Support Analysis of Main Tunnels in Tabas Mechanized Parvadeh Mine at Intersection with Coal Seam

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Article Info	Abstract

The instability of the roof and walls of the tunnels excavated in coal mines has always attracted the attention of the miners and experts in this field. In this work, the instability of the main tunnels of the mechanized Parvadeh coal mine in Tabas, Iran, at the intersection with coal seam is studied. The main tunnels of this mine show significant horizontal displacements due to the complex ground conditions and great depth. The behavior of the rock mass surrounding the tunnel is investigated using various experimental methods, and according to the results obtained, the surrounding rock mass has squeezing conditions. In order to analyze the stability of the main tunnels, a series of 2D and 3D numerical modelings are performed using the FLAC2&3D finite difference software, and the results obtained are compared with the actual displacement values recorded in the walls of the main tunnels of the mine. The analysis results show that the tunnels under study are unstable with a steel frame support, and therefore, the use of different support systems for the stabilization is investigated. The results of modeling different types of support systems show that the use of shotcrete instead of galvanized sheet (as strut) does not have a significant effect on the reduced displacements. Also although the installation of steel sets is very effective in preventing the displacement of the walls, due to the swelling problems in the tunnel bottom and the placement of the conveyor and haulage rail, it cannot be used in practice. Finally, the use of truss bolt has yielded good results, and it can be proposed as a new support system in these tunnels. In addition, the modeling results show that in case the coal seam is higher than the tunnel foot, less displacement will occur in the tunnel walls compared to the other cases. In other words, changing the tunnel level in the future excavations can help reduce the displacements.

1. Introduction

The first scientific definition of the squeezing and swelling rocks was provided by Terzaghi: the squeezing rocks slowly advance into the tunnel without significantly increasing the volume. In contrast, the advance of the swelling rocks into the tunnel is mainly associated with the increase in volume [1]. In order to determine the squeezing conditions, Jethwa et al. [2] have expressed the degree of squeezing, obtained by dividing the compressive strength of the rock mass by the in-situ stress. They presented a table that assessed the ground conditions in terms of the squeezing

degree obtained. Singh et al. [3] have used the relationship between the overburden height and the rock mass quality index (Q) in order to assess the tunneling ground conditions. Aydan [4] has introduced five classes for squeezing based on the values of elastic, plastic, softening, and failure strains and the amount of strain calculated by the closed-form solution methods. In order to estimate the squeezing conditions by the experimental method of Aydan, the radial displacements obtained by the Carranza-Torres and Fairhurst's method were used [4]. Goel et al. [5] have

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proposed the rock mass number to avoid uncertainty in determining the SRF score. This number is obtained from the relation of the quality index of rock by setting the value of one for the SRF parameter. Palmstrom [6] has used the rock mass index (RMi) parameter in order to evaluate the squeezing potential in the continuous environments. In massive rocks, the value of the RMi parameter is considered equal to 50% of the compressive strength of an intact rock, and in non-massive rocks, this value is calculated using Equation (1) [7].

$$\text{RMi} = 0.2\sigma_{ci} \sqrt{J_l (J_r / J_a)} V_b^{D}$$
$$D = 0.37(J_l (J_r / J_a))^{0.2}$$

(1)

where J_l denotes the reliability of the joint and V_b is the volume of the rock blocks (m³). J_r and J_a are determined according to the rock mass engineering classification of Q.

Hoek and Marinos have investigated the squeezing potential, according to Table 1, based on the convergence degree of the tunnel, which is calculated using the numerical methods[7].

Table 1. Classification of squeezing by Hoek and Marinos [7]

Tunnel convergence	< 1	1–2.5	2.5–5	5–10	> 10
Squeezing class	NA	Low	Medium	High	Very high

The experimental methods for predicting the support pressure include the Terzaghi, Deere, Barton, and Unal methods. The Barton method provides an acceptable method for estimating the support pressure under the squeezing and non-squeezing ground conditions in smaller tunnels. Singh et al. [8] have suggested the correction factors for the tunnel depth, closure, and time after the support. They modified the Barton relations for the short-term support pressure, and proposed equations for determining the short-term support pressure in the tunnels under the squeezing conditions [8]. Using the Scandinavian tunnel experiences and the information from Singh et al. and Goel et al, Bhasin and Gristad have introduced a new correction factor for poor qualities in the squeezing conditions [9]. Goel and Jethwa have proposed a relation for estimating the support pressure using RMR. One of the advantages of this relation is that it can be utilized in both the squeezing and non-squeezing conditions without having an advanced knowledge about the ground conditions [5].

Wang et al.[10] have reviewed a suitable support system for several tunnel excavations on the squeezing rocks in the Qingling-Daba mountainous squeezing area. Their proposed a support system including the installation of foot reinforcement bolt (FRB) and the use of a rigid, less deformable main support system that was performed well. Chhushyabaga et al. [11] have studied the effect of the presence of a fault near the tunnel on the design of the support system. They used the geological strength index (GSI) method in order to describe the rock mass and numerical method to analyze the support system. The values of displacement, stress, and yielded elements were studied to study the effect of the fault. In a critical review, Ghorbani et al. [12] have reviewed the main characteristics and support requirements of the squeezing ground conditions and characteristics of the support functions. They introduced different types of energy-absorbing rock bolts and other support elements applicable for ground support in the burst-prone and squeezing grounds. They also discussed the important differences in the choice and economics of ground support strategies in the high-stress ground conditions between the civil tunnels and the mining excavations. Hussein et al. [13] have considered the empirical and numerical methods used to classify the rock masses, and evaluated the status to design a support system. Zhao et al. [14] have installed an intelligent monitoring system in a constructing tunnel in order to investigate the squeezing behavior of the rock mass and its impact on the support system. In their study, the large-scale displacements and pressure on the support system were measured. They considered the implemented monitoring system to be suitable for monitoring in the conditions of high squeezing.

