The Evaluation of Constitutive Models in Prediction of Surface Settlements in Cohesive Soils – A Case Study: Mashhad Metro Line 2

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ABSTRACT: *The current study is aimed at investigating the basic soil behavior involved in a TBM-EPB excavation and the capability of the Modify Cam Clay (MCC) model is verified for the analysis of the soil settlement in cohesive soils. Tunnel excavation in urban areas can engender considerable ground movements, which is known as one of the complicated issues that may have negative effects on the extant structures. In this paper, the construction of the second line of the Mashhad metro is considered as a case study. Each section of the ground was modeled by two constitutive models, namely MCC and Mohr-Coulomb (MC). Afterwards, the results of numerical analyses and monitoring data were compared with each other. In addition, real parameters of soil, such as volume loss and the inflection point, were obtained via empirical approaches verified by tunnel monitoring. Numerical modeling was performed by FLAC3D software. Based on the transverse and longitudinal sections settlement, the MCC model showed high capabilities of predicting the surface settlement in comparison to the MC model. Finally, using the MCC model was chosen as a rational strategy to predict the soil behavior especially for soft clay with low pre-consolidation ratio or normal consolidation.*

KEYWORDS: EPB -TBM, MCC, MC, tunnel monitoring, surface settlement

SITE LOCATION: Geo-Database

INTRODUCTION

Due to the growing of cities and subsequent increase of transportation demand especially in Iran, as one of the developing countries, EPB tunneling is becoming the best strategy for subway development. Consequently, ground surface settlement beneath nearby sensitive structures, such as national monuments located in the heart of the cities, in addition to damaging the underground installation system due to the ground movement can give rise to a tribulation in these areas.

Generally, there is a variety of parameters that would influence the surface settlements including geometric parameters, soil characteristics, geomechanical parameters, and the choice of a suitable constitutive model in the numerical simulation and the design approach. The last parameter is a paramount factor that can play a pivotal role in the precise prediction of settlement. Several methods—such as numerical analyses, empirical approaches, physical modeling and closed form solutions—can be utilized to predict the surface settlements. By considering the capability of numerical methods, such as the finite element (FE) and the finite difference (FD), they could be the best approaches for studying the complex situation of soil, soil-structure interaction, and time-dependent problems such as consolidation. Based on the recent studies, the constitutive model is one of the crucial factors for the prediction of soil deformation (Fargnoli, Boldini, & Amorosi, 2013; Lee, Rowe, & Lo, 1992; Xu, Sun, Sun, Fu, & Dong, 2003), although the simulation results should be checked via back-analysis techniques or compared with tunnel monitoring. One of the common approaches to evaluate the surface settlements is the empirical formulas that were proposed by Peck (Ercelebi, Copur, & Ocak, 2011; Lambrughi, Medina Rodríguez, &

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Castellanza, 2012). Meanwhile, for a precise simulation of TBM tunneling, it should be simulated via a 3D model for which a number of significant items are outlined below (Akhaveissy, 2011; Kasper & Meschke, 2006; Lambrughi et al., 2012; Möller & Vermeer, 2008; Youakim, El-Metewally, & Chen, 2000; Zhao, Gong, & Eisensten, 2007):

- The real behavior of face excavation
- 3D arching of soil
- Distribution of longitudinal settlement
- Temporary heave due to the high value of face pressure

In this study, a FLAC^{3D} code is developed to evaluate the effects of the constitutive model on the soil behavior. In addition, the empirical and analytical methods are used and all results are compared with each other eventually.

Constitutive Modeling Effect
Mohr Coulomb Model (Elastic Perfect Plastic Model)

This statement is generally accepted that simple linear elastic-perfect plastic models lead to a better prediction of surface settlement which is wide and shallow, since it cannot correctly account for the nonlinear soil behavior at small strain. (Brinkgreve, 2011).

The Introduction of Modify Cam Clay Model

The isotropic work-hardening plasticity cap model was first suggested by Drucker et al. from a theoretical point of view. The cap model is suitable for the prediction of soil behavior since it is capable of treating the conditions of stress history, stress path dependency, dilatation, and the effect of the intermediate principal stress (Youakim et al., 2000). This model describes three important aspects of soil behavior:

(i) Strength
(ii) Compression or dilatation (the volume change that occurs with shearing)
(iii) Critical state at which soil elements can experience unlimited distortion without any changes in stress or volume (Wood, 1990).

The yield function of the MCC models is determined from the following Eq (1):

$$\frac{q^2}{p'^2} + M^2\left(1 - \frac{p_0'}{p'}\right) = 0 \tag{1}$$

In two-dimensional space p-q, the yield surface defined by the MCC model formulation is known as the State Boundary Surface (See Figs. 1, 2).

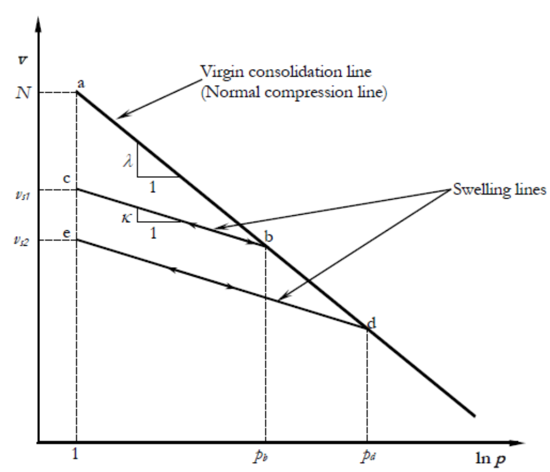


Figure 1. Behavior of soil sample under isotropic compression (Wood, 1990).

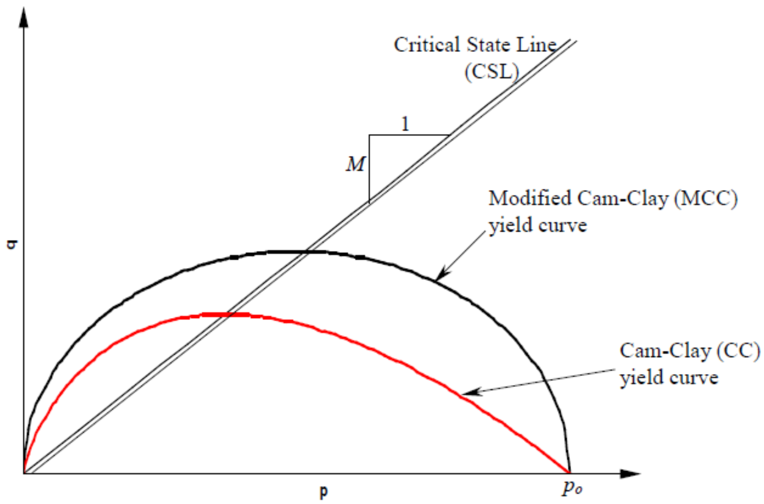


Figure 2. CC & MCC yield surfaces (Wood, 1990).

Closed-Form Solution

The kirsch equations are a set of closed-form solutions, derived from the theory of elasticity, used to calculate the stresses and displacements around a circular excavation (Exadaktylos & Stavropoulou, 2002). According to the Kirsch solution, displacement around a single tunnel can be determined via Eq. (2), assuming the conditions of plane strain holds:

$$U_r = \frac{P_1 + P_2}{4G}\frac{a^2}{r} + \frac{P_1 - P_2}{4G}\frac{a^2}{r}(4(1-\nu) - \frac{a^2}{r^2})\cos 2\theta \tag{2}$$

where *P1* and *P2* are original (pre-tunneling) stress field at the tunnel level, *a* is the radius of the tunnel, *r* is the radial distance to any point, θ is the angular distance to any point, *v* is the Poisson's ratio, and *G* is the shear modulus (Exadaktylos & Stavropoulou, 2002).

