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Tunnel Engineering

Simulation of Complex Support Systems for Large Span Tunnels: Investigation on Support Interferences and Effects of Constitutive Models

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ABSTRACT

Ground settlement control is a critical aspect in underground projects with shallow overburden. In tunnels with large span, the use of common support elements such as shotcrete and lattice girder are not sufficient in order to provide tunnel stability with acceptable safety factor and additional supporting elements may be required. In this research, the effect of combination of multiple support elements, including shotcrete, fore-poling, nailing, and micro-pile to minimise ground settlements, have been investigated. This case study focuses on the Arash-Esfandiar tunnel, a shallow underground passage located in the northern part of Tehran, Iran with a total length of 1532 m. According to the geotechnical report, ground condition varies from silty sandy gravel to dense clay sand. Finite Element (FE) analyses were performed by assuming different constitutive models i.e., Mohr-Coulomb (MC), Hardening Soil (HS) and Hardening Soil with Small strain stiffness (HSS), to investigate the capability of linear and non-linear models on predicting the surface settlement in the study. The results indicate that axial and bending elements as tunnel support measures concurrently, affects more in ground settlement control. On-site measurements and the results of numerical modelling show a significant effect of removing temporary lattice girder on surface settlements. The research is novel in its application of various constitutive soil models – MC, HS and HSS – to predict surface settlement effects. Comparative analysis of FE results with on-site measurements reveals the significant influence of removing temporary lattice girders on surface settlements. It is found that while the MC model is unable to capture the full complexity of the conditions governing the project and the HS and HSS models demonstrate a higher fidelity in representing the soil behavior during the tunneling process. Whereas on-site measurements indicate a higher impact of excavation stages showing larger deformations. Considering the heights of the walls, during the final stage of excavation the invert didn't have much effect on surface displacements.

1. Introduction

Many of the major cities require underground transport systems, particularly in areas facing the challenges of high population density and traffic congestion. The construction of tunnels and underground infrastructure has the potential to affect the stability of the ground, causing ground movement and settlement. These can be significant enough to threaten the ability of nearby buildings and structures to function and remain safe. Therefore, one of the main concerns in tunnel construction is to ensure not only the safety of the construction process itself, but also the well-being of adjacent structures, especially in urban environments.

Among different infrastructures, the performance of tunnels is strongly influenced by soil behavior, identified by physical and mechanical parameters, support systems, and tunnel geometry. So, it is mandatory to be aware of the interaction of structural elements and stress conditions around the opening. Pre-support elements such as spiles and fore-poling became common support measures to provide face stability and reduce surface settlement. In the other hand, micro-piles and rock bolts/nails are used to improve the soil bearing capacity and provide a better stress distribution, respectively. On the excavation of a large-section

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tunnel or a tunnel in the ground, whose bearing capacity or rigidity is insufficient, tunnel supports might settle considerably, causing the expansion of a disturbance zone along the arch of the tunnel. In the case of a tunnel with a very large cross-section or a flat cross-section, this could pose a serious tunnel stability problem. These preliminary supports could cause stress concentration in the tunnel crown. Particularly for earth tunnels in urban areas, therefore, it is becoming increasingly important to develop methods of invert reinforcement as a means of increasing the bearing capacity and preventing tunnel settlement.

Many researchers have investigated the effect of support elements on tunnel stabilities. Oka (1999) described the role of the micropile in bearing capacity and settlement control in a large-section tunnel. Topolnicki et al. (2004) investigated the effects of micropile and anchor combination in Gdańsk road tunnel. Kimura et al. (2005) mentioned two approaches for tunneling in soft ground conditions: i) A complementary technique to limit displacements, involving specialized jet grouting for foundation piles and the use of long steel tubes for forepoling and ii) A drilling method for groundwater drainage that does not interfere with tunnel construction. Particularly for short urban tunnels with shallow overburden, these methods might be a cost effective alternative. According to a previous study of Muya et al. (2006) the use of rock bolts around the tunnel can significantly reduce the stress concentration and the size of the disturbed zone and further improve tunnel stability. Likar et al. (2004) investigated the effectiveness of pipe roof in the Trojane tunnel. The tunnel deformations were monitored optically and extensometers and measuring anchors were installed at selected locations. They used the monitoring data to develop a comprehensive assessment approach aimed at optimizing the excavation and supporting procedures to minimise surface settlement. In addition, Oke et al. (2013) introduced the Umbrella Arch (UA) selection method. The UA support selection method was developed to assist tunnel designers in determining the appropriate method for selecting a UA. This involved analyzing the overall performance of the UA component using calibrated numerical models, based on data from two different tunnel projects. They calibrated numerical models based on in-situ data from two different tunnel projects. The Driskos tunnel project led by Vlachopoulos (2009) and the Istanbul metro project led by Yasitli (2013) were used to investigate the overall performance of the forepoling element. Meanwhile, optimization of specific design parameters was carried out in order to overcome the challenges posed by the highly compacted ground conditions encountered at the Driskos tunnel site (Vlachopoulos and Diederichs, 2014). Oke et al. (2014) demonstrated a step-by-step optimization process for two forepoling design factors: spacing and overlap. They conducted a data-rich case study to identify the most appropriate design, taking into account site-specific factors and the proposed optimization of forepoling element design parameters. It is important to note that the research does not include validation of the proposed method. Calvello and Taylor (1999) evaluated ground reinforcement techniques to reduce ground settlement and improve tunnel stability. They carried out a series of centrifuge model tests as an initial investigation into the effects of using spiles as reinforcement within a tunnel face. The results indicate that incorporating spiles into the design not only improves the stability of the tunnel face, but also results in a reduction in the associated ground movements. A subsequent investigation was carried out to analyze the factors leading to the failure of the initial lining in a deep-overburden, where the collapse of the tunnel's sidewall occurred during excavation. The primary emphasis was on analyzing and contrasting the soil engineering aspects related to the case. The findings have revealed that the tunnel collapse was primarily due to the presence of numerous joint fissures in the surrounding soil (King et al., 2016).

Juneja et al. (2010) conducted centrifuge tests to study the tunnel face stability in clay, based on the forepoling method. Their investigation revealed that the implementation of fore-poling led to a reduction in the extent of settlement ahead of the tunnel face, while the width of the deformation area remained unchanged. Le and Taylor (2017) conducted a series of three-dimensional centrifuge tests to model a reinforced tunnel heading. These tests aimed to evaluate the effectiveness of fore-poling reinforcement by varying the arrangement and bending of the steel pipes at different overburdens. Their research demonstrated that implementing a fore-poling umbrella system resulted in substantial reductions in both the magnitude of ground movements on the surface and subsurface, as well as a decrease in the overall extent of the deformed area.

Chen and Liu (2019) formulated a semi-analytical method to address the problem of undrained circular cavity expansion. This approach incorporated an anisotropic critical state clay plasticity model. The objective of this research was to provide a solution that accounts for both natural and induced anisotropy, with the aim of providing a more realistic perspective for numerical modelling of opening expansion. This development was particularly relevant to advanced critical state plasticity models, as outlined by Chen and Lie (2019). Another semi-analytical approach and finite element simulations to address the tunnel excavation issue, utilizing an enhanced Drucker–Prager framework under drained conditions (Liu et al., 2019). It highlights how the chosen model accommodates soil or rock strain-hardening through internal friction and cohesion. The solution simplifies to a set of differential equations focusing on stress and volume variables. It includes parametric analyses to demonstrate tunnel support and stress distribution, showing strong correlation between numerical results and analytical predictions, affirming the method's precision and dependability.

Cui et al. (2010) described the foot reinforcement mechanism by 2-D elastoplastic FE analyses. The phenomenon of attendant settlement was accurately simulated, and the outcomes showed that this system can effectively mitigate ground subsidence. This is achieved by enhancing the impact of shear reinforcement, redistributing loads, and managing internal pressure. Chen et al. (2016) introduced the analytical method to describe and evaluate the behavior of the feet-lock pipe in combination with the steel frame, which shows reducing surface settlement and load redistribution.

Meanwhile, Salehi et al. (2021) investigated the seismic behavior of complex underground structures.

The detailed FE simulations and comparative analyses with actual measurement data offer a specialized exploration of deformation control – a pivotal aspect of tunnel stability and safety. Those findings, even though that they can only be validated by deformation measurements for this specific case study, give a good overview about advantages and shortcoming of the used constitutive models which can be applied to other study areas.

In this context, geotechnical engineers frequently employ numerical analysis to make the most accurate predictions regarding tunnel performance. They validate these predictions by comparing them with field-recorded data and on-site observations. Consequently, this paper seeks to delve into a case study involving the intricate support system of a large-span tunnel. The case study focuses on the construction of the Arash - Esfandiar tunnel, an underground passage located in the northern region of Tehran, Iran. Meanwhile, Field measurements results have been used to validate the FE analysis.