Parts of the main tunnels of the mechanized Parvadeh mine in Tabas are excavated in the coal seam with a thickness of about 2 m. Therefore, the difference in the strength properties between the coal seam and the surrounding layers made of siltstone causes the squeezing behavior. In this case, the rocks move into the tunnel and cause problems in the support. This causes problems in the support bases or walls due to the displacements that occur in the rocks around the tunnel as a result of the obvious difference in their strength. In this work, the ground behavior in the

studied section of the main tunnels of the Parvadeh mine in Tabas was evaluated using the experimental methods, and the squeezing intensity was determined. Then using the numerical finite difference method of the FLAC$^{2\&3D}$ software, the displacements of the existing support system were analyzed and the various types of support systems were examined. The results of the constructed models were validated by comparing the observations and measuring the displacement values in different parts of the tunnel. Finally, based on the observations obtained from the displacements of different parts of the tunnel and the support analysis with numerical modeling, a solution was provided to improve the support in the points of the tunnel that were related to the coal seam.

2. Empirical evaluation of surrounding rock behavior

According to the International Society for Rock Mechanics (ISRM), squeezing is the time-dependent cut of a rock that causes the surrounding rocks to move towards the tunnel. This phenomenon is associated with large deformations and time-dependent convergences during the tunnel excavation. The squeezing conditions in the selected section of the tunnel are evaluated using various methods as what follows.

2.1 Singh et al. criterion

This method is based on the experiences of 39 case studies. According to the studies collected from the rock mass quality index (Q) and overburden (H), Singh have provided a boundary between the squeezing and non-squeezing rocks, as shown in Figure 1. Equation (2) shows this criterion [8].

$$H = 350Q^{\frac{1}{3}} \tag{2}$$

In Figure 1, the area above the line indicates the squeezing-prone conditions, while the area below the line indicates the points that do not have a squeezing potential. In the tunnel under study, the value of the rock mass quality index is 0.18 and the overburden height in the section under study is 400 m [15]. Therefore, as indicated in Figure 1, the rock surrounding this tunnel will have the squeezing conditions.

2.2 Jethwa et al. criterion

Jethwa et al. [2] have defined the squeezing intensity of rock mass (N_c) as Equation (3) using the uniaxial compressive strength parameter:

$$N_c = \frac{\sigma_{cm}}{\gamma H} = \frac{\sigma_{cm}}{P_0} \tag{3}$$

where σ_{cm} is the uniaxial compressive strength of the rock mass, H is the depth of the tunnel from the ground level, P_0 is the in-situ stress, and γ is the unit weight of the surrounding rock.

Based on the N_c parameter, they introduced a table in order to predict the squeezing value and the intensity. According to this table, if the N_c parameter obtained is less than 0.4, the tunnel will have a high squeezing behavior. With increase in the value of N_c, the squeezing intensity decreases so that in the values above 2, the behavior of the rock mass will fall within the non-squeezing range.

Based on the information obtained for the tunnel under study, where γ = 26.5 kN/m^3, H = 400 m, and σ_{cm} = 2Mpa, it leads to N_c = 0.19. Therefore, the squeezing conditions are met in this work.

2.3 Hoek and Marinos criterion

Hoek [7] has used the ratio of the uniaxial compressive strength of rock mass (σ_{cm}) to the in-situ stress (P_0) as an indicator in the tunnels, and Hoek and Marinos [7] have developed an efficiency diagram in order to evaluate the squeezing problem in the tunnels (Figure 2). According to this figure, with decrease in the ratio of the rock mass strength to the in-situ stress, and consequently, increasing the strain, the squeezing intensity of the tunnel increases. It can be observed that according to this criterion, as shown in Figure 2, the studied rock mass has squeezing conditions.
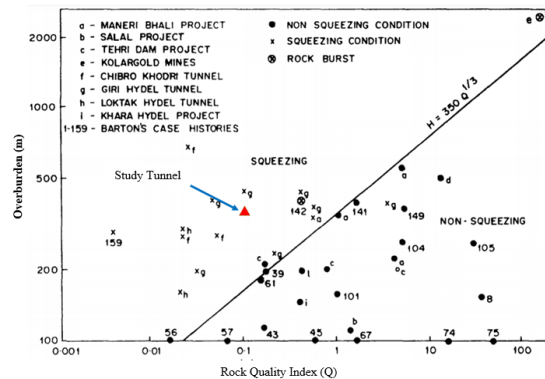

Figure 1. Squeezing criterion of Singh et al. [8]; the point indicating this study was added to the figure.

3. 2D numerical modeling

In order to construct the model, it was attempted to use all the behavior conditions that may exist in the environment to bring the results closer to the reality. The dimensions of the model gradually changed with multiple iterations, and in each case, the stress contours in the vicinity of the external boundaries of the model were examined. Finally, the dimensions of the model were considered as 50 × 50 m. In the mentioned dimensions, the stress contours in the vicinity of the external boundaries of the model are very close to the in-situ stresses, and as a result, the selected boundary

is sufficiently far from the tunnel impact area. In such cases, it is possible to ensure that the model boundaries do not affect the results. The constructed model was meshed by 250 units in the length direction and by 300 units in the height direction (Figure 3). The lateral boundaries were fixed with a roller abutment, and the lower boundary with an articulated abutment. In other words, the sides of the model are limited in the horizontal direction, and the lower part of the model is limited in the horizontal and vertical directions.
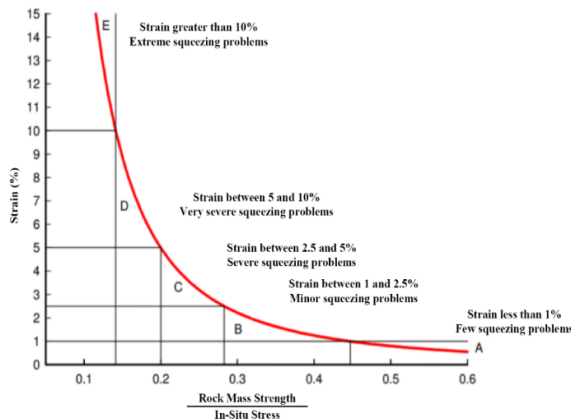

Figure 2. Classification of squeezing behavior according to the Hoek criterion[7].

Figure 3. 2D model geometry.

It should be mentioned that the distance between the studied face and the stopes is more than 300 m, and the stopes are located at the top level of the tunnel. Therefore, exploitation operations have no effect on the modeling results and are not modeled.

