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Simulation techniques and 3D modeling of TBM tunneling in soft and shallow grounds: A case study of Tehran metro line 6

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Abstract

In modern urbanized areas, the increasing demand for underground structures has led to an increase in the number of tunnelling projects in soft and shallow grounds. Tunneling in such conditions poses significant challenges, primarily due to the ground's low shear capacity and high deformation potential, leading to tunnel face instability and surface settlement. Accurate calculation of surface settlement is crucial in order to prevent probable damage to nearby structures. Consequently, various field measurement techniques and analytical solutions have been developed to estimate the maximum surface settlement resulting from tunneling activities in urban areas. These methods provide valuable insights to mitigate the risks associated with excessive settlement, ensuring the integrity and safety of surrounding infrastructure. Therefore, this study employs the numerical approach to gain a comprehensive understanding on surface settlement and the tunneling operation with tunnel boring machines. The analysis considers various performance parameters, including applying pressure around the shield, grout pressure, face pressure, parameters of machine advancement, and shield conicity. The aim of this research is to compare the results obtained from the numerical model with those obtained from precision instruments. Firstly, a comprehensive background on the importance of the considered subject is outlined, and the procedure of numerical modeling is elaborated in detail which is followed by outputs, comparisons, discussion and conclusions. The findings of this study demonstrate that the difference between surface settlement obtained from the results of instrumentation and numerical modeling for the eight investigated sections was an average of 11%, which was deemed negligible. These results highlight the potential of three-dimensional modeling and simulation techniques to support accurate estimation of surface settlement in tunneling operations, thereby minimizing potential damage to the surrounding structures. The reliable assumptions and simulation techniques used in this research can be useful for future tunneling projects to reduce the construction costs.

Keywords: Tunneling, Surface settlement, Soft grounds, Numerical modeling

1. Introduction

In recent decades, the use of earth pressure balancing shields has become a common method for tunneling in urban areas due to its ability to reduce settlements, high excavation rate, and safety. This machine allows for excavation below the underground water level by utilizing the excavated soils to create balance in the tunnel face. However, like any other tunneling machine, the earth pressure balance machine can also cause settlement in loose ground [1]. Hence, it is essential to conduct a comprehensive geological study in the area by performing borehole investigations to ascertain crucial ground parameters, including physical and mechanical properties as well as the underground water level. Moreover, technical factors such as tunnel depth, geometry, and diameter must be carefully evaluated. During the tunnel construction process, both financial and technical aspects need to be taken into account to ensure cost-effectiveness while minimizing settlement in the surrounding region. Furthermore, operational parameters, including excavation method, face pressure, grout pressure, advance rate, tunnel lining resistance, excavation sequence, and thrust force, play a pivotal role in influencing ground deformations and should be meticulously considered.

Given the numerous variables influencing the magnitude of surface settlement, such as soil stratification, tunnel depth, and dimensions, researchers in this field have proposed various relationships categorized into three main groups: empirical, analytical, and numerical methods. These methods aim to provide accurate predictions of the maximum surface settlement. Moreover, several parameters play a crucial role in the precise estimation of surface settlement. These parameters include cohesion, internal friction angle, Poisson's ratio, Young's modulus, depth of the groundwater level, and face pressure. By considering and incorporating these parameters

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into the analysis, it is possible to obtain reliable predictions of the maximum surface settlement induced by tunneling activities. The following methods can be listed as approaches for predicting surface settlement: (i) Formulas derived from empirical methods are primarily based on the collective knowledge gained from numerous tunneling projects and their associated experiences [2-6]. (ii) Analytical method, useful for predicting short-term maximum surface settlements, but cannot accommodate all important parameters [7-15]. (iii) Due to the complexity of the formulas involved in empirical analysis [6, 16-20], numerical methods employing finite difference and finite element techniques have gained widespread use in the fields of soil mechanics and rock mechanics [21-25]. In order to obtain precise results, it is necessary to take into account a greater number of parameters [24, 26-32]. (iv) In recent years, the application of artificial neural networks has emerged as a valuable tool across various engineering disciplines. This approach can be employed to predict tunnel settlement based on the geo-mechanical characteristics of the surrounding ground [27, 33-37]. However, existing studies in this field often consider only a limited set of influential parameters, and further investigation is required to comprehensively analyze surface settlement. To accurately predict surface settlements, it is essential to consider various factors that can influence them, such as the characteristics of soil layers, tunnel dimensions, and excavation methods. Over the years, researchers in this field have proposed various relationships and formulas to estimate surface settlement values. However, the effectiveness of each relationship depends on the accuracy of the input parameters used in the calculation. All theories related to calculating surface settlements indicate a relationship between the volume of soil that has been weakened as a result of tunnel excavation and fills the cavity, and the volume of settled mass on the surface. Therefore, it is crucial to carefully analyze and quantify the relevant parameters when applying these relationships to real-world tunneling projects.

In this study, a full 3D model with all details was built to simulate the TBM tunneling process and compare the settlement results for eight sections with those obtained from monitoring instruments. It is carried out by focusing on assumptions and simulation techniques to obtain the most precise results. The validation of the numerical models was conducted using instrument data from the southern extension of line 6 of Tehran metro, as presented in this paper.

2. Description of the considered project

The southern development plan of Tehran metro line 6 project holds significant importance for the city of Tehran, given its extensive route, complex geological conditions, and strategic location by passing through a densely populated and historic urban area. Therefore, controlling probable settlement and adherence to safety guidelines during excavation was crucial. The excavation of this tunnel is carried out using a Herrenknecht earth pressure balance (EPB) machine with an excavation diameter of 9.19 m for a total length of 6.6 km route (Figures 1 and 2).

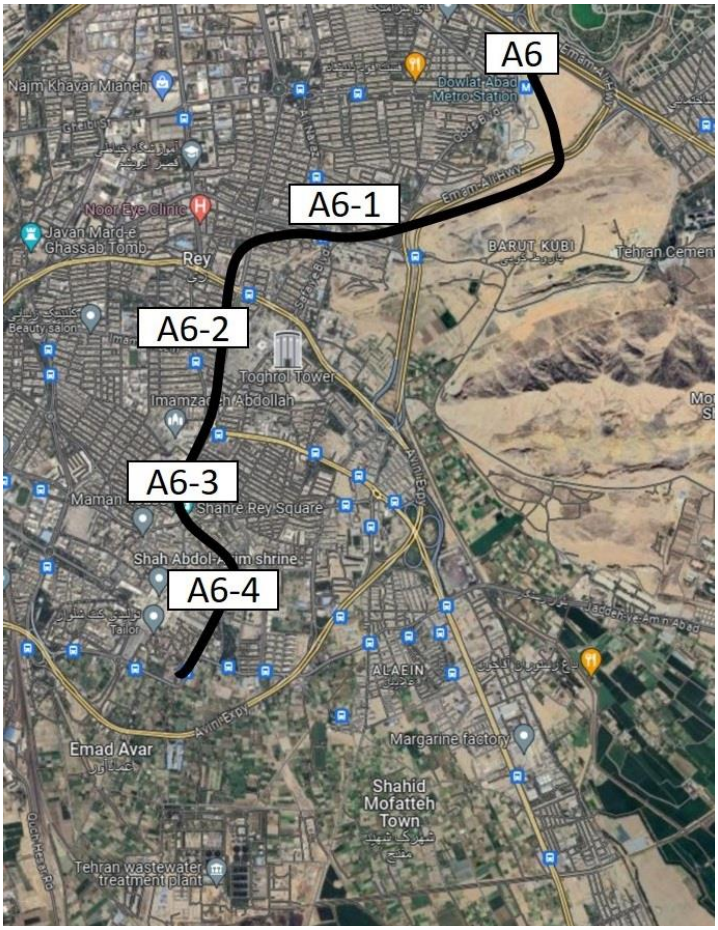

Figure 1 Route of the Tunnel

According to Figure 1, the southern extension of this line commences from Dolat Abad station (A6) and follows an arc-shaped path, passing through four stations (A6-1 to A6-4), ultimately terminating in the southern vicinity of Abd Al-Azim Haram.