A closed-form solution was described by Chow (Chow, 1987). This method accounts for volume loss and is based on incompressible irrotational fluid flow solutions. Chow relation presents a solution for the calculation of vertical displacements via Eq (3):

$$S = -\frac{\gamma D^2 {z_0}^2}{4G(y^2 + z_0^2)} \tag{3}$$

where *S*, γ, *G* represent the vertical displacement, soil density, and shear modulus respectively. *D* indicates the tunnel diameter, z_0 is the depth of the tunnel from the surface and *y* is the horizontal displacement of every point from the tunnel centerline. Meanwhile, closed-form solutions can, in the best case scenario, only provide a rough estimation of ground behavior while providing a useful and quick method for prediction of settlement.

General Information About Line 2 Metro of Mashhad

Mashhad Metro Line 2 is the second metro line that is being created to facilitate passengers' transport in Mashhad, Iran. This metro line is situated beneath the street level in a tunnel running in a Northeast-Southwest direction, as seen in Fig. 3. In total, this line includes 12 stations. Furthermore, Metro Line 2 is connected to Mashhad Metro Lines 1 and 3 as well as the national railway line in Iran. The total length of Line 2 is about 14.3 km. A part of the tunnel running from Station A2 through L2 and going further to the TBM exit shaft is going to be constructed with mechanized tunneling methods, such as the Tunnel Boring Machine or TBM. The TBM excavates the ground in front of the cutter head while pushing itself forward. The tunnel is built up inside the TBM from concrete segments (Arteh, 2009). A section of this tunnel is depicted in Fig. 4.

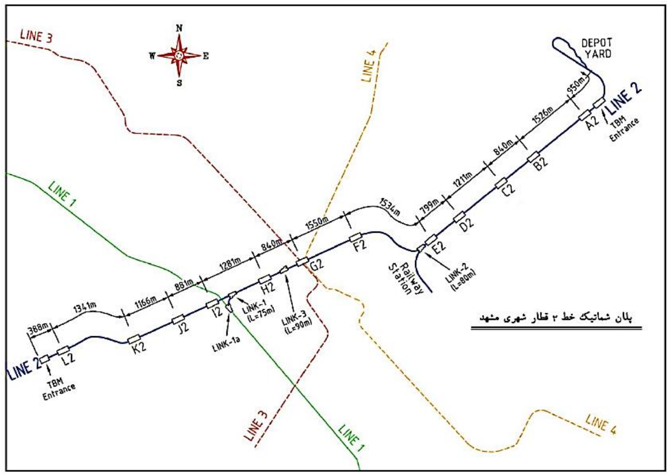


Figure 3. Plan of Line 2 Metro of Mashhad.

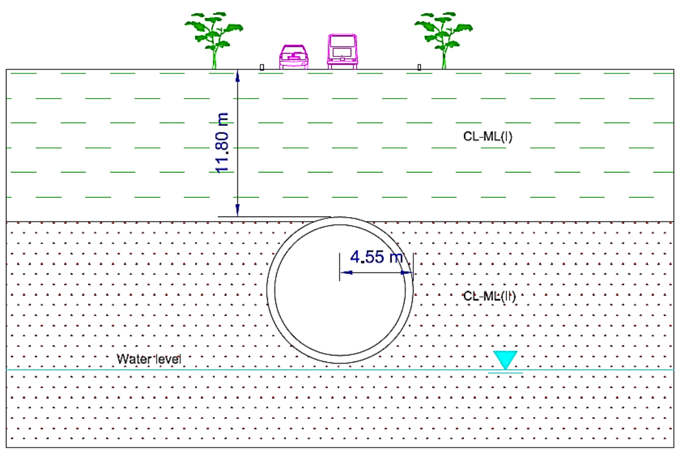


Figure 4. Section ground of Line 2 Metro of Mashhad.

Soil Condition

The characteristics of Mashad's soil are illustrated in Tab.1

- Medium clay-silt (CL-ML l): The uppermost layer is the soft clay soil by low plasticity and low moisture percentage. The average thickness is about 10 m in most areas.
- Medium clay-silt (CL-ML ll): The low layer is the soft clay soil by high plasticity and high moisture percentage. This layer can be found at depths of 10-35 m.

The results of the oedometer test in km 1+260 (DH11 Borehole-depth 8-8.5m) are shown in Fig.5:

Table 1. Soil characteristics - Km of 1+260

Layer No.	Description	γ_d	ω_d	C'	φ'	C_U	φ_U
	Unit	KN/m^3	%	KPa	deg	KPa	deg
I(A)	CL-ML	17.00	17.00	10	25	10	25
I(B)	CL-ML	17.50	18.00	30	23	12	20

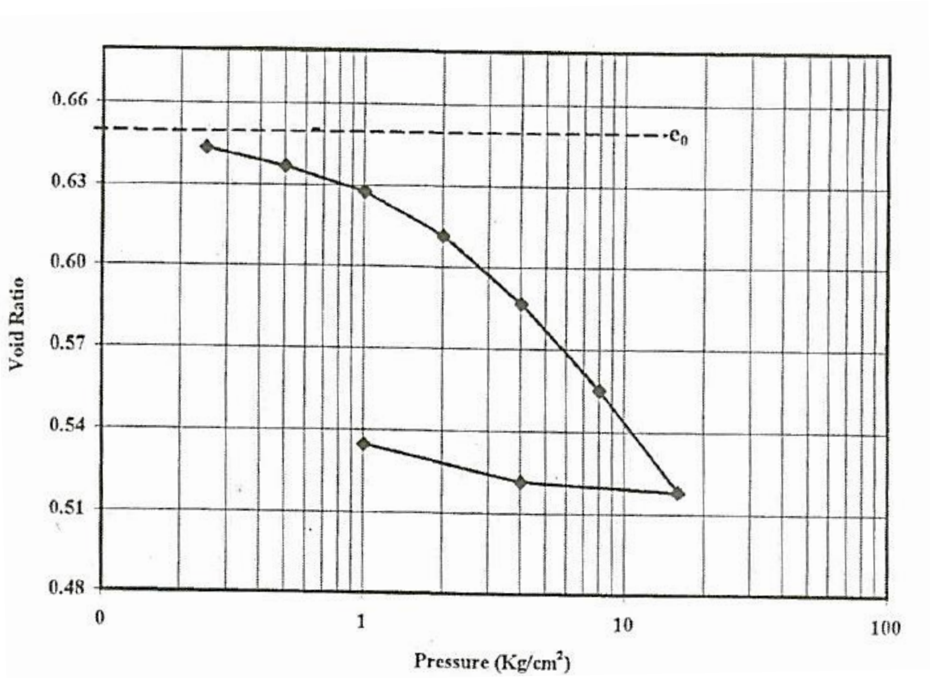


Figure 5. Oedometer test result – km +2+650(DH-15).

The detailed geotechnical investigations were performed by the excavation of 61 boreholes (a total length of 2,487.7 m) and 16 test pits (a total length of 296.95 m). These investigations mainly included some field tests and surveys, laboratory tests, and desk studies. The field tests included a plate loading test (PLT), in-situ shear test, pressuremeter test, standard penetration test (SPT), Lufran permeability test, and in-situ density test. The laboratory tests comprised the direct shear test, triaxial test, particle size analysis, Atterberg limits test, consolidation, permeability, and the Los Angeles Abrasion test. The desk studies included the collection of the existing data such as previous reports, in-situ test results, and data processing and analyzing. The geological section of the project is illustrated in Fig. 6. The soil sample of the considered section is shown in Fig. 7, which is obtained from the site investigation (DH09 Borehole).