In this work, the elastic–perfectly plastic Mohr-Coulomb model was used for siltstone and sandstone, which is common in the modeling of rock mechanics. The modeling was carried out by defining the discontinuity between the coal seam and siltstone and defining the creep behavior model for the coal seam. Note that the creep behavior was reported only for coal in this mine. According to the information available in the mine, the ratio of the horizontal to vertical stress in the numerical model is equal to 1.14. The rock mass parameters listed in Table 2, reported by Planning report of the Tabas mechanized Parvadeh coal mine technical department (1997) for the Parvadeh coal mine, are used in the model.
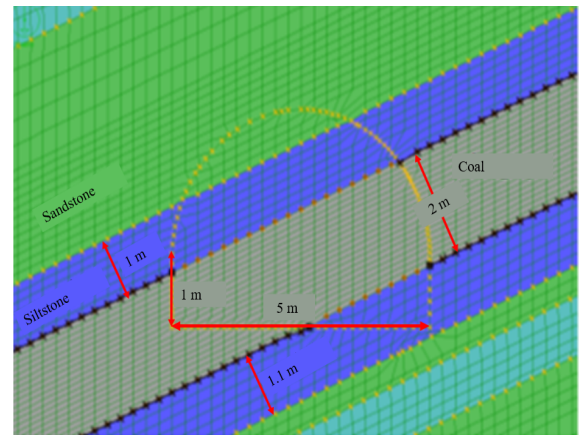
Table 2. Rock mass parameters used in the model [15].

Parameters	Sandstone	Siltstone	Coal
Tensile strength (MPa)	6.3	2.5	0.002
Internal friction angle (Degree)	21.75	24.12	15
Dilation angle (Degree)	5	5	2
Shear modulus (GPa)	2.112	1.126	0.126
Bulk modulus (GPa)	3.52	1.970	0.210
Cohesion (MPa)	8.69	1.3	0.016

As the coal was very brittle, the sample preparation was nearly impossible for this work. Therefore, some typical discontinuity characteristics and creep properties, reported by Xie and Zhao [16-18], were used in the model. According to them, coal has a discontinuity friction angle of 10 degrees and the normal and shear stiffness of 17 GPa and 1.8 GPa, respectively. Moreover, the creep properties of coal in Burgers model are $E_1 = 2.68$ GPa, $E_2 = 12$ GPa, $\eta_1 = 0.54$ GPa.h, and $\eta_2 = 24$ GPa.h.

3.1 Excavation of tunnel space and application of support system

Since the support in the mine is installed immediately after excavating the tunnel, the support installation in the model is performed after the excavation using the Beam command. The data to be given in this command is the cross-sectional area of the frame, moment of inertia, density of steel, and modulus of elasticity of steel. The mine support system consists of a U-shaped steel frame with galvanized sheet metal lagging. The cross-section of the tunnel is horseshoe shaped with 5 m' width and a height of 3.5 m. The common support system in the mine includes the U-shaped steel frames with galvanized sheets. The specifications of the steel frames used in this mine and also in the numerical modeling are shown in Table 3. Also similar to what is done in the mine, the distance between the frames is considered to be 1 m.

3.2 Displacement in rock mass

The 2D modeling was performed for three different locations of coal seam in the tunnel section. The displacement contours for these different positions are given in Figures 4 to 6. Also the displacement values in the walls and bottom of the tunnel in each case are shown in Table 4.

It can be observed that according to the location of the coal seam relative to the tunnel, the tunnel convergence is different, and in the case where the coal seam is located in the upper half of the tunnel

close to the tunnel roof, the displacement of the tunnel walls is less than the other cases.

The purpose of 2D modeling is to find the appropriate position of the coal layer in the tunnel section for future excavations. Due to the location of the coal layer relative to the tunnel, it can be seen that the convergence of the tunnel is different, and in the case where the coal layer is located at the upper half of the tunnel and close to the tunnel roof, the displacement of the tunnel walls is less than the other cases.

Table 3. Specifications of steel frame (TH29) [15].

Weight per meter (kg)	29
Area (cm^2)	37
H (mm)	124
B (mm)	150
b (mm)	44
h$_1$ (mm)	33
h$_2$ (mm)	15
I$_x$ (cm^4)	616
W$_x$ (cm^4)	94
I$_y$ (cm^4)	775
W$_y$ (cm^4)	103
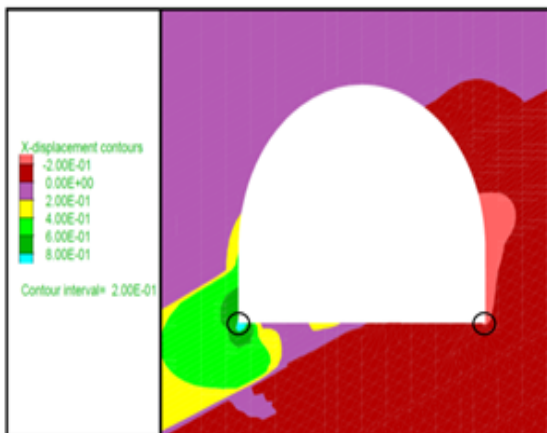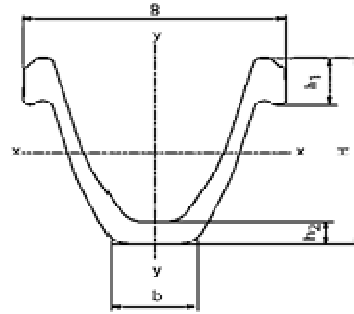
Figure 4. Horizontal displacement in the presence of coal seam in the center of tunnel.
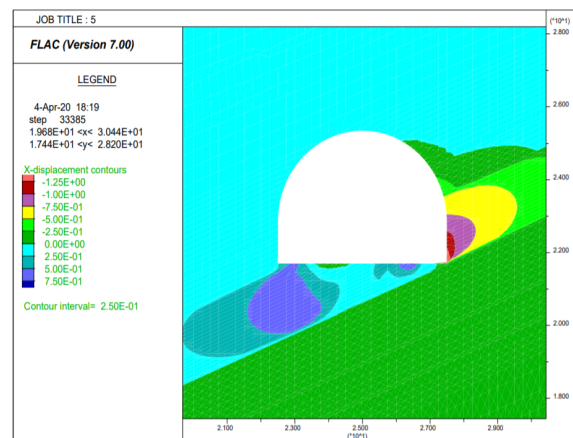
Figure 5. Horizontal displacement in the presence of coal seam at the bottom of tunnel.

Table 4. Displacements obtained in 2D model of tunnel with steel frame support (TH29).

