

Using Umbrella Arch Method in Design of Tunnel Lining, Case Study: Water Transfer Tunnel of Kani-sib, Urmia lake

Naser Pashaye*, Sina Fard Moradinia**, Adel Ferdousi***

ARTICLE INFO

RESEARCH PAPER

Article history:
Received:
February 2021.
Revised:
July 2021.
Accepted:
October 2021.

Keywords:
Umbrella arch,
pre-support,
Lining,
FLAC3D,
Forepoling

Abstract:

The access tunnel of the Urmia Lake for water transfer and restoration project (Kani Sib) is located in the south of West Azerbaijan Province, Iran. Part of this tunnel is located on weak and very loose soil, which in some areas, cannot be stabilized, despite the use of step drilling, and may lead to ceiling collapse, face collapse and even deformation in support system. In these cases, it is necessary to adopt the pre-support method of the umbrella arch. Tunnel stability analysis is one of the important factors in tunnel design and support system. Indeed, the type of support system is chosen according to the required stability and permitted displacement for the tunnel. In the present article, first the permitted displacement for the tunnel is calculated by Sakurai correlation. Then, the ground reaction curve is plotted using the numerical method of finite difference, namely, FLAC3D software, and the convergence-confinement method (CCM) is used to determine the acting instant for the support system. Finally, the safety level of the proposed support system is investigated considering different safety factors. The results of this study indicate that the Sakurai displacement correlation is more reliable than the other graphs presented. The results derived from numerical modeling are verified accurately against visual observation and instrumentation results. A suitable umbrella arch pre-support system with Lattice and Shotcrete support system is recommended. The umbrella arch pre-support system encompasses forepoling pipes of 90 mm diameter with 9 m in length and 2.5 m in overlap length.

1. Introduction

Tunneling in alluvial areas has its own complexities and problems. Due to the low slag depth, the ground is often soft and non-cement. Therefore, the safety and sensitivity of the land subsidence make it more crucial to install the support system immediately after tunnel drilling. In recent years, the use of the umbrella arch and the Austrian method (NATM) have attracted much attention for tunneling on loose soil. This solution is used in cases where rapid support and control of deformation is considered to provide safe conditions. The umbrella arch method is an efficient method, compared to costly and time-consuming support methods, such as

ground freezing, pipe jacking and jet grouting. The tunnel construction causes changes in ground stress, as well as strains and displacements. If the stress exceeds the strength of the soil, it causes high displacements resulting in complete failure. The potential for tunnel breakdown risk is related to the distance from the face and the most critical condition is immediately after excavation of the tunnel crown. For this reason, the stability of the crown is vital. Expansion of the plastic zone, as well as its geomechanical properties, also depends on the approach and form of drilling, and its proper selection can play a significant role in reducing this phenomenon during drilling. Among special auxiliary support systems, umbrella arch pre-support techniques can be inferred. Despite the increasing use of this method, few studies are carried out on its performance and no specific model is proposed for the associated proper design. This method has been widely used in Italy and Japan since the early 1980s. According to Muraki as drilling progresses and the subsidence increases, if the subsidence reaches to 30% - 40% of the total amount at a specific point in the workday,

* MSc, Department of Civil Engineering, Tabriz Branch, Islamic Azad University, Tabriz, Iran.

** Corresponding author: Assistant professor, Department of Civil Engineering, Tabriz Branch, Islamic Azad University, Tabriz, Iran, fardmoradinia@iaut.ac.ir

*** Assistant professor, Department of Civil Engineering, Tabriz Branch, Islamic Azad University, Tabriz, Iran.

no action will be able to restrict the subsidence of the face. Therefore, pre-drilling support operation is an efficient technique to reduce subsidence of face [1].

Fig[2], shows the effectiveness of the umbrella arch method. Aksoy and Onargan investigated the effect of umbrella arch and face bolts on the surface subsidence in the populated area of Izmir, Turkey. Their method is the use of in situ measurements and three-dimensional finite element numerical methods[2]. Ochak [3], investigated the surface subsidence in Istanbul's subway Line 2 using two umbrella arches with and without experimental methods. The results demonstrated that the use of umbrella method is effective in controlling the subsidence, as it results in three times less subsidence than the Austrian method. Although implementing the umbrella arch method is not cost-effective, its costs are lower than the costs of subsidence and damage to surface structures and for this reason, it is extensively studied by various researchers [4, 5, 6, 7, 8]. In addition, historical studies of this method show that in many tunnels around the world, the umbrella arch method is used to stabilize the face [9, 10, 11, 12, 13, 14, 15, 16].

Volkman and Schubert. [21,20,19,18], used behavioral scintigraphy and inclinometers, in the Trojan project, to study the deformations induced by drilling under the umbrella. The number and diameter of pipes are important factors in design. These two parameters determine the residual space between the pipes. Doi et al. [23], investigated the effect of inter-pipe spacing using discrete element method and PFC2D software. In another study by Shinji et al. [23], A large-scale umbrella method is simulated in the laboratory with a three-dimensional numerical finite element method to investigate the mechanism of consolidation and improvement of the design method. Doi et al. [23], after modeling, showed the relationship between adhesion, internal friction angle, distance between pipes and arch height. Junijah et al. [25], conducted studies on the physical modeling of the umbrella arch method using centrifuges to investigate the effect of umbrella arch length on subsidence. In another study, Asadollahi and colleagues [26], modeled station E on line 2 of the Karaj subway and the subsidence caused by its drilling using FLAC3D software. The subsidence at the ground level of the pilot tunnel excavation is compared for two cases of modeling and field measurements and the model is validated. By using three-dimensional numerical modeling in ABAQUS finite element software, Palasi and Moravatdar [27], investigated the effect of pipe length on reducing the tunnel subsidence and determined that choosing the optimal length, about one and a half times the size of the tunnel diameter, subsidence values are further reduced Hashemi and Kamali [28], experimentally and numerically studied the behavior and performance of umbrella arch techniques on reinforcing tunnels and presented the most recent achievements in the design, implementation, and analysis of this method.

In this research Numerical modeling of tunnel drilling and maintenance was performed in two cases; with and without pre-consolidation system. Then the Forepoling umbrella

system was equated by equal-legs angles. The efficiency of the pre-consolidation system was evaluated by the displacement values obtained from the numerical model and the efficiency of the temporary maintenance system after obtaining the axial forces and bending anchors from FLAC3D software and also drawing the interaction curve of the maintenance system by SECTION BUILDER software. Finally, the Sakura hazard level curve as well as the tunnel convergence curves were used to verify the results obtained from the numerical model.

2. Materials and Methods

The access tunnel of the Urmia Lake for water transfer and restoration project (Kani Sib) is located in the south region of West Azerbaijan Province. To access the main tunnel, an access tunnel must be constructed. To this end, an access tunnel of 1.354 km with a negative slope of 10.25% is designed. Approximately 200 meters of this tunnel is located in weak and very loose soil, which in some areas, despite the use of step drilling, cannot be stabilized and may cause ceiling collapse, face collapse and even deformation in support system. In these cases, it is necessary to use the pre-support method of the umbrella arch. The umbrella arch method used in access tunneling is of forepoling type, which is modelled using numerical finite difference method and FLAC3D software. The FLAC3D software is a 3D version of the FLAC software, which allows the user to model different structures in 3D, allowing for more accurate answers. It has the benefits of 3D modeling, including rotation, analysis from different angles, precise determination of actual stresses and alike. This software makes it possible to achieve more accurate answers than its two-dimensional version and embedded FISH programming language provides it with great capabilities. The program simulates the behavior of soil, rock and other structures that may undergo plastic flow when they reach the yield point. Materials are represented by grids or zones that form a grid (volume) to follow the intended purpose. Each grid element responds to the force applied to the limited boundaries based on the linear or nonlinear stress-strain law against. Materials can yield or flow and the grid can deform and move with the materials shown. The explicit Lagrangian calculation and the zoning methods used in FLAC3D provide the required accuracy for the simulation of collapse and plastic flow with high accuracy.