2. Description of the New Austrian Tunneling Method

2.1 Concepts and Assumptions of the New Austria Tunneling Method

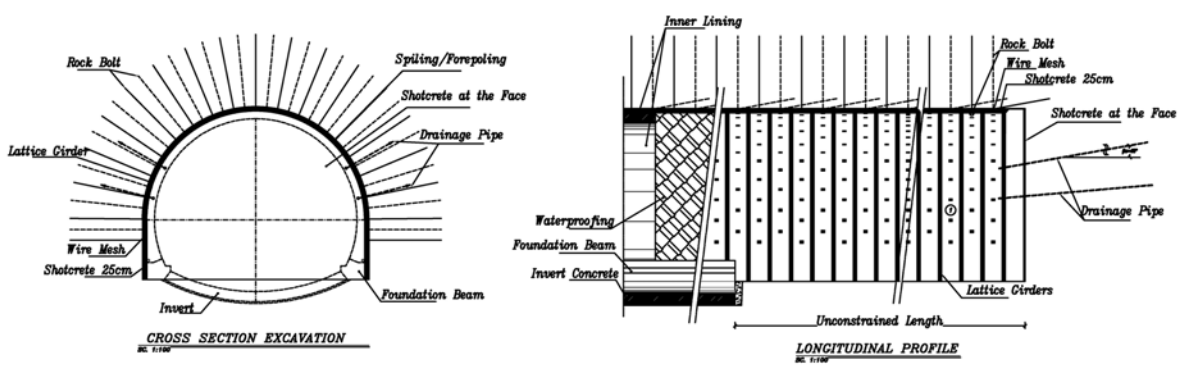
New Austrian Tunneling Method (NATM) was introduced in Austria during 1957-1965 by Professor Rabcewicz. The idea of this method is to allow a certain amount of deformation in the surrounding rock or soil to reduce the stresses acting on the final support of the tunnel (FHWA, 2009). The NATM provides versatility in terms of tunnel geometry, allowing it to adapt to nearly any size of opening. Typically, the cross-section used is generally ovoid in shape, which facilitates the even redistribution of stress in the surrounding ground when a new opening is created.

Nowadays, extensive studies are conducted by rock mechanics specialists to present an assured design for supporting underground spaces, in order to design the support system in such a way that in addition to being safe, it is economically feasible, too. The results of these studies emphasize the necessity of using observation methods in tunneling and flexibility in design. NATM is a method based on using soil/rock mass behavior under excavation and monitoring the operation, mainly the deformations. This type of tunneling method uses the following features:

1. Utilizing the natural strength of the rock mass – This approach relies on preserving the inherent strength of the surrounding rock mass as the primary means of tunnel support. The initial support is designed to assist the rock in supporting itself.
2. Shotcrete protection involves minimizing loosening and excessive deformation of rock while ensuring work safety. This is achieved by using of shotcrete layer, immediately after the tunnel excavation.

Measurement and monitoring are essential components of the NATM construction process. It requires careful tracking of potential deformations in the excavation. Advanced measurement instruments are installed within the lining, surrounding ground, and boreholes. Additional supports are only installed when necessary upon observation of any movements. This approach enhances the overall cost-efficiency and safety of the project.

1. Support is provided using a flexible approach rather than a fix one. The tunnel's reinforcement is achieved through an adaptable mix of rock bolts, wire mesh, and steel ribs, as opposed to using a heavier concrete lining. The initial support should be flexible and allow the soil/rock to deform in a certain range. In this way, it is possible to use a lining with less thickness in comparison to stiff support.
2. Closing of the invert is particularly vital in soft ground, quick sealing the invert (the lower part of the tunnel) is key as it forms a supportive ring. This approach benefits from utilizing the natural strength of the surrounding rock mass, enhancing the tunnel's stability.
3. The Classification and categorization of rock mass, which varies from very hard to very soft, dictates the minimum necessary support measures. This helps prevent the financial waste associated with overly robust support systems. For each major category of rock, there are specific support system



(a) (b)

Fig. 1. Example of an Excavation of a Tunnel Section by NATM Method: (a) Applied Supports in Cross Section, (b) Applied Supports in Longitudinal

designs. These designs act as the standard for reinforcing tunnels.

4. Displacement control behind the face – Given that most of the internal and surface displacements on each cross section begin ahead of the tunnel face, controlling this part has a remarkable impact on reducing deformations.

2.2 Tools to Approach NATM Goals

The NATM is a flexible and economical approach to tunnel construction that leverages of the rock mass natural strength. This method depends on adaptive support elements like rock bolts and fore-poling systems to manage stress and prevent excessive deformation, thus avoiding the need for thick concrete linings (Fig. 1). In practice, NATM's responsive design allows for efficient resource use while ensuring structural stability.

To mitigate ground movement during open-face tunnel excavation, the fore-poling umbrella system is employed. This technique involves steel pipes arranged to form an arch that reinforces the tunnel heading, particularly in softer or less stable soil conditions, providing crucial support before the tunnel lining is installed. Design principles posit that fore-poling pipes, in conjunction with grout, function as longitudinal support beams spanning the tunnel. These beams, anchored in shotcrete on one end and embedded in the ground on the other, help manage deformation at the tunnel front (Fig. 2). The goal is to maintain control over the forces impacting the tunnel face (Muraki, 1997). This system is engineered to absorb vertical stresses and avert tunnel face collapse, with the fore-poling acting as a semi-flexible beam, anchored in the stable ground and linked to tunnel supports. The support configuration aids in adjusting stress around the tunnel, promoting stability within the safe parameters of the Mohr-Coulomb model.

The installed pipes should be sufficient to take up the vertical stress acting over it and prevent face sliding (Fig. 2(a)). For the analytical model, the umbrella is considered as a pipe with one end fixed and supported by the firm ground ahead of excavation while the other end is hinged by tunnel supports in the form of lattice girder or steel ribs as shown in Fig. 2(b). Fig. 2(c), shows how fore-poling support can influence the stress condition. Any reduction in the vertical stress or increase in the horizontal stress will cause a decline in the radius of the stress circle. Such stress redistribution will provide a safe zone under the Mohr-coulomb failure envelope.

3. Research Methodology

To address the needs and fill the gap in the body of knowledge, a research is conducted to evaluate the different constitutive soil models (MC, HS, and HSS) on tunnel stability and settlement. The study meticulously incorporates axial and bending elements as tunnel support measures and investigates the critical support required to stabilize the tunnel. This study will try to explain the innovative integration of multiple support elements like shotcrete, fore-poling, nailing, and micro-piles, and their interferences. The numerical analysis results will be validated against on-site measurements, showcasing the importance of choosing appropriate constitutive models and support systems for accurate prediction and control of tunnel-induced ground movements. Tunnel structure and its surrounding soil are simulated in PLAXIS 2D finite element code simultaneously. Stage construction and continuous implementation of tunnel coverage, match real stages of construction and have been considered in the model.

3.1 Project Overview of the Case Study

In this contribution, an underground project was investigated which is located in the capital of Iran, Tehran metropolitan.

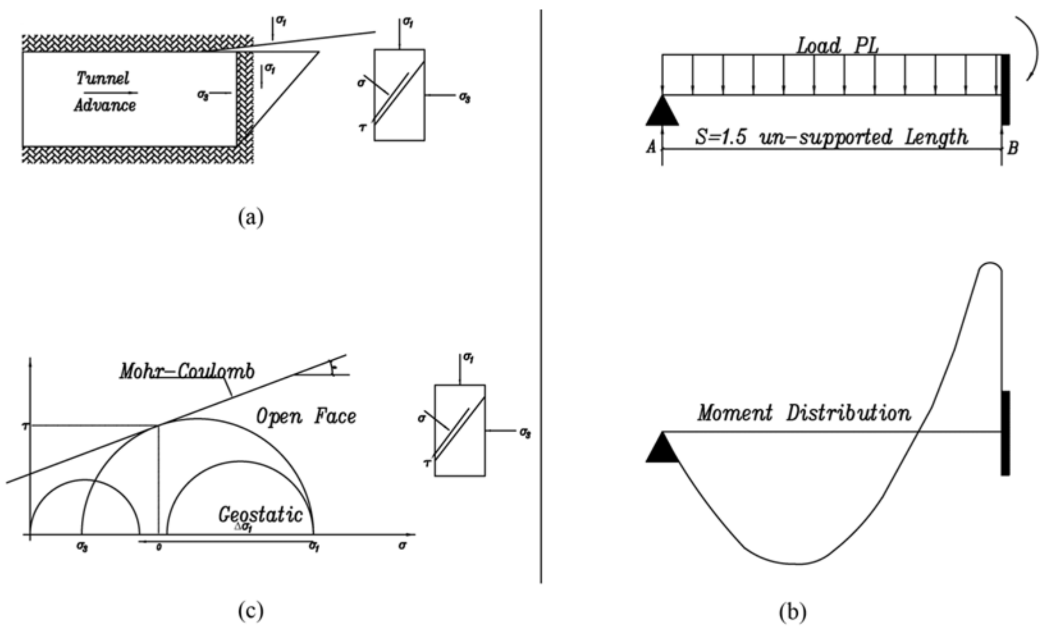


Fig. 2. Theory of Fore-Poling Function: (a) Influence Fore Poling on Face Stability, (b) Structural Concept of Fore Poling, (c) Shear Strength Improvement

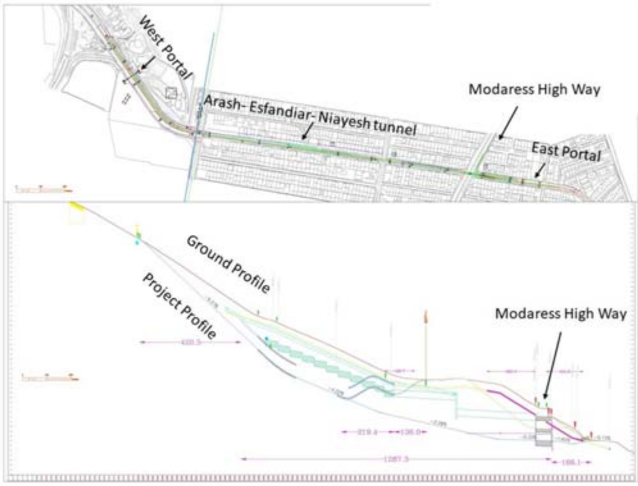


Fig. 3. Plan of Arash-Esfandiar Tunnel

Arash-Esfandiar tunnel in the third district of Tehran Municipal and is being implemented to provide east-to-west access in this area, between the East Arash Street and the beginning of Niayesh Highway (Fig. 3).