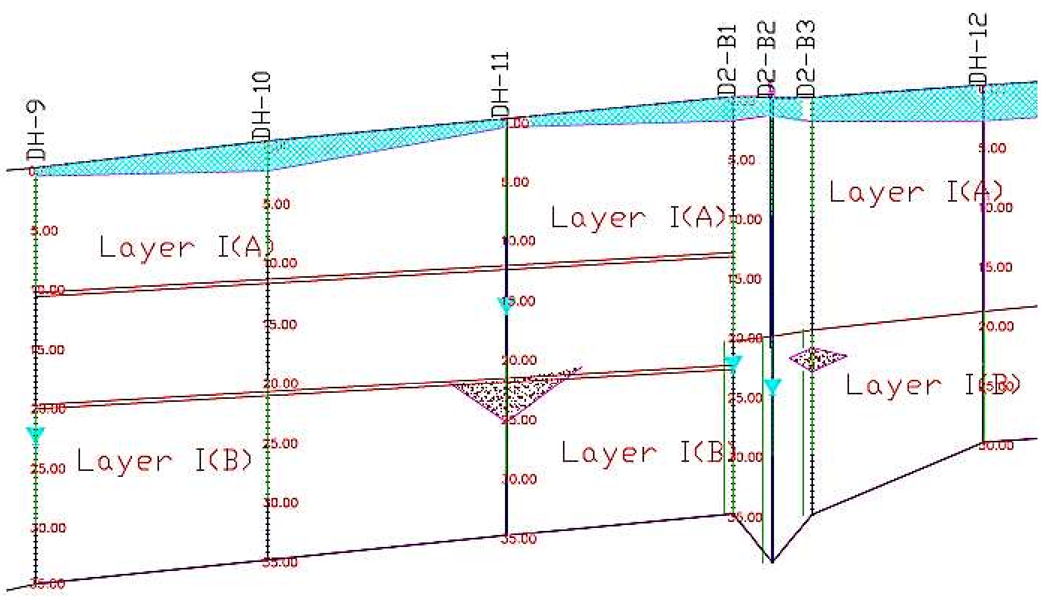


Figure 6. Geological section of Mashhad Metro Line 2 – km of 12+500.

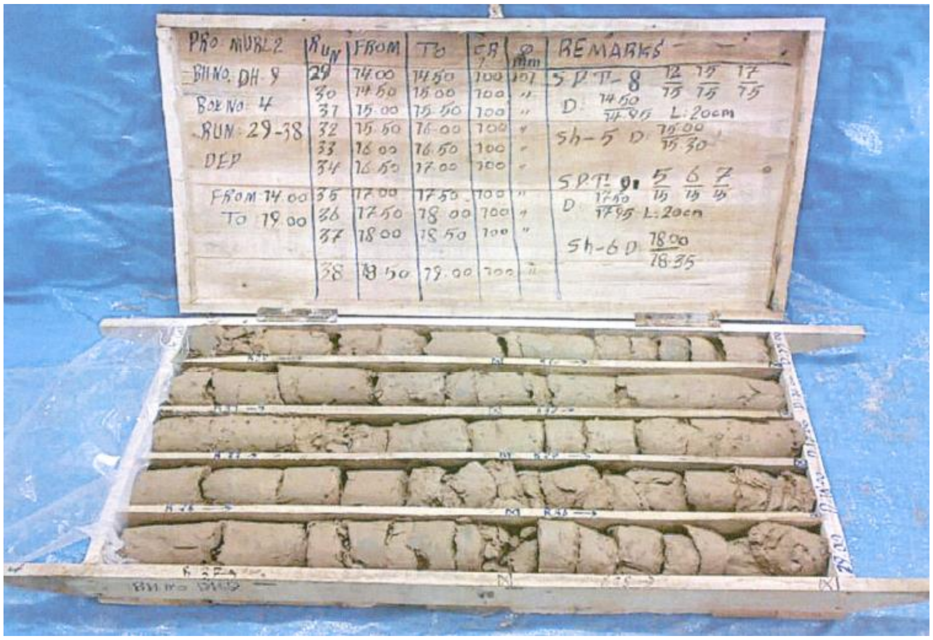


Figure 7. Box sample of log DH-9-depth 14-19 m.

The calculation of the MCC parameter was performed based on the elasticity rule shown in Eqs. 4-8 as follows (Arteh, 2009):

$$G = \frac{E}{2(1+\nu)} \tag{4}$$

$$K = \frac{E}{3(1-2\nu)} \tag{5}$$

$$\kappa = C_S .\ln 10 \tag{6}$$

$$\lambda = C_C .\ln 10 \tag{7}$$

$$v_0 = 1 + v \tag{8}$$

Soil characteristics and the respective parameters are summarized in Tables 2 and 3. A part of our site investigation results was obtained from a consolidation test which was performed in Km 1+260. Frictional constant (*M*) was calculated based on the slope of the critical state line.

Table 2. Consolidation test result – km 1+260 (DH-11)

	Layer i	Layer ii
Type of sample	Undisturbed	Undisturbed
Depth of sample (m)	7-7.5	12-12.5
Sample No	DH -11	DH -11
P_C (KPa)	60000	78000
C_C	0.10	0.12
C_S	0.015	0.019

Table 3. *Parameters of cam clay model in Mashhad soil*

Parameter	Description	Values for soil layer	
		i(A)	i(B)
E (MPa)	Young modulus	100	120
G (MPa)	Shear Modulus	40	48
K_{max} (MPa)	Max elastic bulk modulus	70	84
ρ (KN/m^3)	Density	19.85	20.65
M	Frictional constant	0.983	0.898
K	Slope of swelling line (equal to slope of consolidation)	0.0345	0.044
Λ	Slope of normal compression line	0.23	0.28
p_c (KN/m^2)	Pre consolidation pressure	60000	78000
ν_0	Initial specific volume	1.27	1.27
ν	Poisson ratio	0.27	0.27
p1 (KN/m^2)	Reference pressure	16000	16000
ν_{ref}	Specific volume at reference pressure, on normal consolidation line	1.61	1.63

Numerical Modeling

For accurate modeling of a tunnel in soft ground by FEM methods, some of key parameters that affect the surface settlement such as constitutive soil model, tunnel lining, over excavation, and shield element should be considered. In this study, the result of field tests, in situ measurements, and laboratory data is utilized to describe two different constitutive models. Since there is a complicated correlation between the target parameter (surface settlement) and other factors, the input parameters of constitutive models should be considered accurately (Kasper & Meschke, 2006).

To obtain a rational result, all main elements of mechanized excavation should be modeled such as: TBM's shield, concrete tunnel lining, support face pressure, tail void grouting, and over excavation. Therefore, FLAC^{3D} (Version 3.0) code, a commercial software package based on the generalized finite difference method, was used to develop the numerical simulation (Cundall, 1995). In this software, dynamic equations of motion were solved at each calculation step in the small strain mode, even for semi-static problems. An explicit solution scheme was adopted, with a mixed-discretization formulation.

The shield of TBM was modeled using a shell element, since it had a considerable stiffness moment in the inner plan. For modeling this element, a simplified cylindrical geometry was considered, while TBM has a cone-shape shield. Additionally, elastic behavior is assigned to the shield body (Lambrughi et al., 2012).

The segmental lining and the shield element were modeled by the elastic constitutive model. The details of these support elements (lining and shield) are shown in Fig. 8; moreover, the characteristics of the TBM machine in Line 2 Metro of Mashhad are illustrated in Table 4.