In this study, the structural performance of this method at different drilling stages, as well as stress and force distribution over the pipes located in the tunnel crown and around it are studied and juxtaposed. After modeling the umbrella arch system, the necessity of using these two umbrellas in the alluvial section of the access tunnel is studied considering the occurred displacement and subsidence values. Sakurai risk levels and convergence data are examined and finally the interaction curve of the support

system (bending moment-axial force) is plotted to evaluate the stability of the support system.

Simultaneously with the excavation and execution of the tunnel, the excavated sections are immediately maintained by methods such as rock bolt or shaktrit, after which the main and permanent wall of the tunnel must be executed. This wall is usually made of concrete and is called tunnel lining or tunnel inner covering. Lining operations can be performed by special tunnel molds. This formwork is made based on the general considerations of formulating underground structures as well as the specific considerations of the project.

2.1 Umbrella method maintenance mechanism

Stabilization of the face is done by creating an arched area in both longitudinal and transverse directions. In the longitudinal direction of the load, the part that has just been drilled is inserted by a beam, one end of which is on the ground and the other end on the maintenance system. And in the transverse direction, the arch-shaped area that acts as a frame bears the load of the earth (Fig.1&2).

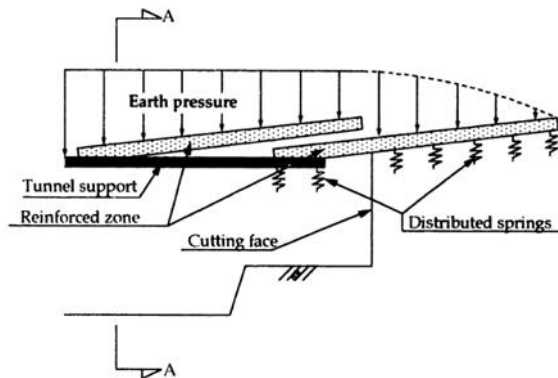


Fig. 1: Increase stability in the longitudinal direction [1]

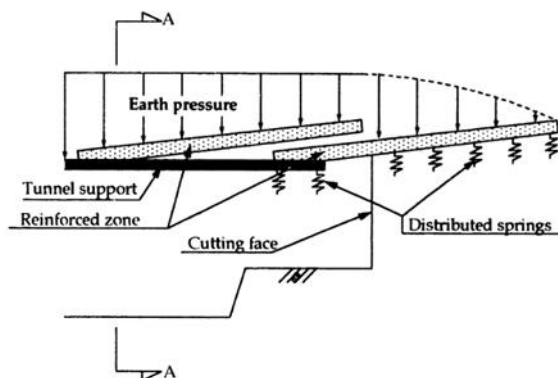


Fig. 2: Increase stability in the transverse direction [1].

2.2 Increasing stability in the face using umbrella method

As shown in Figure 3, if the umbrella method is not used, the stress concentration is associated with a reduction in vertical tension to zero in the pectoral (point A). The amount of stress in the unsaved section (between points A and B) should be zero. With the installation of the maintenance system at a certain distance behind the work face, an increase in vertical

tension is expected again. In Figure 4, which uses the umbrella method, the unmaintained cross section of the tunnel is covered with an umbrella structure. This structure withstands overhead pressure. The vertical stresses applied to the umbrella structure, such as the lining and ground connection, depend on the relative stiffness between the structure and the ground. Finally, this method has reduced the concentration of tension in the front and back of the face. If the umbrella method is not used, with increasing tension, the mohair circle will break the failure and lead to failure. While using this method, the increase in stress is controlled and does not lead to failure.

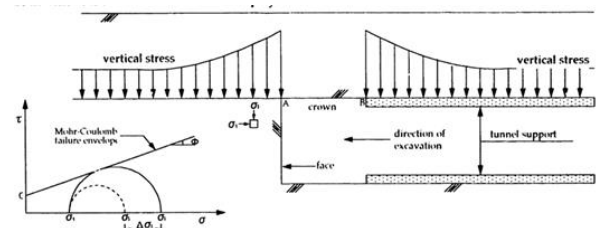


Fig. 3: Distribution of vertical stress in the crown of the tunnel and the position of stress in front of the face (no use of umbrella method)

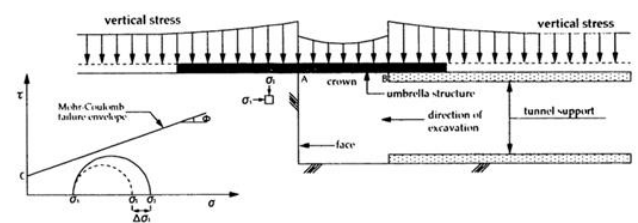


Fig. 4: Distribution of vertical stress in the crown of the tunnel and the position of stress in front of the chest (using the umbrella method)

2.3 Geotechnical studies carried out on the access tunnel area

In the study area, for the purpose of obtaining the geotechnical information required for drilling and stabilizing the entrance trench and alluvial section of the tunnel, a geotechnical borehole is excavated in the access tunnel entrance with a depth of 30 m, by the Ministry of Roads and Urban Development, Technical & Soil Mechanics Lab. Co. These boreholes are rotated by continuous core drilling and in-bore experiments are performed. Specifications and borehole logs are presented in Table 1 and Figure 5, respectively.

Table 1: Exploration borehole characteristics of the tunnel area

Bore No.	position	depth		
		alluvium	rock	total
1	Access tunnel	30	0	30

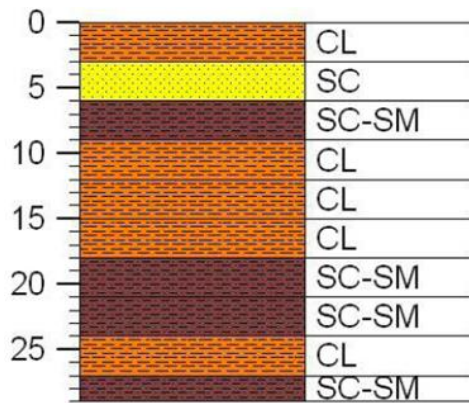


Fig. 5: Normal Logs

The results of the laboratory experiments performed on the samples of this borehole are presented in Table 2:

Table 2: Results of laboratory experiments on borehole samples

Depth		Soil type	γ_d (g/cm ³)	γ_w (g/cm ³)	E (kg/cm ²)	Direct shear	
from	until					C(kg/cm ²)	ϕ
0	3	CL	2.03	2.08	76.8		
3	6	SC	1.96	2.03		0.11	27
6	9	SC-SM					
9	12	CL	2.07	2.24		0.15	27
12	15	CL	2.05	2.10		0.15	29
15	18	CL	1.98	2.03	35.67		
18	21	SC-SM		2.08			
21	24	SC-SM	2.00	2.08		0.09	31
24	27	CL		2.08			
27	30	SC-SM		2.08			

In Table 2, γ_d denotes the dry density, γ_w is the wet density, E represents the elastic modulus, C is the adhesion and Φ refers to the internal friction angle of the materials. In addition, to ensure the input parameters of the stability analysis, intact sampling of the face and wall of the tunnel is performed and the samples are tested, the results of which are presented in Table 3. The most noticeable properties of the materials in this area of the tunnels are their Cohesion and plasticity index. Considering the materials and properties presented above, it can be deduced that these materials are highly susceptible to touching, which can cause the materials to fall in different parts of the tunnel. In Table 3, γ_d is the dry density, C denotes adhesion, and Φ is the internal friction angle of the material. Plastic limit and PI is the plasticity index of material.

