

Numerical modeling of an umbrella arch as a pre-support system in difficult geological conditions: a case study

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Abstract In order to prevent the collapse of ceilings and walls of large tunnels, especially in difficult geological conditions, either a sequential excavation method (SEM) or ground reinforcing method, or a combination of both, can be utilized. The first part of the adit tunnel in northwestern Iran is being drilled in alluvium material with very weak geotechnical parameters. Despite applying an SEM in constructing this tunnel, analyzing the numerical modeling done using FLAC3D, as well as observations during drilling, indicate tunnel instability. To increase operational safety and to prevent collapse, a pre-support system was designed and implemented. The results of the numerical modeling accompanied by monitoring during operation, as well as the results of instrumentation, indicate the efficacy of this method for preventing collapse in alluvium material along the tunnel route. Modeling the behavior of the umbrella arch shows that the location of the maximum compressive force will change with a change in the tunnel arc location. Moreover, displacement, force and moment exerted on the pipes will change during drilling steps according to a certain pattern.

Keywords SEM · Alluvium material · Numerical modeling · Pre-support system · Monitoring · Instrumentation

Introduction

While drilling tunnels in loose ground with low stability and especially in tunnels with relatively large cross sections, drilling leads to risks such as the possibility of collapse, environmental hazards, and unauthorized surface subsidence. Two suggested solutions for such situations are the application of ground reinforcing methods or sequential drilling, or a combination of both, depending on the conditions.

In some cases, the tunnel face is partitioned to have temporary drifts in order to promote face stability and to reduce surface deformations and settlements; this method is referred to as the sequential excavation method (SEM) (FHWA 2009).

Another suitable method is applying special reinforcement techniques such as the umbrella arch method (UAM) that is commonly used to ensure tunnel stability (Ocak 2008). For this purpose, longitudinal bars are installed ahead of the tunnel from the periphery of the face, typically over the upper third or quarter of the excavated profile (Fig. 1) (Hoek 2004). This method has been widely studied (Carrieri et al. 1991, 2002; Lunardi 2000; Yoo and Shin 2003; Kim et al. 2005). Moreover, many field cases of excavated tunnels that used the UAM have been reported (Barisone et al. 1982; Gangale et al. 1992; Murata et al. 1996; Shin et al. 1999; Haruyama et al. 2001; Sekimoto et al. 2001; Kamata and Mashimo 2003; Shin et al. 2008; Aksoy and Onargan 2010).

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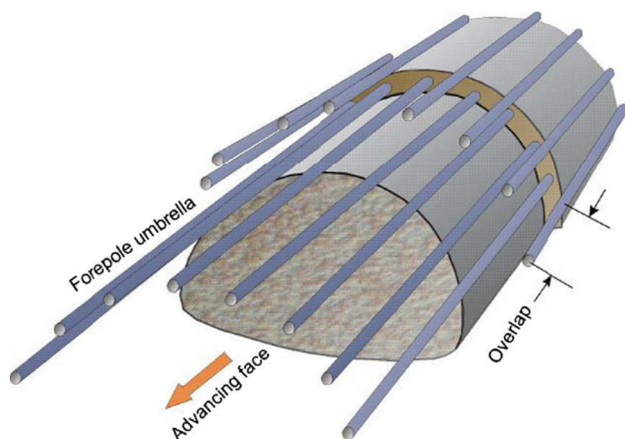


Fig. 1 Application of the umbrella arch method (Hoek 2004)

As design of underground excavations becomes larger and more complex, numerical analyses are required to address difficult ground conditions under which pre-support reinforcement may be required (Volkman and Schubert 2007). 2-D and 3-D numerical analyses have been used in the design of umbrella arch systems, and design parameters of forepole elements related to site-specific considerations have been determined (Oke et al. 2014).

Pre-support methods such as the UAM, jet grouting, mechanical pre-cutting and sub-horizontal fiberglass face reinforcement are widely used in difficult conditions and large scale excavations. In fact, pre-support methods include methods that insert structural elements into the rock mass ahead the tunnel face with the purpose of creating a pre-support before the excavation (Peila and Pelizza 2005).

Application of the UAM reduces the area of influence of a tunnel excavation and decreases the impact of the tunnel on surface and subsurface structures. The umbrella pipes compensate for the pressure applied to the tunnel ceiling and, moreover, it is possible to inject support material into the periphery of the pipe and into cavities in the foundation as they are perforated. Another important impact of tunnel excavation is that ground settlement occurs after the face movement. In order to prevent the face from moving into the tunnel, different lengths of face bolts are applied at different intervals depending on the face mass' condition and cross sectional area. Therefore, as the stability of the face increases, movement of both faces is reduced and, consequently, ground settlement is also reduced (Aksoy and Onargan 2010).

Certain reinforcement systems cannot be literally distinguished. Thus, the pipe-arch reinforcement process should be separated from other systems. The pipe arch process is one of the safest and most efficient ways of constructing large-diameter tunnels in poor geologic conditions. The pipes can be applied on the ground by two different pre-drilling and case-drilling methods. Both

Table 1 Common variation of umbrella arch dimensions

Characteristics	Amount
Pipe size	OD 114 mm, $t = 6$ mm
Pipe length	12–30 m
Overlapping length	1–4 m
Pipe distance	0.33–0.8 m
Water/cement ratio	0.5–2
Pipe inclination	$<15^\circ$

methods possess advantages and disadvantages (Aksoy and Onargan 2010). Designing the pipe-reinforced heading requires determining the length, stiffness, and cross section (thickness and diameter) of the pipes, the installation angle, the distances between pipes, the overlapping length and grouting pressure, etc. Generally, steel pipes with a diameter of about 10–15 cm are used due to constructability. The distance between pipes is governed by grouting pressure. Consequently, in many cases, the determination of pipe length is the main concern in the design of pipe reinforcement with grouting (Shin et al. 2008).