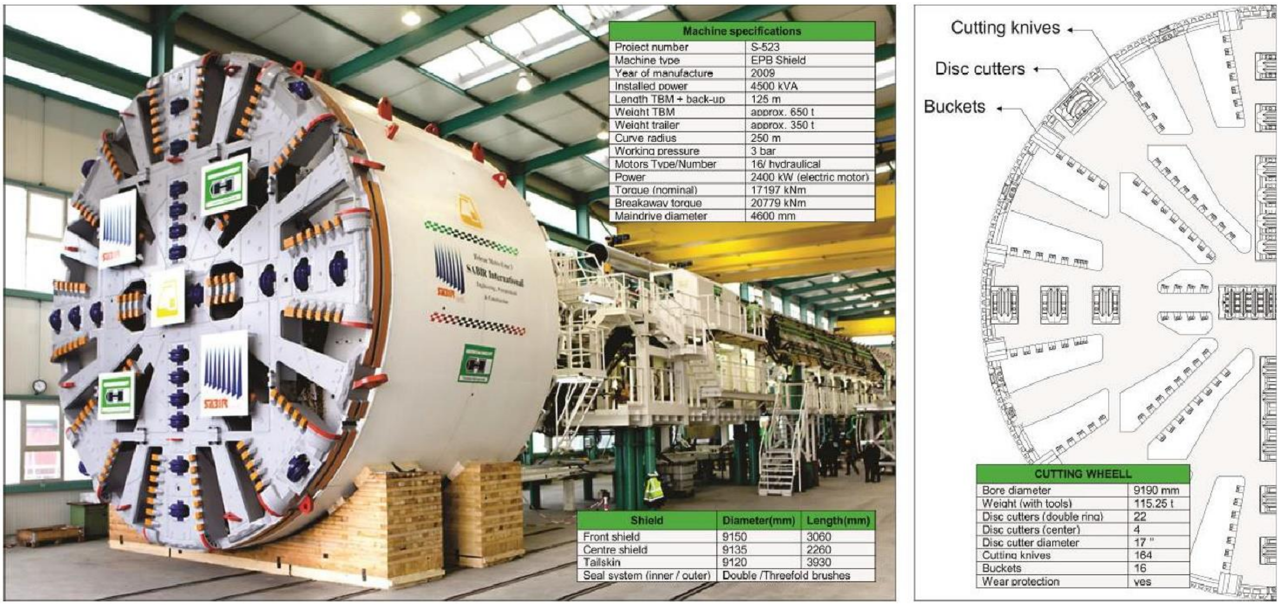

Figure 2 Herrenknecht EPB TBM considered for the studied case [38].

A total of 595 meters of exploration drilling were excavated in 20 boreholes (BH) around the tunnel alignment and different sites. In addition, comprehensive geotechnical tests, including; standard penetration, permeability, uniaxial compressive strength, triaxial compressive strength, shear strength, sieve analysis, and physical properties of the soil samples were employed to differentiate each distinctive site. Based on the results of field and laboratory studies, the soil layers along the tunnel route have been classified into four engineering geological units (Table 1). Additionally, hydrogeological investigations of the tunnel route show that the depth of the water table increases from north to south. It is anticipated that water will enter the tunnel from 640 meters onwards. In rock-dominated areas, the difference between the water level and the tunnel was roughly 8 meters; however, from chainage 2400 to 3300 the underground water level is at least 10 meters above it.

Table 1 The range of values for engineering geological units along the tunnel route [37].

Items/ Unit Name	ET-1	ET-2	ET-3	ET-4
Passing sieve no. 200	<15 %	<30 & >10 %	<60 & >25 %	>50 %
Passing sieve no. 4	>45%	<50 & >20 %	<50 & >10 %	<20 %
Soil type (USCS standard)	GW, GW-GM, GP-GC, SW & SP	SC, SC-SM & GC	SC, SM & CL	CL, ML & CL-ML
Cohesion (kPa)	19–24	30–35	35–40	40–50
Friction angle (deg.)	38–42	35–40	25–32	18–22
Young modulus (MPa)	50–60	50–60	45–55	40–50
Poisson ratio	0.3	0.32	0.32	0.35
Unit weight (gr/cm3)	2.05	1.95	1.95	1.8

3. Materials and methods

3.1 Numerical modelling

To determine suitable sections for validation, critical sections were first identified along the tunnel route for analysis and modeling. The selection of appropriate sections, along with the correct assessment of geotechnical conditions and surface loads, is crucial to gaining a comprehensive and accurate understanding of ground settlement in the selected areas. This approach allows for the identification of sections with a higher potential for settlement, as well as critical areas that require particular attention during design and implementation operations. To select critical sections along the southern expansion route of Tehran metro line 6, the guidelines and recommendations provided in reference [39] were utilized. This involves considering factors such as maximum and minimum surcharge, minimum and maximum groundwater height level, the weakest soil unit of the tunnel, surface and subsurface structures in the tunnel area, and minimum and maximum loads applied to the tunnel. A numerical simulation was performed to investigate ground surface settlement during excavation using an earth pressure balance shield device. The simulation was conducted using the finite element method and PLAXIS 3D software. The soil was assumed to be homogeneous and isotropic, with a fully plastic-elastic Mohr-Coulomb failure criterion and a hardening soil resistance criterion. The lining rings and the machine shield were modeled using an elastic behavior. To avoid repetition and due to similar steps in the modeling process, only one section was fully investigated, including the numerical modeling methodology, the application of face and grout pressure, the constitutive model used, and whether the state was drained or undrained.

The EPB machines are equipped with pressure gauge sensors installed on the pressure wall that indicate the soil pressure inside the excavation chamber. The operator observes the increase or decrease in pressure and adjusts the advance rate or rotation of the spiral conveyor to bring the pressure closer to the set value. Figure 3 illustrates the location of the pressure gauge sensors related to grout pressure and face pressure on the pressure wall. EP1 to EP7 are sensors related to face pressure, and P1 to P7 are sensors related to grout pressure. Table 2 and Table 3 provides complete information about the parameters related to the grout pressure and working face pressure applied in this section, as well as the geometric and geotechnical parameters. The information related to this section and all sections is extracted from the TBM data logger file. It should be noted that the central lines of the tunnel of line 6 are located at section 3320, which is 27.88 meters below the surface (the depth of the tunnel axis).

