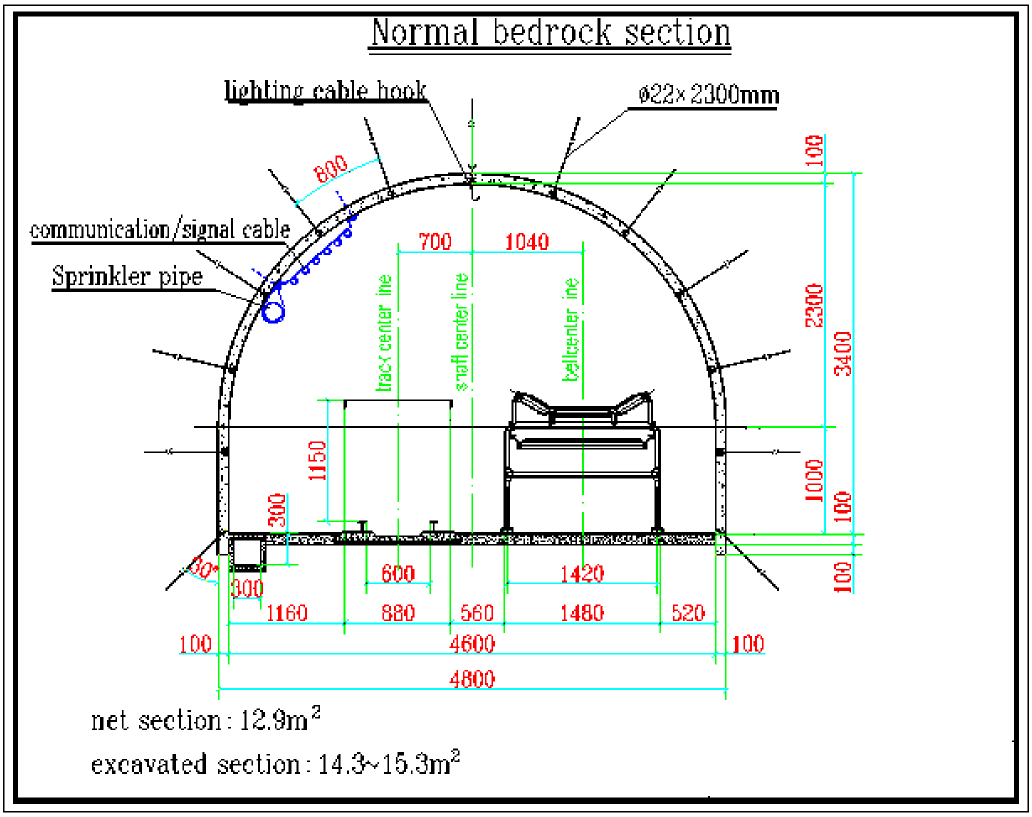


**Fig 2.** Cross-section of main inclined shaft (CMC, 2018)

## III. 3- METHODOLOGY

The Barton method is utilized to evaluate stability and determine the most appropriate support system for tunnels and underground mines. This method is based on an analysis of rock mass properties and their behavior under various loads, particularly those resulting from excavation and geological pressures (Putra, 2021).

The Barton method provides an analytical model utilizing various criteria such as the physical and mechanical characteristics of the rock mass, climatic conditions, and existing stress patterns in the excavation area. This allows engineers to select an appropriate support system. These systems may include various types of support structures, such as metal meshes, timber frames, or precast concrete systems (Yagiz, 2008).

One of the main advantages of the Barton method is its capability to predict the behavior of rock masses under different conditions. By employing this method, engineers can conduct accurate assessments of tunnel support needs and optimize designs based on the obtained results. Furthermore, this method aids in risk analysis and identifies weaknesses in design, ultimately leading to enhanced safety and reduced operational costs (Zou et al., 2024).

In general, the Barton method is recognized as an efficient analytical tool for determining tunnel support systems, assisting engineers in providing suitable solutions tailored to the specific conditions of each project.

To assess the stability of the inclined shaft, the Sakurai criterion was used, as per Eq. (1) (Sakurai, 1997).

$$\log \varepsilon_c = -0.25 \log E - 1.22 \quad (1)$$

Where $\varepsilon_c$ is the critical strain under unconfined compressive conditions around the incline, and E ($kgf/cm^2$) is the modulus of elasticity. To evaluate the structural stability, the maximum shear strain developed in the structure should be compared with the critical shear strain, according to Eq. (2) (Sakurai, 1997).

$$\gamma_c = (1+\nu).\varepsilon_c \quad (2)$$

Where $\gamma_c$ is the critical shear strain, and $\upsilon$ is Poisson's ratio.

The installation time of the support system at each stage can be calculated using Eq. (3) (Carranza-Torres and Fairhurst, 2000).

$$\frac{u_r}{u_{max}} = [1+\exp(-\frac{x/R}{1.1})]^{-1.7} \quad (3)$$

Where $u_r$ is the displacement at a distance $x$ from the face, $u_{max}$ is the maximum displacement, $R$ is the tunnel radius, and $u_r/u_{max}$ is the ratio between the displacement occurring up to the point of support installation and the maximum displacement due to complete stress relaxation.