In design practice, the length of a pipe is defined at the tunnel crown. However, appropriate pipe length at the spring line also needs to be evaluated. In practice, however, generally the pipe length at the spring line is the same as that of the crown (Shin et al. 2008). The most common variations of the umbrella arch dimensions are presented in Table 1.

In this study, after studying the results of numerical modeling and ensuring the applicability of this method, the designed umbrella arch was implemented.

Case study

The great tunnel of Zagros is located in northwest Iran. An adit tunnel will be constructed to access the main tunnel. This adit tunnel has been designed to be 1.354 km long and have a downward slope of 10.25 %.

Geological conditions

In the area under study, a borehole 30 m deep was drilled to obtain the required geotechnical data. Borehole specifications are given in Table 2. Results of the lab experiments

Table 2 Borehole specifications

BH no.	Location	Depth		
		Alluvium	Rock	Total
1	Adit tunnel	30	0	30

Table 3 Geotechnical properties of borehole material

Depth	Soil type	γ_d (g/cm ³)	γ_w (g/cm ³)	E (kg/cm ²)	Direct shear		
					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	2.1		0.15	29
15	18	CL	1.98	2.03	35.67		
18	21	SC-SM		2.08			
21	24	SC-SM	2	2.08		0.09	31
24	27	CL		2.08			
27	30	SC-SM		2.08			

Table 4 Results of geotechnical tests on undisturbed samples

Sample 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

on the samples extracted from the borehole are given in Table 3, where γ_d is the dry density, γ_w is the wet density, E is the elastic module, C is the cohesion and Φ° is the friction angle of the medium.

To ensure the quality of the input parameters in the stability analysis, undisturbed samples were selected from the working face and tunnel walls. These samples were then studied and the results of these tests are presented in Table 4, where γ_d is the dry density, C is the cohesion, Φ° is the friction angle, LL is the liquid limit and PI is the plasticity index of the medium.

The most significant features of the material in this section of the tunnel are its incoherence and very low plasticity index. Regarding the above facts and the material parameters, it is clear that the materials are highly sensitive to disturbance, which can lead to their collapse in different parts of the tunnel. The geological profile of the tunnel route is given in Fig. 2.

Excavation and installing support system

Regarding the studies conducted and the weakness of the geological parameters of the alluvial material in the beginning of the adit tunnel route and in order to maintain operational safety, the drilling of the first part is based on the SEM. Therefore, first, the top section of the tunnel is drilled and the primary support system implemented. Characteristics of the support system are presented in Table 5. The top section includes a semi-circle with a radius of 4.05 m and a rectangular area as large as 60 cm under the semi-circular area. When this section was drilled about 15–30 m, the drilling of the lower section (bench) will start. After installing the bench support system, invert concreting as high as 60 cm will start. Also, to neutralize the bending moments and shear forces on the system, two

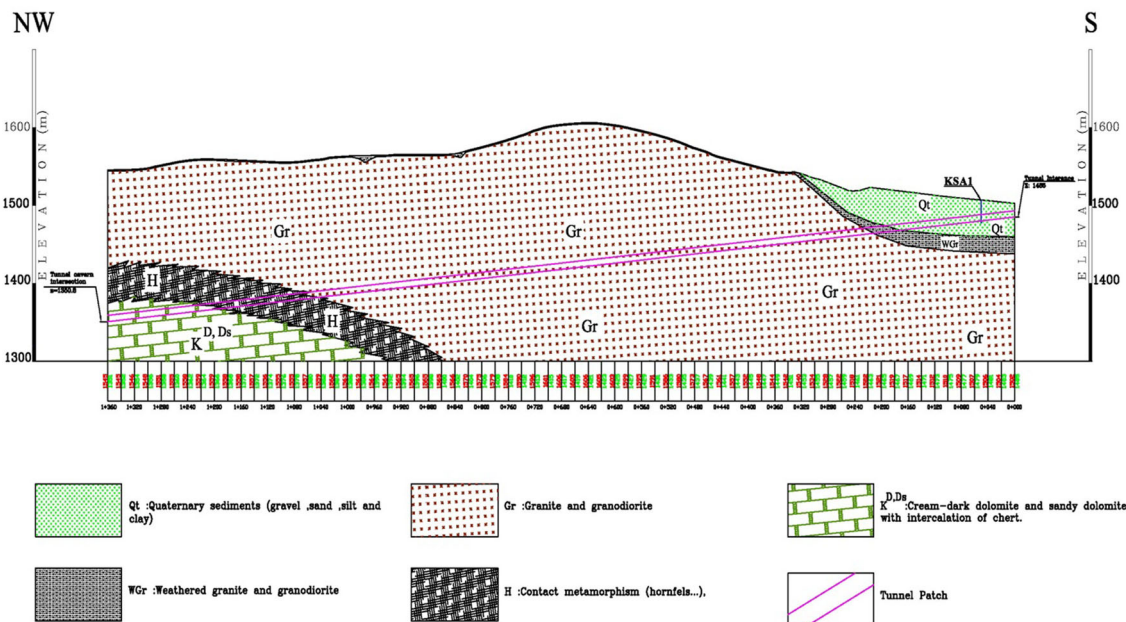


Fig. 2 Geological cross-section along adit tunnel alignment

Table 5 Components of the initial support system

Support component	Number
Steel support	Lattice girder by 0.5 m spacing
Systematic bolt	2 @ 32, L = 3.0 m
Wire mesh	φ6 @ 10 mm × 10 mm
Shotcrete thickness (cm)	30
Excavation round (m)	0.5

rows of nailing (one after installing a lattice ceiling girder, and the other after installing a lattice girder on the bench wall) in 1.3 m parts and 1.5 m away from each other were

installed. The cross-section of the tunnel is shown in Fig. 3 and some images of the top and bench sections of the tunnel are shown in Fig. 4.