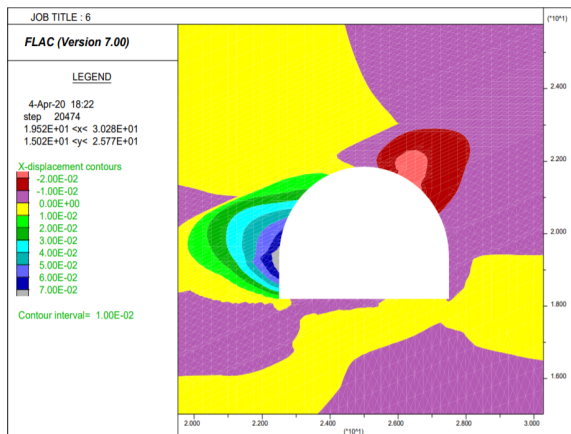| Location of coal seam in tunnel section	Max. horizontal displacement (cm)		Max. vertical displacement (cm)
	Right wall	Left wall	Bottom of tunnel
Center	20	70	150
Bottom	125	0	250
Roof	2	7	8

Due to the fact that in most cases of the intersection of the main tunnel with the coal seam in the Parvadeh mine in Tabas, the location of the coal seam is similar to the situation shown in Figure 4, the location of the coal seam relative to the tunnel is modeled in 3D in the next section.

4. 3D modeling
4.1 Model geometry

In the geometry of the 3D model, the three-node mesh is used, which has a higher density around the tunnel. Figure 7 shows the model geometry including 2 m thick coal, 5 m thick siltstone seams above and below the coal seam, and sandstone that forms the rest of the environment. This model is developed with the length and height of 50 m and thickness of 20 m. According to what is done in the mine, was has been made to
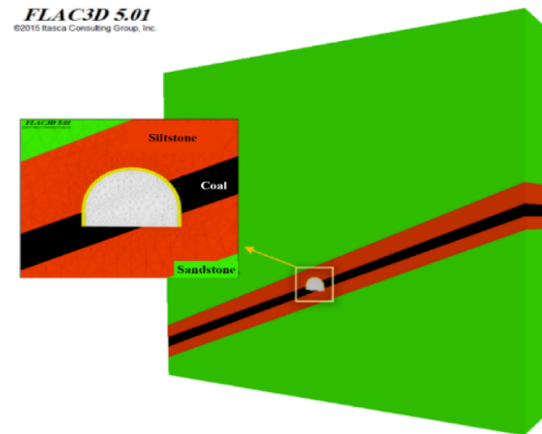
apply the boundary conditions in the form of fixed displacement boundaries at the bottom of the model as fixed boundaries, and to apply the roller boundary conditions on the sides of the model. In the upper part of the block, according to the pressure from the existing overburden, the upper part boundary was applied as a stress boundary with a load equivalent to γZ, where γ is the rock mass density and Z is the overburden height. The depth of the studied tunnel in the numerical model is 400 m. The vertical stresses were considered as a product of the density and height of the overburden. Due to the density of the rock mass, which was about 2650 kg/m³, a vertical stress of 8.87 MPa was entered in the model. The horizontal shear stress in the Y and X directions is a product of the vertical stress entered in the model during the experimental analysis.

Figure 6. Horizontal displacement in the presence of coal seam in the tunnel roof.

Figure 7. 3D model geometry.

4.2 Application of support system

The support system of the main tunnels of the mine is a horseshoe-shaped steel frame with a radius of 2.5 m and a height of 3.5 m, where the lateral distance of the frames is 1 m. The sandbags were used in order to fill the empty space behind the frames (20 cm). The distance between the steel frames was covered with 2 mm thick galvanized sheets. The properties of the

galvanized sheet used in the numerical model included the Young's modulus and Poisson's ratio equal to 210 GPa and 0.3, respectively. By applying the support, the results of the horizontal displacements after the equilibrium of the model are shown in Figure 8, and the comparison with the location of the initial frame is schematically shown in Figure 9.
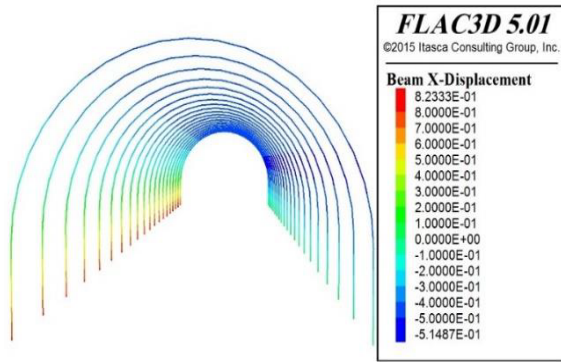
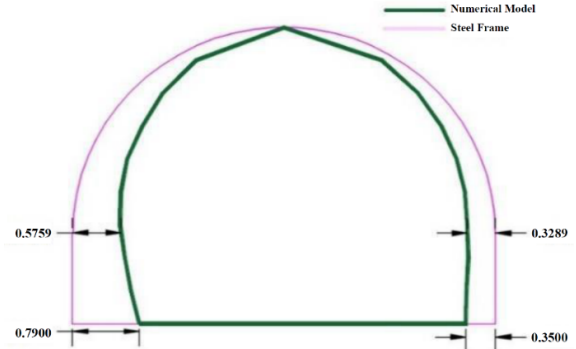**Figure 8. Horizontal displacement in the steel frame support system (unit: m).**

Figure 9. Comparison between the horizontal displacements in the numerical model and the main frame (unit: m).

4.3 Numerical model validation

The model was validated using the surveying data obtained from the displacement of the steel frame in the mine (Figure 10).