In this study, geomechanical parameters of the surrounding rock masses are first estimated using borehole data and Roclab software (Roclab - User's guide, 2002). Subsequently, utilizing the Barton empirical method, the required support system is assessed, and considering operational conditions, the optimal support system is selected. Following this, based on the mechanical properties and current status of Block 3 of the Parvadeh 4 mine, a three-dimensional model of the inclined shaft is created. Using FLAC3D software, the appropriate timing for installing the support system is determined, and the stability of the tunnel is analyzed.

The steps for numerical modeling in FLAC3D are as follows:

1. Drawing the model in the software.
2. Entering in-situ stresses.
3. Excavating the structure and performing calculations using the software.
4. Installing the maintenance system.
5. Conducting additional calculations using the software.

### A. *Stability Analysis Methods and Support System Design*

The methods for evaluating the stability of underground excavations, which ultimately inform their engineering design, can be categorized into four groups: observational methods, empirical methods, analytical methods, and numerical methods. This research employs the empirical method proposed by Barton, followed by numerical modeling utilizing the finite difference method.

Barton et al (1974) have presented the stability condition of underground structures and their required support systems based on two parameters: Q and equivalent dimension (Eq. (4)), as illustrated in Fig. 3.

$$D_e = \frac{B}{ESR} \quad (4)$$

Where De is the equivalent dimension, B is the span, diameter, or height of the excavation space, and ESR is the tunnel support ratio.

Accordingly, the length of rock bolts can be obtained from Eq. (5) (Hoek, 2007).

$$L = \frac{2+0.15B}{ESR} \quad (5)$$

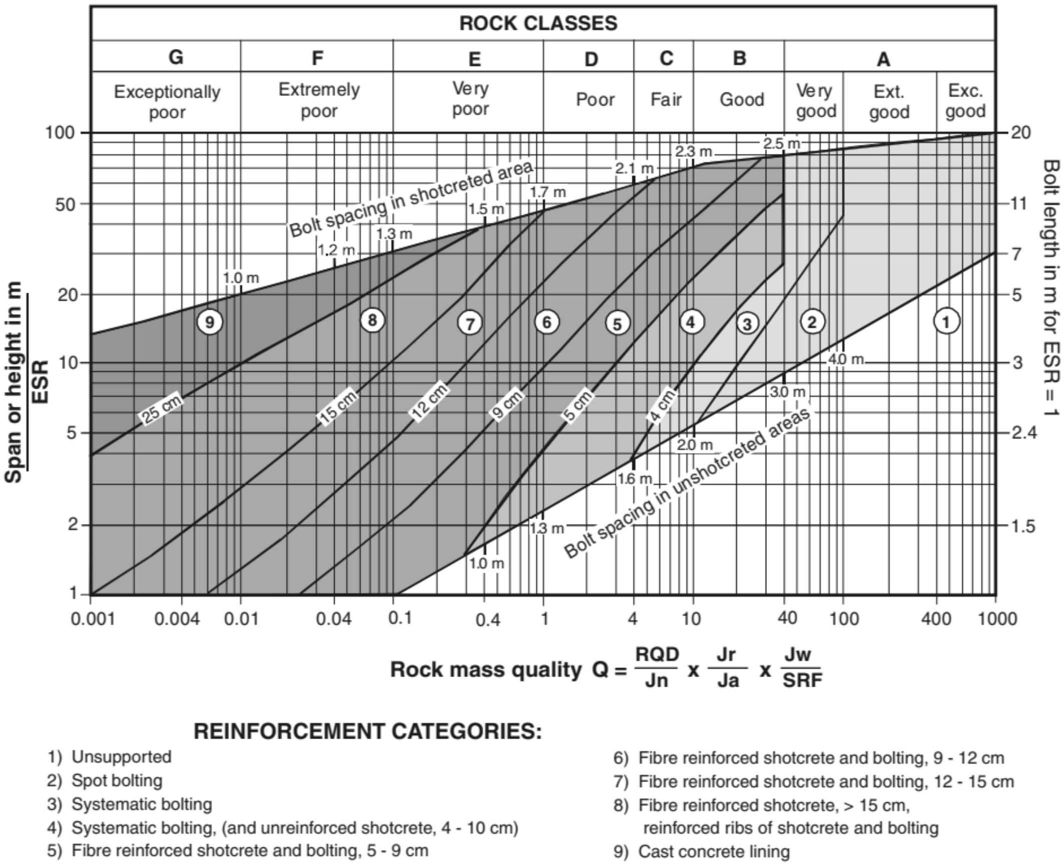


**Fig 3.** The Q support chart (Barton et al., 1974)

*B. Estimation of Geomechanical Parameters of the Rock Mass*

In this area, the non-coal strata are predominantly composed of siltstone and sandstone, exhibiting a dip of 5 degrees and a density of 2600 $kg/m^3$.