Numerical modeling

In this study, FLAC3D was used for the stability analysis. The failure criterion assumed in these analyses was the Mohr–Coulomb criterion. To solve the model, after the geometry of the model was created, the following conditions were defined for the program: initial and boundary conditions, material properties along the tunnel route (SC,

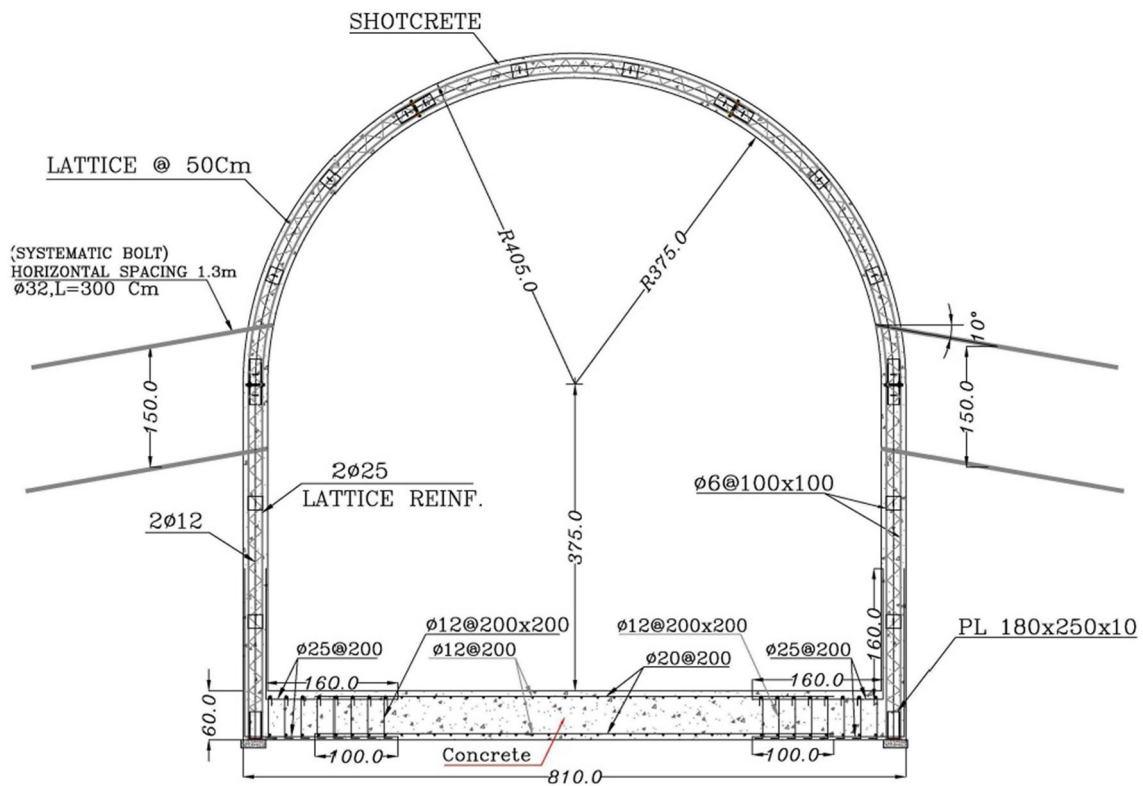


Fig. 3 Adit tunnel cross-section in the alluvium part

Fig. 4 Top and bench excavation and installation of the support system



Table 6 Lattice girder properties

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

Table 7 Shotcrete properties

E (GPa)	γ (kg/m ³)	σ_c (MPa)	ν
20	2,200	21	0.15

Table 8 Nailing properties

E (GPa)	Diameter (mm)	Length (m)	Tensile strength (KN)
200	32	3	0.16

SM, etc; according to Table 2), boundary conditions, and fixing the model boundaries. Then the model was run until it reached the primary balance. The next phase in modeling would be drilling the top section of the tunnel with a 0.5-m step, and implementing the support system and solving the model after each step of the drilling up to the 20-m point. Characteristics of the support system, including the buried triangular lattice girder, shotcrete and nailing, are shown in Tables 6, 7 and 8, respectively.

Finally, the following steps were taken: simultaneous drilling of the top and bench sections of the tunnel with steps as long as 0.5 m, implementing the support system, and running the model after each step of the drilling (in each phase of running the model, the distance between the top and the bench sections is constant and equal to 20 m) (see Fig. 5).

Stability analysis without the pre-support system

A plan of the plastic zone formed around the tunnel is shown in Fig. 6. As can be seen, the vastness of the plastic

zone around the tunnel leads to an increased dead load on the tunnel support system. This could lead to instability in the working face of the tunnel, in regards to the geological conditions of the site.