3.2 Used Tunnel Geometry

Given on the one hand, the necessity of providing multiple accesses to the project site from West Arash Street, and Esfandiar and on the other hand, due to project restrictions, it was inevitable to use several different geometric sections during construction. Under the Modares Highway, due to the need to supply a two-way traffic option and due to limited space available, a two-level section was selected as the final option. Therefore, Modares two-level tunnel was implemented as part of the Arash-Esfandiar tunnel project. The tunnel section is 12.6 m in width and 13 m in height (Fig. 4). Considering the very low overburden and the passing through main urban arteries in proximity of high buildings, the New Austrian Tunneling Method was considered for this project.

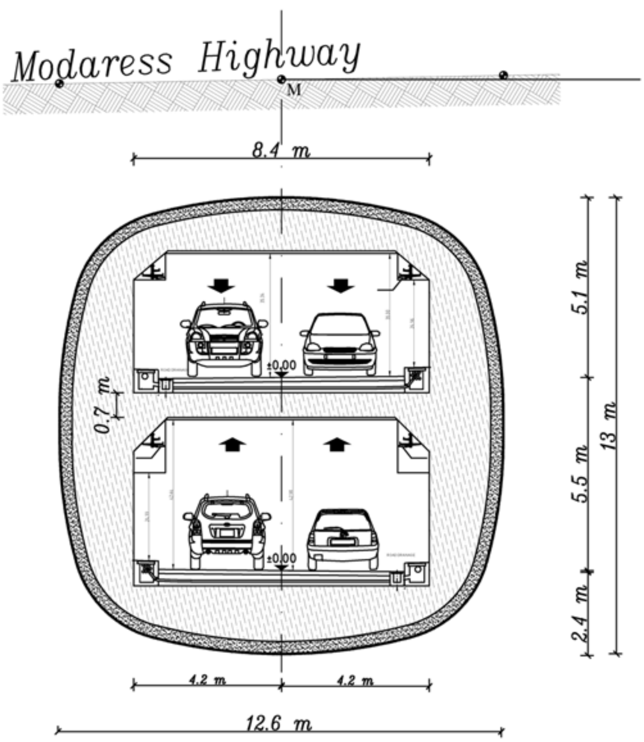


Fig. 4. Modares Two-Level Tunnel Section

3.3 Involved Materials

3.3.1 Geotechnical Conditions and Soil Parameters

Tehran is built on an alluvial structure. Therefore, during different geological periods, a sequence of homogeneous and regular layers of sand and gravel has been placed on top of each other. The geological and geotechnical characteristics of different soil types were established through laboratory tests performed on various samples collected from seven boreholes and four test pits. Four bore holes (each with 60 m depth) and one test pit (with 19 m depth) are placed on the subject part of the project for this study. The borehole locations (BH) and test pits (TP) are shown in Fig. 5. According to the laboratory and field tests, the soil were classified as cemented granular and as follows: dense silty sandy, very dense sandy gravel, dense clay sand (located beneath the gravel layer), and thin layers and lenses of clay.

Based on geotechnical tests (triaxial, uniaxial, direct shear, and in-situ shear tests) and field investigations the geotechnical properties of the soil types were determined. Table 1 gives an overview of the geotechnical parameters, essential for this research.

3.3.2 Support Elements and their Characterization

In the current research according to the large span of the tunnel and the geotechnical conditions, a combination of 4 different support elements was proposed. A classification of the suggested support system and their combination is provided in Table 2. Table 3 show the support structures and their representation in the numerical model.

In the two-level section (Modares Tunnel), the main walls will be supported by shotcrete with 35 cm thickness and lattice girder truss metal structures. If necessary, fore-poling element will

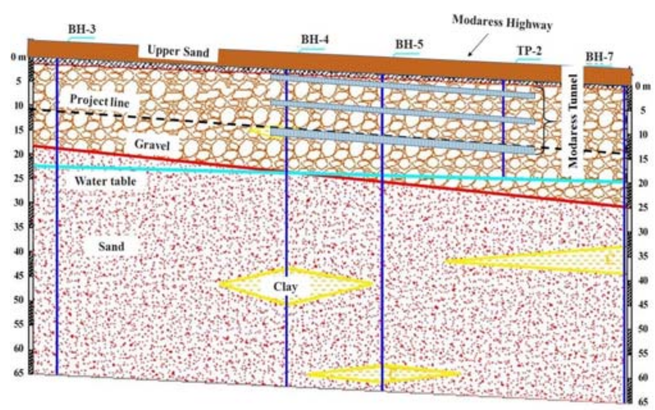


Fig. 5. Soil Profile in the Project Site

Table 1. Geotechnical Parameters Used in Modares Two Level Tunnel Simulations

First layer (0 to -3.5 m)	Second layer (-3.5 to -40 m)
$\varphi = 30°$	$\varphi = 37°$
$c = 9.8$ kN/m^2	$c = 24$ kN/m^2
$\gamma = 17$ kN/m^3	$\gamma = 18$ kN/m^3
$Dr = 35\%$	$Dr = 46\%$
HS and HSS parameters	
$v_{ur} = 0.2$	$v_{ur} = 0.2$
$E_{ref\,(50)} = 4.0$ e04 kN/m^2	$E_{ref\,(50)} = 7.0$ e04 kN/m^2
$E_{ref\,(oed)} = 4.0$ e04 kN/m^2	$E_{ref\,(oed)} = 7.0$ e04 kN/m^2
$E_{ur\,(HS)} = 12.0$ e04 kN/m^2	$E_{ur\,(HS)} = 21.0$ e04 kN/m^2
$m = 0.5$	$m = 0.5$
$G_{0\,(ref)} = 5.856$ e04 kN/m^2	$G_{0\,(ref)} = 11.7$e 04 kN/m^2
$\gamma = 0.1\%$	$\gamma = 0.1\%$
M–C parameters	
$v = 0.2$	$v = 0.2$
$E_{oed} = 4.0$ e04 kN/m^2	$E_{oed} = 7.0$ e04 kN/m^2
$G_{ref} = 1.15$ e04 kN/m^2	$G_{ref} = 2.625$ e05 kN/m^2

Table 2. Classification of the Examined Support Systems in Modares Two-Level Tunnel

No.	Support system	Class
1	Lattice + shotcrete	A
2	Lattice + shotcrete + forepoling	B-I
3	Lattice + shotcrete + micropile	B-II
4	Lattice + shotcrete + nailing	B-III
5	Lattice + shotcrete + forepoling + nailing	B-IV
6	Lattice + shotcrete + forepoling + micropile	C-I
7	Lattice + shotcrete + nailing + micropile	C-II
8	Lattice + shotcrete + forepoling + nailing + micropile	D

Table 3. Support Elements

Structural Elements	Representation in the numerical model	Function
Shotcrete+ Lattice Girder	Plate	Axial and bending
lf-Drilling Nail	Plate	Axial
Micropile	Plate	Axial
Forepoling	Cluster	Bending

be added to micro-piles and nails. Moreover, temporary walls are supported by 25 cm shotcrete and lattice. The shotcrete support is simulated with 7- and 28-day parameters in the numerical model (Elastic Modulus: 18.7 + 06 kPa and 23.4 + 06 kPa, respectively). In addition to ground load, the load caused by the traffic of Modares Highway is applied in the model (20 kN/m). The tunnel initial lining had a thickness of 35 and 25 cm. The shotcrete and lattice girder are designed for strength class of C25, and grade AIII steel bars, respectively. The lining structure was modeled with a linear elastic behavior and plate element was adopted for the reinforced shotcrete material. Meanwhile, nail and micropile are modeled by the plate element with elastic behavior. Thus, the equivalent bending and axial capacity is calculated as a combination of steel and cement grout. Based on the recommendation of the software developer (Plaxis Bulletin, 2009), the definition of the interface in the nail and micropile has been neglected.

3.3.2.1 Fore-Poling Elements

Equation (1) was used to calculate the elasticity modulus E_{eq}, specific gravity γ_{eq}, and area adhesion C_{eq} for forepoling elements (Plaxis Bulletin, 2009).

$$E_{eq} = \frac{(E_{grout} \cdot n \cdot V_{grout}) + (E_{Soil} \cdot V_{Soil})}{V},$$

$$\gamma_{eq} = \frac{(\gamma_{grout} \cdot n \cdot V_{grout}) + (\gamma_{Soil} \cdot V_{Soil})}{V},$$

$$C_{eq} = \frac{(C_{grout} \cdot n \cdot V_{grout}) + (C_{Soil} \cdot V_{Soil})}{V}, \quad (1)$$

where

C_{grout} = The tensile strength of cement grout,
E_{soil} = Elasticity and soil modulus,
n = The number of injected fore-poles that here is 60,
V_{grout} = The average volume of grout,
γ_{grout} = The specific gravity of cement grout.

According to Eq. (1), the following parameters are used to simulate the fore-poling implementation:

C_{eq} = 84 kN/m^2,
E_{eq} = 3.46 e 05 kN/m^2,
γ_{eg} = 20 kN/m^3.

3.3.2.2 Nail and Micropile Elements

Calculating axial and bending stiffness of the nail and micropile elements were conducted based on user manual's (Plaxis, 2009) recommendation using Eq. (2):

$$E_{eq} = E_n\left(\frac{A_n}{A}\right) + E_q\left(\frac{A_q}{A}\right),$$

$$EA\left[{}^{kN}\!/_{m}\right] = \frac{E_{eq}}{S_h}\left(\frac{\pi D_{DH}^2}{4}\right),$$

$$EI\left[{}^{kN.m^2}\!/_{m}\right] = \frac{E_{eq}}{S_h}\left(\frac{\pi D_{DH}^2}{64}\right), \quad (2)$$

where

A = Cross section area of bore hole,
A_n = Cross section area of nail or micropile,
A_n = Cross section area of grout around nail or micropile,
D_{DH}: = Bore hole diameter,
S_H = Distance between nails or micropiles.