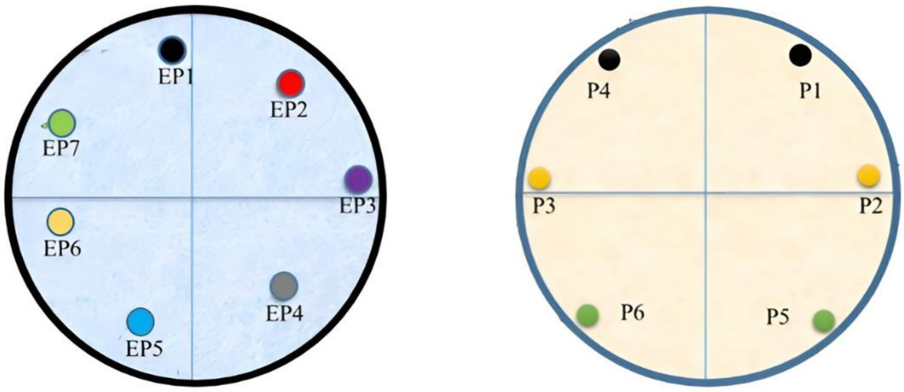

Figure 3 Positioning of face pressure and grout pressure sensors in the southern expansion project of Tehran metro line 6 [40]

Table 2 Associate pressure in EP1, EP2, EP3, P1, P2 and P3

Settlement (mm)	EP1 (bar)	EP2 (bar)	EP3 (bar)	P1 (bar)	P2 (bar)	P3 (bar)
21	1.15	1.54	2.04	2.3	1.8	2.3

Table 3 Specifications of section 3320 extracted from TBM data logger

Z_0 (m)	C (m)	H_w (m)	E (MPa)	c (KPa)	φ deg.	V_L (%)	I
27.88	23.28	14.24	50	40	20	1.04	13.2

3.1.1 Geometry and modeling of section 3320

The constructed model exhibits dimensions of 100 meters in the y-direction, 40 meters in the z-direction (height), and 50 meters in the x-direction (width). These dimensions are chosen to ensure that the model size is at least 10 times greater than the tunnel radius. Specifically, in section 3320, the overburden measures 23.285 meters, which is twice the diameter of the tunnel (H > 2D). It is important to note that this section lies below the groundwater level with two soil layers (ET-4 and ET-3). The underground water level in the studied area is 14.24 meters below the ground surface, which is considered as pore water pressure in the modeling. Due to the axial symmetry and the circular structure of the tunnel, as well as in order to increase the modeling speed, only half of the desired model has been simulated. The dimensions of the model are considered in such a way as to prevent the influence of model boundaries on the obtained results and prevent the increase of calculation time. Figures 4 and 5 depict the boundary conditions, geometry and layering. Figure 6 also illustrates the meshing of the constructed model.

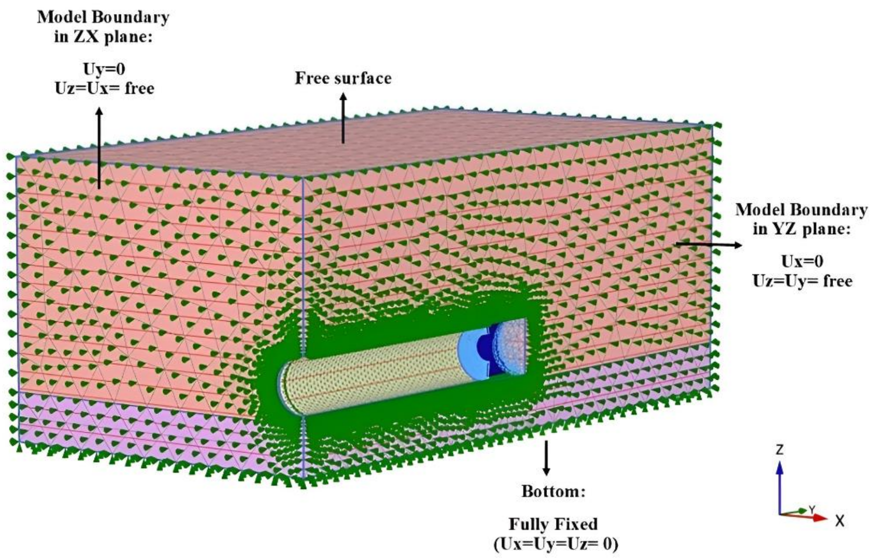

Figure 4 Boundary conditions of the model

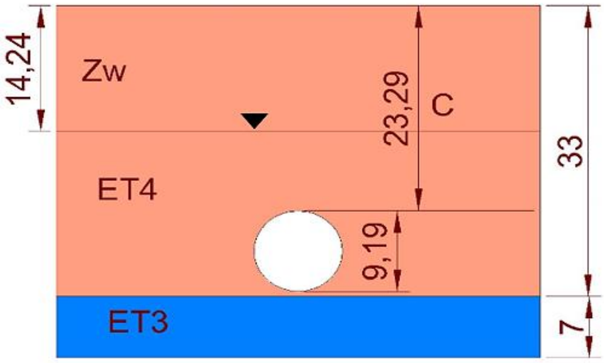

Figure 5 Layering and geological conditions of section 3320

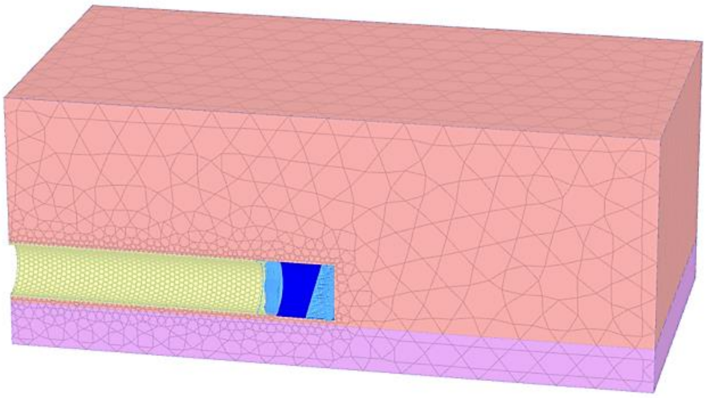

Figure 6 Meshing of the model created from section 3320

3.1.2 Boundary conditions

The boundary conditions of a numerical model involve field variables, such as stress and displacement. In the program, the bottom and sides of the model are fixed by selecting appropriate boundary conditions. The boundary condition is applied as a roller support on the edge of the geometry, and along the x-axis, U_x=0. The boundary conditions for structural plates, such as a shield, are designed to act as roller supports. In order to achieve this, the four sides of the model, denoted as the artificial lateral boundaries (-x, x, y, -y), as well as the bottom of the model (z = -40), are constrained using roller boundaries. This means that these boundaries allow only rotational displacements, while preventing any translational movements. Conversely, the upper part of the model remains unrestricted, allowing for free displacement.