To determine the geomechanical parameters of the rock mass at the inclined shaft site, exploratory borehole data were analyzed. Subsequently, the rock mass parameters for the area were determined using RocLab software. Within this software, geomechanical parameters are calculated based on the Hoek-Brown failure criterion, using input parameters such as uniaxial compressive strength ($\sigma_{ci}$), Geological Strength Index (GSI), $m_i$ (material constant), and $D$ (disturbance factor). Table 1 presents the lithological information and corresponding geomechanical parameters for the rock layers in the inclined shaft excavation area.

*C. Finite Difference Numerical Modeling with FLAC3D*

The model geometry, which is based on the proposed shape and dimensions for the inclined shaft design—a semi-circular arch with a width of 4.8 meters and a height of 3.4 meters—was implemented in the model. Accordingly, the model length was set to 50 meters. The height of the model above the top of the inclined shaft is 32 meters, and the model's width is 23 meters. The strata have a dip of 5 degrees. Fig. 4 illustrates the developed geometry. The model dimensions were chosen such that stress changes resulting from the inclined shaft excavation return to the initial state (in-situ stresses) at the model boundaries.

The Mohr-Coulomb plasticity model was selected as the appropriate constitutive model for the problem, representing materials that yield only under shear stress. This model is commonly used for plasticity analysis in rock and soil mechanics.

The normal stress, resulting from the overburden weight with a density of 2600 kg/m$^3$, was applied to the model. To determine the horizontal stress, the equation proposed based on 204 hydraulic fracturing tests in 49 Chinese coal mines was used, as shown in Eq. (6) (Kang, 2017).

$$K = \frac{116.5}{H} + 0.7 \tag{6}$$

Where K is the ratio of horizontal to vertical stress, and $H$ (m) is the depth of the structure. With a depth of 100 meters, the value of this coefficient in this study is 1.865. Fig. 5 shows the stress state before the structure excavation.

Table 1. Geomechanical parameters of rocks around the inclined shaft. Here, υ is Poisson's ratio, C is cohesion, $E_m$ is Young's modulus, $\sigma_{tm}$ is rock mass tensile strength, and φ is angle of internal friction.

| Rock type | Depth (m) | *GSI* | $\sigma_{ci}$ (MPa) | $\sigma_{tm}$ *(MPa)* | $E_m$ *(GPa)* | C (MPa) | φ (º) | *υ* |
|---|---|---|---|---|---|---|---|---|
| Siltstone | 53-57 | 35 | 32 | 0.016 | 2.4 | 1.095 | 23.26 | 0.26 |
| Coal | 57-58 | 25 | 25 | 0.015 | 1.18 | 0.648 | 19.23 | 0.25 |
| Sandstone | 58-62 | 45 | 73 | 0.066 | 6.8 | 4.169 | 34.84 | 0.26 |
| Siltstone | 62-66 | 35 | 32 | 0.016 | 2.4 | 1.095 | 23.26 | 0.26 |
| Sandstone | 66-70 | 55 | 73 | 0.129 | 11.4 | 4.735 | 37.5 | 0.26 |
| Siltstone | 70-82 | 35 | 32 | 0.016 | 2.4 | 1.095 | 23.26 | 0.26 |
| Coal | 82-83 | 25 | 25 | 0.015 | 1.18 | 0.648 | 19.23 | 0.25 |
| Siltstone | 83-85 | 35 | 32 | 0.016 | 2.4 | 1.095 | 23.26 | 0.26 |