3.4 Application of the Finite Element Model

To model the layers of the ground, 15-node triangular elements

Table 4. Support Element Parameters in Modares Two Level Tunnel

Element	Shotcrete	Micropile pipe	Micropile bar	Nail (Self-Drilling)	Forepoling
Diameter	25 / 35 cm	7.6 cm	3.2 cm	IBO R32S	7.6 cm
Yield Strength (kPa)	2.5 e + 04	23.5 e + 04	40.0 e + 04	40.0 e + 04	35.0 e + 04
Elastic Modulus (kPa)	23.4 + 06	200.0 e + 06	200.0 e + 06	200.0 e + 06	#200.0 e + 06

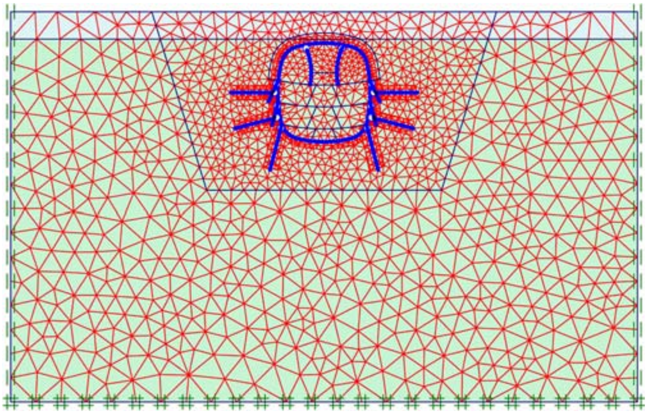


Fig. 6. Meshing Model in Modares Two Level Tunnel

have been used (Fig. 6). Meanwhile, to fully understand the range affected by the construction of underground structures, a model with a 50 m width and 80 m height was built (10-times the tunnel diameter in width and 5 times in height). Tunnel overburden within the investigated sections is at least 4 m, which has been considered in the geometric definition of the structure.

In this research, three constitutive soil models are considered: 1. MC, 2. HS, 3. HSS. The MC model is a linear elastic – perfect plastic model which requires specific parameters to explain the stress-strain behavior (Young's modulus E, Poisson's ratio, friction angle φ, and cohesion c, and dilatancy angle ψ). This model, is selected because of liner formulation as well as limited material parameters required as input. Generally, the 'real' soil behavior exhibits the following characteristics:

1. Non-linear stress-strain relationship, starting at very low-stress levels.
2. Strain-dependent stiffness

By comparison, the MC model fails to satisfy the above characteristics. Hence, the Hardening Soil model also will be used as the second constitutive model in this research.

The HS model is a non-linear elastoplastic soil model that is used for simulating both stiff and soft soils. HS also links stiffness parameters to the stress-level and describe the development of plastic strains during compressive loading. In soils with high relative density, the dilation (or expansion) behaviour during shearing might not be adequately captured by the HS model. In our case, the HS model is a reliable model for soils with a moderate relative density where its assumptions align more closely with the soil's actual behaviour. This constitutive model has been studied in the other section of this project and proved its validity (Golshani et al., 2018). The HS model assumes that the material behaves linearly during unloading and reloading processes. However, soil behavior can only be considered linear within a limited strain range. As strain amplitude increases, soil stiffness decreases non-linearly. The HSS model is an improved version of the HS model, accounting for the increased stiffness of soils at low strain levels. This stiffness increases non-linearly with strain. The HSS model follows the same structural principles as the HS model, including the Mohr-Coulomb failure criteria. However, this model enables the evaluation of alterations in the shear modulus concerning small-strain conditions.

When designing tunnels, it is important to consider the stability of the ground, surface settlements, deformations of the tunnel cavity, and the forces exerted on the lining. The evolution of stresses and deformations can be a complex three-dimensional challenge, but in engineering practice, simpler empirical methods and 2-dimensional FE analyses are commonly used. To represent the third dimension of a tunnel in 2D models, certain assumptions are necessary. In this paper, the stress reduction or relaxation method will be used. Based on previous studies (Golshani et al., 2018), the relaxation factor for each part of the excavation are as follows:

1. Top: relaxation of 35%
2. Bench: relaxation of 20%
3. Invert: relaxation 15%

Meanwhile, the safety factor is analyzed with the strength-reduction method. This idea is based on the gradually reduction of soil strength, until failure occurs. The relation between the soil's strength at failure and the original soil's strength can be considered as a factor of safety.

3.5 Tunnel Excavation Steps of the Finite Element Model

After providing stable initial conditions and activating the loads due to traffic, numerical modeling of excavation and stage stabilizations were conducted. It should be mentioned that the term *top* refers to stages 1 to 3 of excavation, *bench* refers to stages 4 to 9 and, *invert* refers to stages 10 to 12, presented in Fig. 7. Each single part from the left to right sections is called a side drift. In the following, all 42 calculation stages of modeling are presented:

1. Installing fore-poling pipes (if required)
2. Excavation of the upper part of the right drift of the tunnel by removing the elements inside the excavation area (section II)
3. Activating shotcrete + lattice girder structures in this area
4. Excavation of the upper section of the left drift (section I)
5. Activating shotcrete + lattice girder structures in this area
6. Excavation of the upper part of the middle section (IIIa)
7. Activating shotcrete + lattice girder structures in this area
8. Removing the temporary walls of the middle section
9. Excavation of the right side of the first bench (section V)

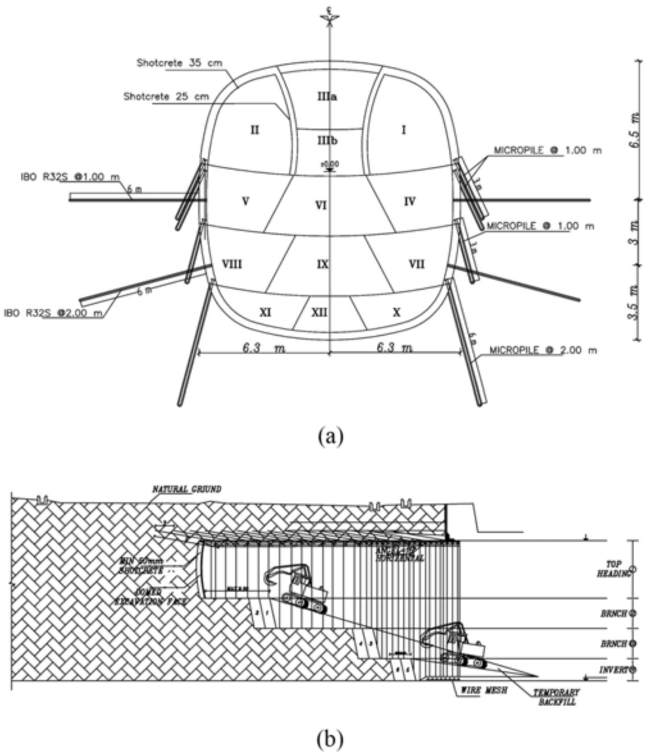


Fig. 7. Excavation Stages and Implementing the Initial Support of Modares Two-Level Tunnel: (a) Cross Section, (b) Longitudinal Profile

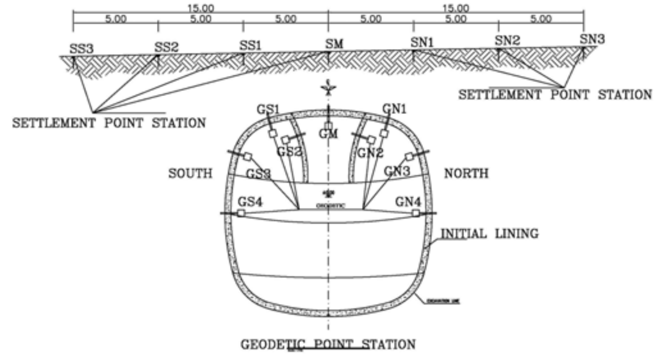


Fig. 8. Instrumentation Section of Modares Two-Level Tunnel

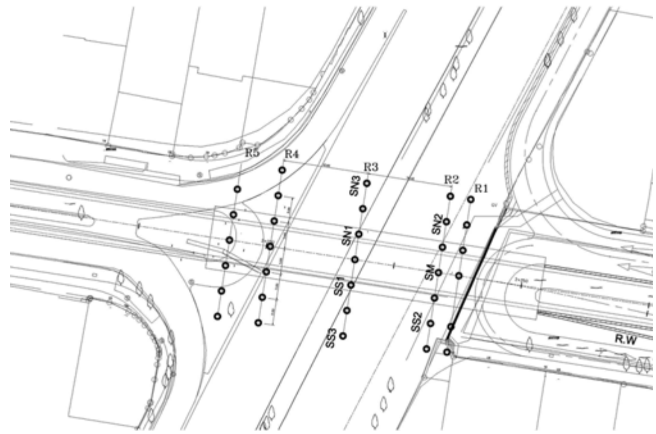


Fig. 9. Instrumentation Plan of Modares

10. Activating shotcrete + lattice girder structures in this area (and micropiles and nails, if required)
11. Excavation of the left side of the first bench (section IV)
12. Activating shotcrete + lattice girder structures in this area (and micropiles and nails, if required)
13. Excavation of the middle part of the first bench (section VI)
14. Excavation and installation of the shotcrete + lattice girder structures (and micropiles and nails, if required) in the second bench (sections VII, VIII and IX)
15. Excavation the tunnel invert in the right and left sides (respectively) by deactivating the soil elements (section X and XI)
16. Activating shotcrete +lattice girder structures in this area (and micropiles, if required)
17. Excavation of the invert core (section XII) and activating the invert shotcrete + lattice girder structures in this area.