Table 4. Characteristic of the EPB machine in Line 2 Metro of Mashhad

Element	Diameter (cm)	Length (cm)	Thick (cm)	Weight (KN)	Poison ratio
Prior shield	937	340	6	468.38	0.15
Middle shield	936	288	5	330.62	0.15
Head shield	935	492	4	451.85	0.15
Precast segment	934	150	35	355.860	0.25
Face pressure (bar)			1.2		
Tail void grout (bar)			1.5		
Over excavation (cm)			3.5		
Overburden (m)			15.5		

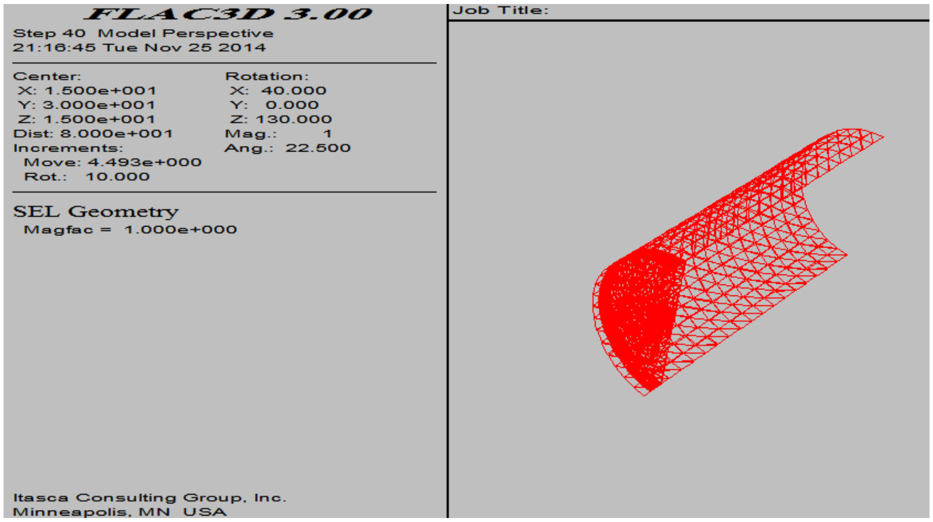


Figure 8. Shell element for modeling of segment and shield.

In all the outputs, units of measurement were Newton (N) for force, and meter (m) for length. The effect of virtual boundary on the results was neglected because the model had a longitudinal dimension (in y direction) of 65 m (approximately 6.5 D, D tunnel diameter), an extension under the tunnel axis (in z direction) of 30 m (about 3D), and a transverse extension of 40 m (in + x direction), which must be at least 4D (Chakeri, Ozcelik, & Unver, 2013) (Fig. 9).As the underground water table in this project is lower than the project line, all analyses have been performed in drain condition.

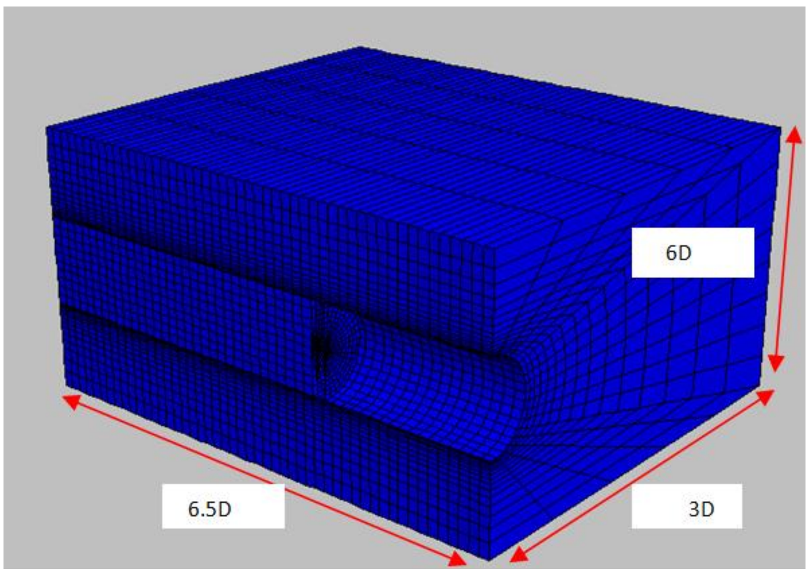


Figure 9. Dimension of the 3D simulation in FLAC3D.

In a full aspect 3D-finite difference simulation, the mechanized excavation process includes the soil excavation, providing the adequate pressure, segment installation and gap filling. The order of excavation integrated into the models is as follows:

Step 0: Application of traffic load and initial condition of ground to get the stable situation.
Step 1: Excavation of tunnel (about 1.5 m).
Step 2: Application of face pressure by the TBM on the new excavation face of the tunnel.
Step 3: Excavation of the tunnel by driving the EPB machine.
Step 4: Generation of both gap filling and segment elements performed after excavation of the tunnel whose length is equal to the width of the segment.
Step 5: Removing the previous face pressure on the tunnel face.

Step 6: Repeating the steps 1 to 5 until the TBM reaches its destination.

The result of deformation by the aforementioned process of numerical modeling in FLAC^{3D} is illustrated in Figs. 10 and 11. Generally, one of the important factors affecting the surface settlement is the relaxation of soil in front of the cutter head (front loss). As it can be seen clearly in Figs. 10 and 11, when supporting face pressure is applied to the soil, an infinitesimal movement occurs in front of the machine, compared to cases where this face pressure is not applied and there is a remarkable settlement at the ground surface.

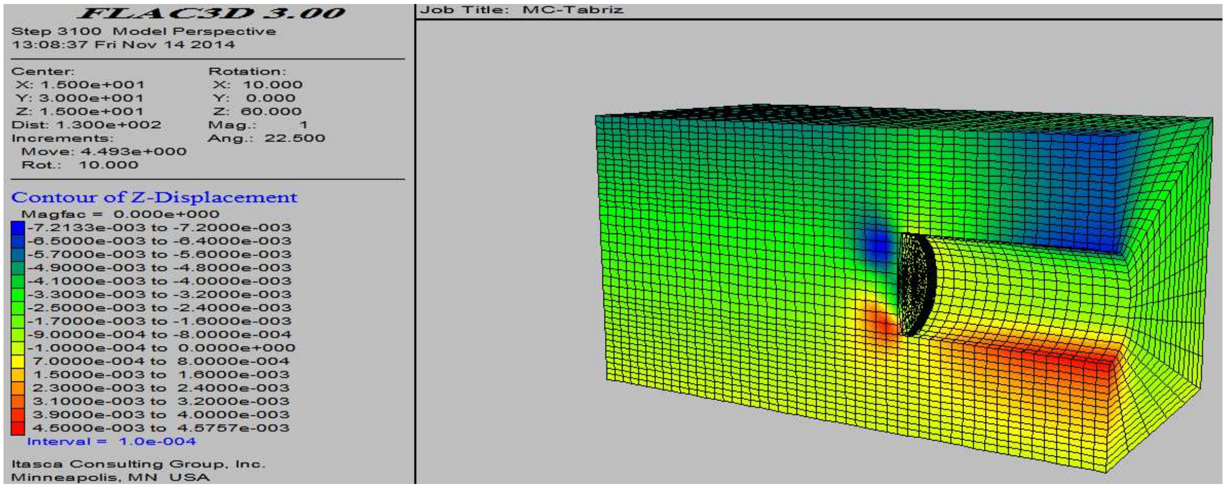


Figure 10. Vertical displacement after tunneling.