The results of the numerical modeling were compared with the actual displacement values of the steel frames in Figure 11. The displacement values at the four points shown in this figure including the foot of the steel frame and the points at the height of 1 m from the foot were measured and compared in Table 5 with the actual values

measured at the mine. It is noteworthy that the surveying data available in the mine is related to the present time of the tunnel, while the frame displacements still occurred at a slower rate than before. However, the results of the numerical model are related to the time when the model reaches the equilibrium, and gives the latest displacements in the section, and this time difference can cause a difference in the modeling results and the values measured in the mine.
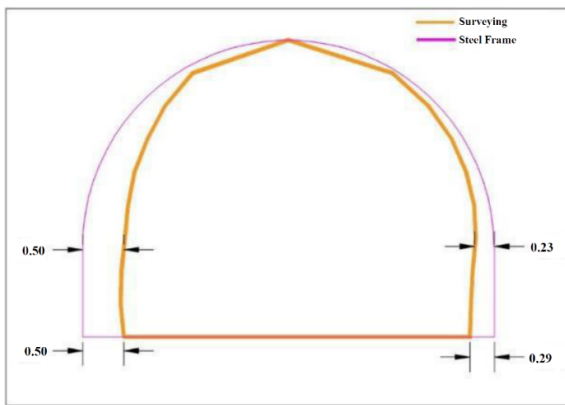

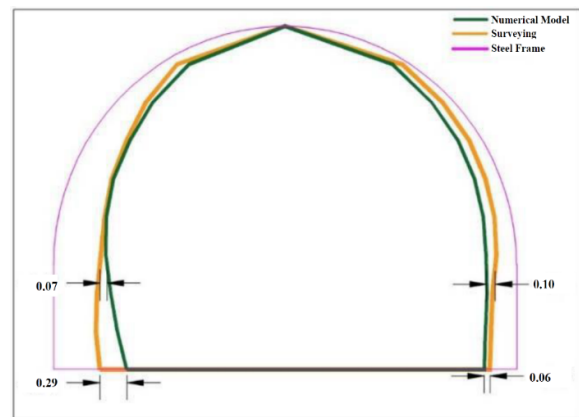**Figure 10. Displacements measured from the surveying data (unit: m).**

Figure 11. Comparison between the surveying data and the numerical model (unit: m).

Table 5. Comparison between the numerical model results and the measured surveying data.

Examination method	Right wall		Left wall	
	Foot	1 m from bottom	Foot	1 m from bottom
Numerical model (cm)	35	33	79	57
Surveying	29	23	50	50
Difference (%)	17	30	36	12

## 4.4 Application of various types of support in numerical model

In order to apply the other types of support, a combination of steel frame with shotcrete, fiber-reinforced shotcrete, and rock bolt arrangement was used. The properties of the modeled rock bolt and shotcrete are given in Tables 6 and 7, respectively. The support combinations used for this tunnel in the model included changing the distance between the steel frames, replacing galvanized sheet with shotcrete, using fiber-reinforced shotcrete, and using rock bolt. Figure 12 shows the arrangement of 13 rock bolts with two different lengths of 2 m and 4 m in the model.

Table 6. Rock bolt specifications used in the model [15].

Modulus of elasticity (GPa)	210
Density (kg/m³)	7000
Cross-sectional area (m²)	0.00037994
Yield limit (kN)	152
Mortar cohesion (MPa)	0.8
Mortar stiffness (MN/m)	5640

Table 7. Shotcrete specifications used in the model [15].

Modulus of elasticity (GPa)	Poisson's ratio	Thickness (cm)
27	0.25	10
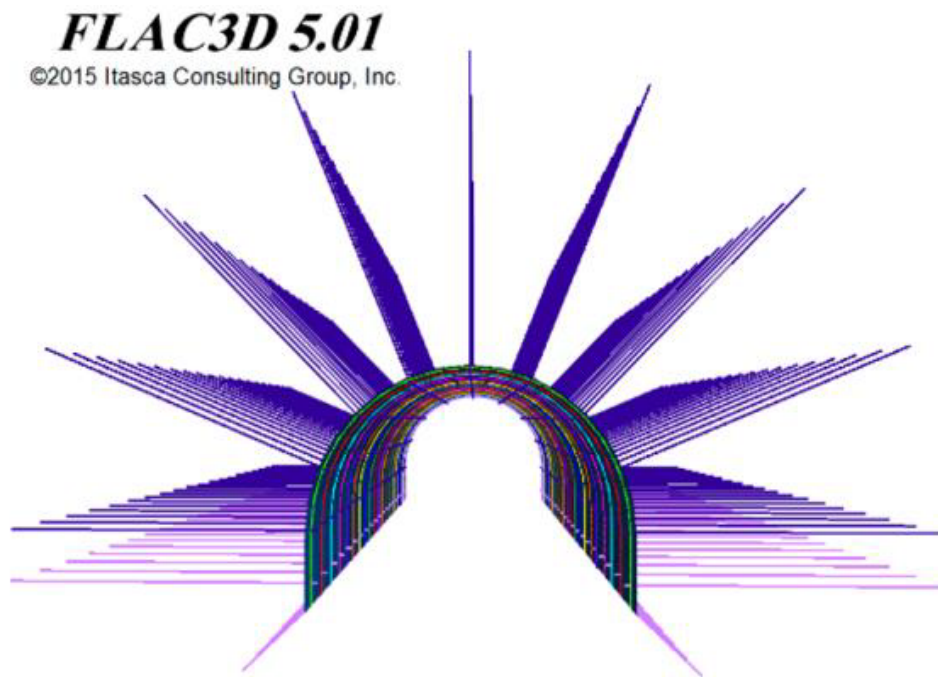
Figure 12. Arrangement of 13 rock bolts in the model.

4.5 Comparison between results of installing various types of support

The amount of displacement in the numerical models with various types of support system was measured for the four points specified in Figure 11 and summarized in Table 8. The results shown in this table are described below. The numbers shown in the titles of the following sections refer to the support number in Table 8.

a) Main support used in mine (No. 1)

The main support used in the main tunnels of the mine consists of a steel frame with the spacing of 1 m and galvanized sheet. In this model, the displacements in the right and left feet are 35 and 79 cm, respectively, and in the walls at the height of 1 m from the tunnel bottom on the right and left are 32 and 57 cm, respectively.

b) Main support by reduced spacing between frames (No. 2)

In this support, the distance between the frames was reduced to 0.5 m. It can be observed that reducing the spacing between the frames reduces the displacements by 10% compared to the previous case.

Table 8. Comparison between various support types applied in the numerical model.