3.6 On Site Measurement Instrumentation

In the Arash-Esfandiar tunnel project, in order to know the trend of ground settlements and also to verify the numerical model, various measurement techniques have been used in the different profiles. In the two-level profile of the project, a settlement measurement system has been used for recording the surface deformations (Settlement South (SS), Settlement Middle (SM), and Settlement Nord (SN)). The instrumentation plan can be seen in Figs. 8 and 9. Fig. 8 represents profile R3 of Fig. 9 which is in the middle of Modarres Highway. The inside deformation is measured by optical reflectors and since the internal convergence is not part of this research, it will not be discussed in this paper.

4. Results

After solving the model, it was found that in the case of using just shotcrete and lattice as temporary support, the tunnel will collapse during the second bench excavation and further excavation is impossible. Fig. 10 shows forming shear and collapse zones based on both constitutive soil models. In all three scenarios, based on different constitutive models, Developing lateral shear strain, causes instability on the model. The color-coded shear strain results show a gradient where warmer colors (yellows and reds) indicate higher shear strain levels, and cooler colors (blues) indicate lower levels. In conducted scenario with the HS model, the distributed shear strain is on the right side, with a clear peak near the middle.

The MC has a significantly higher scale of shear strain values and shows a more intense and focused area of shear strain near the base on the right side and the tunnel crown. Meanwhile, the HSS model has a similar pattern to HS, but the region of high shear strain appears more extensive.

Hence to provide a stable excavation, 76 mm pipes with 5 mm thickness have been used for fore-poling. The pipes can be simulated with plate behavior or as an improved soil material. The latter

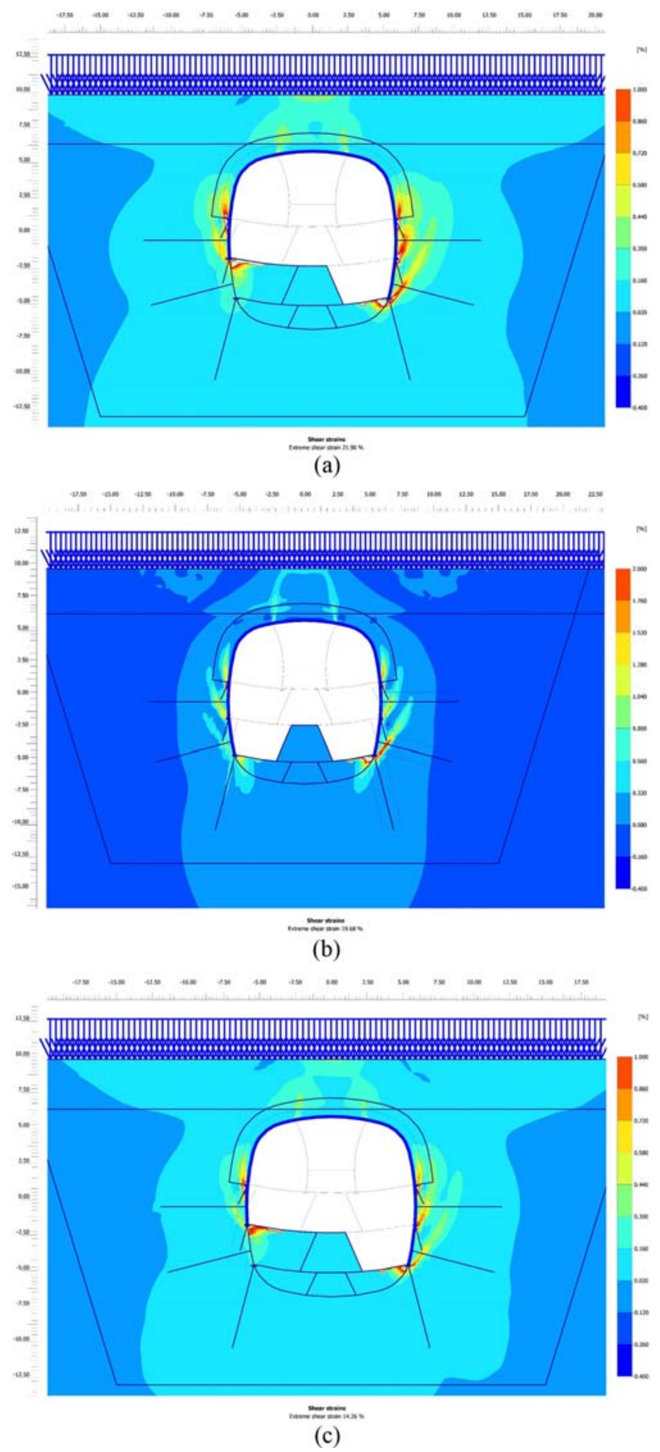


Fig. 10. Shear Strain in Tunnel Excavation Section VII Showing Shear Band Formation (lattice + shotcrete - support system type A): (a) HS Constitutive Soil Model, (b) MC Constitutive Soil Model, (c) HSS Constitutive Soil Model

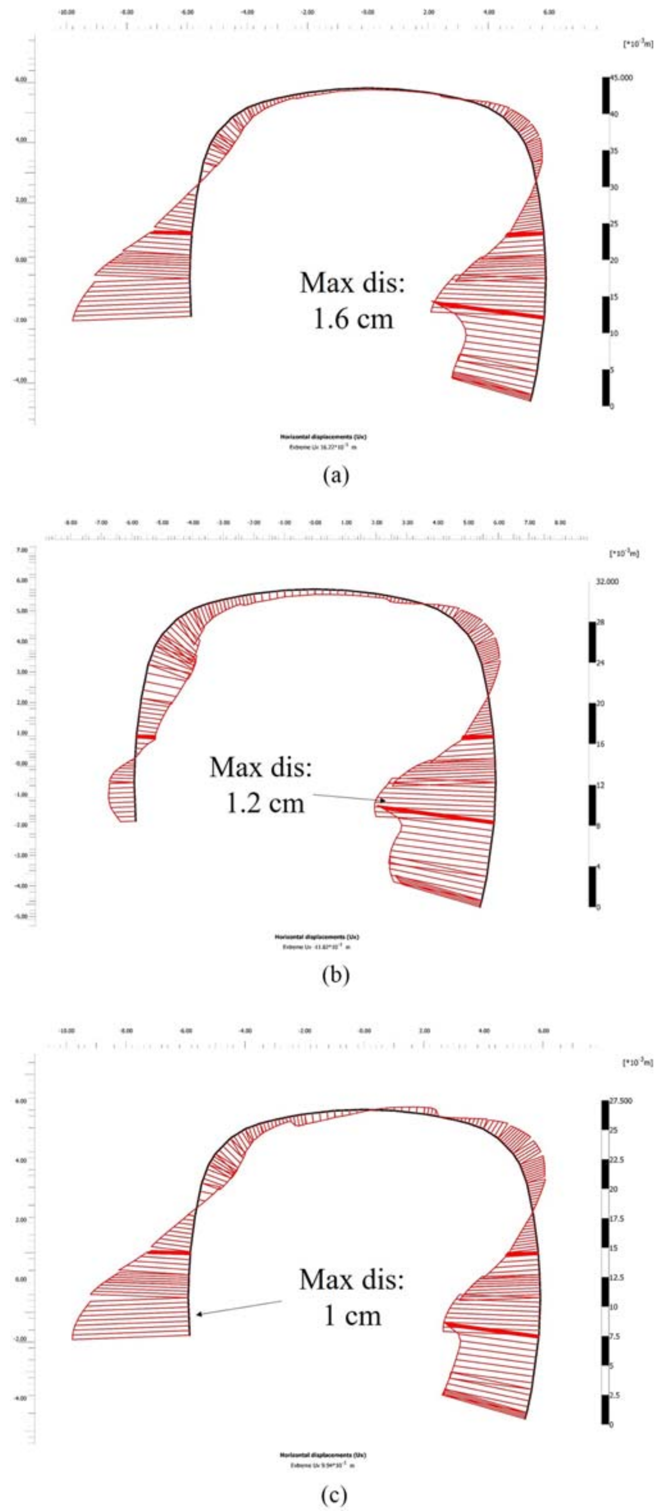


Fig. 11. Shotcrete Horizontal Deformation in Tunnel Excavation Section VIII (lattice + shotcrete - support system type B-I): (a) HS Constitutive Soil Model: 1.6 cm, (b) MC Constitutive Soil Model: 1.2 cm, (c) HSS Constitutive Soil Model: 1 cm

method is chosen, due to suggestions from the FE code developer. Regarding the conducted analysis, despite adding forepoling to the model, structural instability reoccurs in tunnel excavation section VIII. Increasing the unsupported height of the wall during excavation of the second bench resulted in increasing bending stress on the shell of the support structure and development of plastic points, creating a plastic hinge, which will cause the tunnel wall to collapse internally (Fig. 11).

In the HS scenario, horizontal displacements with an extreme value of 1.6 cm, the deformation pattern shows inward bending at the top and outward bulging at the sides, suggesting a compressive force at the crown and expansive forces at the sidewalls. In the MC case, the deformation pattern is quite uniform, with inward movement on both the crown and sidewalls, indicating a compressive environment around the tunnel. On the other hand, the pattern of deformation in the HSS model, is more asymmetric with significant inward deformation on one side, suggesting an uneven distribution of stress or a localized pressure source.