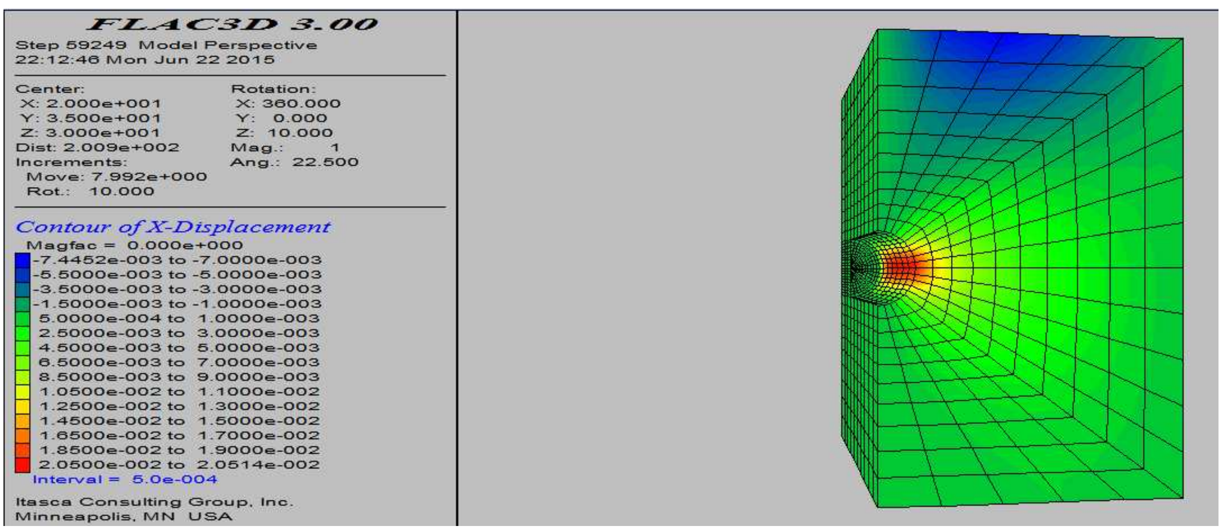


Figure 11. Horizontal displacement after tunneling.

Distribution of Surface Settlements in Longitudinal Section:

Based on Peck formulation, 50% of maximum settlements happen in the tunnel face, while pre-settlement happens in front of the slurry shield; at the Cairo project, for example, there was about 25-30% of maximum settlement by 16m overburden and 9.48 m diameter (Group, 2012; Lambrughi et al., 2012). The results of our study in Line 2 Metro of Mashhad illustrates that this value is about 16% maximum surface displacement. Generally, distribution of surface settlement for tunneling by TBM-EPB consists of:

- Face relaxation (10-20%)
- Over excavation (vacant space between shield and soil) (40-50%)

- Difference between the tail shield and segment diameter (30-50%)

But these values are highly dependent on the construction method. Based on the Peck formula, the longitudinal settlement profile is followed in Eq. 9:

$$S = \frac{V_S}{\sqrt{2\pi}.i}\left\{G\left[\frac{x-x_i}{i}\right] - G\left[\frac{x-x_F}{i}\right]\right\} = S_{\max}.(G_1 - G_2) \tag{9}$$

X: longitudinal position of the considered surface point (m);
Xi: initial position or starting section of the tunnel (m);
X f: position of the tunnel face (m);
G: a function defined as:

$$G(\alpha) = \frac{1}{\sqrt{2\pi}}.\int_{-\infty}^{\alpha} e^{-\frac{\alpha^2}{2}} d\alpha \tag{10}$$

$$\alpha = \frac{x - x_i}{i} \tag{11}$$

The instrumentation and the monitoring of surface pins were used to verify the numerical results. The monitoring activities were conducted by adjusting the settlement pins' network transversally arranged on the tunnel plan. The instrumentation tools in this project were pins with 120 cm length, and three row patterns that were installed with a 10 m longitudinal space and 5 m transverse space on the ground surface (3 pins per cross section). After the installation of the pins in the ground, the result was compared with the reference point during tunnel construction. The installed instrumentation layout in the section under study is illustrated in Fig. 12. The precision of this method was adjusted to about 1 mm. Also, to neglect the effect of the shallow layers of ground and pavement deformation, these pins were rooted 20 cm into the soil (See Fig. 13). Since the settlement profile followed the Gaussian distribution curve, we used monitoring data in km 1200, 1250, 1270 to determine the real value of the soil parameters via back-analysis methods.

The vertical settlement in the longitudinal section is shown in Fig.14. It can be clearly seen that the Peck relation (semi empirical method) results significantly differ from the real data because, based on the Gaussian curve, about 50% of the settlement in the maximum case scenario must happen as pre-settlement. However, this value in the numerical and data monitoring is less than the Peck formulation. In addition, the selection of the best value for the volume loss is highly dependent on the type of the ground and the tunneling method, so the perfect prediction of the longitudinal settlement relies on the design approach and engineering judgment (Lambrughi et al., 2012). Hence, it is preferred to utilize a model with fewer parameters and a linear stress–strain path with constant modulus, so that the designer can get a general overview about the soil mass deformation.

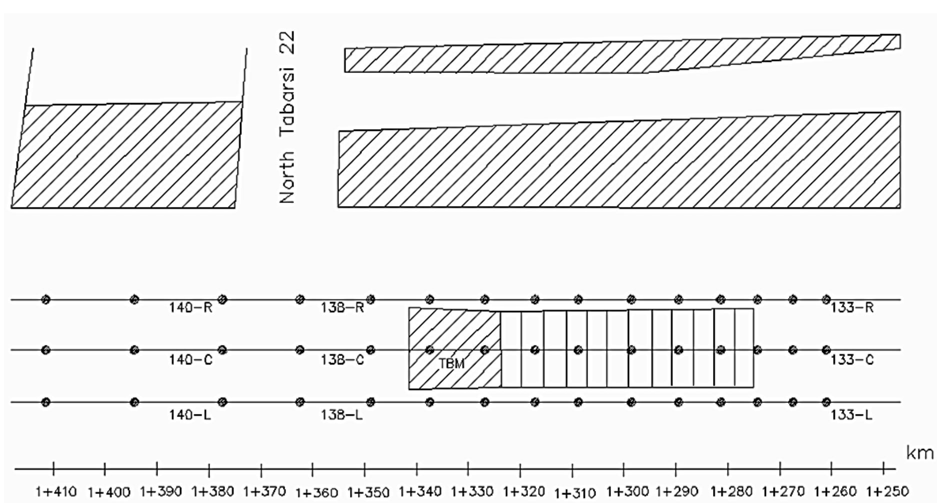


Figure 12. Pattern of the installation pins in the ground.

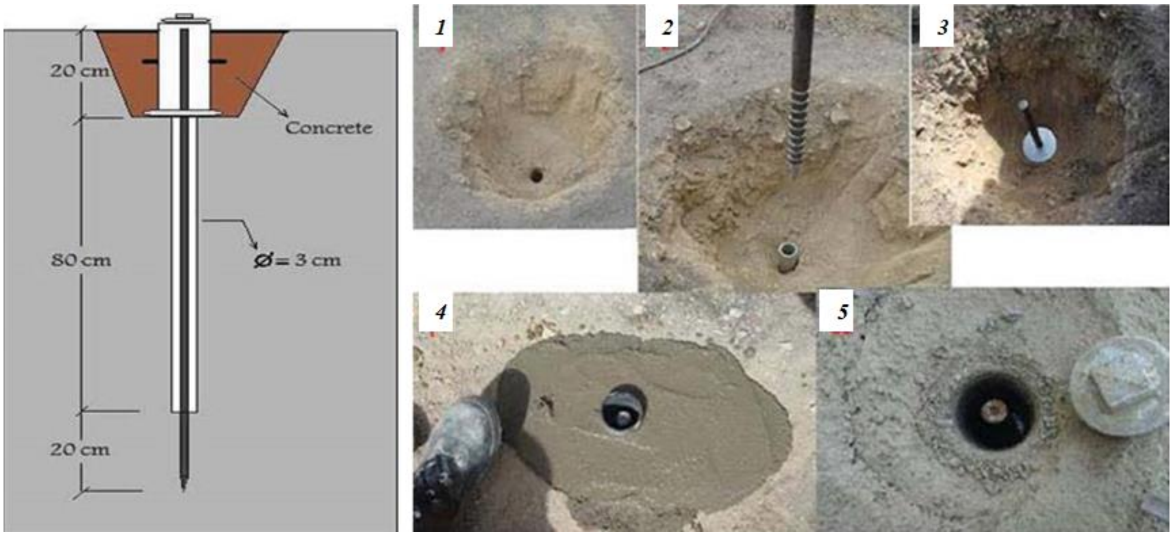


Figure 13. Monitoring benchmarks on each instrument section.

