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Comparing Field Displacement History with Numerical Results to Estimate Geotechnical Parameters: Case Study of Arash-Esfandiar-Niayesh under Passing Tunnel, 2.5 Traffic Lane Tunnel, Tehran, Iran

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***Abstract*—**Underground structures are of those structures that have uncertainty in design procedures. That is due to the complexity of soil condition around. Under passing tunnels are also such affected structures. Despite geotechnical site investigations, lots of uncertainties exist in soil properties due to unknown events. As results, it possibly causes conflicting settlements in numerical analysis with recorded values in the project. This paper aims to report a case study on a specific under passing tunnel constructed by New Austrian Tunnelling Method in Iran. The intended tunnel has an overburden of about 11.3m, the height of 12.2m and, the width of 14.4m with 2.5 traffic lane. The numerical modeling was developed by a 2D finite element program (PLAXIS Version 8). Comparing displacement histories at the ground surface during the entire installation of initial lining, the estimated surface settlement was about four times the field recorded one, which indicates that some local unknown events affect that value. Also, the displacement ratios were in a big difference between the numerical and field data. Consequently, running several numerical back analyses using laboratory and field tests data, the geotechnical parameters were accurately revised to match with the obtained monitoring data. Finally, it was found that usually the values of soil parameters are conservatively low-estimated up to 40 percent by typical engineering judgment. Additionally, it could be attributed to inappropriate constitutive models applied for the specific soil condition.

***Keywords*—**NATM, surface displacement history, soil tests, monitoring data, numerical back-analysis, geotechnical parameters.

I. Introduction

DESPITE different field and experimental tests on soil along with site investigation to well understand the soil behaviour, again it is not far to exist some error or unknown factors. Among different infrastructures, the tunnels performance is strongly under the effect of the soil behaviour identified by physical and mechanical parameters. So, it is necessary to be aware of soil condition around the structure and well estimate the values of soil parameters. The existence of uncertainties in the parameters of soil materials has long been recognized [1]. There are different ways to deal with these uncertainties, such as probabilistic or reliability-based approach [1]. In this regard, geotechnical engineers commonly use numerical back-analysis to best estimate soil input parameters based on field recorded data and site observations. Accordingly, this paper also aims to report a case study on well estimation of the soil properties through the obtained monitoring data and soil test results. The case study is based on Arash- Esfandiar- Niayesh tunnel (a particular under passing tunnel construction in north of Tehran- Iran), starting from Modares highway and running to Niayesh highway. The tunnel project has a total length of 1532 m and constructed based on New Austrian Tunnelling Method (NATM). Considering different aspects of traffic engineering (transportation) and variable topography in project site, the shape of the tunnel section compatibly varied along its route, so that the construction to be easily performed [2], [3]. Hereafter, the intended tunnel is named the main tunnel. It should be noted that the main tunnel is located near a hydraulic canal (Velenjak canal). By the monitoring procedure, three different stations along the tunnel axis were selected, for which a middle point on the ground surface above the tunnel axis was controlled. Figs. 1 and 2 show the project site area and the detailed plan of the tunnel path, respectively. According to the tunnel importance and the sensitivity of the site area, the field measurements were recorded up until the entire initial stabilization and supporting. Finally, considering the settlement difference between numerical and real values, the soil parameters were updated such that the final ground surface settlements and the displacement ratio in numerical model match with the real measurements.

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II. Methodology

A. Steps of Analysis

In this paper, the considered numerical analysis was carried out in two total steps:

- First, developing initial condition; in this step, the in-situ geostatic stresses were modelled, with the coefficient for lateral soil pressure assumed to be $K_0 = 1 - \sin\varphi$ and the existing surface load.

- Second, modelling of the tunnel excavation; as mentioned in the first section, tunneling is a NATM based excavation. Accordingly, since after reaching the equilibrium of in-situ stresses, the soil in top, bench and invert galleries of the main tunnel section were removed with a relaxation factor, followed by the installation of initial tunnel lining. By the sequential excavation method and considering real excavation with three-dimensional modelling effects, one and seven stages were adopted for the Velenjak canal and main tunnel, respectively (as shown in Figs. 3 (a) and (b)). The related relaxation factors for each stage are presented in Table I.