Table 3: Results of geotechnical experiments on intact specimens

Specimen No.	γ_d (g/cm ³)	Direct shear		Soil type	LL	PI
		C(kg/cm ²)	Φ °			
1	1.75	0.07	29	SC-SM	24	5
2	1.65	0.08	27	SC	30	10
3	1.53	0.09	27	SC	32	14
4	1.72	0.07	28	SM		NP
5	1.7	0.08	27	SC	29	11

3. Results and Discussion

The use of numerical-analytical models to investigate the mechanical behavior of the tunnel requires a number of input parameters that must be defined and determined. In this research, the Mohr-Coulomb is selected to analyze the tunnel stability, due to the geological characteristics of the tunnel under study.

The process of modeling and stability analysis is as follows:

- Modeling geometry based on the dimensions and the tunnel path.

- The simulated local and natural conditions of tunnel path and limiting assumptions such as plate strain (2D analysis) are absent in 3D modeling.

- The model size is opted in such a way that the boundary distance of the model from the tunnel wall is greater than 5 times the radius, but this distance may be greater in conditions of low and deep rock mass, which should be controlled when modeling this issue. Thus, the zero displacement conditions at the model boundary can be considered [29].

The initial stresses are gravitational and depend on overburden height. In the access tunnel modeling, the overburden height is assumed to be approximately 30 m.

- Zoning the model.

- Assigning soil properties to the access tunnel environment, including sand or clay sediments (applying Mohr-Coulomb to simulate the structure failure, i.e. study of elastoplastic material behavior).

- Applying boundary conditions and closing model boundaries.

- Solving the model in initial state until it reaches the initial equilibrium (that is, before the tunnel excavation, the model must first be run to reach the initial equilibrium).

- Plotting the unbalancing forces diagram and the history of displacements at different points in the tunnel to ensure that the modeling process is correct.

- Canceling displacements and velocity values in the model to simulate the conditions prior to any drilling, collapse or manipulation in the initial environment (displacement values before excavation of the tunnel is zero).

- Umbrella arch installation (two rows with a distance of 0.25 m and a distance of 0.5 m for pipes from each other).

- Excavating the upper part of the tunnel at 0.5 m steps, applying and maintaining the model after each drilling step to reach 30 m in the three-dimensional model.

- Simultaneous excavation of the upper and lower sections in 0.5 m steps, applying support and solving the model after

each drilling step (at each stage of the model the distance between the upper and lower part of the tunnel is 30 m).

-Extracting the results and comparing them with each other and presenting corresponding plots.

The model created in FLAC3D software has dimensions of 60*55*74 meters (length, width and height) and 88,000 zones. The tunnel is a horseshoe with an excavation width and height of 7.05 meters. A schematic of the modeled block is shown in Fig. 6. The specifications of the support system including the buried triangular fixed lattice, shotcrete and nailing are presented in Tables 4, 5 and 6, respectively.

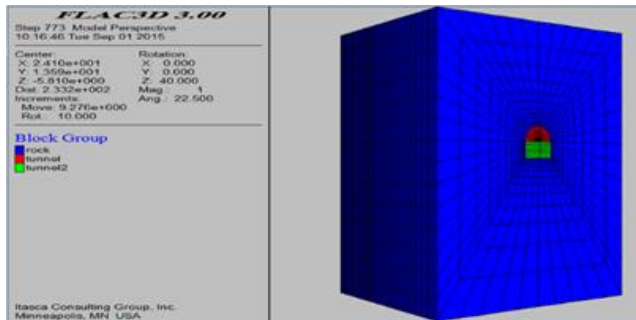


Fig. 6: Schematic of the modeled block

Table 4: Specifications of modeled shotcrete

E (GPa)	γ (kg/m3)	σ_c (MPa)	ν
20	2200	21	0.15

Table 5: Specifications of modeled fixed lattice

E (GPa)	σ_c (MPa)	σ_t (MPa)	ν	Distance (m)
200	240	240	0.3	0.5

Table 6: Specifications of auto excavator nailing

E (GPa)	Diameter (mm)	Length (m)	Tension (KN)capacity
200	32	3	0.16

In Tables 3 through 6, γ is the density of shotcrete, E represents the elastic modulus, σ_c is the uniaxial compressive strength, σ_t is the tensile strength, and ν is the Poisson ratio of support systems.

The unbalancing forces obtained from the three-dimensional model are shown in Fig. 7. As can be seen in this figure, the maximum unbalancing force in the model is 3.352×10^5 N, which is reduced to 6.489×100 N after applying the model. This ratio is less than 0.001 and it can be inferred that the model is in equilibrium. The values for the unbalancing forces are shown on the left side.

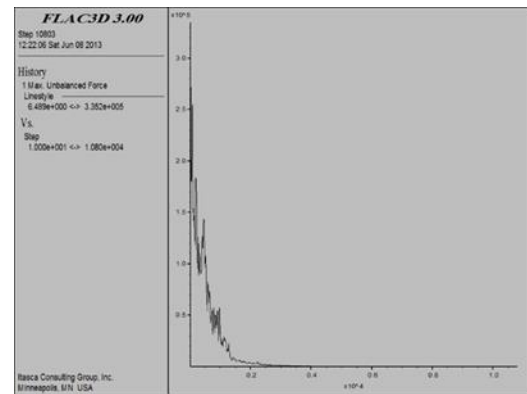


Fig. 7: Unbalancing force-cycle of initial conditions

3.1 Modeling without a pre-support system

In the absence of a pre-support system after drilling each step with a magnitude of half a meter, the corresponding support system is implemented. Figure 8 depicts the support system applied during the drilling of the upper part of the access tunnel (including support with a fixed lattice of 0.5 m and a 30 cm thick shotcrete).

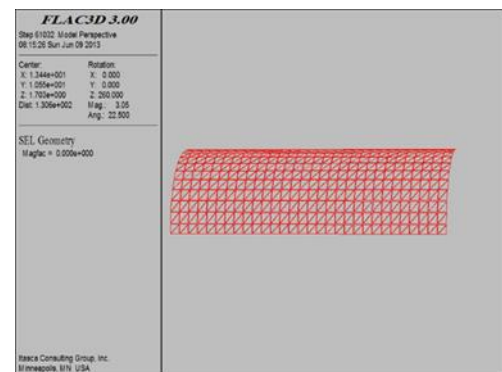


Fig. 8: Modeling of first stage drilling system support

Figure 9 shows the modeling of the support system (including upper and lower retention 0.5 m apart from each other with fixed lattice, 30 cm thick shotcrete and 60 cm bottom concrete) in the simultaneous excavation of the upper and lower parts.

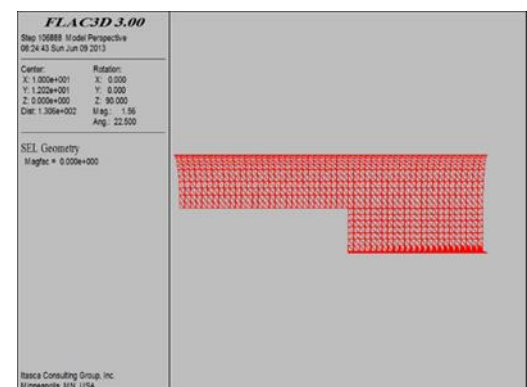


Fig. 9: Modeling of the support system at simultaneous excavation of the upper and lower parts

An overview of the plastic zone created around the tunnel is shown in Fig. 10. As can be seen, the excessive plastic zone around the tunnel increases the dead load exerted on the tunnel reinforcement system. Depending on the geological conditions of the project site, this can cause instability in the tunnel face, as well as deformation of the support system.