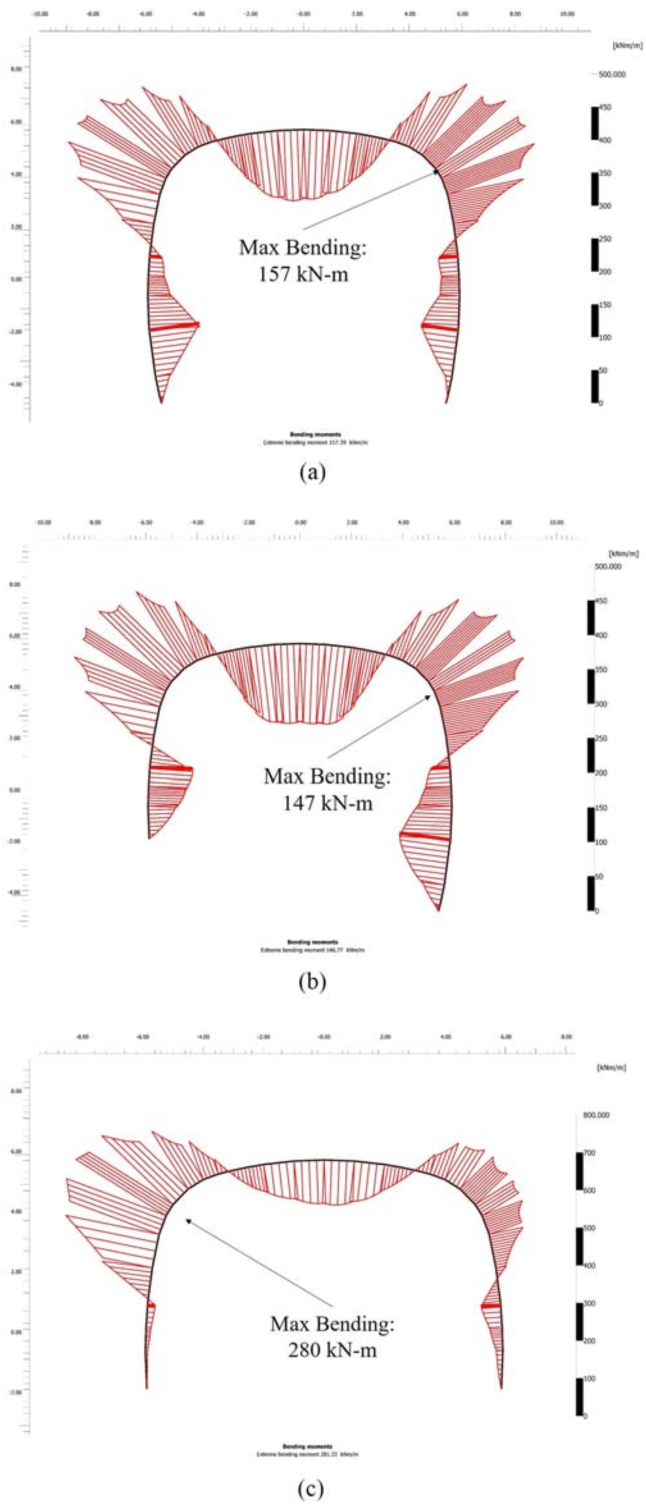


Fig. 12. Bending Moment in Tunnel Excavation Section X (support system type B-II): (a) HS Constitutive Soil Model: 156.4 kNm, (b) MC Constitutive Soil Model: 146.8 kNm, (c) HSS Constitutive Soil Model: 279.6 kNm

To reduce the stresses created in the initial support (shotcrete and lattice), additional axial elements have been used. In this case, self-drilling nails with 32 mm diameter (86 mm drilling diameter) and 1 m distance were selected and implemented at two different heights. Moreover, using micro-pile elements was also examined to control vertical displacements. Calculations of the features of micro-piles were similar to self-drilling nails. In the case of using micropile alone, the model again became unstable in excavation section X.

The non-uniform stress distribution is often the most dangerous as it can lead to creating a plastic hinge and localized failure. In all cases, the bending moment distribution is asymmetric, with the larger moments occurring at the transition from the tunnel's crown to the wall and a significant concentration at the side. The magnitude of the bending moments relative to the tunnel's bending capacity determines whether the lining will remain elastic, yield, or fail. Figs. 12 and 13 show the structural capacity (Bending Moment-Axial Interaction curve) for the shotcrete + Lattice girder+ micropile and induced load before the collapse in this alternative.

Consequently, in the next step, the simultaneous use of forepoling and nails was examined. The calculation results of the internal forces are illustrated in Fig. 14 and summarized in Table 5. Vertical deformation of the crown and maximum surface settlement are extracted on the tunnel central axis. The critical bending moment occurs in the connection between the roof and walls (Fig. 15).

Not shown in Fig. 14 are the support systems A, B-I and, B-II:

1. Support system A: Structural instability of the model in excavation section VII due to the increase of bending moments.
2. Support system B-I: Structural instability of the model in excavation section VIII due to the increase of bending moments.
3. Support system B-II: Structural instability of the model in excavation section X due to the increase of moments in the micropile.

Considering the sensitivity of the discussed project and the fact that it passes under a major urban road, it was decided to stabilize the tunnel using the Class D support system. In the case of combining axial and bending support structures such as forepoling and nails, the magnitude of displacement will be reduced in addition to ensure tunnel stability, even though all factors are safety are high enough to ensure stability alone (Table 6).

5. Discussion

Based on the FE analysis the effect of each stage on plastic zone development has been extracted. Fig. 16 illustrates the development

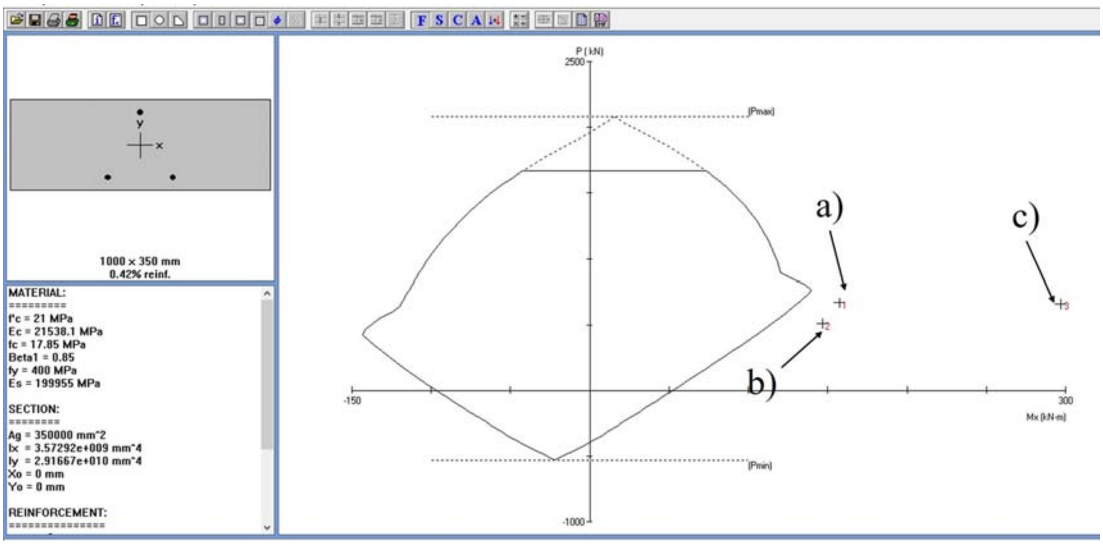


Fig. 13. Bending Moment-Axial Interaction (support system type B-II): (a) HS Constitutive Soil Model, (b) MC Constitutive Soil Model, (c) HSS Constitutive Soil Model

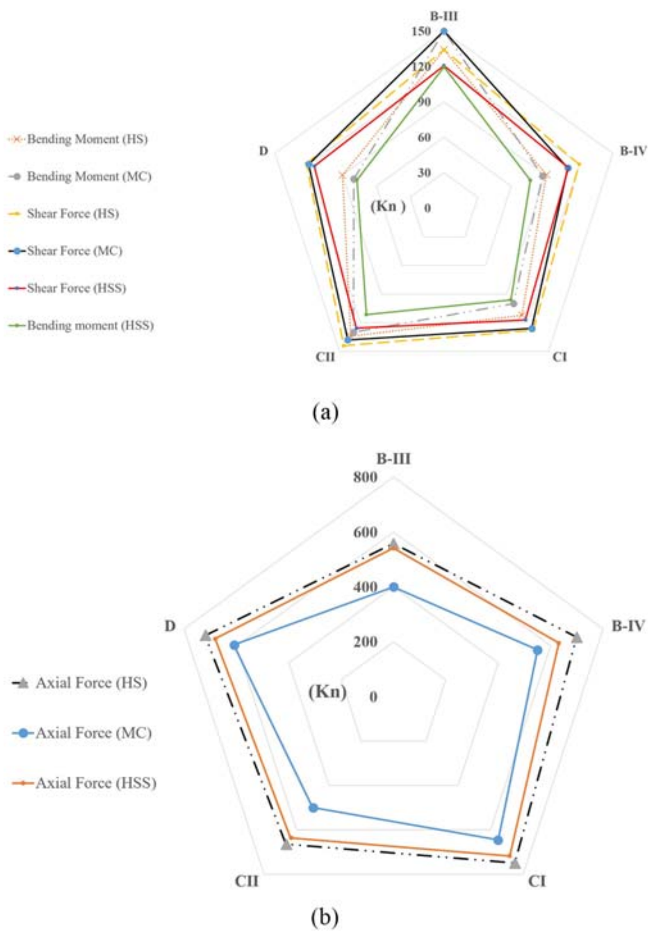


Fig. 14. Initial Lining Intern Force in Various Support Systems, Results of the Finite Element Calculations

of plastic zones around the tunnel during some of the sequential excavation and support phases. In this method, the excavation is carried out in stages, and after each stage, the tunnel is supported,