3.1.3 Considered parameters

Table 4 presents the considered soil design parameters related to section 3320, along with the soil characteristics. Table 5 shows the properties of the EPB machine and tunnel segment. Once the model geometry and boundary conditions have been established, the different layers within the environment should be identified. Based on the geotechnical section and provided information, the subsurface sediments have been divided into two layers. For this study, the soil behavior has been assumed to follow the hardening soil behavior.

Table 4 Soil properties of section 3320

γ (gr/cm3)		**Young modulus (MPa)**	**Internal friction angle (φ)**	**Cohesion (kPa)**	**Engineering geology unit**
γ sat	**γ d**				
1.95	1.90	50	28	38	ET-3
1.80	1.75	45	20	45	ET-4

Table 5 Segment specifications and EPB machine characteristics.

Parameters	**γ (KN/m3)**	**d (cm)**	**Y**	**E (MPa)**
Machine Shield	78.5	17	0.2	200000
Segment	25	35	0.2	30000

3.1.4 Preliminary conditions

After creating the geometry of the model, meshing, and assigning the soil parameters, it is important to create the initial conditions of the ground by applying the gravity and water level to induce the same stress level in soil as reality. Table 6 illustrates the height of the water level from the tunnel bottom, the overburden, the pore water pressure, and the effective stress of the soil at the bottom of the model. Figures 7 and 8 show the amount of effective soil stress and pore water pressure calculated by PLAXIS software for 3+320 section, respectively. As it is clear in Figures 4 and 5, the amount of effective stress and pore water pressure is higher in the bottom of the model.

Table 6 Initial conditions of the modeled section of Tehran metro line 6 tunnel

Effective soil pressure at the bottom of the model (kPa)	Pore water pressure in the bottom of the model (kPa)	Overburden height (m)	The height of the water from the bottom of the tunnel (m)	Tunnel mileage
475	260	23.285	18.235	3+320

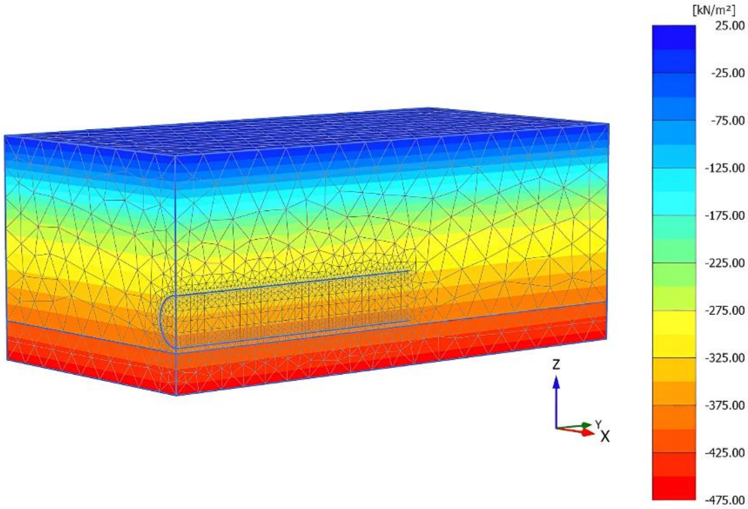

Figure 7 in-situ stresses induced in soil before tunneling

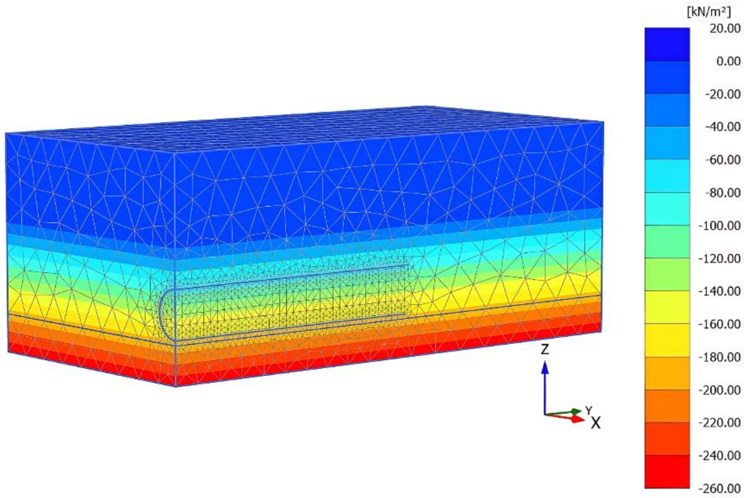

Figure 8 Water pore pressure calculated in model

3.1.5 Defining steps and modeling stages

In engineering operations, a project is often divided into several phases. Similarly, a calculation operation in PLAXIS is also divided into several phases. The modeling and tunnel excavation stages are considered for a 50-meter excavation. In the first phase of calculations, after the displacements are set to zero, the model reaches its initial equilibrium. It should also be noted that there were no building and traffic to consider their associated loads in the model. The drilling step of 1.5 meters is considered according to the width of the segment. The length of the model elements along the tunnel excavation (y) is set to match the width of a 1.5-meter maintenance ring. The reinforced concrete cover and the cement grout surrounding the tunnel segment are modeled using 8-node elements with linearly elastic behavior and shell structural elements. The pressure distribution on the tunnel face exhibits a broad linear load with a certain gradient, increasing from top to bottom in the form of a triangular distribution. The amount of pressure that is applied at the crown of the tunnel is considered as the minimum pressure, while the maximum pressure applied is relevant to the lowest part of the tunnel. The pressure values in the center and bottom of the tunnel are depicted in Figure 9, where the pressure at the center is 175.4 kPa, and at the bottom, it is 248.92 kPa.

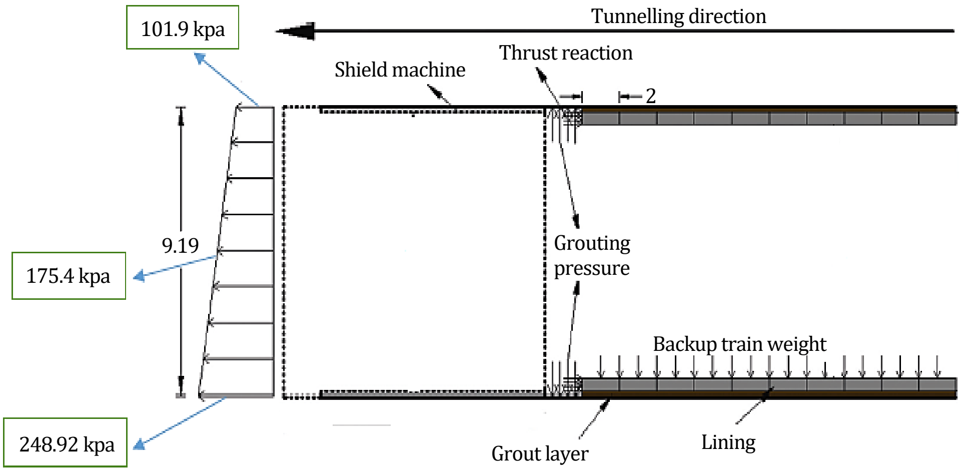

Figure 9 Schematic of TBM and face pressure values

The grout pressure is the pressure required to inject the cement grout into the ground. As it was mentioned, the grout pressure values for the center and bottom of the tunnel are 275.3 and 330.44 kPa, respectively. These values can be seen in Figure 10, which is a graphical representation of the pressure distribution in the tunnel. The grout pressure is an important factor in ensuring the stability of the tunnel and preventing the inflow of groundwater. Therefore, it is crucial to accurately determine the grout pressure and monitor it throughout the construction process to ensure the safety of the tunnel.