In practice too, drilling in the above-mentioned conditions led to some collapses in the walls and the ceiling of the tunnel (Fig. 7). This justifies use of the UAM pre-support system as one of the practical methods in drilling this tunnel.

Determination of umbrella arch influence on tunnel stability

Since the UAM imposes high expenses on a project on one hand, and requires a relatively long time to implement on the other, decisions about the method need to be made according to the requirements of the project. Therefore, the necessity of the umbrella arch was examined using numerical modeling as well as studying the conditions of the tunnel construction without applying this system. Regarding the afore-mentioned facts, a pre-support umbrella arch system with the properties shown in Table 9 was designed.

3-D modeling of the tunnel after implementing the umbrella arch system is shown in Fig. 8 (left). Also an image of the finished umbrella arch is given in Fig. 8 (right).

The process of modeling is similar to the phases mentioned in “Numerical modeling” except that, in the latter case, an umbrella arch system with the features shown in Table 9 is installed prior to any drilling.

The plastic zone around the tunnel with an installed umbrella arch pre-support system is shown in Fig. 9. As can be seen, because of the fore-poling, there has been a significant decrease in the plastic zone around the tunnel and, therefore, the tunnel will be more stable. Moreover, comparison between the displacements indicates that

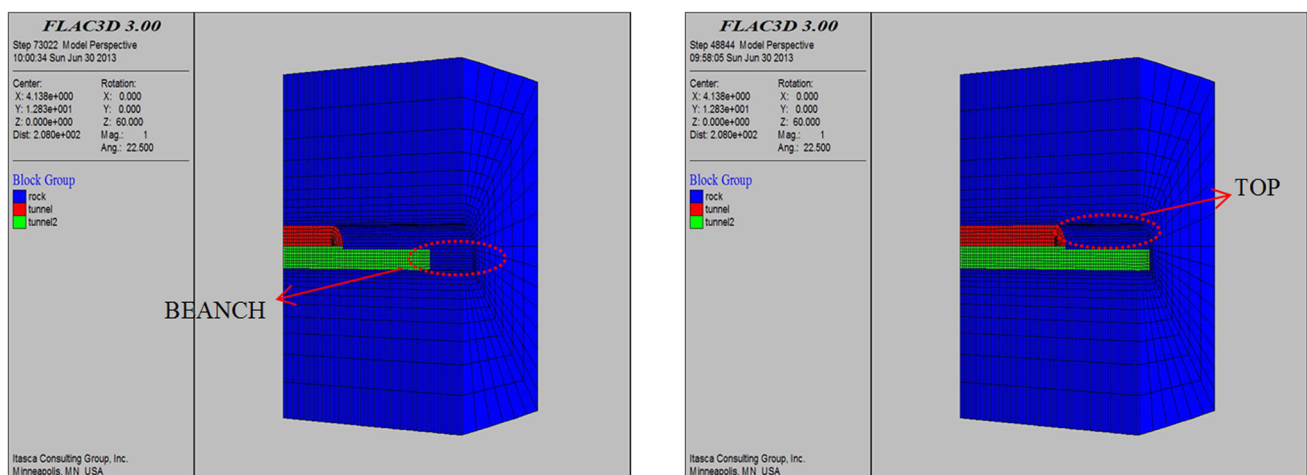


Fig. 5 Sequential excavation method (*top* and *bench*) in modeling with FLAC3D

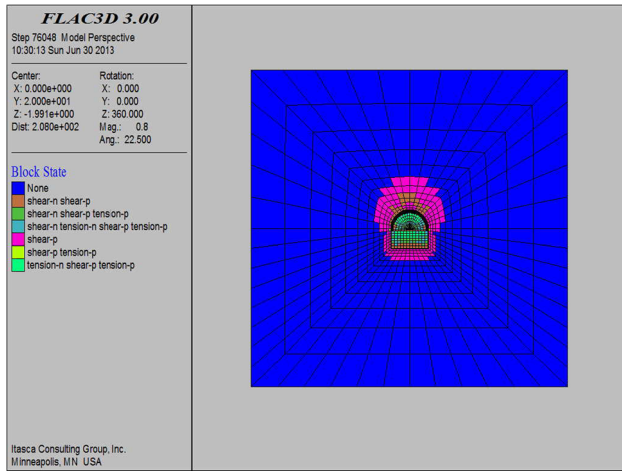


Fig. 6 Plastic zone around the tunnel without the pre-support system

Fig. 7 Tunnel instability (without the pre-support system)



displacement in the crown and tunnel face decreased 30–40 percent.

Results show that deformations and the force on the pipes will differ based on the progression of drilling steps. The maximum force usually occurs near the tunnel face. The maximum deformation occurs from 0.5–0.75 of the pipe length. The location of the maximum force along the vertical axis is 1 m ahead of the tunnel face. The moment distribution related to this force also follows the same trend.