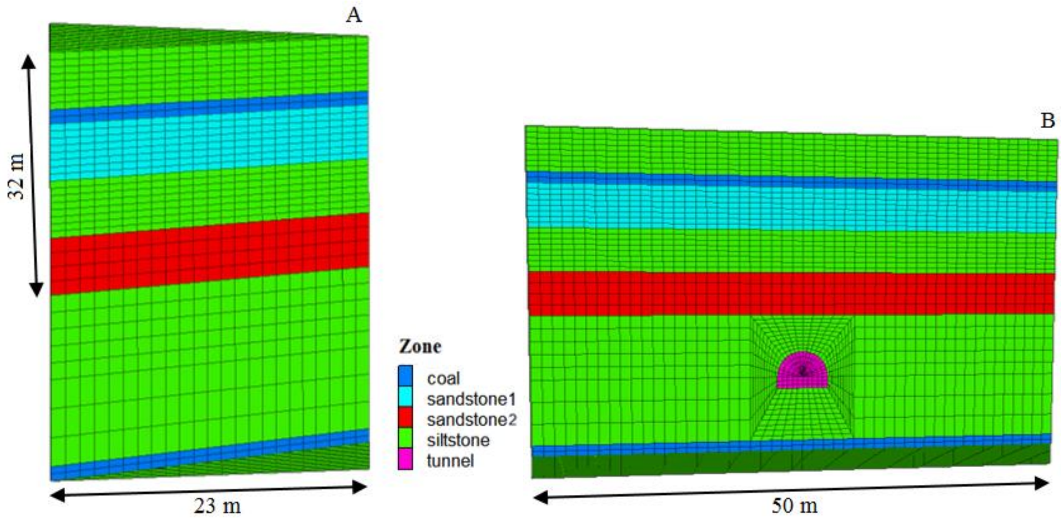


**Fig 4.** Numerical model introduced into FLAC3D (A- side view and B- front view)

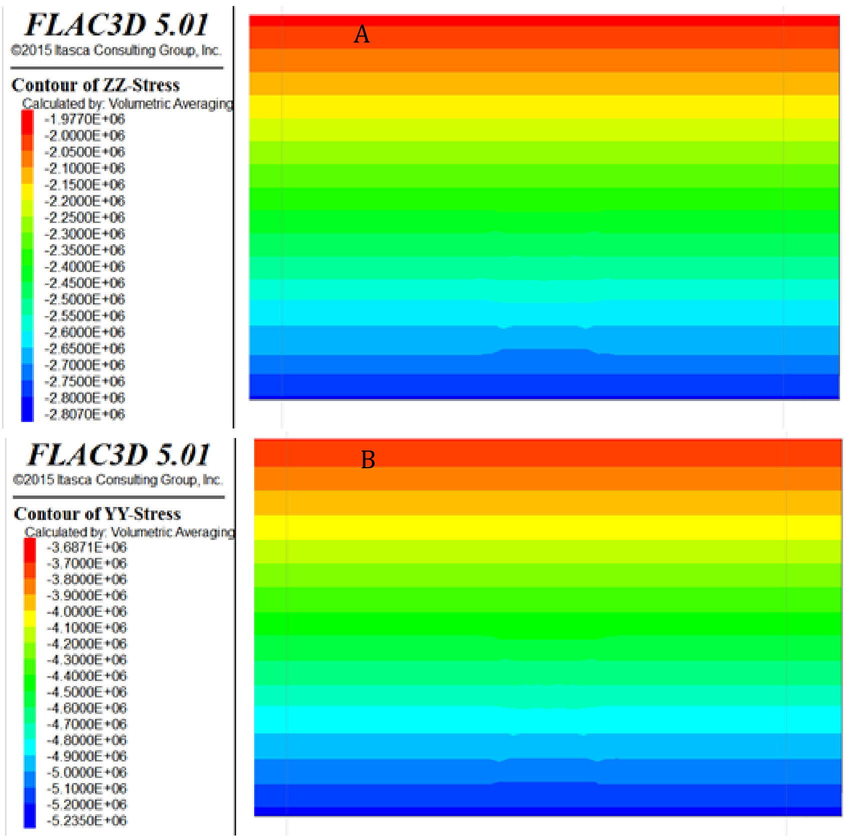


**Fig 5.** Stress status in the rock before excavations (A- vertical stresses and B- horizontal stresses)

## IV. RESULTS AND DISCUSSION

### *A. Stability Analysis and Support System Design Using Barton's Empirical Method*

Barton's empirical method was selected for its high accuracy in stability assessment and adaptability to the specific geological conditions of the Parvadeh 4 mine. This method is particularly effective in mines with high lithological diversity and variable environmental conditions, such as the Parvadeh 4 coal mine, which exhibits various rock types with differing strength characteristics, necessitating a precise stability analysis method. Given that the tunnel is excavated within siltstone, and based on borehole data from the inclined shaft portal area and its path (as shown in Table 2), the Q-value obtained is 1.33. Considering that the main tunnel is part of the permanent mine infrastructure, an ESR (Excavation Support Ratio) of 1.6 was adopted.

The values for each parameter in Table 2 were derived from lithological, geological, and structural features obtained from borehole data, using the tables provided by Barton (2002).

According to Fig. 3, this excavation span is not self-supporting and falls into Class 4, for which the recommended support consists of systematic bolting and unreinforced shotcrete, 4-10 cm.

The rock bolt length was calculated using Eq. (2) as follows:

$$L = \frac{2 + 0.15 \times 5}{1.6} = 1.7$$

Table 3 presents the support recommendations according to Barton's criterion.

**Table 3.** Recommended support system

| Type of support systems | Length of the rock bolt (m) |
|---|---|
| Systematic bolting with bolts spaced 1.8 m apart and unreinforced shotcrete 4-10 cm thick. | 1.7 |

### *B. Stability Analysis and Support System Design Using FLAC3D*

To analyze the stability of the incline, an initial study was conducted on the incline's stability without any support system installed. Considering that the inclines are excavated using drilling and blasting, an average advance of 1 meter per drilling stage was assumed. After each excavation step, the model was allowed to reach equilibrium before proceeding to the next step. This process continued until further excavations no longer affected the area under study. It is important to note that, in order to minimize the impact of boundary conditions, these conditions were examined starting from the fourth step onward. Fig. 6 illustrates the vertical displacement observed after the incline excavation at the fourth step (following 10 meters of excavation).

As shown in Fig. 6, the maximum vertical displacement at the roof is approximately 11 mm.

According to Eq.s (1) and (2), and given that the incline is excavated in siltstone, the critical strain and critical shear strain values are 0.053 and 0.0664, respectively. Fig. 7 illustrates the shear strain in the excavated incline.