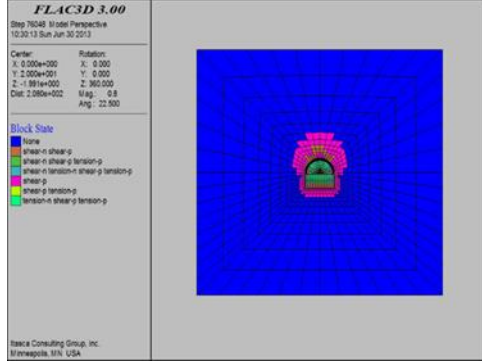


Fig. 10: Expanding the plastic zone around the tunnel (without the use of a pre-support system)

3.2. Modeling of pre-support in the forepoling arc umbrella

As mentioned above, it is necessary to employ an umbrella arch pre-support method for realization safety and stability of the access tunnel. Therefore, in this modeling, the umbrella arch pre-support method with the initial fixed lattice and shotcrete system are used to provide support. The umbrella pipes are simulated by the pile element and the initial support system (fixed lattice and shotcrete) by the shell element.

The used forepoling pipes are 90 mm in diameter, 9 m in length and 2.5 m in overlap length, and these pipes are arranged in two-row staggered arrangement with a 0.5 m pitch. The distance between the two rows of pipes is 0.25 meters. The initial support system also includes a 30cm shotcrete and a fixed lattice.

Increasing the length of the overlapping pipes decreases the amount of subsidence and displacement, but economically, this length must be selected correctly and in accordance with the project requirements. The minimum overlap length is obtained from the correlation of Eq. 1 [1].

$$L_{ov} = D \left(\frac{\sin \left(45 - \frac{\varphi}{2} \right)}{\sin \left(45 + \frac{\varphi}{2} - \theta \right)} \right) \quad (1)$$

Where L_{ov} is the overlap length, θ denotes the angle of the pipe relative to the horizon, D represents the height of the drilling section, and φ is the angle of internal friction of the soil. Overlap length between two adjacent umbrellas in front of the tunnel is controlled by the ground behavior behind the tunnel face. In recent years numerous studies are carried out on strengthening the tunnel face using longitudinal pipes in small-scale laboratory tests via in situ and numerical modeling. The results revealed that the overlap length should

not be less than 0.3-0.4 of the equivalent tunnel diameter[30, 31].

As calculated by Eq. 1, the overlap length is 2.7 m which is considered to be 2.5 m in the modeling based on the drilling step of half a meter.

Details of the components of the support system, including shotcrete, fixed lattice and Auto excavator nailing are given in Tables 4-6. The slurry parameters used for injection into the forepoling pipes are given in Table 7.

Table. 7: Slurry parameters in uniaxial test

Compression strength (MPa)	Young Modulus (MPa)	Compression strength (MPa)	Young Modulus (MPa)	
28 days		7 days		Grout
11.42	2569	8.26	911	w/c=0.5

In the modeling of the umbrella arch, since drilling commences almost a week after the installation of the umbrella, 7-day parameters of slurry are considered.

Implementation and modeling are performed in such a way that before drilling the face, the first two rows of umbrella arches with the above specifications are executed and then drilling and installation of the initial tunnel support system with 0.5 m steps are performed. By continuing drilling and installing the primary cover, the second series of umbrella arches is installed at 6.5 meters to provide an overlap length of 2.5 meters. Figures 11 and 12 illustrate the position and arrangement of the umbrella arch around the tunnel.

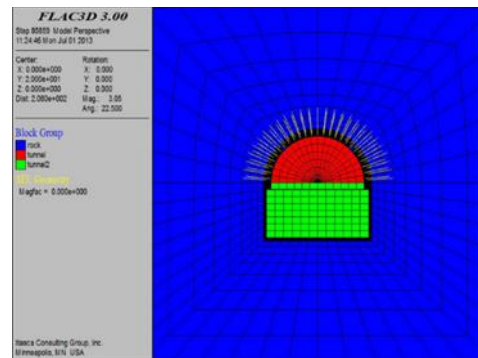


Fig. 11: Arrangement of the umbrella arch around the tunnel

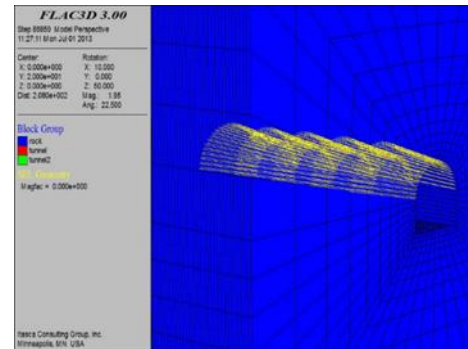


Fig. 12: modeled umbrella arches with two rows

3.3. Examine the necessity of using pre-support system

Since the pre-support method is costly and time-consuming and should be identified by the consulting engineer in accordance with the project's needs, first the use of an umbrella arch should be considered. The plastic zone around the tunnel in the case of forepoling is shown in Figure 13. As can be seen in the case of forepoling, a significant decrease occurred in the plastic zone around the tunnel and thus the tunnel was more stable. Comparison of displacements also shows that the amount of displacement in the crown and face of the tunnel are reduced by 30% to 40% in the case of forepoling. Displacement vectors of the tunnel indicate that tunnel floor displacement and heave potential are high. For this reason, it is necessary to install a support system for the floor. Installation of floor support, in addition to resisting against floor heave, will also resist against a high percentage of compressive loads from the tunnel flanks.

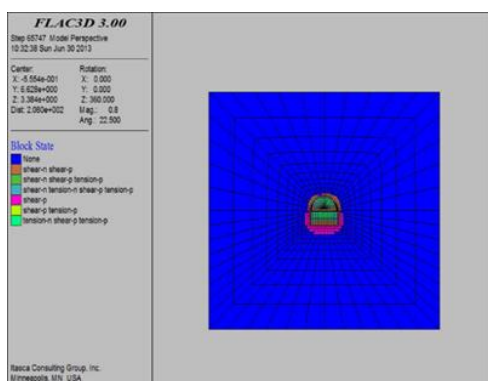


Fig. 13: Expansion of the plastic zone around the tunnel in the case of forepoling

3.4. Structural behavior of the umbrella arch

As mentioned in the previous section, the pile element is used to simulate the umbrella arch. Different analytical models are presented on how umbrella arch pipes function. According to the models presented by the researchers, the modeling performed and the study of the deformation, axial force and bending in this paper, the pipes are considered as a simply supported beam, as the other end is freely located in the soil. In fact, placing the pipe inside the ground and injecting it into the pipe leads to the injection material leaving the other end and can act as a support at the other end of the pipe, but it is still possible to move the pipe with the soil. The distribution of forces is shown in Figure 14 for the model. S is an unsupported end, and point B is a support indicating the initial support system.

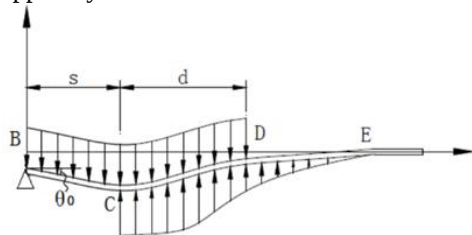


Fig. 14: Force distribution for the model

As the drilling process continues, the deformation and force on the pipes change. Usually the maximum amount occurs near the face. When the length of the pipe in the ground is half or less, the maximum deformation occurs at 0.5 to 0.75 of pipe length.