Figure 10 (left) shows the distribution of the force along the vertical axis on the fore-poling pipes (the Y axis is in local coordinates of the pipes equal to the Z axis of the global coordinates) and Fig. 10 (right) shows the moment of this force. In the top pipe, the vertical force is maximum. It should be noted that the effective forces acting on the

Table 9 Umbrella arch properties

Characteristics	Pipe diameter (mm)	Pipe thickness (mm)	Pipe length (m)	Overlapping (m)	Row no.	Pipe distance (m)	Rows spacing (m)
Amount	90	9	9	2.5	2	0.5	0.25

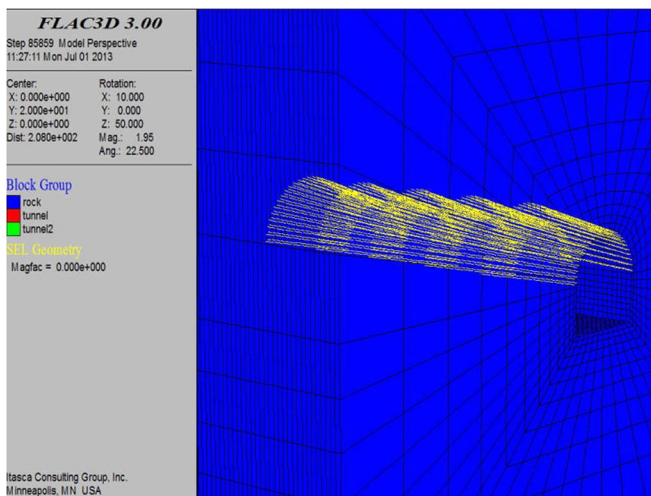


Fig. 8 Umbrella arch model (left) and its implementation (right)

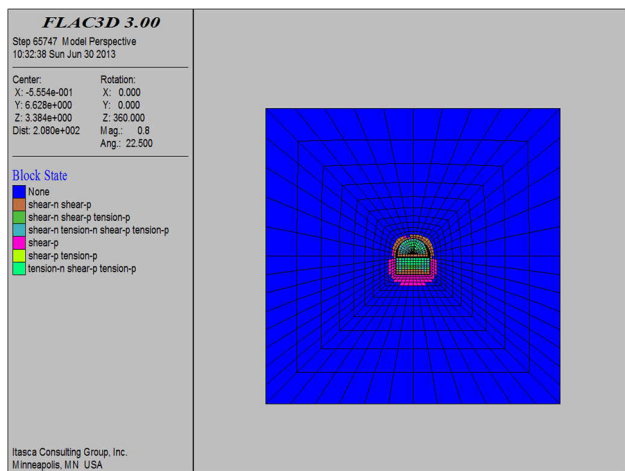


Fig. 9 Plastic zone around the tunnel with the pre-support system

pipe are in the shear form in the vertical and horizontal axes. These forces are the result of the vertical and lateral tensions that cause moment and bending in pipes. Also, the axial force along the pipe is not significant.

The vertical force on the top row pipes in the top 7 m of the excavation is shown in Fig. 11. According to Fig. 11, the top pipe has the maximum vertical force. In vertical loading from pipe 1 to pipe 6 (top half of the tunnel arc), the maximum compressive force will occur within the first one-third of the pipe. However, from pipe 7 to pipe 12, this force will occur within the first quarter of the pipe. In fact, the location of the maximum compressive force will change with the change in the tunnel arc location.

To study the possibility of yield and failure in the umbrella arch, first its strength is calculated using a combination of pipe and steel with a weight average. Regarding the 28-day strength of the grout with a water-cement ratio

of 0.5 ($\sigma = 0.5$) and a 240 MPa yield strength of the steel, the strength of the umbrella arch is equal to 170 MPa.

The maximum amounts of force, moment and stress on the pipes during the top and bench excavation are listed in Table 10.

In Table 10, σ_y and σ_z are from the bending, and σ_x is from the axial force. Although these values did not occur simultaneously in one section, for comparison, all stress values were added up, being equal to 123.4 MPa. It can be seen that the aggregate stress value is less than the pipes' strength. Therefore, the pipes will not reach the yield point value and will not fail.

Since the tunnel lining is meant for permanent support, the primary support system including the lattice girder and shotcrete was designed with a safety factor of 1.2. The interaction curve for the primary support system, when fore-poling is being performed in different tunnel sections, is shown in Fig. 12. As clearly shown, all measures are within the acceptable limits of axial force and bending moment. Also, it should be noted that the diagrams are for excavation of both the top and bench part, and, in all cases, the force and moment values are within an envelope with a safety factor of 1.2.

Results validation

In order to monitor the tunnel behavior, a five-point convergence meter during construction of the tunnel is suggested. The convergence pins in each tunnel section include one on the crown of the tunnel and four on the two side walls; these stations are repeated every 20 m along the tunnel. Installation and measurement are carried out simultaneously with the drilling and in the minimum distance from the working face in order to be able to record the minutest changes. Regarding the fact that construction of

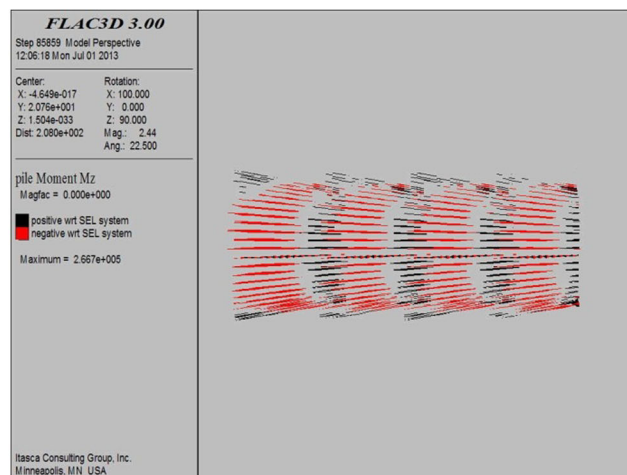
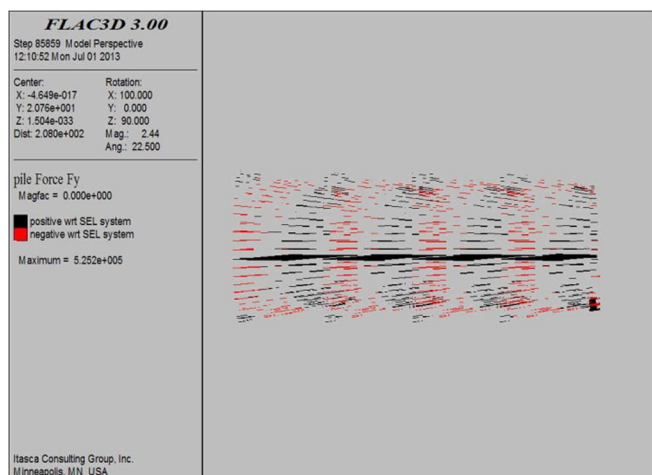