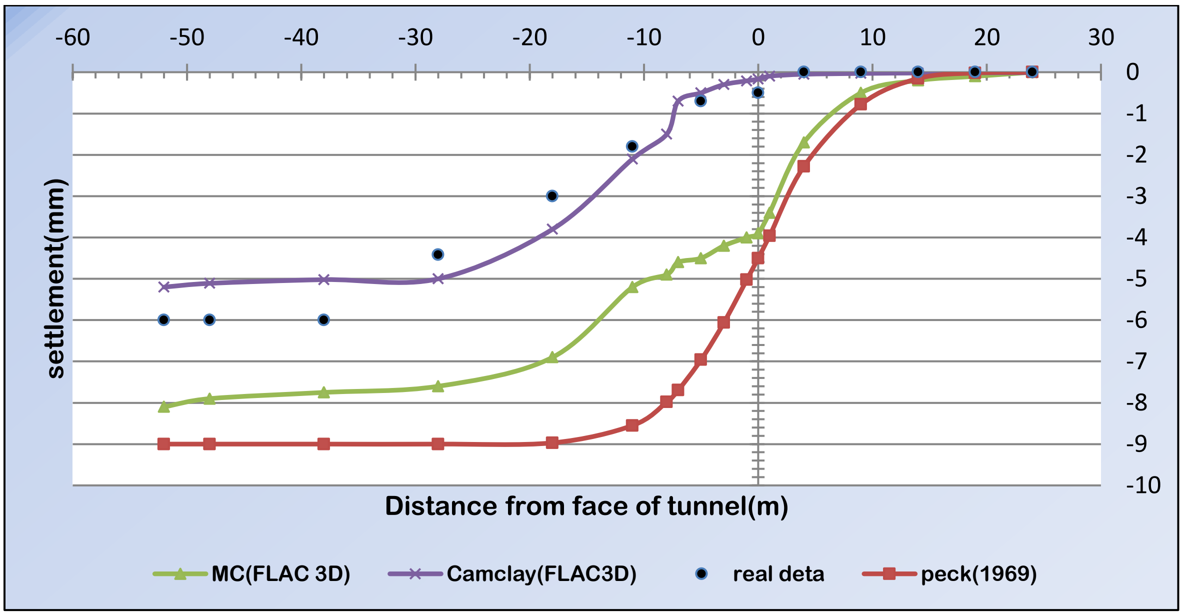


Figure 14. Profile of longitudinal section (km 1+200 -1+290).

Distribution of Surface Settlements in Transverse Section:

The semi-empirical relation of Peck was obtained in Eqs. 12-14, seen below, showing the shape of transverse settlement.

$$S = S_{\max} \exp\left(\frac{-y^2}{2i^2}\right) = \frac{V_L}{i\sqrt{2\pi}} \exp\left(\frac{-y^2}{2i^2}\right) \tag{12}$$

$$V_S = \int_{-\infty}^{+\infty} S_{\max} e^{\frac{-y^2}{2i^2}} = \sqrt{2\pi} i S_{\max} \tag{13}$$

$$V_L = \frac{V_S}{V_0} \times 100 \tag{14}$$

In the above equations, S is the vertical surface settlement at (y) location (m); y is the distance of the considered point from the tunnel axis [m]; V_s is volume of the settlement per meter of tunnel advancement (m3/m), defined as a percentage of the unit volume V_L of the tunnel; i is trough width parameter, expressed as: $i = k\ z_0$, where "k" is a dimensionless constant, depending on soil type; and "z_0" is the depth of the tunnel axis below surface (Guglielmetti, Grasso, Mahtab, & Xu, 2008). (See Fig.15.) The volume loss V_L is the volume of the settlement per unit length expressed as a percentage of the total excavated volume of the tunnel, whereas V_0 is the volume required to construct the tunnel. This is based on the assumption that soil movements occur under constant volume.

For calculating volume loss and i parameters, Loganathan & Poulos (1998) offered the Eqs. 15 and 16 in shield tunnels.

$$\epsilon_0 = v_l = \frac{\pi(R + g/2)^2 - \pi R^2}{\pi R^2} = \frac{4gR + g^2}{4R^2} \tag{15}$$

$$g = G_p + U_{3D}^{*} + \omega \tag{16}$$

In Eq. 15, ϵ_0 and v_l are volume loss, R is radius of the tunnel, and g is the gap parameter which is the function of three other parameters and is calculated based on the Eq. 16. In Eq. 16, G_P is the difference between the outer diameter of shield and soil, U^*_{3D} is an elasto-plastic parameter that is related to the tunnel face in three dimensions, and ω is the parameter that depends on the operator's skill.

O'Reilly & New (1982) showed that point of inflection (trough width parameter) i had a linear relation with depth of tunnel and they suggested Eqs. 17 and 18.

i= 0.43Z_0+ 1.1 For cohesive soil (17)

i= 0.28Z_0 - 0.1 For granular soil (18)

For places where the effect of consolidation is not problematic, Tan & Ranjith (2003) suggested, this relation for the calculation of trough width parameter.

i = (0.57+0.45 Z0) ± 1.01 (19)

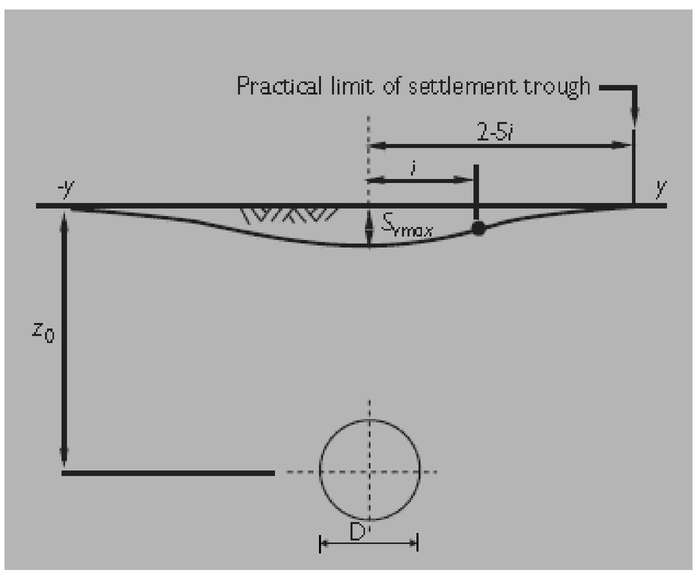


Figure 15. Relevant parameters for Peck relation (Guglielmetti et al., 2008).

Since the Longathan has considered all aspects of the mechanized excavation, we used it for the calculation of the volume loss. In Eq. 19, set U^*_{3D} and the ω (which is related to the skill of the operator) to zero to control the face deformation in the EPB machine. Based on recent studies, the gap parameter was considered to be 7% of total vacant space section of the tunnel, and eventually the final value for the volume loss was determined to be 1.54%.