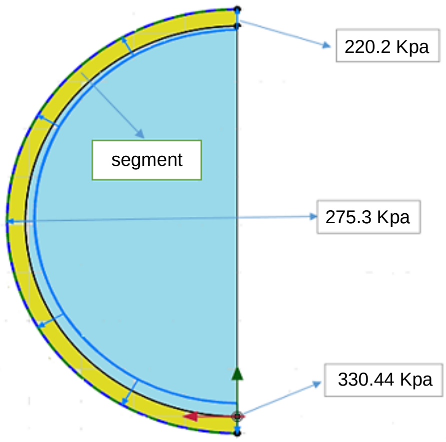

Figure 10 Schematic of triangular distribution of grout pressure modelling stages

To simulate the stepped state behind the shield and its conical shape, the axial increment parameter is used, which is applied at 1 meter from the shield and negatively along the length of the model. For this project, the axial increment parameter value is 0.0466, which takes into account the length of the shield (9 meters), the conical shape of the shield, and the drilling step of 1.5 meters. The shield enters the tunnel completely in six steps. Therefore, the simulation is performed in the same way until the sixth stage, when the 9-meter tunnel is excavated. However, in each stage, the amount of additional excavation and the conical shape of the shield must be accurately modeled. The construction parameter starts from zero for the first 1.5 meters and increases by 0.07 at each stage, reaching the value of 0.35 at the sixth stage. The calculations are also carried out using a step-by-step method, which stops when soil yielding occurs. To apply water pore pressure in the excavated cross-section, it is necessary for the tunnel to be free of water. At this stage, the water inside the tunnel is deactivated, allowing the tunnel to be dry, and a new pore pressure is then applied to the model. In the seventh step, the shield advances 1.5 meters, resulting in 10.5 meters of excavation. The seventh drilling section and the shield element in the first section are deactivated, while the shield element in the seventh section is activated. At this stage, the length of the shield is 9 meters and grouting is performed behind the shield. Additionally, the face pressure in front of the drilled section is applied. In the final stage, in addition to advancing the tunnel, grout pressure and segmentation operations are performed, resulting in 12 meters of drilling. The eighth drilling section and the shield element in the second section are deactivated, and the segment element in the first section and the shield element of the drilling machine in the eighth section are activated. At this point, the length of the shield is 9 meters, and grout pressure is applied behind the shield. Additionally, at least the face pressure in front of the drilled section is applied. The modeling process is continued up to stage 34, which corresponds to a tunnel excavation of 51 meters in this case. Figures 11 and 12 illustrate the different stages of drilling, as well as general schematics of grout and face pressure.

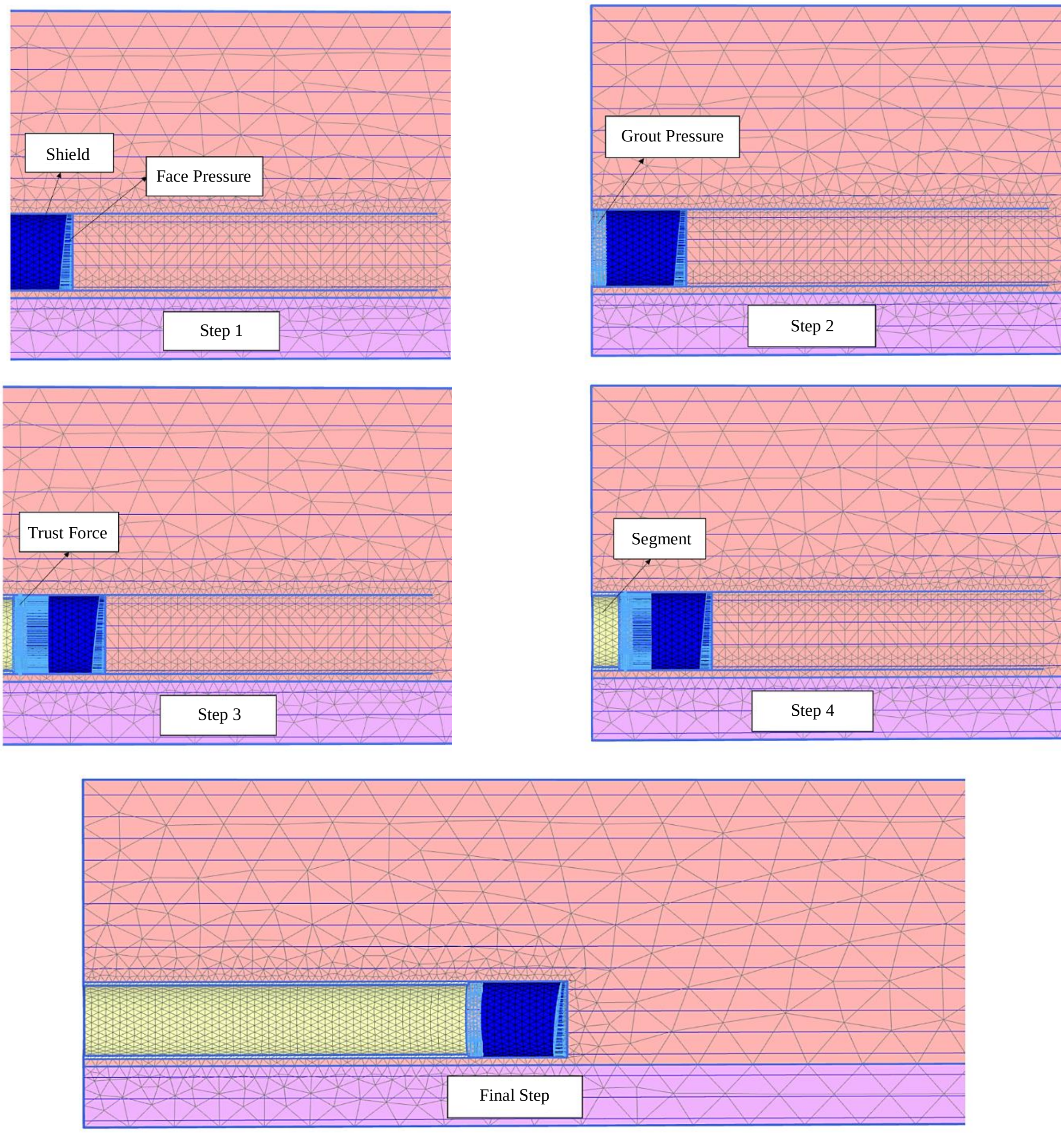

Figure 11 Simulation of mechanized excavation, face pressure, grout pressure, jack pressure, and concrete cover (segment) after 50 meters of excavation advancement.