Fig. 1 Overall view of the project site (Satellite map)

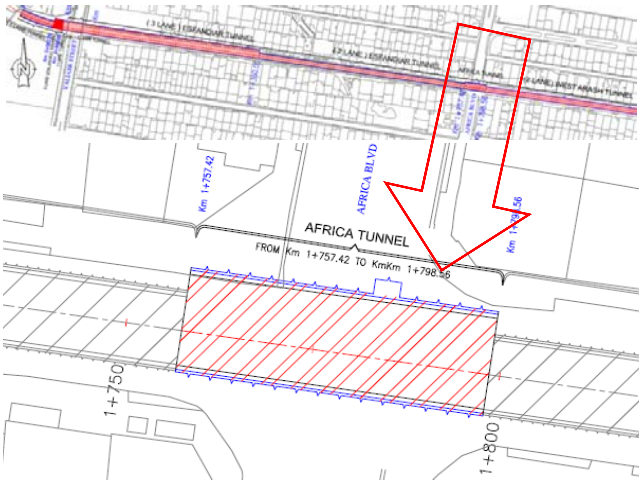
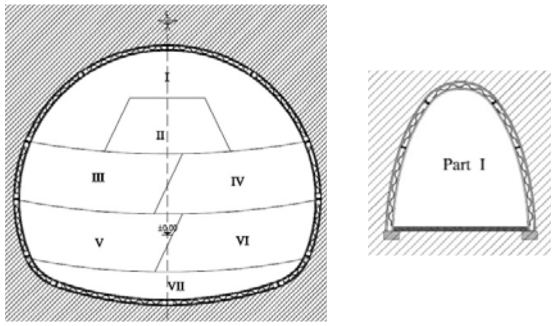


Fig. 2 General and detailed plan of the tunnel route

B. Model Properties

The numerical model was based on a 2D finite element program (PLAXIS-V8) [4]. By the software feature, mesh generation made by triangular 15-node elements. The model geometry is schematically shown in Fig. 4. The intended tunnel runs next to a hydraulic canal (Velenjak canal) at a horizontal distance of nearly 2.6 m. In accordance with the construction order, first the main tunnel and secondly the Velenjak canal were excavated and initially supported. According to longitudinal profile along the project rout, the maximum overburden is approximately 11.3 m. Figs. 5 (a) and (b) shows the developed numerical model and generated mesh, respectively. The tunnel crosses under the Africa Street, thus the maximum considered surcharge load equals2 ton/m^2(i.e., the equivalent maximum traffic load).



(a) (b)

Fig. 3 Sequence of the tunnel excavation; (a) The main tunnel, (b) Velenjak canal

TABLE I
THE AMOUNT OF RELAXATION FACTORS

Parts of The Main Tunnel	Relaxation Factor
Top Heading (part-I)	35%
Core area (part-II)	100%
First Left Bench (part-III)	20%
First Right Bench (part-IV)	20%
Second Left Bench (part-V)	20%
Second Right Bench (part-VI)	20%
Invert (part-VII)	15%
Parts of The Velenjak Canal	**Relaxation Factor**
Total section (part-I)	35%

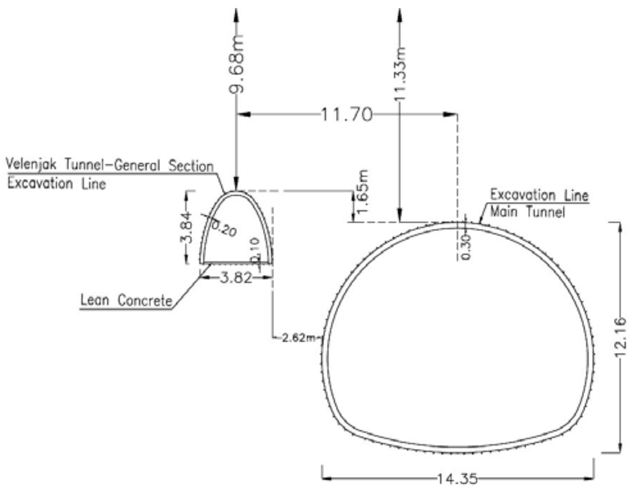
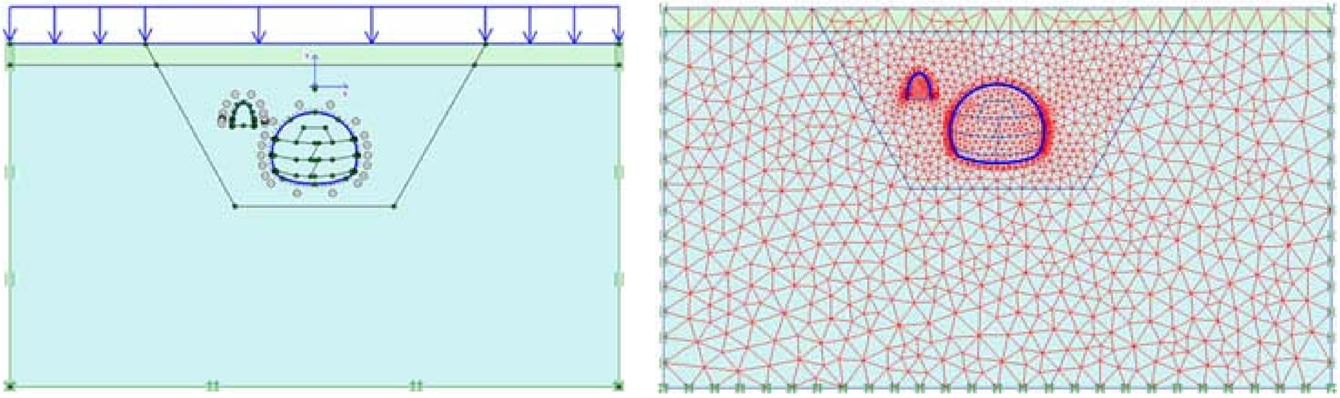