Fig. 10 Distribution of the force along the vertical axes (left) and moment (right) on the fore-poling pipes

Fig. 11 Vertical force on the top row pipes in the top 7 m of the excavation

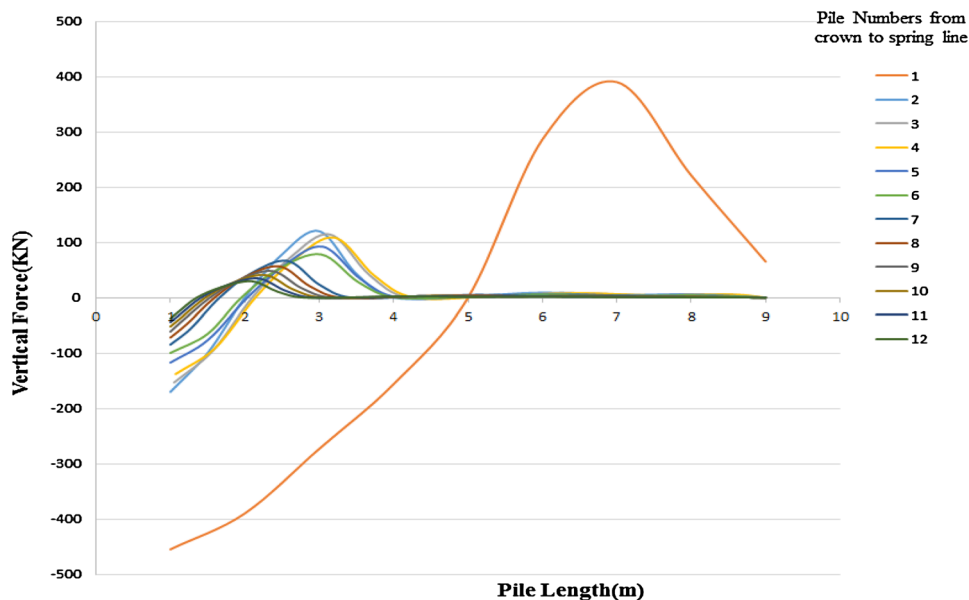
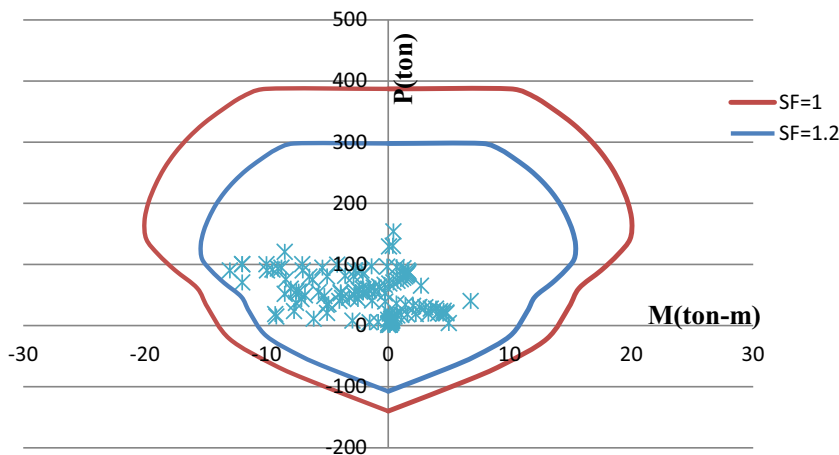


Table 10 Maximum amounts of force, moment and stress on the pipes during the top and bench excavation

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

Fig. 12 Axial force–moment (P–M) interaction diagram for the primary support system



the access tunnel utilizes the top and bench drilling method in which the top half is drilled where the deformations occur, three convergence pins were installed in this area, according to the timetable. In the next stage, when the lower section is being drilled, two other pins are installed in this area to control the total displacement (Fig. 13).

Within the part of the tunnel in which the pre-support system of the umbrella arch is installed, there are two monitoring stations (tunnel sections) at 0 + 134 and 0 + 154 km. The convergence trend of these sections

along with the reasons of the sharp changes in diagrams is given in Figs. 14 and 15. The results from reading the convergence of the spring lines (L1–R1) of the two stations show that horizontal convergence values for the two stations are 55 and 54 mm. Also, the horizontal displacement calculated from the numerical modeling for each end point of the arch (spring line) is equal to about 29 mm (Figs. 16, 17). Therefore, the results from numerical modeling closely matched those from periodical readings of the instrumentation.