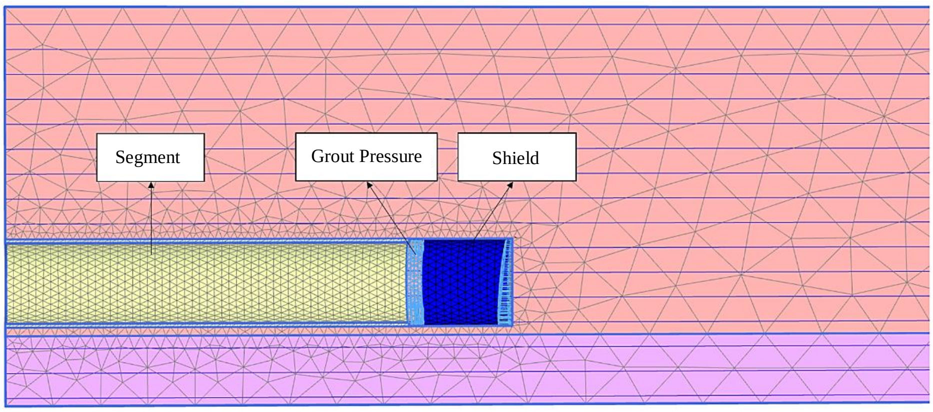

Figure 12 Schematic of shield, grout pressure and segmental support after 50 meters of tunnel excavation

The focus of this study is to examine the settlement of the section and displacement; therefore, the outputs are of the displacement type. Figures 13 and 14 illustrate the outputs related to vertical displacement and horizontal displacement of the modeling for the 3+320 km tunnel.

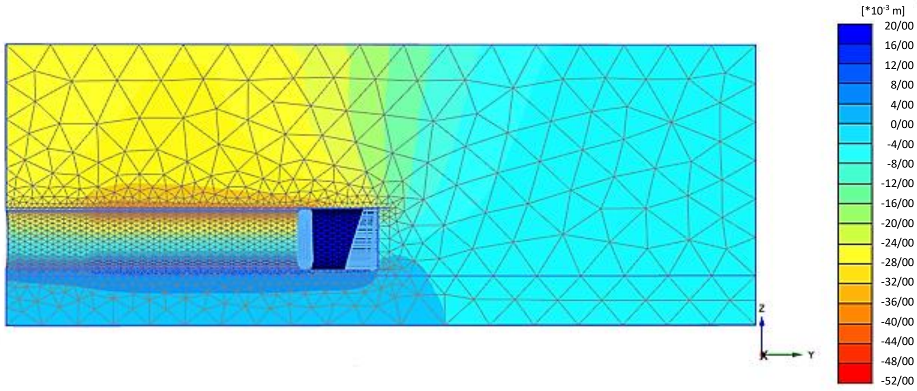

Figure 13 Vertical displacement contour after advancing 50 meters with face pressure of 101.9 kPa and grout pressure of 220.2 kPa

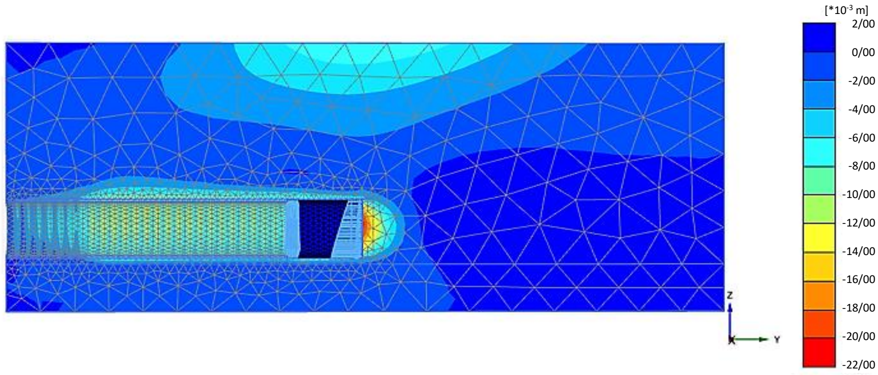

Figure 14 Horizontal displacement contour after advancing 50 meters with face pressure of 101.9 kPa and grout pressure of 220.2 kPa

Alignment method is used to measure the settlement and elevation of the earth's surface. This process involves creating distinctive points on the surface of the earth and continuously reading them before and after the tunnel passes through these points. This enables the vertical displacement of these points to be measured, allowing for the determination of the amount of settlement or elevation of the surface due to tunnel excavation. In urban environments, the surface of the ground is often covered with paving stones or asphalt, which provides structural resistance. Therefore, to accurately record the behavior of the ground and possible settlements, marking points must be placed underneath the surface to avoid errors. In the southern expansion project of Tehran metro line 6, land surface settlement was evaluated using mapping pins. The placement of these pins can be categorized as single-point and multi-point installations. The single-point pins are positioned along the tunnel axis at a minimum interval of 8-15 meters. On the other hand, the multi-point pins are installed perpendicular to the tunnel axis, forming a configuration of either 3 or 5 points, with a spacing of 30-60 meters. The pins were installed before the TBM reached the point under investigation, for example, 50-100 meters in front of the work face, and their recording began. The settlement recording continued until a long distance after the TBM passed, for example, 50 meters behind the face, and the recording continued until relative stability was reached. The accuracy of the measurement with the mapping tool is 0.01 mm.

To verify the accuracy of the numerical modeling process, the 3320 section was used. In this section, engineering and geological information and TBM data logger information of the desired section in line 6 of the southern development have been used. Figure 15 illustrates the maximum settlement values obtained from two numerical models and instrumentation. According to the results, the transverse profile in this section is estimated to experience a maximum settlement of 21 mm and 24 mm in the constructed model. Accordingly, the FEM model predicted the ground settlement with high accuracy.

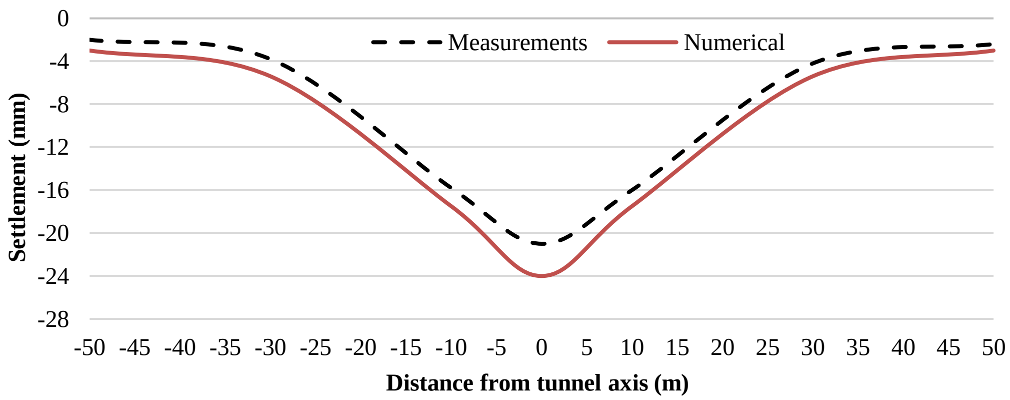

Figure 15 comparison of maximum lateral settlement results for 3+320 km obtained by numerical method and field instruments

Table 7 presents the final settlement values for the whole sections investigated in this study. As shown in Table 7, the average error is about 11 percent, which is quite satisfactory considering the small displacement values. Moreover, in Figure 16 transverse settlement curve is illustrated.