Fig. 4 Geometrical properties of the numerical model (The existence of Africa tunnel next to Velenjak canal)



(a) (b)

Fig. 5 (a) Developed numerical model, (b) Generated finite element mesh used for the analysis

C. Soil Model

At the beginning of the project, different in-situ and laboratory tests were performed in site area depending on geotechnical engineering judgment and the area limitations. The main field tests are including Standard penetration, Pressuremeter, in-situ shear box tests and some other laboratory tests like permeability, tri-axial shear, and shear box test. The location of site investigation boreholes is presented in Fig. 6. Through the geotechnical study, three identification boreholes numbered BH-A1, A2, A3 and four test pits numbered TP-A1, A2, A3, A4 are distributed along the project path. Then, by the performed site investigations and engineering judgment on, preliminary soil parameters are estimated. Also, some images of soil specimens are presented in Appendix. Accordingly, a deep section of the geological layer for the site area is illustrated in Fig. 7, schematically. Through a glance on Fig. 7, a thick filling material layer of 2 to 6 m is observable in ground surface. And, the rest are mainly of sandy gravel material. Based on characteristic of the soil in around and design experiences in analogous projects, it is considered that the soil material behaves as hardening soil in numerical model. With regards to this constitutive model, the soil properties are summarized in Table II [5]. It is an advanced hyperbolic soil model formulated in the framework of hardening plasticity [4]. The main difference with the Mohr-Coulomb model is the stiffness approach. Here, the soil is described much more accurately by using three different input stiffnesses: tri-axial loading stiffness E_{50}, tri-axial unloading stiffness E_{ur}, and the oedometer loading stiffness E_{oed}. Apart from that, it accounts for stress-dependency of the stiffness moduli, all stiffnesses increase with pressure.

TABLE II
PARAMETERS OF SOIL MATERIALS (PRIMARILY ESTIMATED)

Symbol	Quantity	First Layer	Second Layer
ϕ	Internal friction angle (degree)	30	37
C	Cohesion (kg/cm^2)	0.1	0.25
γ_m	Natural density (gr/cm^3)	17	18
P_{ref}	Reference vertical effective stress (kN/m^2)	10	33
ν_{ur}	Poisson ratio of unloading/reloading	0.2	0.2
E_{50}	Secant deformation modulus (kg/cm^2)	400	700
E_{ur}	Unloading stiffness (kg/cm^2)	1200	2100
ψ	Dilatancy angle (degree)	0	7

Fig. 6 Plan of bore holes (BH) and test pit (TP) locations (red marked points)