A summary of all relations suggested in the literature is presented in Table 5. The behavior of surface settlement in transverse section follows the Gaussian distribution. Based on this assumption, a Gaussian curve is fitted to the data monitoring outputs. As a result, the Gaussian distribution is back analyzed for obtaining trough width parameter *i*, which is about 7.41 m. This value is very close to the Oreilly & New relation whose error value was about 3.91% (See Fig. 16).

The transverse profile of the surface settlement was compared with the numerical results obtained from the MCC model and the MC model. It can be clearly seen that results of the MCC model have the best fit to the data points. The MC model substantially differs from data monitoring outputs, thus the elasto-plastic model (e.g., the MCC model) is considered to be suitable for this type of soils. Also, empirical and analytical methods can predict the surface settlement in the green field; for example, the Chow relation is very close to the maximum settlement, but the distribution of settlement doesn't fit well with real data (S_{max} 5.77 mm).

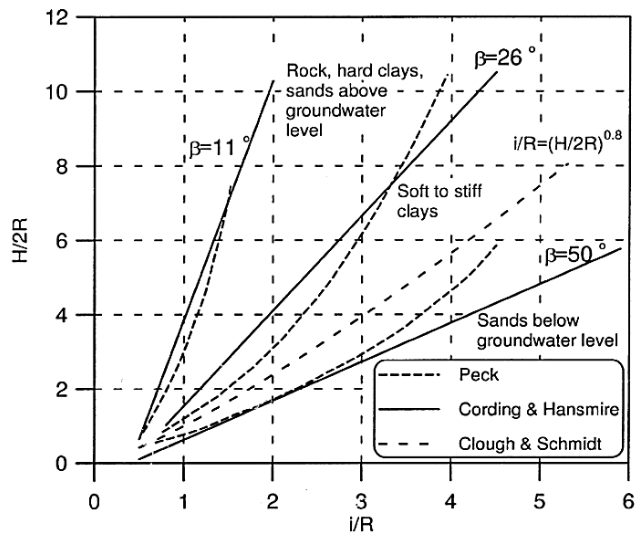


Figure 16. Gaussian curve fitted by real data (Aoyagi, 1995).

Table 5. Calculation of inflexion point distance based on other researches.

No	researcher	Relation	Z_0(m) Depth of tunnel	R (m) Radius	K_0 Lateral pressure	i (m) Point inflexion	real value	Error value%
1	(Peck, 1969)	$i/R=(Z_0/2R)^n$, n=0.8-1	14	4.55	-	6.42-7.00	7.41	9.44
2	(Clough & Schmidt, 1981)	$i/R=(Z_0/2R)^{0.8}$	14	4.55	-	6.42	7.41	13.36
3	Cording & Hans maire (Aoyagi, 1995)	Fig.16	14	4.55	-	5.55	7.41	25.10
4	(Atkinson & Potts, 1977)	$i=0.25(1.5Z_0+0.5R)$	14	4.55	-	5.81	7.41	21.59
5	(O'Reilly & New, 1982)	$i=0.43Z_0+1.1$	14	4.55	-	7.12	7.41	3.91
6	(Mair & Taylor, 1999)	$i = (0.4\text{-}0.5) Z_0$	14	4.55	0.4-0.5	5.6-7.00	7.41	14.97
7	(Mair & Taylor, 1999)	$i=0.5Z_0$	14	4.55	0.5	7.00	7.41	5.53
8	(Rankin, 1988)	$i=0.5Z_0$	14	4.55	0.5	7.00	7.41	5.53
9	(Attewell & Farmer, 1974)	$i/R=(Z_0/2R)$	14	4.55	-	7.00	7.41	5.53
10	(Tan & Ranjith, 2003)	$i= (0.57+0.45 Z_0) \pm 1.01$	14	4.55	-	5.86-7.88	7.41	7.28

To predict the surface settlement, the MCC model is proposed in soft clay with a low over consolidation ratio or normal consolidation similar to the soil in this site. In other words, where the shear modulus is independent of the shear strain, the surface settlement has a wide and shallow profile. Since the over consolidation clay exhibits non-linear stress strain behavior at the small strain prior to crossing the plastic yielding, it is very important to consider the behavior of these kinds of soils under small strain condition. Nevertheless, the shear modulus in the MC model is constant and the shear strain doesn't change with shear stress; this is probably the main reason for the difference between the results. Based on the results of Bolton for the prediction of surface settlement, we must implement strain non-linearity within the elastic domain (Bolton, Dasari, & Britto, 1994; Lambrughi et al., 2012). (See Figs. 17, 18, 19.)

We studied 17 transverse sections between Stations B2 and C2, then the result of the settlement in the center and right pins were assessed as in Figs. 20 and 21. Vertical displacement was determined by calculating the average values in these sections. Finally, we found that the MCC and MC models, by about 9.6% and 41% error respectively, have a difference from real data.

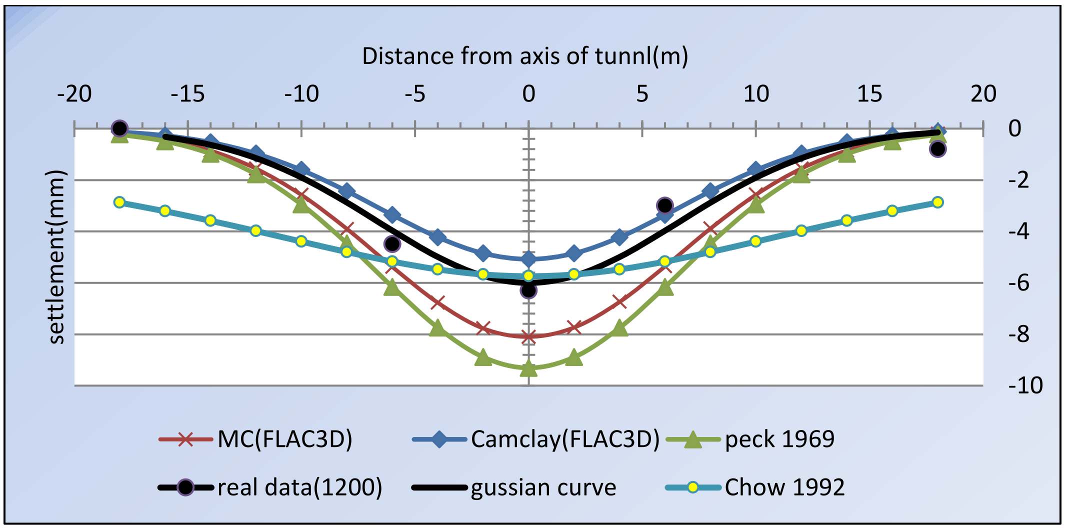


Figure 17. Settlement at transverse section-km 1+200.