Table 7 Predicted and actual maximum settlement for each section

Section	Surface settlement (mm)		Error (%)
	Actual	Predicted	
2280	14	16.5	15
2475	5	5.4	7.5
2544	7	6	14
3015	8	8.9	10
3240	4	4.35	8
3255	17	15	12
3320	21	24	12.5
3560	34	30	12

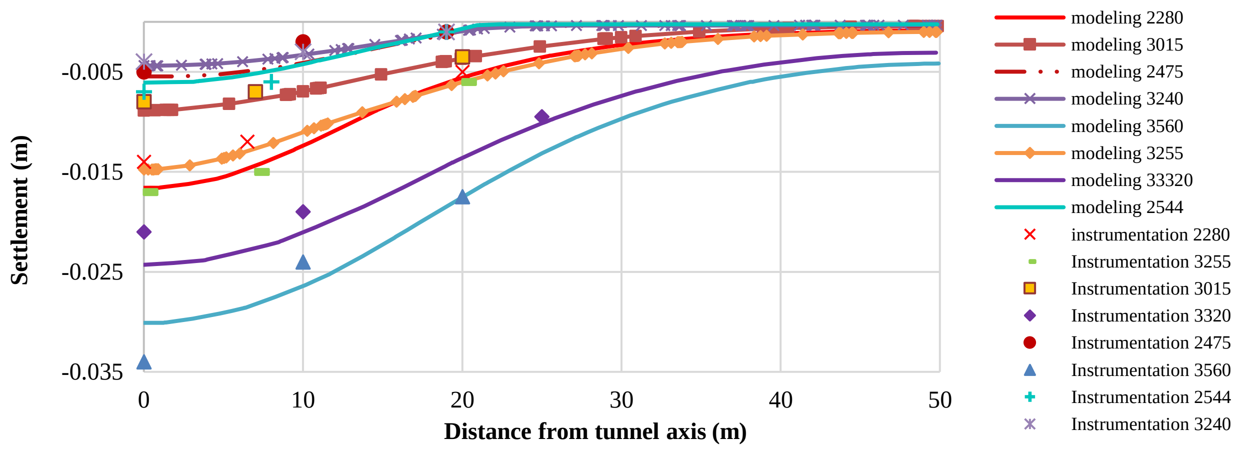

Figure 16 Comparison of transverse profile of modeled section settlement with instrumentation results

3.2 Estimation of surface settlement by analytical and empirical methods

In empirical methods, the ground displacements around tunnels are usually associated with surface settlements. The sources of settlement are investigated separately, and then their effects are combined. The first presented formulas for calculating ground settlements due to tunnel excavation in soft soils were introduced by "Peck" in 1969. Peck believed that the shape of the settlement distribution curve on the ground surface and above the tunnel follows a Gaussian curve, and it can be estimated using field measurements through equation (1) [3].

$$S(x) = s_{max} \exp\left(\frac{-x^2}{2{i_x}^2}\right) \tag{1}$$

In this equation, $S(x)$ represents the settlement at a distance x from the tunnel centerline, Smax is the maximum settlement value at the tunnel axis, and i is the distance from the inflection point of settlement to the tunnel centerline.

Directly using this equation is not possible because, in this relationship, the settlement magnitude at each point of the profile is a function of the inflection point length (i) and the maximum settlement value. Until we have an accurate estimation of these two parameters, we cannot use the provided equation. Numerous studies have been conducted on estimating the maximum settlement parameters and the inflection point length by different researchers. Below, we refer to some of these relationships that have been developed for estimating the inflection point length.

$$\mathrm{i} = \frac{\mathrm{i_1 + i_2 + i_3 + i_4}}{4} \tag{2}$$

$$\mathrm{i_1} = 0.5\mathrm{Z_0} \tag{3}$$

$$\mathrm{i_2} = 0.43\mathrm{Z_0} + 1.1 \tag{4}$$

$$\mathrm{i_3} = \mathrm{R}\left(\frac{\mathrm{Z_0}}{2\mathrm{R}}\right)^{0.8} \tag{5}$$

$$\mathrm{i_4} = 0.9\mathrm{R}\left(\frac{\mathrm{Z_0}}{2\mathrm{R}}\right)^{0.88} \tag{6}$$

As shown in equations (2-6), Peck proposed a dimensionless relationship between the width of settlement trough, i.e., $\frac{i}{R}$, and the tunnel depth, $\frac{Z_0}{2R}$, for tunnels excavated in different materials, where R is the tunnel radius and Z is the depth of the tunnel center from the ground surface. Mair et al. [41] suggested equation (3) for estimating the parameter i in cohesive soils. This relationship was proposed for cohesive soils. O'Reilly and B. New [6] conducted a case study on tunneling projects in clayey soil and obtained the parameter i using linear regression in equation (4) for cohesive soils [6]. Clough and Schmidt [42] proposed equation (5) for determining the parameter i for shield tunneling in clayey soil. Arioglu [19] proposed various relationships for estimating the parameter i based on precise instrument data. He obtained the parameter i for shield machine excavation in different types of soils in equation (6) [19]. Finally, the parameter i was calculated to be 12.3 based on equation (2). In this study, the values of tunnel radius (r) and tunnel axis depth (z_0) were considered 4.595 and 27.88 meters, respectively.

Several empirical relationships have been proposed by different researchers for calculating the maximum surface settlement induced by tunneling. Some of them are mentioned below. For calculating the maximum settlement for individual tunnels, Schmidt [43] suggested equation (7) [4]; where i is related to the inflection point parameter of the settlement curve, and R is the tunnel radius. In this formula, the parameter K is obtained using the relationship provided in equation (8).

$$S_{max} = 0.0125K\left(\frac{R^2}{i}\right) \tag{7}$$

$$K = 0.87e^{0.26(\frac{\gamma_n z_0+\sigma_s-\sigma_T}{c_u})} \tag{8}$$

In which γ_n represents the unit weight of soil (kN/m³), σs is the surface load (kPa), σt is the overburden pressure (kPa), and cu is the undrained shear strength of the soil (kPa). The maximum calculated settlement by this method is determined to be 56.6 millimeters. Another equation that has been proposed for calculating the maximum settlement resulting from tunnel excavation for individual tunnels is the Herzog equation [17]. Herzog [17] suggested equation (9) for calculating the maximum surface settlement. In this equation, E represents the modulus of elasticity (kPa).