Support No.	Support type						Horizontal displacement (cm)			
	Steel frame		galvanized sheet	Shotcrete	Rock bolt		Right side		Left side	
	Spacing (m)	Invert	Thickness (m)	Thickness (m)	Length (m)	Spacing (m)	Foot	1 m height from foot	Foot	1 m height from foot
1	1		0.002				35	32	79	57
2	0.5		0.002				30	28	70	46
3	1			0.1			32	30	75	56
3.1	1			0.1+mesh			30	28	61	45
4	0.5			0.1			31	32	69	45
4.1	0.5			0.1+mesh			29	22	65	41
5	1		0.002		2	1	30	29	56	43
5.1	1		0.002		2	1	30	29	54	41
6	1		0.002		4	1	28	27	47	38
6.1	1		0.002		4	1	29	26	46	36
7	1		0.002		2	0.5	30	28	43	33
7.1	1		0.002		2	0.5	30	27	41	33
8	1		0.002		4	0.5	29	26	40	30
8.1	1		0.002		4	0.5	28	25	39	30
9	1			0.1	2	1	30	25	57	42
9.1	1			0.1+mesh	2	1	27	24	52	35
10	1	✓	0.002				2	8	2	4
11	1	✓		0.1			4	10	5	7
11.1	1	✓		0.1+mesh			3	8	4	5
12	1	✓	0.002		2	1	2	7	2	5

c) Using shotcrete instead of galvanized sheet (No. 3 and 3.1)

In this type of support, instead of using the galvanized sheet, the shotcrete with the thickness of 10 cm is used, which has no significant effect on the results. However, in practice, the galvanized sheet is preferred due to the ease of installation. Although the use of mesh-reinforced shotcrete further reduces the displacement compared to shotcrete, the galvanized sheet is still preferred due to the insignificant impact.

d) Reduced spacing between frames with shotcrete (No. 4 and 4.1)

In this type of support, the displacements are reduced by 10% compared to the case where the frame with the distance of 1 m and shotcrete are used. Using the mesh-reinforced shotcrete, the displacement was not improved compared to the initial spacing of the frames.

e) Using rock bolt (No. 5 to 9.1)

This model uses 13 rock bolts, where the number of rock bolts in the walls and roof is 11, and there are two rock bolts in the feet. This system is not efficient enough to reduce the displacements. In addition, according to the results presented in Table 8, the installation of rock bolt under steel frame, increasing the length

of rock bolt and reducing the spacing were not so efficient in reducing the displacements.

f) Using steel inverts (No. 10 to 12)

The use of steel inverts in the original model significantly reduced the displacements. In the models consisting of steel inverts, the shotcrete and mesh-reinforced shotcrete are used instead of the galvanized sheet, and the results obtained are compared with the galvanized sheet. As with the previous cases, the use of galvanized sheet is recommended. Figure 13 shows the displacements in the steel frame with the presence of inverts.
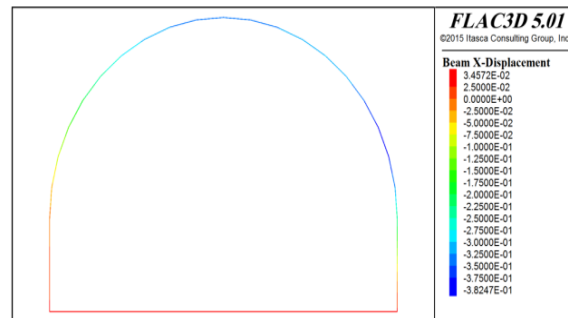

Figure 13. Displacement in steel frame with inverts.

Comparing the results of installing various types of support, which are summarized in Table 8, it can be seen that the installation of the inverts with a steel frame has the greatest effect on the reduced

displacement. The inverts prevent the convergence of the feet in the steel frame and considerably reduce the displacements.

Although the installation of steel inverts is very effective in preventing the displacement of the walls, due to the swelling problems in the tunnel bottom and the placement of the conveyor and haulage rail, it cannot be used in practice. Therefore, another approach should be adopted in order to solve the convergence problem of the main tunnels.
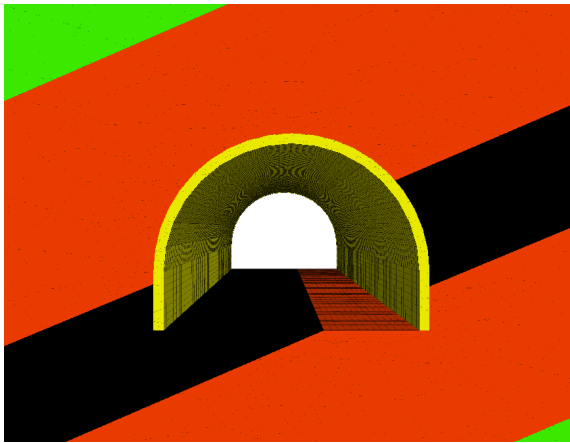

Figure 14. Model geometry and position of coal seam.

The displacement values resulting from the modeling of each type of truss bolt are shown in Figure 16, and the results obtained are compared in Figures 17 to 20 and Table 9 with the displacement values obtained from the model with steel frame support (as the main support system of the mine).

It can be observed that the use of truss bolt significantly reduces the displacement of tunnel

5. Truss bolt as a different support system

Another support system that can be used is to install the truss bolt in the walls of the tunnel. In order to simulate the installation of the truss bolt in the numerical model, the tensile force applied to the rock mass by the truss bolt was calculated and introduced into the nodes at the desired points in the model. In this work, the truss bolts with the length of 6 m and four different capacities of 60 tons, 80 tons, 100 tons, and 120 tons were modeled. The model geometry including the position of the coal seam relative to the tunnel cross-section is shown in Figure 14, and the position of the installed truss bolts is shown in Figure 15.
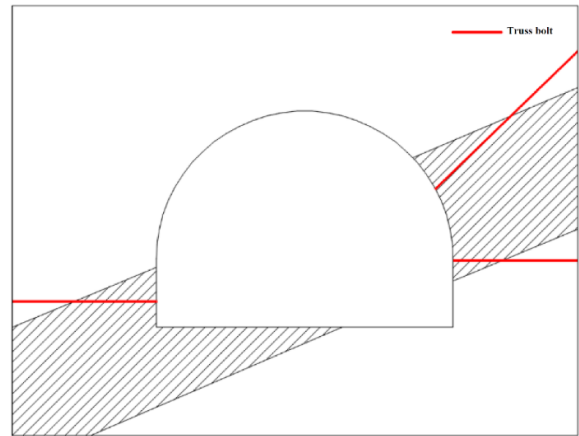
Figure 15. Installation position of truss bolts.