**Table 2.** Characteristics of the rock surrounding the inclined shaft

| Rock type | Depth (m) | RQD | $j_n$ | $j_r$ | $j_a$ | $j_w$ | SRF | Q |
|---|---|---|---|---|---|---|---|---|
| Siltstone | 70-82 | 80 | 3 | 0.5 | 1 | 1 | 10 | 1.33 |

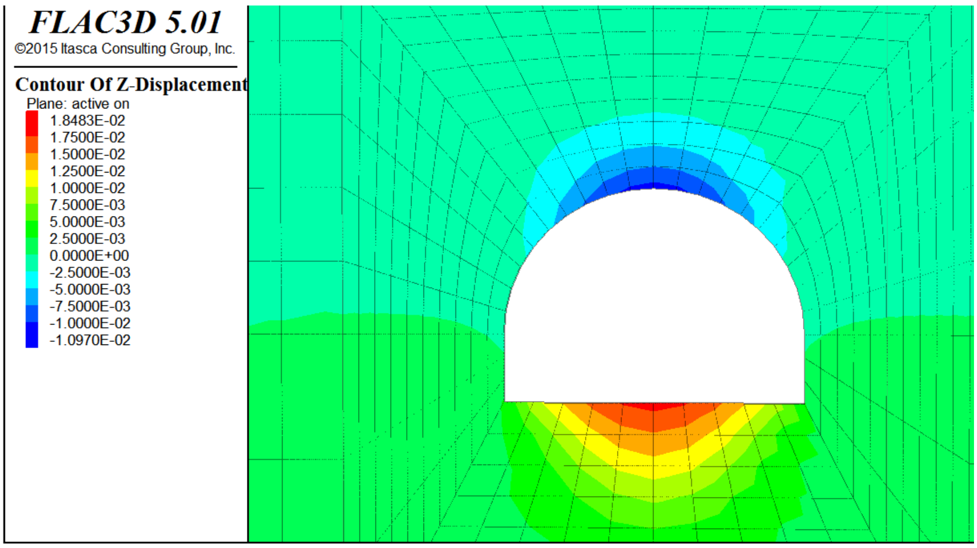


**Fig 6.** Vertical displacements around the inclined shaft

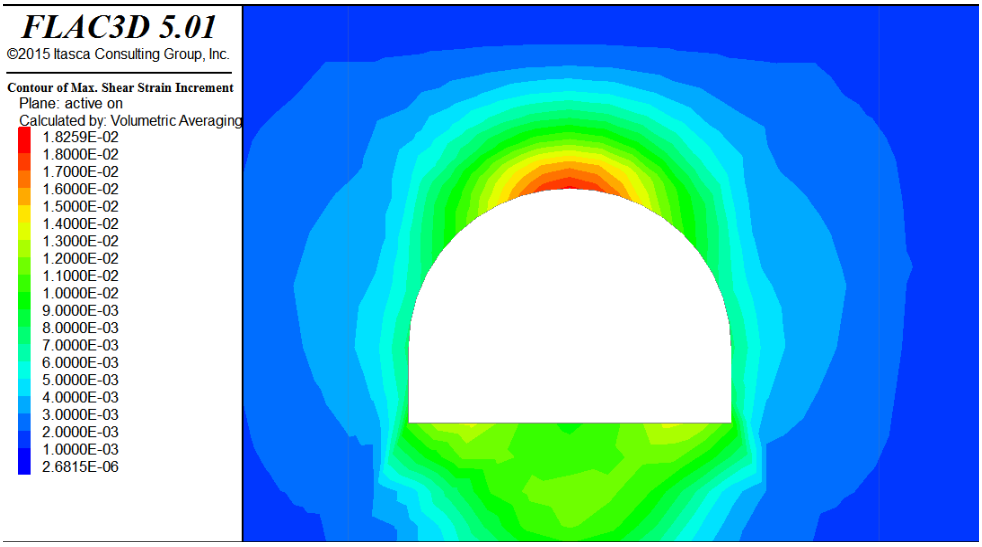


**Fig 7.** Shear strain caused by the excavated inclined shaft

As shown in Fig. 7, the maximum shear strain at the inclined roof is 0.0182. Therefore, it is concluded that the shear strain at the inclined roof exceeds the critical shear strain, indicating an unstable condition. This finding confirms the results of the empirical assessment using Barton's method. Consequently, support systems must be implemented to stabilize the structure.

To evaluate the status of the support system recommended by Barton's method, its performance was assessed in FLAC3D based on the specifications in Tables 4 to 6.

**Table 4.** The bolts properties (CMC, 2018)

| Parameter | Quantity | Unit |
|---|---|---|
| Diameter | 23 | mm |
| Modulus of elasticity | 200 | GPa |
| Tensile yield | 250 | kN |
| Bolt spacing | 2 | m |

**Table 5.** Shotcrete properties (CMC, 2018)

| Parameter | Quantity | Unit |
|---|---|---|
| Thickness | 100 | mm |
| Modulus of elasticity | 24 | GPa |
| Tensile strength | 2.7 | MPa |
| Compressive strength | 27 | MPa |
| Poisson's ratio | 0.2 | - |

**Table 6.** Wire mesh properties (CMC, 2018)

| Parameter | Quantity | Unit |
|---|---|---|
| Diameter | 6 | mm |
| Modulus of elasticity | 200 | GPa |
| Tensile strength | 400 | MPa |
| Compressive strength | 400 | MPa |
| Poisson's ratio | 0.25 | - |
| Spacing | 150 | mm |

With an advance rate of 1 meter and a bolt spacing of 2 meters, bolt installation was performed during odd-numbered excavation steps. To determine the timing of shotcrete application using Eq. (3), the model is solved with bolts installed at 2-meter intervals until the axial force in the bolts exceeds their tensile strength.