$$s_{max} = 0.785(\gamma_n z_0 + \sigma_s)\left(\frac{D^2}{iE}\right) \tag{9}$$

The obtained settlement value from this equation is equal to 53 millimeters. Analytical methods have been developed for calculating settlements based on stress and strain fields. Numerous analytical methods have been proposed regarding the relationships between stress and strain fields. However, analytical methods for predicting ground surface settlement induced by tunneling in soft soils are not very prevalent. Some of the most important relationships proposed for estimating settlements include those presented by Loganathan N and Poulos HG [9], Chi et al. [10], Chou and Bobet [12], Park [13], and Lee et al. [21]. In this study, the Loganathan and Poulos method [9] was used to calculate the surface settlement. Loganathan and Poulos [19] introduced their relationship by modifying the equation proposed by Rowe et al. [4], which was used to calculate ground loss based on the gap parameter. They introduced the equivalent ground loss parameter according to equation (10).

$$S = 4(1-\nu)R^2\left(\frac{Z_0}{{Z_0}^2+x^2}\right)\left(\frac{4gR+g^2}{R^2}\right)exp\left[\frac{-1.38x^2}{(z_0+R)^2}\right] \tag{10}$$

In this equation, ν represents the Poisson's ratio of the soil, R is the tunnel radius, x shows the horizontal distance from the tunnel center, and Z_0 is the vertical distance of the tunnel center from the ground surface. The gap parameter (g) is calculated using equation (11).

$$g = G_P + U_{3D} + w \tag{11}$$

In which G_P represents the annular gap between the tunnel boring machine diameter and the outer diameter of precast concrete segments in circular tunnels, which is defined according to equation (12).

$$G_p = 2\Delta + \delta \tag{12}$$

Where Δ represents the shield thickness of the tunnel boring machine and δ is the required space for installing precast concrete segments. Additionally, in equation (11), w denotes the skill coefficient of personnel, which is typically taken as 0.6 times Gp. U_{3D} represents the three-dimensional elastoplastic deformation of the tunnel lining. Considering that an EPB machine is used for tunnel excavation, this parameter is assumed to be zero. The calculated value of surface settlement obtained by this method is equal to 36 millimeters.

In 2014, Chakeri and Ünver [25] used numerical modeling approach to develop new equation to predict maximum surface settlement;

$$S_{\text{max}} = 3198.744\left(\frac{D}{Z_0}\right) \times ((\frac{\gamma Z_0 + \sigma_s - (c + 0.3\sigma_T)}{E})(1-\nu)(1-\sin\Phi))^{0.8361} \tag{13}$$

in which S_{max} is the maximum surface settlement (mm), D is the tunnel diameter (m), E is the Young's modulus (kPa), ν is the Poisson's ratio, Φ is the angle of internal friction σ_T represents the required face support pressure (kPa) and σ_s presents the surface surcharge (kPa). Similarly, by considering the values according to the considered case study, the surface settlement obtained from this equation is equal to 12 mm which represents a poor estimation.

4. Discussions

In this section, an attempt has been made to compare the settlements obtained from these formulas with the values obtained from the precise instruments and numerical modeling. Among the three formulas presented in the previous section, the Schmidt [43] and Arioglu [19] method shows a higher settlement compared to those obtained using the Loganathan and Poulos [9], and Herzog [17] methods and actual measured values.

Among the mentioned empirical and analytical methods, the Loganathan and Poulos [9] method provides settlements close to the precise instrument values. By accurately determining this parameter, numerical modeling can provide results that are closer to the actual values. It is worth noting that the accuracy of numerical modeling using PLAXIS software is contingent upon the precise calculation of the volume loss parameter. Despite this limitation, the developed model demonstrates its ability to produce reliable and trustworthy results. The settlement values obtained from the different methods used in this study are presented in Table 8, allowing for a direct comparison of their accuracy and suitability for different scenarios.

Table 8 Maximum surface settlement values calculated for three analytical, Empirical and numerical modeling methods for section 3320

	Prediction method	Maximum surface settlement (mm)
Numerical	PLAXIS 3D	24
Analytical	Loganathan and Poulos [9]	36
Theoretical	Arioglu [19] and Schmidt [43]	56.6
	Herzog [17]	53
Measured	Instrumentation	21

According to Table 8, only the numerical method provides a result closer to the actual settlement compared to the other methods, indicating that the numerical modeling has been performed well. It should be noted that the formula proposed by Chakeri and Ünver [25] provides considerably poor prediction and it is not reported in the Table 8.

5. Conclusions

Nowadays, efforts are made to minimize the associated deformation in the surrounding environment of underground structures, especially in densely populated urban areas. One effective technique to prevent tunnel convergence and unauthorized changes in the ground surface is the use of TBMs. Numerical modeling is a reliable method for evaluating and quantitatively interpreting field data in order to assess designs or major construction projects. Although numerical analysis may not be able to model all the boundary conditions and interactions between the soil and tunnels, they could provide fairly acceptable results. In numerical simulation, the main challenge is to create a model with sufficient details considering the main assumptions, boundary conditions and interactions between elements so that the output would be reasonably comparable with real monitored data. The results of settlement values obtained from the numerical modeling of Line 6 of Tehran metro's south development project compared to the results of precise instruments demonstrate that numerical modeling is a useful tool for simulating such complex structures. Due to the time and cost-effectiveness of numerical approaches, most researchers attempt to use 2D numerical software for their analysis. Although these software packages are capable of performing tunneling operations, they are not recommended for tasks that require more realistic results considering the third dimension. In previous studies, the simulation of the surrounding soil often involves the utilization of an elastoplastic behavior model. However, in this particular study, a more advanced hardening soil behavior model was employed to accurately represent the characteristics of the surrounding soil. This behavior model possesses exceptional capabilities in accurately simulating the behavior of both hard and soft soils, making it a robust choice for the analysis.

As it was previously investigated, the numerical simulation performed in this study has the capability to accurately simulate the excavation phases and tunnel lining with good details. Therefore, it can be concluded that the use of such powerful simulation software enhances the predictability of tunneling projects, minimizes construction costs, and assists decision-makers in selecting the optimal solution for future tunnel projects.

In the southern development project of Tehran metro line 6, ground level settlement was evaluated using mapping pins. However, since the instrument reading was taken at the ground level and 50 meters away from the working face of the tunnel, settlement readings were obtained in the numerical modeling after the TBM had advanced 50 meters beyond the tunnel face. The obtained results indicated that the average error percentage in the modeling for eight sections after 50 meters of progress (33 steps of 1.5 meters) compared to the instrumentation was around 11%, which is considered a negligible error.

It is also expected that in the future years to come, by employing applied Machine Learning (ML) and Artificial Intelligence (AI) techniques in the tunneling projects, more opportunities will be available for making predictions and real-time assessments of associate settlements with underground operations.

6. Glossary of abbreviations and acronyms

AI	Artificial Intelligence
BH	Borehole
c	Cohesion
C	Overburden height
E	Elasticity modulus
EP1	Face pressure at point 1
EP2	Face pressure at point 2
EP3	Face pressure at point 3
EPB	Earth Pressure Balance
Hw	Distance between the ground surface and the groundwater level
I	Horizontal distance factor

ML Machine Learning
N Stability number
P1 Grout pressure at point 1
P2 Grout pressure at point 2
P3 Grout pressure at point 3
TBM Tunnel boring machines
USCS Unified Soil Classification System
VL Volume loss
Z0 Tunnel axis depth
φ Internal friction angle

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