The location of the maximum force (perpendicularly) is within one meter of the face, and approaches the end of the pipe and reaches the maximum overlap length in the 0.5 to 0.75 of the pipe length. The distribution of moment resulting from this force also follows this trend. It should be noted that the positive values indicate compression and negative values show tension. Figure 15 shows the force distribution in the vertical direction (the y direction in the local coordinates of the pipes is equal to the z direction of the main coordinates). At the highest pipe, the vertical force has the maximum value. Figure 16 also shows the moment due to this force.

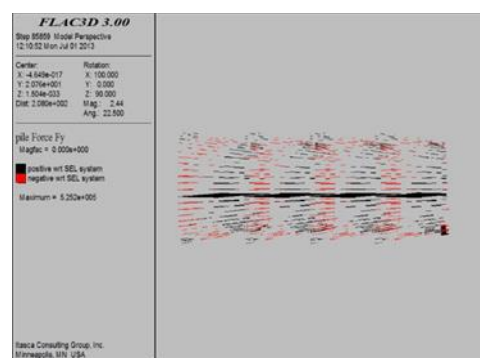


Fig. 15: Axial force distribution in the umbrella arch pipes in the y direction

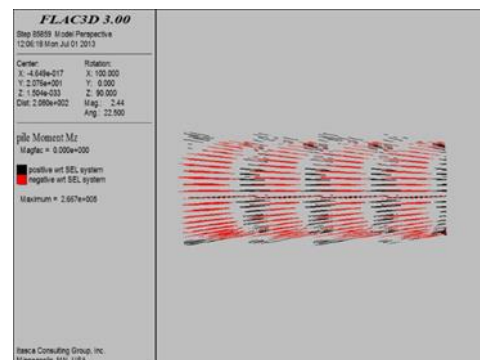


Fig. 16: Bending moment distribution of umbrella arch pipes along z direction

It should be noted that the effective forces acting on the pipe are shear in both vertical and horizontal directions and thus vertical and lateral stresses are applied, which force moment and bend the pipes. The axial force applied to the pipe is not significant. In Figures 9 to 11 the force is in Newton. The vertical force applied to the upper row pipes in the 7-meter arch drilling is shown in Fig. 17. Given its shape, it is clear that the highest pipe has the maximum vertical force. In vertical loading of pipes 1 to 9 (half of the upper tunnel arch) the maximum compressive force occurs in the first third of the pipe length. However, from pipes 7 to 12, the maximum occurs in the first quarter of the pipe length. In fact, by

changing the arch of the tunnel, the location of maximum force also changes.

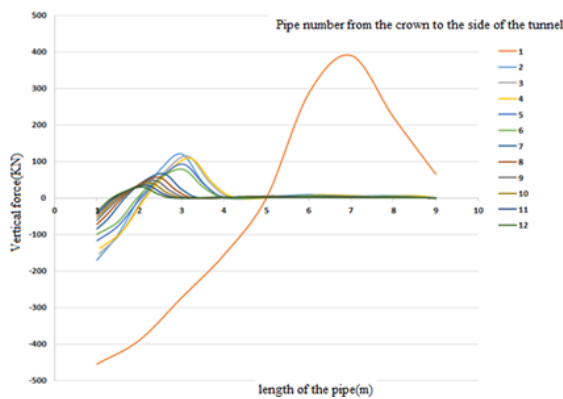


Fig. 17: The force applied to the pipes in a vertical direction by drilling a 7-meter arch

In order to investigate the failure of the umbrella arch, first its resistance is calculated by means of a weighted average combination of steel and pipe. With a 28-day slurry with water-cement ratio of 0.5 and a yield strength of 240 MPa for steel, the umbrella arch strength was 170 MPa. The maximum amount of force, moment and stress applied to the pipes during the drilling of arch and springing are in accordance with Table 8. σ_y and σ_z are due to bending and σ_x is due to axial force. Although these values did not occur simultaneously at one cross-section, the total stress values are summed to be 123.4 MPa for comparison. It can be seen that the sum of stress values is lower than the resistance of pipes. Therefore, pipes do not reach yield point and no failure occurs.

Table 8: Maximum force, moment and stress in pipes

N_x (MN)	N_y (MN)	N_z (MN)	M_y (MN-m)	M_z (MN-m)	σ_x (MPa)	σ_y (MPa)	σ_z (MPa)
0.418	0.525	0.155	0.325	0.267	3.34	42.05	78.04

3.5. Replacement Forepoling umbrella system by equal -legs angles.

Due to some operational problems, an angle may be used instead of Forepoling pipes. Forepoling pipes were equated with equal angles of 10 * 100 (thickness 10 mm and wing width 100 mm) and 80 * 8 (thickness 8 mm and wing width 80 mm) using SECTION BUILDER software. The results are given below.

3.5.1. Stress distribution in Forepoling pipes

The contour of normal and shear stresses for Forepoling pipes (including steel pipe and grout injected into it) under axial load and certain bending moment is shown in Figures 18 and 19. The study of this figure shows that the range of axial stresses is in the range of -2900 to 2910 and the range of shear stresses is in the range of 0.687 and 19.9 N / mm².

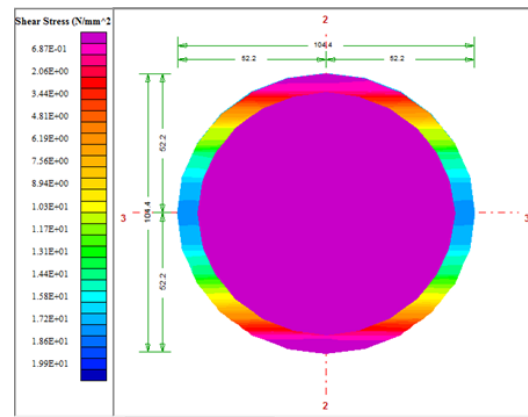


Fig. 18. Shear stress distribution in the Forepoling section

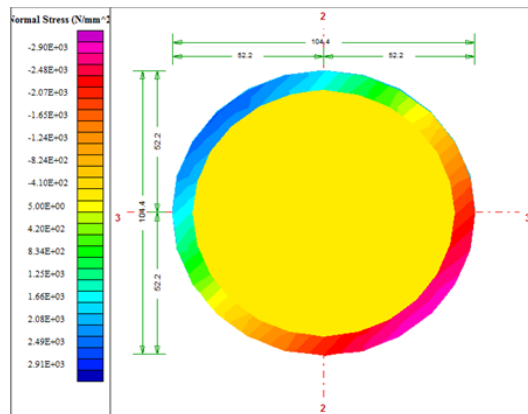


Fig. 19. Axial stress distribution in the Forepoling section

Also, according to Figures 17 and 18, it can be seen that the values of normal and shear stresses in the injection slurry are less than that of the steel pipe.

3.5.2. Equivalence of Forepoling pipe with single and double legs sections

Table 9 summarizes the geometric characteristics and Table 10 shows the axial and shear stress distributions of single and double sections (including foreplay, 80 * 8 angle and 100 * 10 angle). According to Tables 9 and 10, it can be seen that the equivalent cross section for Forepoling to have the area and moment of inertia in the same range, is the double angle sections of 100 * 10 (both T- Section and box arrangement). These two sections have more area and moment of inertia than the furling section. Comparing the characteristics of the sections as well as the distribution of axial and shear stresses with the furling section, it is observed that none of the single sections of 80 * 8, 10 * 100 and double 80 * 80 (both T-Section and box arrangement) correspond to the area, and moment Inertia and stress distribution are not required as they are not able to withstand the loads on the propulsion system. Therefore, for stabilization of the access tunnel, only 100 x 10 10 corner sections (T-Section or box arrangement) should be used as a substitute for Forepoling. Also, the comparison of two T- Section and box arrangements of double wing angle sections equal to 80 * 8 and 100 * 10 shows that in boxy mode, the amount of normal and shear stresses distributed in the cross section is less, so the double legs angle cross section is equal to 10 * 100 with boxy

arrangement has more resistance than spray arrangement. However, due to the easier implementation of the T-Section arrangement, in the access tunnel, a double cross section of the angel equal to 100 x 10 is used with the T-Section arrangement.