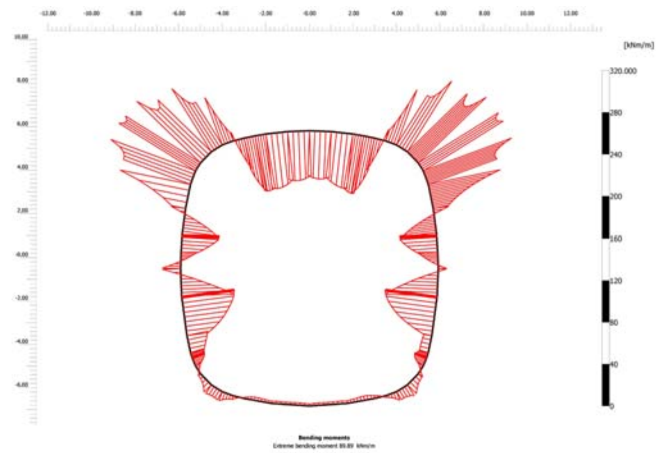


Fig. 15. Bending Moment Distribution (support system type D)

which allows for the controlled deformation of the surrounding area and the mobilizing the self-supported arch zone. The placement of supports (nail and micropile) is timed correctly to ensure that the plastic zone does not propagate beyond a controllable limit. Each part of Fig. 16 represents a different stage of excavation and support installation, with corresponding changes in the plastic zones.

a and b) After excavation of side drifts a significant plastic deformation is created at the middle drift and bench of the tunnel. However, by removing the upper part of the middle drift the plastic point pattern has changed and concentrated under the side walls (Fig. 16(b)).

c) A noticeable increase in the plastic zones at the top, is becoming visible after cutting the temporary wall. Also, plastic points are developed under the side wall. This impact is recorded during site monitoring too (Fig. 18).

d) By excavating the first and second benches the supports effectively contain the plastic deformation and a noticeable

Table 5. The Results Obtained from Numerical Modeling: Maximum Values

Soil Model	Support System	Deformation at tunnel crown (cm)	Surface settlement (cm)	Bending moment (kN-m)	Shear force (kN)	Axial force (kN)
HS	B-III	4.8	4.4	134	130	558
	B-IV	3.5	3.2	91	120	666
	C-I	3.4	3.0	112	128	750
	C-II	3.8	3.6	134	144	665
	D	2.9	2.7	90	122	708
MC	B-III	4.5	3.8	150	145	400
	B-IV	2.8	1.8	87	110	550
	C-I	2.4	1.6	100	126	646
	C-II	3.2	2.4	130	138	500
	D	2.0	1.4	80	120	610
HSS	B-III	3.5	3.0	119.4	120.6	540.7
	B-IV	2.8	2.4	76.6	110.6	631
	C-I	3.2	2.8	238	182	718
	C-II	2.8	2.3	111.4	125.6	636.1
	D	2.2	2.1	77.2	115	682.2

Table 6. Safety Factors of Examined Options

Support system	Class	HS	MC	HSS
Lattice + shotcrete + nailing	B-III	2.46	3.02	2.6
Lattice + shotcrete+ forepoling + nailing	B-IV	2.57	3.05	2.63
Lattice + shotcrete + forepoling + micropile	C-I	3.08	3.3	3.5
Lattice + shotcrete + nailing + micropile	C-II	3.14	3.53	3.3
Lattice + shotcrete + forepoling+ nailing + micropile	D	3.31	3.73	3.8

growth in the plastic zones under the side walls occurs.

e) In the last bench the plastic zones have diminished significantly, with only a few remaining at the lower sidewalls. This suggests that the tunnel is nearing stability, but attention is still needed for the lower regions.

f) It shows the plastic zones have been minimized to a few isolated points at the invert of the tunnel. The rest of the tunnel appears stable with the support system effectively controlling the deformation and maintaining the integrity of the surrounding ground.

Figure 17 shows the impact of excavation stages and temporary walls on surface settlement. This effect was also recorded during construction (Fig. 18).

In the settlement measurements above the tunnel, absolute values of maximum settlement correspond with the results of the numerical analysis (e.g., for the removal of the temporary wall: 1 cm measured settlement). It should be noted that almost in all stages, a significant increase in settlements (change in slope) occurs at an earlier stage in the results of FE analysis than in the monitoring diagram. This issue can be due to a lack of considering time effects in the analyses and assuming all settlements occur instantaneously in the related phase. However, in practice, deformation and settlements are time dependent.

The results of the FE analysis show in total 2.7, 1.5, and 2. cm vertical displacement for Hardening Soil, Mohr-Coulomb, and Small Strain Hardening Soil models, respectively. According to the on-site settlement points, vertical displacements in the ground surface were recorded to be max. 2.6 cm (Fig. 18). The total value of the settlements is chosen to display increasing subsidence due to excavation stages.

The FE analysis using the HSS model closely tracks the instrument data up to a certain point before diverging, which may be attributed to its sensitivity to the small strain stiffness of soil, providing a more accurate early-stage prediction. The HS model predictions maintain a conservative profile, underestimating the deformations throughout the recording period, which could be due to its less refined representation of early-stage soil stiffness. The MC model's predictions are consistently lower than the recorded data, which could lead to an unsafe design if not corrected. This underestimation is likely to be due to the simplistic approach of the model, which does not take into account the complexities of soil behavior under load/unload conditions due to tunnel excavation.

In the diagrams related to instruments SN1 and SS1 (Fig. 19), the numerical results of both sides (northern and southern drifts) are almost the same.

The FE analysis using the HSS model demonstrates a closer alignment with the SN1 instrument data, particularly in the initial and mid-phases of the recording. This suggests that the HSS

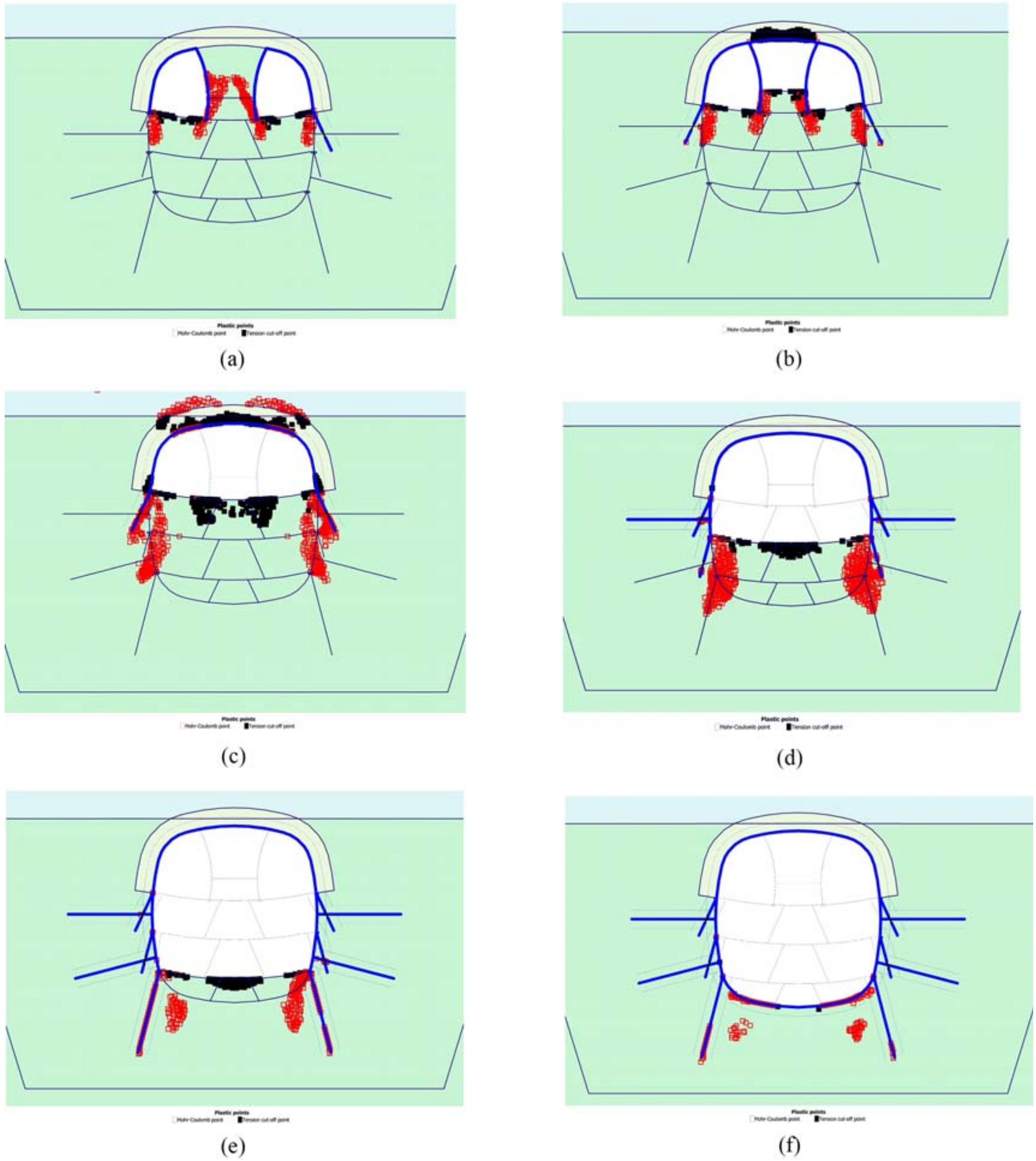
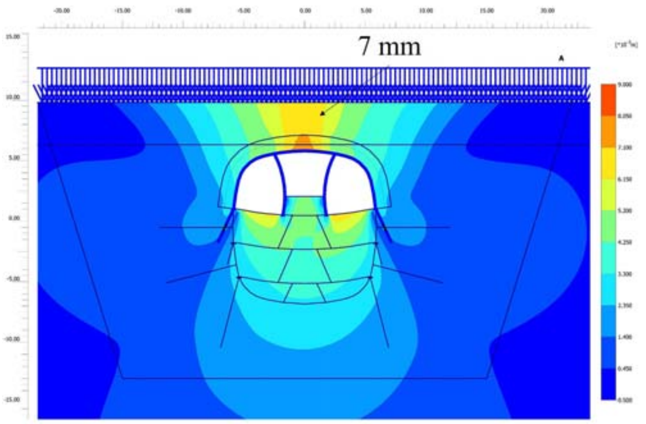