Conclusion

According to the obtained results, it can be concluded that:

- Based our modeling and studying the plastic zone and the displacements around the tunnel, using an umbrella

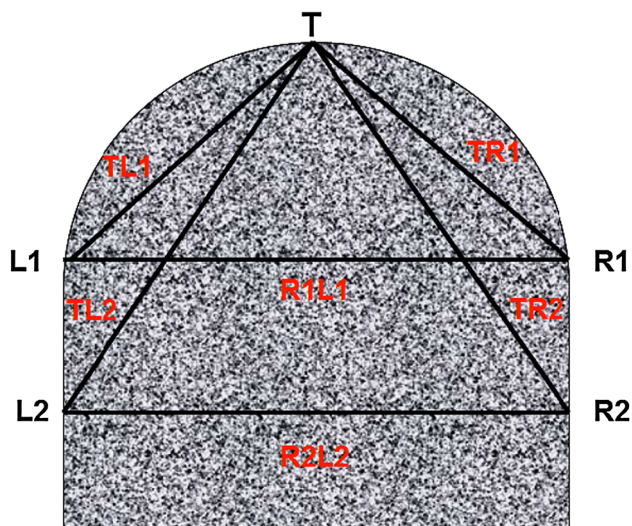


Fig. 13 Directions of measurement in the five-point convergence meter

arch for guaranteeing the stability of the alluvial regions of the adit tunnel seems necessary. The use of fore-piling leads to a significant decrease in the plastic zone as well as in the displacements around the tunnel and, therefore, the tunnel will be more stable.

- The structural behavior of the pipes in the umbrella arch is similar to that of a beam with one end cantilevered and the other end in the ground. Displacement, force and the moment exerted on the pipes will change as the drilling progresses.
- When the length of the pipe inside the ground is half or less of the total length, the maximum deformation occurs at the 0.5-mark and 0.75-mark of the pipe’s length. In overlap areas, the maximum displacement will shift to the middle parts of the pipe and it is the next series of pipes that will deform near the tunnel face.
- The distribution of the forces and the displacement in the central pipe on the crown of the tunnel are different from those in pipes around the tunnel. In the top pipe, the vertical force is the maximum. In vertical loading of the pipes in the top half of the tunnel’s arch, the maximum force occurs at the first one-third of the pipe, whereas, in the pipes in the lower half of the arch, the maximum force occurs in the first quarter of the pipe. In fact, the location of the maximum force will change with the change in the tunnel arc location.