Table. 9: Specifications of Forepoling and double corner legs of equal wings

Section	Area(mm ²)	Moment of inertia-The main axis(mm ⁴)
Forepoling	2198.6	2.61×10^8
Angle 80*8	1216	0.737×10^6
Angle 100*10	1900	1.8×10^6
Double Angle (T-Section) 80*8	2432	1.47×10^6
Double Angle (T-Section) 100*10	3800	3.6×10^6
Double Angle (Boxy) 80*8	2432.4	2.18×10^6
Double Angle (Boxy) 100*10	31600	4.44×10^7

Table. 10: Stress distribution in Forepoling pipes and single and double edge sections

Section	Axial tension N/mm ²		shear stress N/mm ²	
	Min	Max	Min	Max
Forepoling	-2900	2910	0.687	19.9
Angle 80*8	-11100	11100	1.62	46.9
Angle 100*10	-5700	5710	1.04	30
Double Angle (T-Section) 80*8	-4410	4320	0.754	21.9
Double Angle (T-Section) 80*8	-2260	2210	0.472	13.7
Double Angle (Boxy) 80*8	-3570	3620	0.746	21.6
Double Angle (Boxy) 100*10	-1840	1860	0.471	13.7

According to Figure 20 and 21, it can be seen that in the shield arrangement, the range of axial stresses is in the range of -2260 to 2210 and the range of shear stresses is in the range of 0.472 and 13.7 N / mm². Also, the study of Figure 22 and 23 shows that in the box arrangement, the range of axial stresses is in the range of -1840 to 1860 and the range of shear stresses is in the range of 0.471 and 13.7 N / mm².

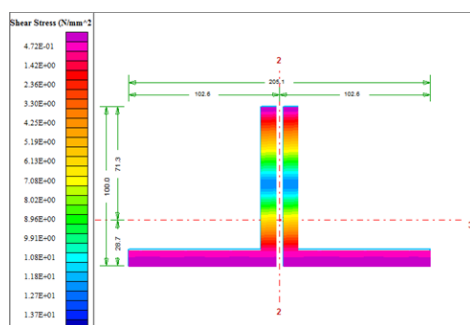


Fig. 20: Normal stress distribution at wing angle equal to 10 * 100 (T-Section arrangement)

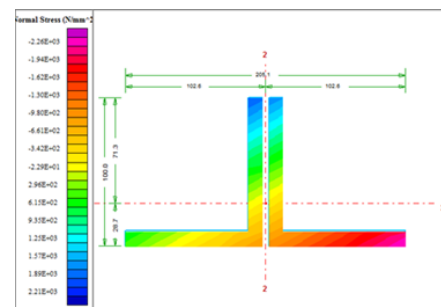


Fig. 21: Shear stress distribution at wing angle equal to 10 * 100 (T-Section arrangement)

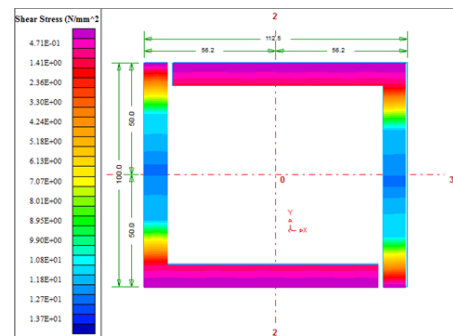


Fig. 22: shear stress distribution at wing angle equal to 10 * 100 (box arrangement)

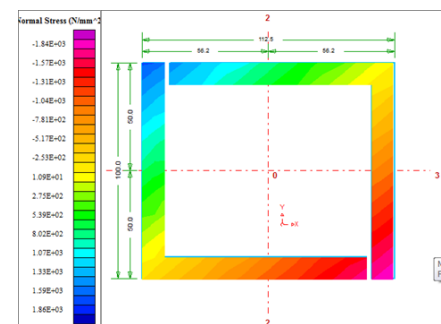


Fig. 23: Axial stress distribution at wing angle equal to 10 * 100 (box arrangement)

3.6. Investigation of section interaction curve

The P-M curve of the holding capacity of each of these sections is shown in Figure 24. As it is clear, only double angled sections of 10 x 100 (both spray and boxing arrangements) are able to withstand more loads than the foreplay section, so only these two sections can replace the foreplay system.

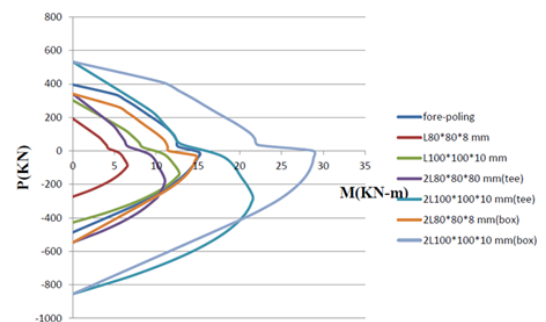


Fig. 24: Axial force-bending moment curve (P-M) of furpoling and angle sections

3.7. Verification of results

For behavioral monitoring of the access tunnel during excavation, the use of 5-point convergence monitoring operation is suggested. The position of the converging pins in the cross section includes the installation of 1 pin in the tunnel crown and 4 pins on both sides of the wall (Fig. 25) and is repeated every 20 meters along the tunnel. It is necessary to elaborate that the installation operations are executed and measured simultaneously with drilling and with the shortest distance from the face, so that the smallest changes can be recorded. Since the Top & Bench drilling method is used to implement the access tunnel and due to the fact that the upper half is drilled first and deformation occurs in this area, three converging pins are installed in this area and they are read in accordance with the scheduler. Next, when the lower half of the tunnel is drilled, two more pins are installed in the lower half to control the overall displacement. There are two sections of 0+134 km and 0+154 km of the fore poling length of access tunnel. The convergence trend of these sections is illustrated in Figures 26 and 27, along with the reasons for the changes in the graphs. The convergence results of the L1-R1 horizontal edges of the two stations in the mentioned range show that the convergence values of these two stations are 55mm and 54mm. In addition, the horizontal displacement obtained by numerical modeling for each point of the spring line is approximately 29 mm (Fig. 28 and 29) resulting in 58 mm convergence. Therefore, the results obtained from numerical modeling showed good agreement with the results obtained from periodic instrumentation readings [32].

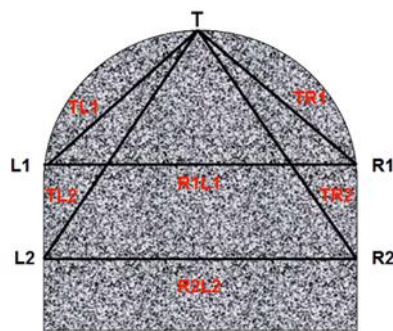


Fig. 25: Measuring strands in five-point convergence monitoring

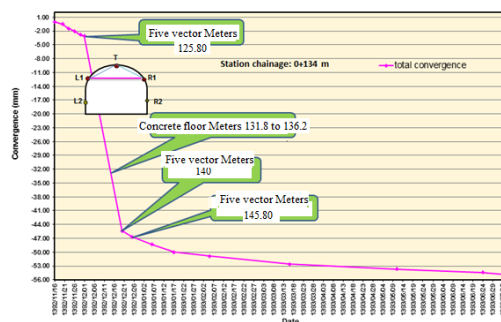


Fig. 26: Changes trend of convergence monitoring edge at 0+134 kilometerage section