Fig. 16. Developing the Plastic Zone in Different Stages: (a) Excavation the Side Drifts, (b) Excavation the Upper Part of Middle Drift, (c) Removing Temporary Walls, (d) Excavation First Bench, (e) Excavation Second Bench, (f) End of the Construction Phases

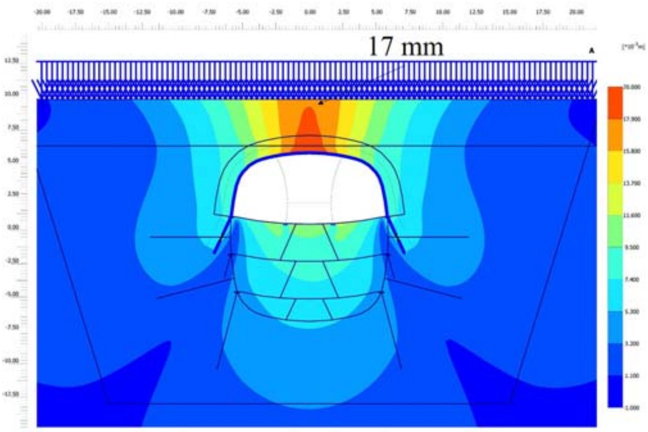
model effectively captures the soil behavior's stiffness at small strains, which is critical in the initial stages of tunnel excavation where such strains are most prevalent. The HS model shows a conservative trend, with displacements less than those recorded by the SN1 instrument. This may indicate that while the HS model accounts for strain hardening, it might underestimate the soil's initial stiffness. The MC model shows the least correlation with the actual recorded data, particularly in the early stages, highlighting its limitations in accurately modeling non-linear soil behavior near the surface. The discrepancy between the FE results and SS1 instrument data is more pronounced here. The HS and HSS models both predict a gradual increase in displacement that somewhat mirrors the trend of the instrument data but deviates in magnitude, especially after removing the temporary wall (beyond the 100 recording mark).

The MC model, again, shows significant deviation from the actual recorded data, particularly after the initial excavation stages, underestimating the displacements. This might be due to the MC model's inability to capture the combined soil-structure interaction and the effect of stress path changes during excavation.

Since an equal relaxation factor is applied for both sides in numerical modeling, similar displacement values are also obtained, and the impact of the excavation order is not observed. In order to correct this shortcoming in 2D modeling, one can assign greater values of relaxation factor to the face, which is excavated earlier. Therefore, the assumption of an equal relaxation factor in

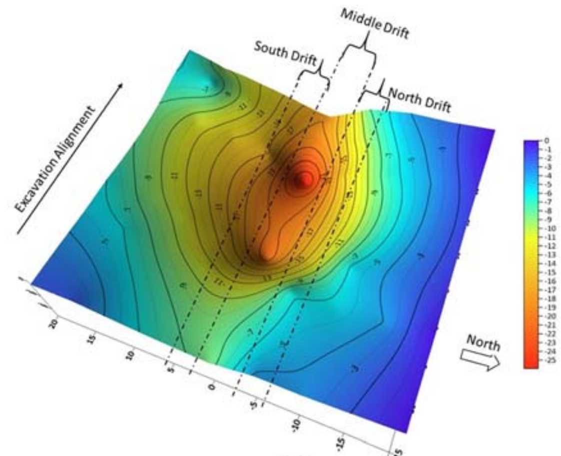


(a)

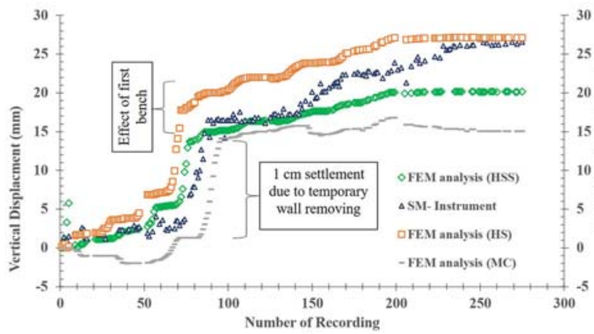


(b)

Fig. 17. Effect of Removing the Temporary Wall on Vertical Displacement, Calculation Results (HS): (a) Before Removing the Temporary Lattice – Surface Settlement 0.7 cm, (b) After Removing the Temporary Lattice - Surface Settlement 1.7 cm

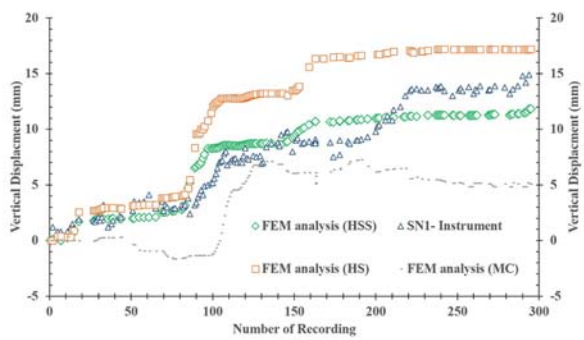


(a)

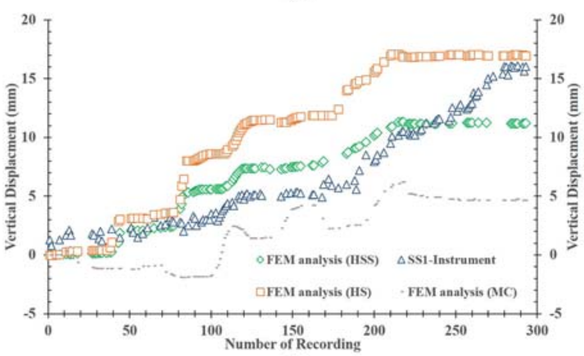


(b)

Fig. 18. Surface Settlement: (a) Normalized Charts 3D Plot Displacements Recording, (b) Comparison of Onsite Measurements and FE Results in SM Station (above the tunnel axis)



(a)



(b)

Fig. 19. Comparing the Measured Settlements with FE Modeling Results at the Final Stage of Excavation: (a) SN1, (b) SS1

two according to excavation sections is not sufficient to model effects related to excavation order and would require further investigations, especially on the role of the stress reduction factor. The almost linear slope at the end of the graphs shows the low effects of the last steps on surface settlements.

Based on the finding in this study, Mohr-Coulomb model can provide reasonably accurate predictions for simple tunnel geometries and shallow tunnels, however, it may not capture the complexities of tunneling-induced stress redistribution and deformations accurately in all cases. In Figs. 18 and 19, the prediction made by this model in the early stages of excavation is completely different from the results recorded by the instruments and can be extremely misleading. Also, the final value of the settlement is underestimated as shown by the 15 mm in tunnel axis. Such interpretation can be generalized to the induced loads, however, in this study there is no suitable instrument to justify the latest claim. On the other hand, Hardening Soil, due to its ability to consider the non-linear stress-strain behavior, strain hardening, and a more accurate representation of soil behavior (e.g., different stiffnesses for loading and unloading/reloading), provides more reliable predictions of tunnel deformations in complex tunnels. It is evident that still there is a difference between the actual behavior of the ground and simulated situation. But the changes in ground settlement are well simulated and even the effect of removing the temporary wall is visible in Fig. 18. The shift between the actual deformation and FE result (in HS cases) can be due to simplifications for 2D analysis, such as chosen relaxation factors. Therefore, in future research, three-dimensional simulation can be used in order to reach more accurate results.

6. Conclusions

This study examines Arash-Esfandiar's two-level tunnel as a case study. The results of analysis and field data indicate the significant impact of tunnel geometric shape on stability and appropriate stress distribution.

The research combined the use of axial and bending support elements in the form of shotcrete, lattice girders, fore-poling, nails, and micro-piles for large span tunnels with a shallow overburden. This composite system is a breakthrough in addressing the high bending moment and structural requirements such tunnels. The investigation reveals that considering conventional support systems alone are inadequate for large-span tunnels with shallow overburden. The research validates the implementation of a composite support system, reducing surface settlement to a minimum of 2.7 cm, as opposed to a potential collapse with conventional methods.

The application and comparison of linear (Mohr-Coulomb) and non-linear (Hardening Soil and Hardening Soil with small strain stiffness) constitutive soil models offer an understanding of soil behavior in response to tunneling. The Mohr-Coulomb model's limitations are quantitatively showcased, with it consistently underestimating displacements across various stages of excavation, sometimes by as much as 1.5 cm at the tunnel axis. In contrast, the Hardening Soil model and the Hardening Soil with small strain stiffness models demonstrate closer agreement with field measurements, indicating their superior predictive capability.

Finite element analysis using the HSS model correlates closely with the on-site instrument data in the early and mid-phases, particularly in capturing the non-linear stiffness behavior of the soil at small strains. On the other hand, The HS model's predictions are aligned with the actual vertical displacements recorded on-site, with a maximum discrepancy of around 0.1 cm, substantiating its applicability in tunnel excavation scenarios.

The study quantitatively illustrates the effect of sequential excavation on ground settlement. In particular, it shows that after the removal of the temporary wall, the FE analysis and site records indicate an increase in settlement of up to 1 cm.

Accurate tunnel modeling requires 3D analysis, and field studies, especially for complex geometries and support systems. While 2D modeling is sufficient for simpler tunnels and stable geometries, the intricate nature of quasi-rectangular sections with high sidewalls requires the precision of 3D modeling. The use of monitoring tools such as extensometers, load cells, and stress cells, together with careful planning, and design, will improve project outcomes.

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