walls, and thus reduces the deformation of steel frames. In addition, the greater the bearing capacity of the truss bolt, the greater the impact on the reduced displacement of the tunnel walls. The displacement changes of the tunnel walls in terms of the bearing capacity of truss bolt are shown in Figure 21.
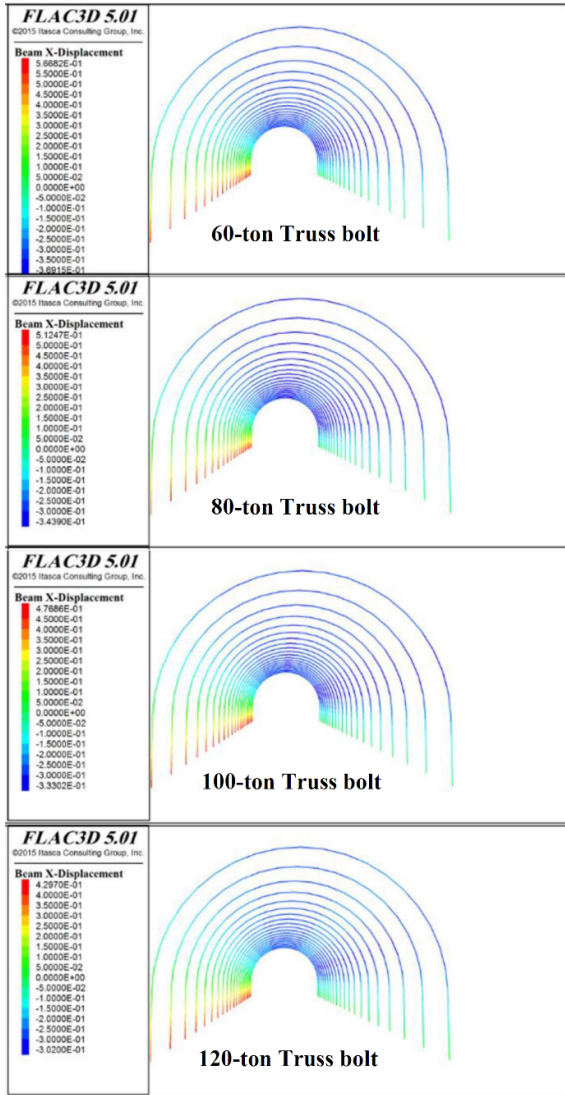
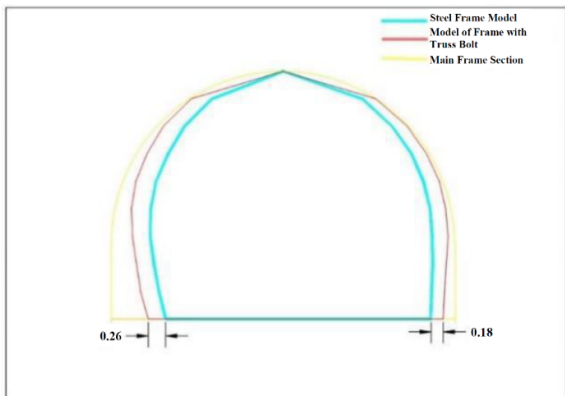**Figure 16. Horizontal displacements in truss bolt models.**

Figure 17. Horizontal displacement for the model with 60-ton truss bolts (unit: m).
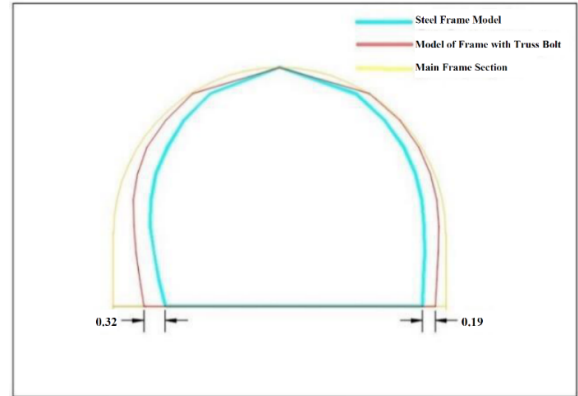

Figure 18. Horizontal displacement for the model with 80-ton truss bolts (unit: m).
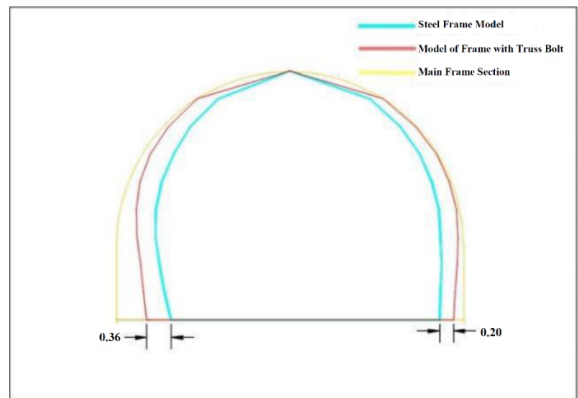

Figure 19. Horizontal displacement for the model with 100-ton truss bolts (unit: m).
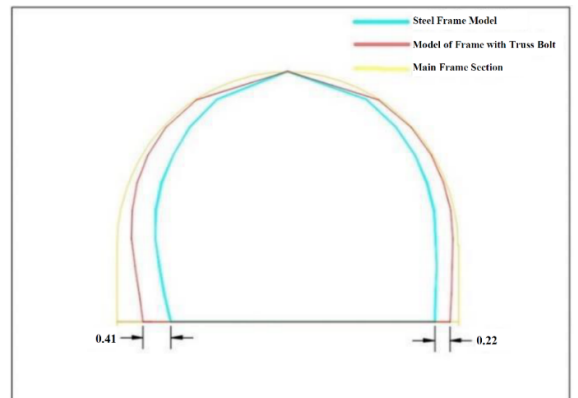