In the first step, the tunnel is excavated by 1 meter, and after 39% of the final displacement (obtained from the model without support) occurs, the bolts are installed. In this case, the maximum axial force is 57 kN, which is less than the tensile strength (Table 2) (Fig. 8). The maximum axial force in the second step is 128 kN, still less than the tensile strength of bolt (Fig. 9). In the third step, this value increases to 153 kN, indicating it exceeds the tensile strength, leading to instability (Fig. 10). Consequently, shotcrete should be applied to the first excavation stage at this point. To this end, the model is re-solved with bolts installed at this stage and shotcrete applied to the first stage. The axial force in the third step after installing bolts and shotcrete for the first stage is 141 kN, which is less than their resistance, and the tunnel is stable (Fig. 11). Thus, shotcrete application should occur two steps back, i.e., in the first excavation stage. In subsequent steps, excavation and support installation proceed as before. For an accurate stability analysis, the model was excavated up to the thirteenth stage until there was no impact on the first stage. Fig. 12 illustrates the axial force of the bolts after 13 excavation steps.

To assess the stability of the shotcrete, an axial force interaction diagram is used. Fig. 13 shows the axial force and bending moment applied to the shotcrete.

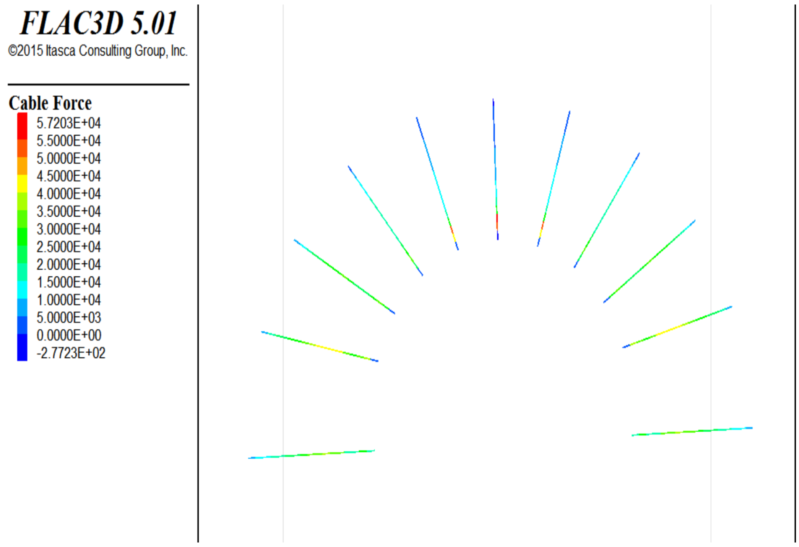


**Fig 8.** Axial force on bolts in the first excavation step, without shotcrete installation

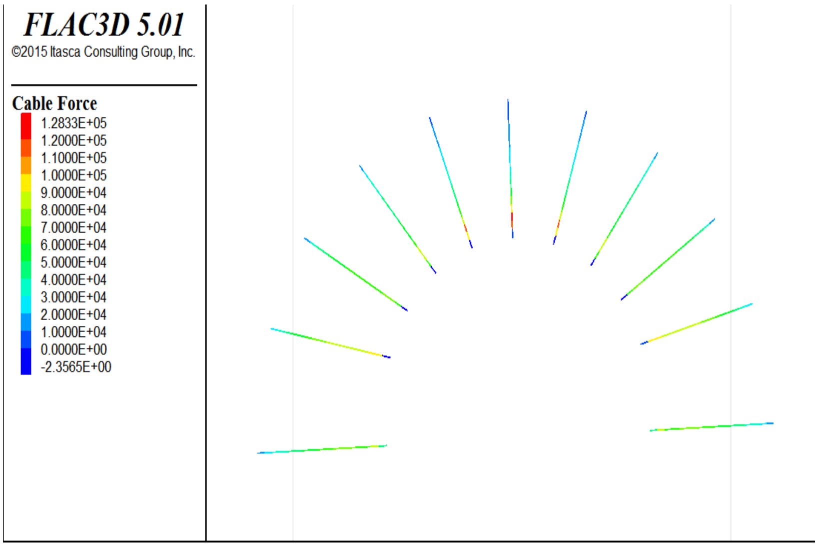


**Fig 9.** Axial force on bolts in the second excavation step, without shotcrete installation

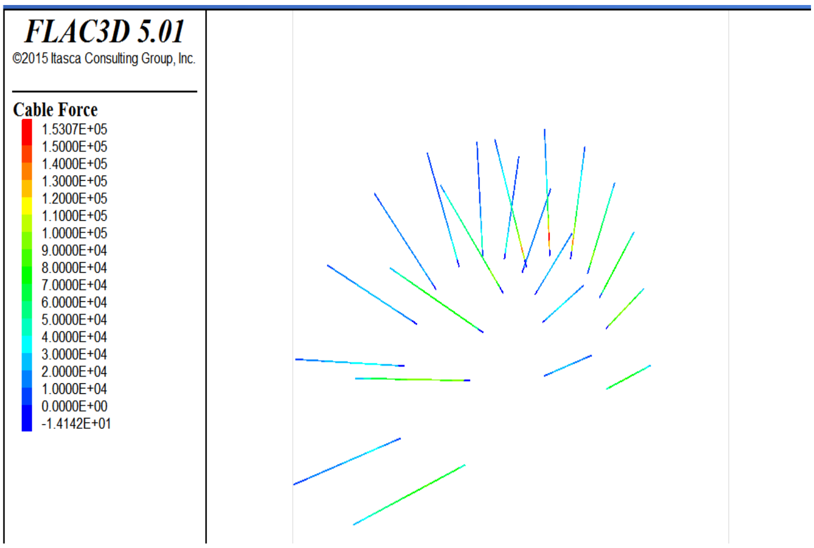


**Fig 10.** Axial force of bolt in the third excavation step, without shotcrete installation

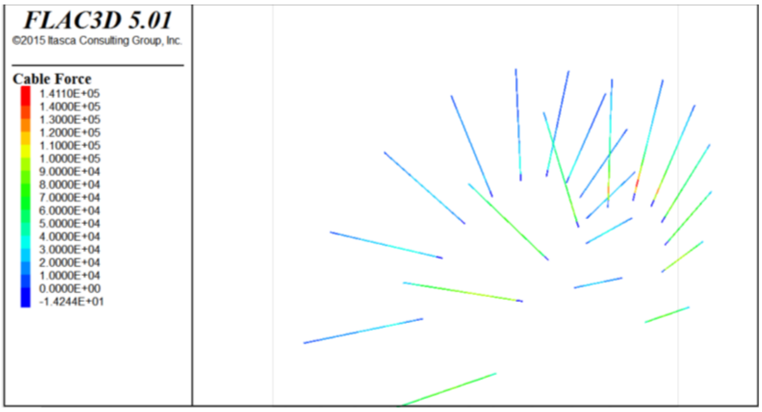