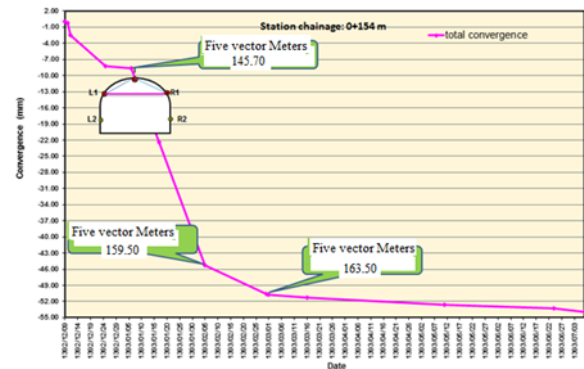


Fig. 27: Changes trend of convergence monitoring edge at 0+154 kilometerage sectional

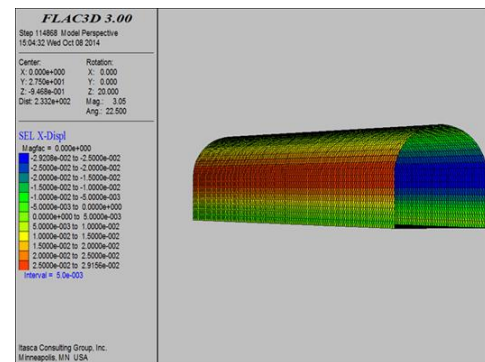


Fig. 28: The displacements occurred around the tunnel after implementing the pre-support system

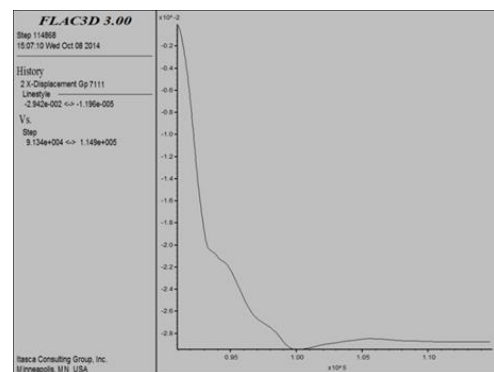


Fig. 29: Displacement history of spring line

Sakurai [32], proposed a method for evaluating the stability of underground structures considering the critical strain defined by uniaxial compressive tests on different rocks. This method is often used as the standard method for calculating the convergence (relative displacement of tunnel walls) obtained during tunnel monitoring. According to the graph presented by Sakurai, the underground space has no stability problem when the strains are less than the alert-level 1. The underground structure will encounter serious trouble if the strains created in the structure go to alert-level 3. Sakurai proposed alert-level 2 as the basis for tunnel design, and alert-level 1 and 3 are permitted as upper and lower limits for tunnel stability based on permitted strain. The critical displacement can be obtained by determining the critical strain and using Eq. (2).

$$\varepsilon_c = \frac{u_c}{a} \quad (2)$$

Where a is the radius of tunnel and u_c is the permitted displacement. Based on the documentation and results of the experiments carried out at the project site and the materials of the tunnel area, the permitted displacement values for the Sakurai alert-levels are calculated (Table 10) and compared with the results of the monitoring operations. This yields a clear view of the stability of the tunnel. According to the Sakurai criterion, the tunnel displacement value is close to the alert-level 2 (Fig. 30), but given the slope of the convergence diagrams approaching horizontal, it can be assured that the static stability of the support system is met.

Table. 10: Permitted Displacement and Sakurai Alert-Levels

Material types	Critical Strain (%)			Allowable displacement based on hazard alert level 1 (cm)	Allowable displacement based on hazard alert level 2 (cm)	Allowable displacement based on hazard alert level 3 (cm)
	Hazard Alert Level 1	Hazard Alert Level 2	Hazard Alert Level 3			
SC	0.007	0.016	0.038	2.576	6.039	14.157
CL	0.009	0.020	0.048	3.265	7.653	17.940

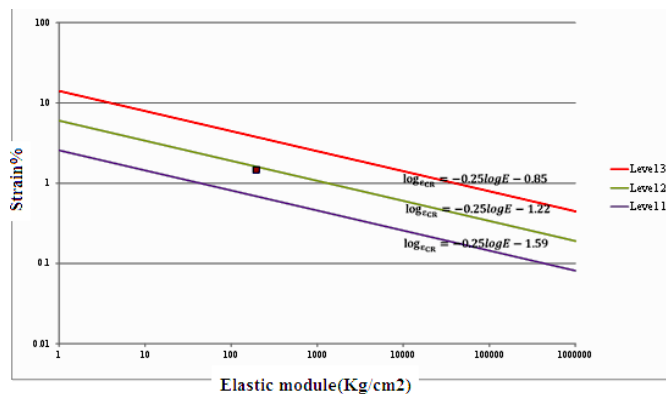


Fig. 30: Stability assessment of access tunnel by Sakurai Alert-Levels

Since tunnel lining is considered as permanent support, the initial support system, including lattice and shotcrete with a safety factor of 1.2 is designed by SECTION BUILDER software (Figure 31). Fig. 32 shows the interaction curve of the initial support system in the forepoling operation at different sections of the tunnel. As it is obvious, all the mentioned conditions are within the permitted range of axial force, bending moment and shear force. It should be noted that these diagrams are for both cases of upper and lower drilling, and in all cases the points are within the range of coverage factor of 1.2.

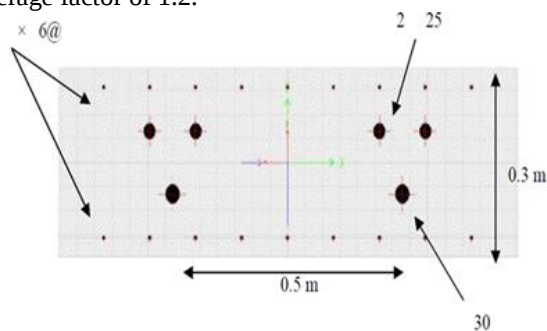


Fig. 31: Initial support system consisting of two layers of wire meshes with a triangular lattice buried in 30 cm shotcrete

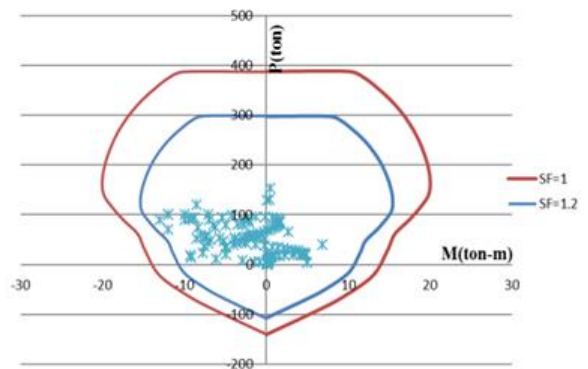


Fig. 32: Bending force-moment curve of the initial support system

4. Conclusion

Based on the modeling performed, the visual observation, the results of the instrumentation, and the analyses performed on the modeling, the following findings can be presented.

Despite the use of step drilling in the access tunnel construction process, analyses using numerical modeling and visual observation during drilling indicate that tunnel instability may occur. Therefore, it is necessary to use an umbrella arch to sustain tunnel stability.

In the access tunnel and when using the umbrella arch (forepoling), a significant mitigation occurred in the plastic zone around the tunnel, which resulted in a more stable tunnel. Comparison of displacements also showed that the amount of displacement in the crown and face of the tunnel is reduced by 30 to 40% in the case of forepoling.

Increasing the length of pipes has a positive effect on tunnel stability. Increasing the length by greater than 1.5 times the diameter of the tunnel has no significant effect on tunnel stability, due to moving away from the failure zone. Values less than the diameter of the tunnel do not fully cover the failure area. The desired value is 1 to 1.5 times the tunnel diameter.