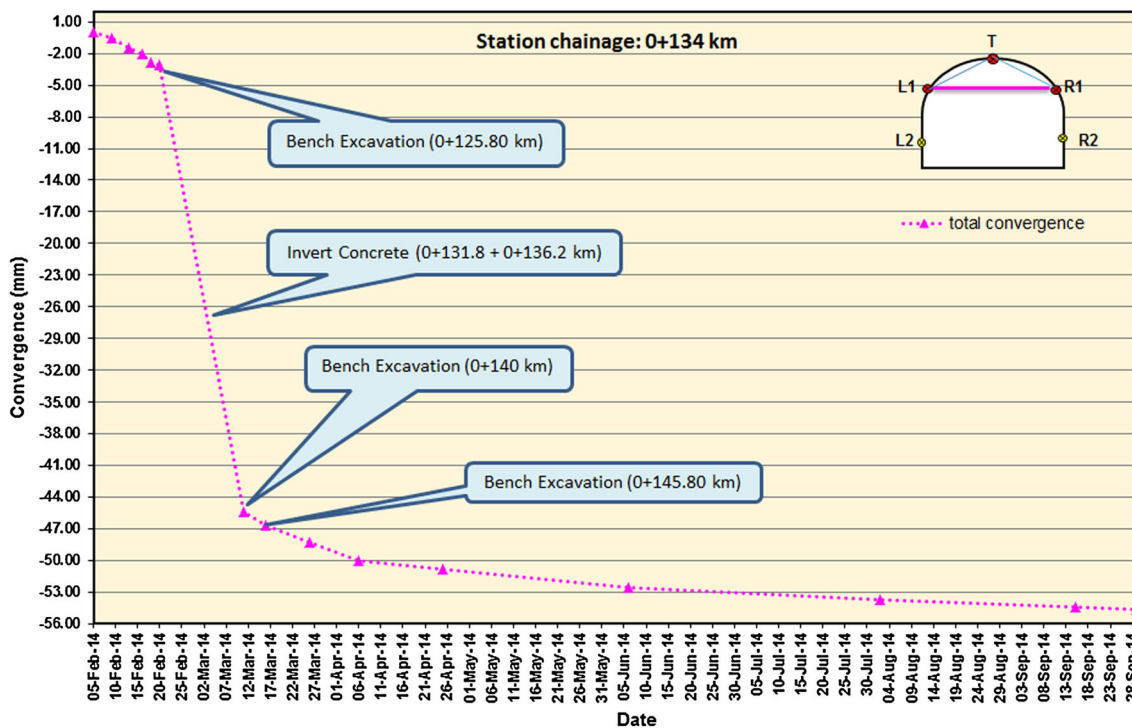


Fig. 14 Convergence of L1–R1 at the 0 + 134 km station

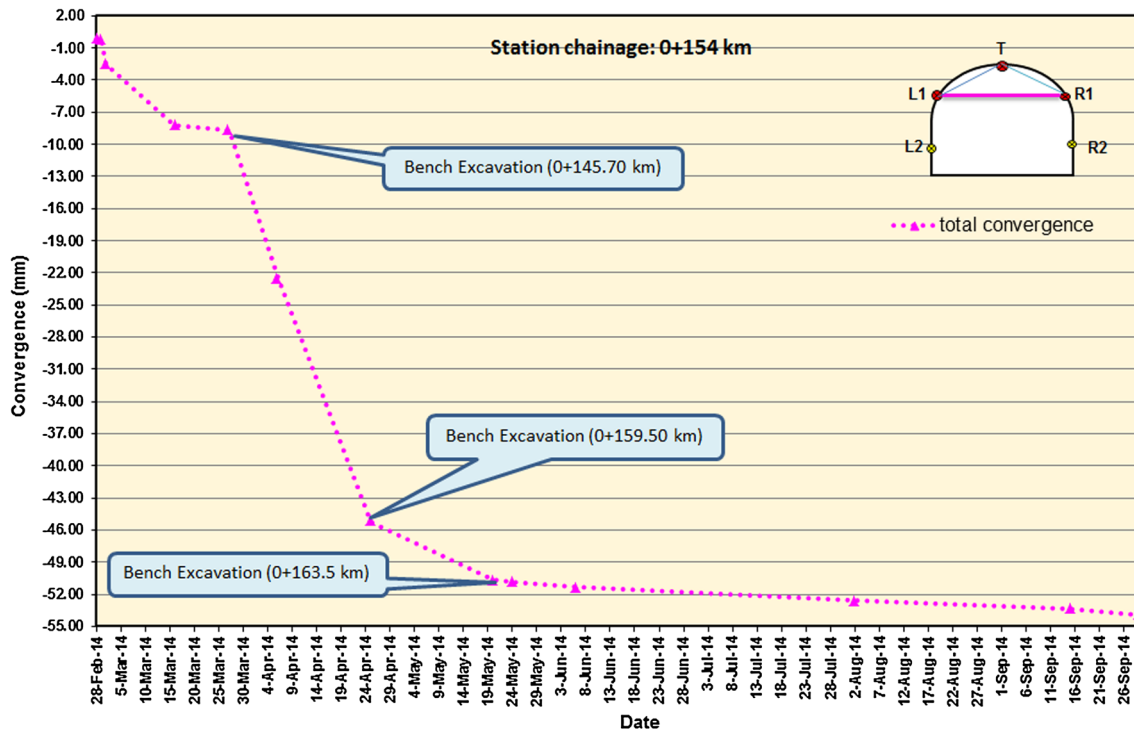


Fig. 15 Convergence of L1–R1 at the 0 + 154 km station

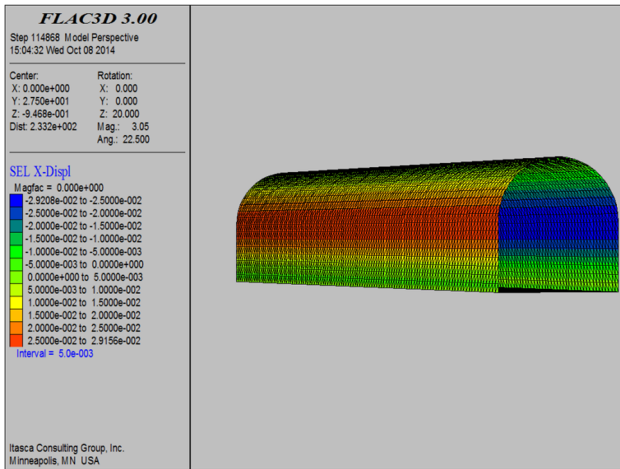


Fig. 16 Horizontal displacement around the tunnel with the pre-support system

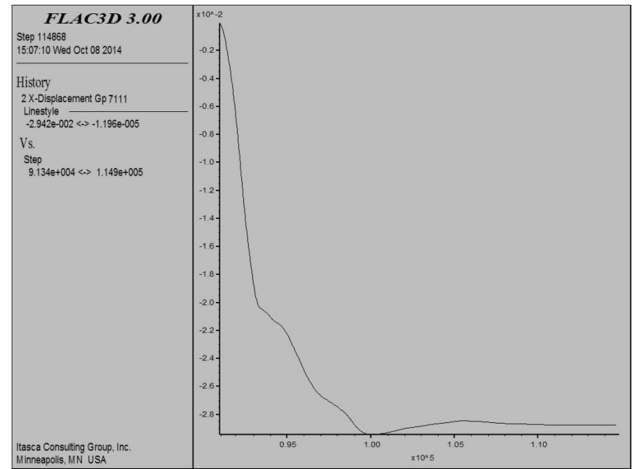


Fig. 17 Displacement history of the spring line (the node at the end of the arc)

- A water-cement ratio of 0.5 produces more strength than a 0.7 ratio.
- An increase in the pipe length has a positive effect on the stability of the tunnel. An increase in the length more than 1.5 times the diameter of the tunnel has no significant effect due to the distance from the plastic zone. Values less than the diameter will not cover the plastic zone completely either. The optimum amount is 1 to 1.5 times the tunnel’s diameter.
- Due to the decrease in the cantilever effects of the grout at the end of the pipe and the decrease in the length of the pipe in the disturbed zone in front of the working face, it is necessary to provide the second series of the pipes with appropriate overlap. The minimum length of overlap is equal to the length of the disturbed zone in front of the working face. With regard to the disturbed area of the adit tunnel face and the results of the modeling, the optimum length of overlap is 2.5 m.

- The pipes used have diameters of 90 mm, lengths of 9 m, and overlap lengths of 2.5 m; they were arranged in two rows 0.5 m away from each other in a zig-zag arrangement. The distance between pipes is 0.25 m.

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