**Fig 11.** Axial force of bolt in the third excavation step, with shotcrete installation in the first excavation step

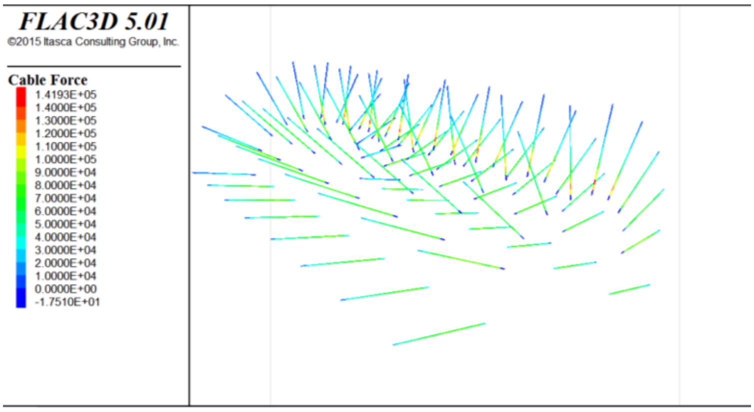


**Fig 12.** Axial force on bolts after 13 excavation steps

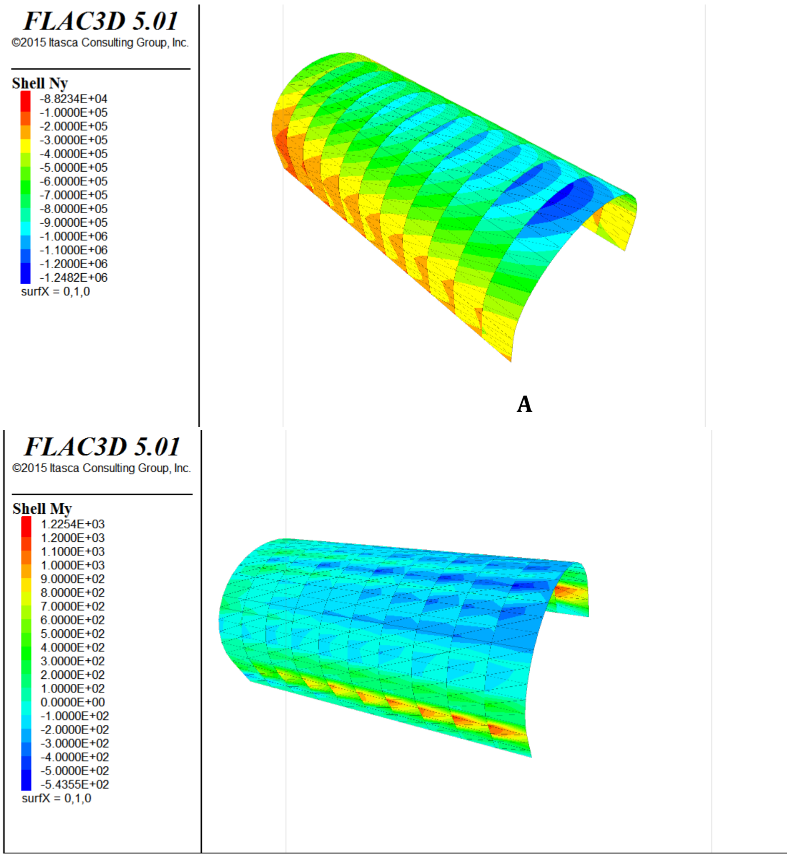


**Fig 13.** Loads on the shotcrete (A- Axial force contour and B- Bending moment contour)

**B**

To control the forces and bending moments on the composite lining calculated by FLAC3D, the bearing capacity diagram and the bending moment/axial force interaction diagram generated by the software are used. For this purpose, SAP2000 software was used to determine the bearing capacity and the bending moment/axial force interaction diagram. First, the properties of shotcrete and wire mesh were entered into the software (Fig. 14). Next, the geometric characteristics of the shotcrete and wire mesh were applied. Then, using SAP2000, the bending moment/axial force interaction diagram was drawn based on ACI 318-09, and the axial force and bending moment values from FLAC3D at points around the tunnel were also specified (Fig. 15).

As shown in Fig. 15, all points from the numerical modeling are within the interaction diagram, confirming stable conditions.

## V. Conclusion

In this study, Barton's empirical method and numerical modeling using the finite difference method were used to analyze the stability and specify the support system for the main inclined shaft of the Parvadeh 4 Block 3 mine. The most significant results from this paper are as follows:

- Results from the Barton empirical method indicate that the tunnel is not self-supporting and requires a support system consisting of 4 -10 cm shotcrete and rock bolts on 2 m spacing that are 2 m long.