Increasing the distance between the pipes leads to a decrease in stability, but taking into account the economic constraints and the fact that the operational problems of increasing the number of pipes are significant in terms of time and cost, it is possible to use fewer pipes. It may be reasonable to use the least number of pipes with higher diameter.

The structural behavior of the umbrella arch pipes is similar to a simply supported beam, as the other end is freely located in the soil. The displacement, force, and moment of the pipes change with the continuation of the drilling process.

Due to the reduced support effects of slurry at the end of the pipe and the reduction of the pipe length in the turbulent zone in front of the face, it is necessary to provide a second series of pipes and proper overlap.

Reference:

- [1] Muraki Y (1997). The Umbrella Method in Tunnelling. M.S, Civil Engineering, Massachusetts Institute of Technology.
- [2] Aksoy CO, Onargan T (2010). The role of umbrella arch and face bolt as deformation preventing support system in preventing building damages. *Tunnelling and Underground Space Technology*, vol. 25, pp. 553-559.
- [3] Ohcak I (2008). Control of surface settlements with umbrella arch method in second stage excavations of Istanbul Metro. *Tunnelling and Underground Space Technology*, vol. 23, pp. 674–681.
- [4] Peila D (2005). Ground Reinforcing for Tunnelling The Example of Steel Pipe Umbrella. ITA/AITES – Train-ing Course tunnel engineering.
- [5] Hefny AM, Zhao j, Ranjith P, Tan WL (2000). Numerical analysis for umbrella arch method in shallow large scale excavation in weak rock.
- [6] Carrieri G, Grasso PG, Mahtab A, Pelizza S (1991). Ten years of experience in the use of umbrella-arch for tunnelling. In: *Proceedings of the congress on soil and rock improvement in underground works*, Milano 1:99–111.
- [7] Carrieri G, Fiorotto R, Grasso P, Pelizza S (2002). Twenty years of experience in the use of the umbrella-arch method of support for tunnelling. In: *Proceedings of 4th international workshop on Micropiles*, Venice, Italy.
- [8] Lunardi P (2000). The design and construction of tunnels using the approach based on the analysis of con-trolled deformation in rocks and soils. In: *Tunnels and tunneling international special supplement, ADECO-RS approach*, May 2000, pp 3–30.
- [9] Yoo C, Shin HK (2003). Deformation behavior of tunnel face reinforced with longitudinal pipes-laboratory and numerical investigation. *Tunn Undergr Space Technol* 18:303–319.
- [10] Kim SH, Baek SH, Moon HK (2005). A study on the reinforcement effect of umbrella arch method and prediction of tunnel crown and surface settlement. In: *Proceedings of the international world tunnel congress and the 31st ITA*, Istanbul, pp 245–251
- [11] Barisone G, Pigorini B, Pelizza S (1982). Umbrella arch method for tunnelling in difficult conditions-analysis of Italian cases. In: *Proceedings of the fourth congress international association of engineering geology*, New Delhi 4:15–27
- [12] Gangale G, Corona G, Pelizza S (1992). The Messina-Palermo motorway: complex rock masses and tunnel-ing problems. In: *Proceedings of the international congress towards new worlds in tunneling*, Acapulco, vol 2, pp 873–880
- [13] Murata M, Okazawa T, Harunaka K, Tamai A (1996). Shallow twin tunnel for six lanes beneath densely residential area. *North Am Tunn* 96:371–380
- [14] Shin HS, Han KC, Sunwoo C, Choi SO, Choi YK (1999). Collapse of a tunnel in weak rock and the optimal design of the support system. *Proceedings of the Ninth International Congress on Rock Mechanics*, Paris
- [15] Haruyama K, Teramoto S, Harada H, Mori M (2001). Construction of urban expressway tunnel with special large cross-section by NATM Metropolitan Inter-city Highway (ken-O-Do) Ome Tunnel. In: *Proceedings of international symposium, Modern Tunnelling Science and Technology* 693–698.
- [16] Sekimoto H, Kameoka Y, Takehara H (2001). Countermeasures for surface settlement in constructing shal-low overburden urban tunnels penetrated through active fault. In: *Proceedings of modern tunneling sci-ence and technology*, pp 711–716.
- [17] Aksoy CO, Onargan T (2010). The role of umbrella arch and face bolt as deformation preventing support system in preventing building damages. *Tunn Undergr Space Technol* 25(5):553–559
- [18] Volkmann GM, Schubert W (2006). A Contribution to the Design of Tunnels Supported by a Pipe Roof. The 41st U.S. Rock Mechanics Symp.
- [19] Volkmann GM, Schubert W (2008). Tender document specifications for pipe umbrella installation methods. The World Tunnel Congress - Underground Facilities for Better Environment and Safety, india.
- [20] Volkmann GM, Schubert W (2007). Geotechnical Model for Pipe Roof Supports in Tunneling. The 33rd ITA-AITES World Tunneling Congress.
- [21] Volkmann GM, Krenn F (2005). Back-Calculated Interacting Loads on Pipes of Pipe Umbrella Support Systems.
- [22] Shin JH, Choi YK., Kwon OY, Lee SD (2008). Model testing for pipe-reinforced tunnel heading in a granular soil. *Tunnelling and Underground Space Technology*, vol. 23, pp. 241-250.
- [23] Doi Y, Otani T, Shingi M (2009) The Optimum Distance of Roof Umbrella Method for Soft Ground by Us-ing PFC. *Analysis of Discontinuous Deformation: New Developments and Applications*, pp. 461-468.
- [24] Shinji M, Sato T, Doi Y, Ootani T (2010). The Optimum Distance of Roof Umbrella by Using the Distinct Element Method.
- [25] Juneja A, Hegde A, Lee FH, Yeo CH (2010). Centrifuge modelling of tunnel face reinforcement using fore-poling. *Tunnelling and Underground Space Technology*, vol. 25, pp. 377-381.
- [26] A. Asadollahi, M. a Mossavi, Mohammad Hossein Khosravi (2018). Numerical Investigation of the Influence of Cement Injection on Ground Settlement due to Excavating in Soft Soils (Case Study: Station E on Line 2 of Karaj Metro)
- [27] Placie, M. Merciful, AS. (2016). Evaluation of the effect of forging pipe lengths on tunnel shrinkage, 9th National Congress of Civil Engineering, Ferdowsi University of Mashhad, Iran
- [28] Hashemi Esfahanian, M. Kamali Dehkordi, P. (2015). Investigation of Tunnel Maintenance Types by Umbilical Arch Method, Tenth International Congress of Civil Engineering, Tabriz, Tabriz University, Faculty of Civil Engineering.
- [29] A. Laki Rouhani and S. Jolfaei (2018), Complete three-dimensional modeling of tunnel excavation and cover installation with emphasis on step drilling, comparison of three-dimensional and two-dimensional analysis, *Amirkabir Journal of Civil Engineering*, 50(1) (2018) 51-52 DOI: 10.22060/ceej.2016.708

[30] Maleknia, A. (2010), "forepoling in tunnel excavation", Report of Behrooz Construction Company, Tehran,

[31] Sdegi A, Sahraki N. (2016). " The Impact of Purpleling Method on the Stability of the Tunnel Working Front (Case study: C2 metro station, Karaj)", Ninth National Congress of Civil Engineering., Ferdowsi University of Mashhad, Iran

[32] Sakurai S, Kawashima I, Otani T (1995). A criterion for assessing the stability of tunnels. ISRM Intern. Symp, vol.2, pp. 969-973.



This article is an open-access article distributed under the terms and conditions of the Creative Commons Attribution (CC-BY) license.