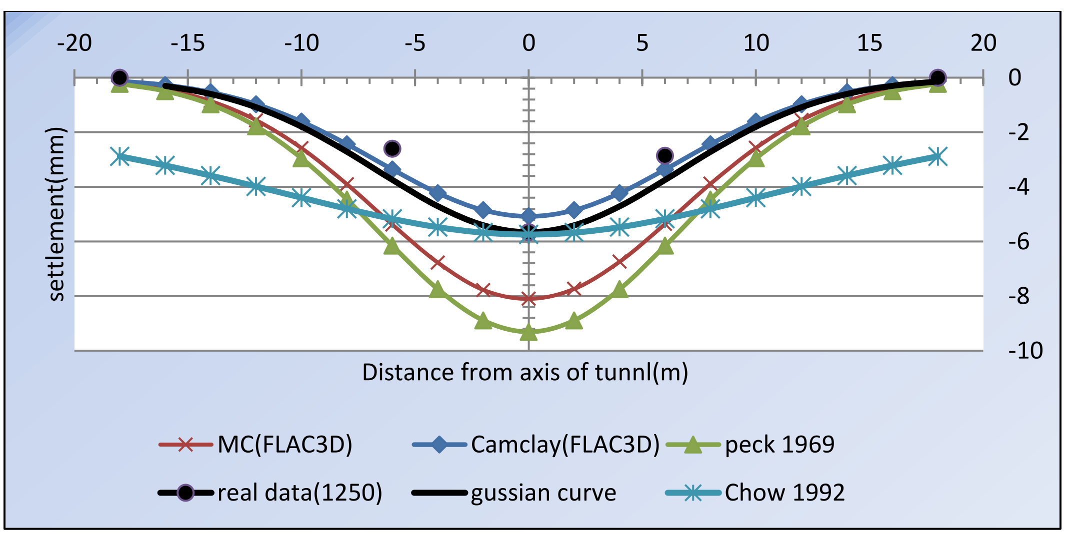


Figure 18. Settlement at transverse section-km 1+250.

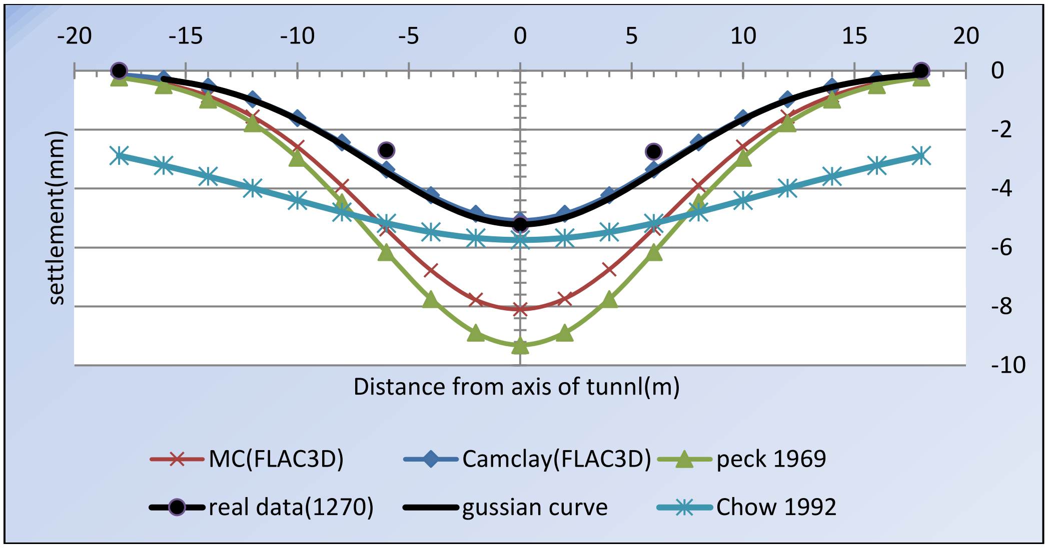
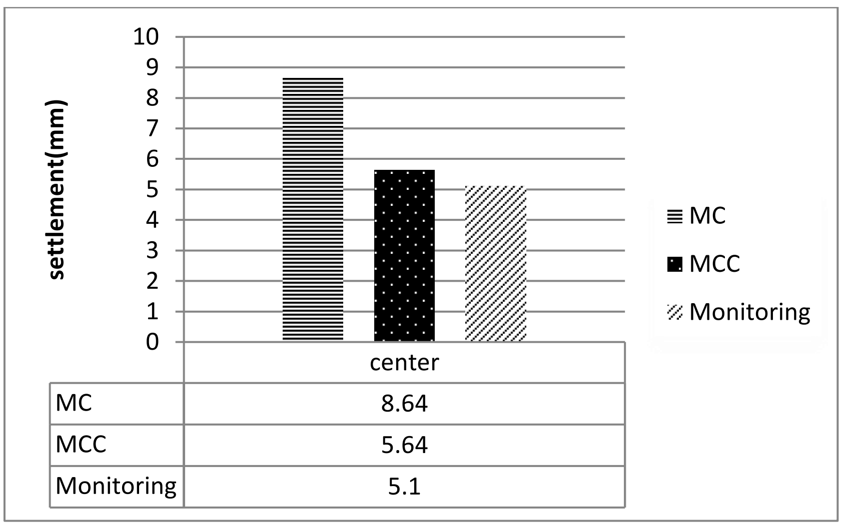
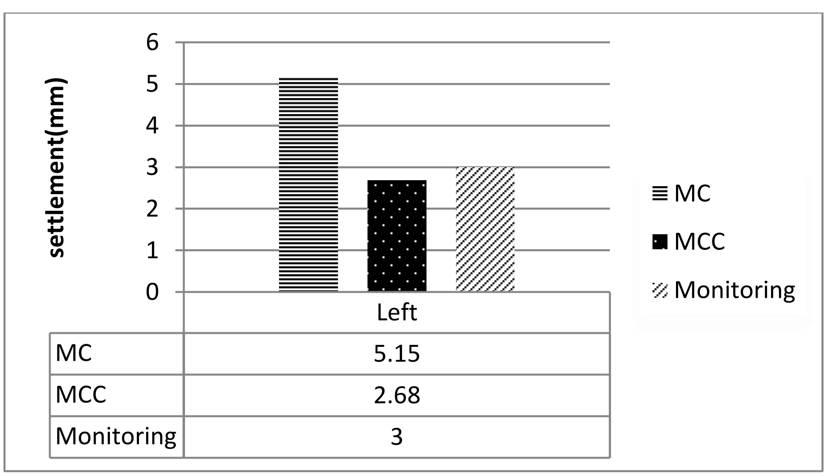


Figure 19. Settlement at transverse section-km 1+270.



	center
MC	8.64
MCC	5.64
Monitoring	5.1

Figure 20. Static comparison of the constitutive model effect at center line of tunnel.



	Left
MC	5.15
MCC	2.68
Monitoring	3

Figure 21. Static comparison of the constitutive model effect in left hand of tunnel (-5 m from the center line).

CONCLUSIONS

In this paper, a 3D numerical model was developed. In addition, all of the parameters affecting the surface settlement (target parameter) were considered for simulation during the TBM-EPB excavation. These parameters included over excavation, shield and lining element, tail void grouting, and face pressure. We then drew this conclusion that the selection of the constitutive model was a considerable step in the modeling procedure to determine the accuracy of surface settlement.

It was proven that simple linear elastic-perfect plastic models could lead to the prediction of a surface settlement profile that was over-estimated, since they could not correctly account for the nonlinear soil behavior which was shown to occur at small strains, which is an important feature of soil-structure interaction. In this study, 17 transverse sections of ground in Mashhad Metro Line 2 were selected and analyzed by using two constitutive models: MC and MCC. The main conclusions from this paper are outlined below:

1- In the analysis process, the semi empirical method does not yield a precise prediction of ground settlement and this approach must be used only to give a general overview to designers.
2- By considering the complicated condition of the soil, the result of numerical simulation and its parameters should be controlled by an inspection during the tunnel construction. The implementation of the MCC in the soft clay with low consolidation is suggested by the authors. The shear modulus in this model is dependent on the shear strain, while, in the MC model, the shear modulus is independent of the shear strain. In other words, the MCC model has a high capability to consider the small strain in the elastic domain, especially in tunnel simulation where the maximum shear strain occurs in a small strain.
3- The MCC model has a relatively precise prediction of the surface displacement in clay, either by normal consolidation or low OCR value.

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