- The non-self-supporting status of the inclined shaft was evaluated in FLAC3D using the Sakurai criterion, and the results indicate that the tunnel is not self-supporting and requires the installation of a support system.
- Through various modeling scenarios in the software, the optimal time for shotcrete application was determined to be the first excavation step. Bolt installation is performed immediately after excavation, with stress relaxation applied.
- In FLAC3D, the proposed system with 10 cm thick shotcrete and a 6 mm diameter wire mesh with a 15 cm spacing was evaluated. To control the forces and bending moments generated in the composite tunnel lining, interaction diagrams of bending moment/axial force prepared by SAP2000 were used. By modeling the lining and applying the forces and bending moments (calculated by FLAC3D), it was concluded that the lining with this type of wire mesh adequately ensures tunnel stability and withstands the axial forces and bending moments applied to the tunnel lining.
- It is suggested that the results obtained from the Barton method be compared not only with FLAC3D but also with other numerical methods such as the finite element method (FEM) or discrete element methods (DEM) in future studies.
- It is recommended that the investigation of support systems under more complex conditions or the use of hybrid finite element and discrete element models be considered in future studies at this mine.

It is suggested that studies be conducted to develop new support methods based on real data from the Parvadeh 4 coal mine.

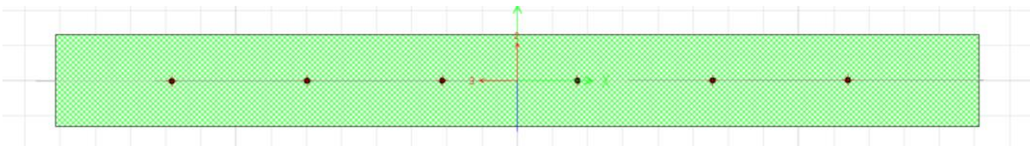

**Fig 14.** Cross-section of wire mesh with Shotcrete

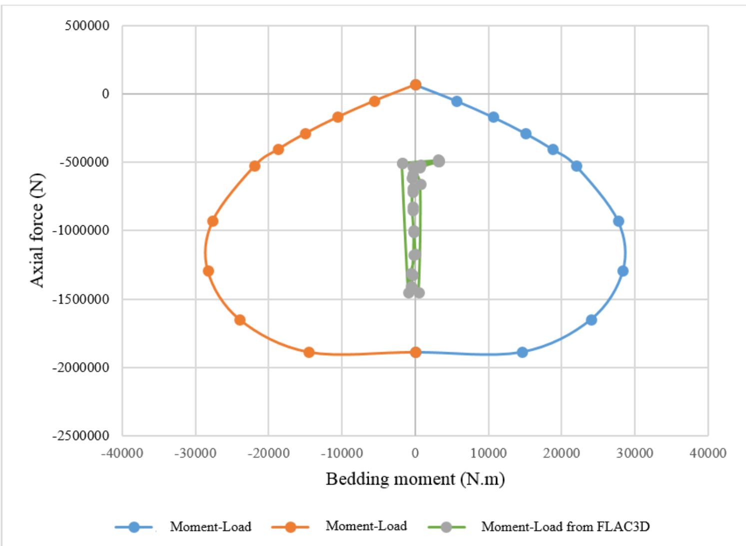


**Fig 15.** Bending moment/Axial force interaction diagram in the tunnel lining

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