Figure 20. Horizontal displacement for the model with 120-ton truss bolts (unit: m).
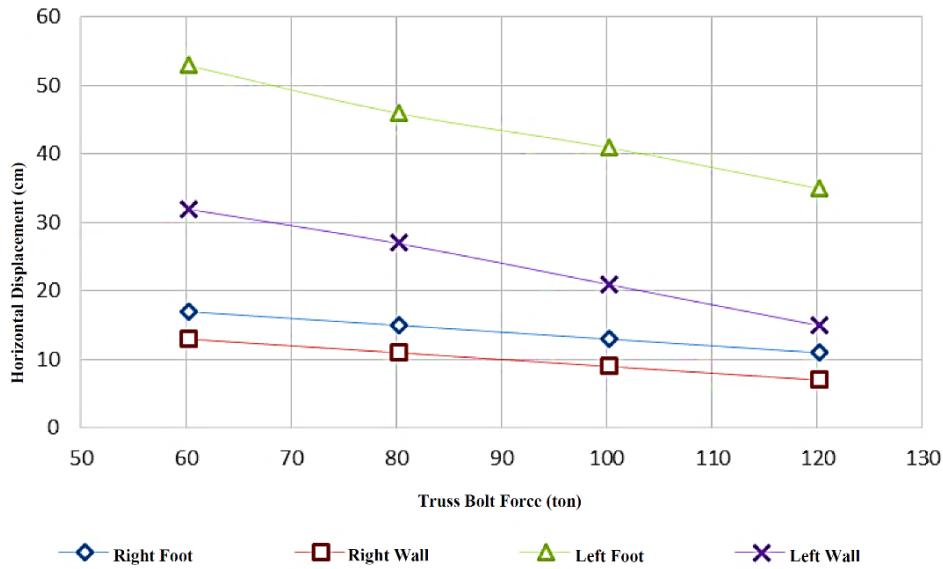
Figure 21. Diagram of horizontal displacement with truss bolts.

Table 9. Effect of truss bolts on displacement reduction in tunnel walls.

Type of support		Steel frame	60-ton truss bolts	80-ton truss bolts	100-ton truss bolts	120-ton truss bolts
Horizontal displacement in right wall (cm)	Foot	35	17	15	13	11
			-51%	-57%	-63%	-68%
	1 m height from foot	32	13	11	9	7
			-59%	-66%	-72%	-78%
Horizontal displacement in left wall (cm)	Foot	79	53	46	41	35
			-33%	-42%	-48%	-56%
	1 m height from foot	57	32	27	21	15
			-44%	-53%	-63%	-74%

6. Conclusions

In this work, the instability of the main tunnels of the Parvadeh mine in Tabas at the intersection with coal seams was studied and evaluated. According to the experiments based on the experimental methods of Singh et al. (1992), Jethwa et al. (1982), and Hoek and Marinos (2000), it was found that the rock mass surrounding the tunnels had squeezing conditions. In this regard, a series of 2D and 3D modelings were performed using FLAC[2&3D], and the results obtained were analyzed.

The 2D modeling was performed for three different locations of coal seam in the tunnel cross-section, and the displacement values in the tunnel walls were compared. The location of the coal seam was found to be effective in the tunnel convergence. In the case where the coal seam was located in the upper half of the tunnel close to the tunnel roof, the displacement of the tunnel walls was less than that in the other cases.

The 3D modeling was performed for the location of the coal seam, which included most cases of the main tunnel intersection with the coal seam in the Parvadeh mine of Tabas. In this modeling, the effects of different types of common support systems including shotcrete, steel frame, rock bolt, steel inverts, and their combination on the displacement of tunnel walls were investigated. It was observed that the installation of inverts with a steel frame had the greatest effect in reducing the displacement. However, due to the swelling problems in the tunnel bottom and the location of the conveyor and haulage rail, it could not be used in practice.

Finally, the use of truss bolts was evaluated as a different support system. For this purpose, the truss bolts with the length of 6 m and four different capacities of 60 tons, 80 tons, 100 tons, and 120 tons were used for the modeling. The results obtained showed that the use of truss bolts significantly reduced the displacement of tunnel walls, and thus reduced the deformation of steel frames. In addition, the greater the bearing capacity of the truss bolt, the greater the impact on the reduced displacement of the tunnel walls.

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تحلیل نگهداری تونل‌های اصلی معدن مکانیزه پروده طبس در محل تقاطع با لایه زغالسنگ

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چکیده:

موضوع ناپایداری سقف و دیواره‌های تونل‌های حفر شده در معادن زغالسنگ همیشه یکی از موضوعات مورد توجه معدنکاران و متخصصان این رشته بوده است. در این تحقیق ناپایداری تونل‌های اصلی معدن مکانیزه زغالسنگ پروده طبس در محل تقاطع با لایه‌زغالسنگ مورد مطالعه قرار گرفته است. تونل‌های اصلی این معدن، بدلیل شرایط پیچیده زمین و عمق زیاد، جابجایی‌های افقی قابل توجهی را از خود نشان می‌دهند. رفتار توده‌سنگ درون‌گیر تونل با استفاده از روش‌های تجربی مختلف مورد بررسی قرار گرفته و مشخص شد که توده‌سنگ درون‌گیر دارای شرایط مچاله‌شوندگی است. به منظور تحلیل پایداری تونل‌های اصلی، یکسری مدلسازی عددی دوبعدی و سه‌بعدی با استفاده از نرم‌افزار تفاضل محدود FLAC2&3D انجام شده و نتایج آن با مقادیر واقعی جابجایی ثبت شده در دیواره‌های تونل‌های اصلی معدن مقایسه شد. نتایج به دست آمده از تحلیل‌ها نشان داد که تونل‌های مورد مطالعه با نگهداری قاب فولادی ناپایدار بوده و بنابراین به منظور پایدارسازی، استفاده از سیستم‌های نگهداری مختلف مورد بررسی قرار گرفت. نتایج مدلسازی انواع مختلف سیستم‌های نگهداری نشان داد که استفاده از شاتکریت به‌جای ورق گالوانیزه (به عنوان لارده) تأثیر قابل ملاحظه‌ای بر روی کاهش جابجایی‌ها ندارد. همچنین، اگرچه نصب کف‌بند فولادی در جلوگیری از جابجایی دیواره‌ها بسیار تأثیر گذار است، امّا بدلیل وجود مشکلات آماس در کف تونل و قرار گیری نوارنقاله و ریل باربری، استفاده از آن عملا امکان پذیر نخواهد بود. نهایتا، استفاده از تراس بولت نتایج مطلوبی را نشان داده و به عنوان یک سیستم نگهداری جدید در این تونل‌ها پیشنهاد شده است. علاوه بر آن، نتایج مدلسازی نشان داد که در حالتی که لایه زغالسنگ بالاتر از پاشنه تونل باشد مقدار جابجایی کمتری در مقایسه با حالات دیگر، در دیواره‌های تونل اتفاق خواهد افتاد. به‌عبارت دیگر تغییر تراز تونل در حفریات آتی می‌تواند به کاهش جابجایی‌ها کمک کند.

کلمات کلیدی: تحلیل پایداری، مدلسازی عددی، سیستم نگهداری، معدن زغالسنگ پروده طبس، تراس بولت.