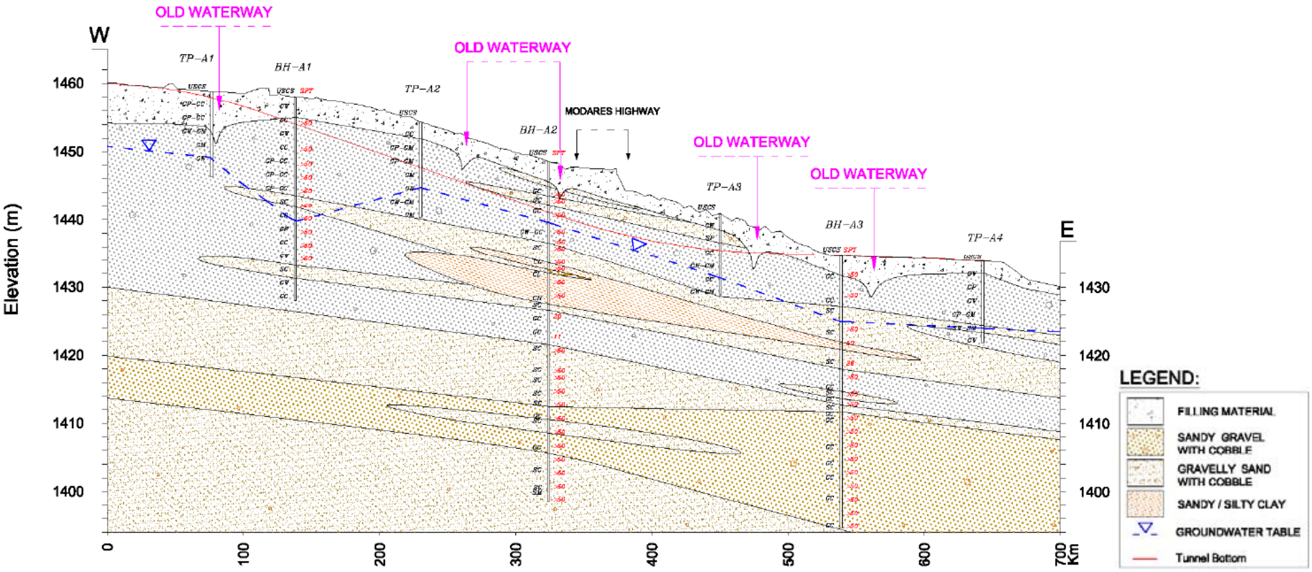


Fig. 7 Schematic geological layers of the project site

D. Tunnel Model

As explained in Section II, part B, two tunnel structures are involved in this numerical model. The main tunnel had a total length of 41m, 14.5 m wide, and 12.2 m height. Its initial lining had a thickness of 30cm. Also, the Velenjak canal had a 3.82m wide and 3.84 m height, initially stabilized with a 20-cm thick shotcrete lining. The properties of lining structure are summarized in Table III. The lining structure was modeled with plate element and a linear elastic behavior was adopted for the concrete material. The reinforced concrete shall be of class C25, and steel bars shall be of type III [6].

TABLE III
PARAMETERS OF TUNNEL LINING STRUCTURE

Symbol	Quantity	The main tunnel	The Velenjak canal
E_c I	Bending stiffness $E_c \times (bh^3/12) \times 0.5 (m^2/m)$	26859.6	7958.40
E_c A	Axial stiffness ($E_c \times b \times h$) (kN/m)	7162558.9	4775039.27
w	Weight ($h \times b \times \gamma$) (kN/m/m)	7.2	4.8

Ec: the elastic modulus of concrete $15100\sqrt{f'_c}$ equals to 23875196.33 (kg/cm^2)

h, b: the thickness and length of initial lining section (h=30 cm, b= 100 cm)

III. MONITORING PROCEDURE

In this project, recording field data during the main tunnel construction took about four months (i.e., started from 04/2016 till 08/2016). Relative displacements are typically the main variable that can be measured at the ground surface on top of the main tunnel. In this procedure, three stations along the main tunnel axis were considered in which the relative vertical displacements for a middle point on the ground surface was recorded continuously, till the entire initial stabilization. The locations of the monitoring stations are presented in Fig. 8.

According to the field measurements (as shown in Fig. 9), the largest surface settlement occurred at stage No. 1794 as high as 15 mm.

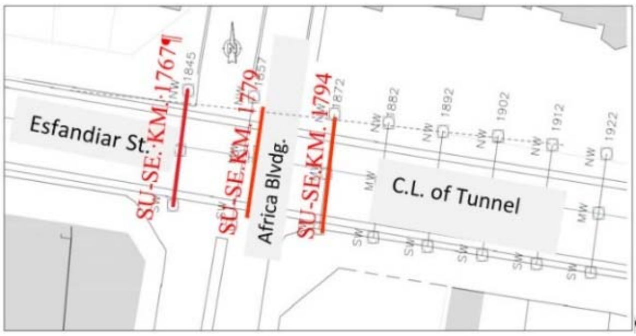
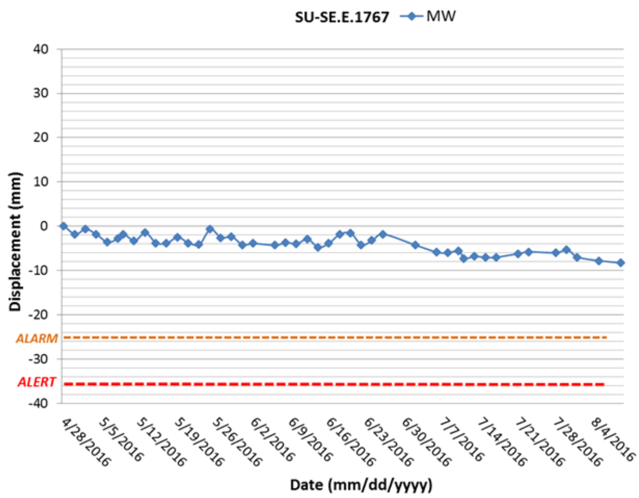
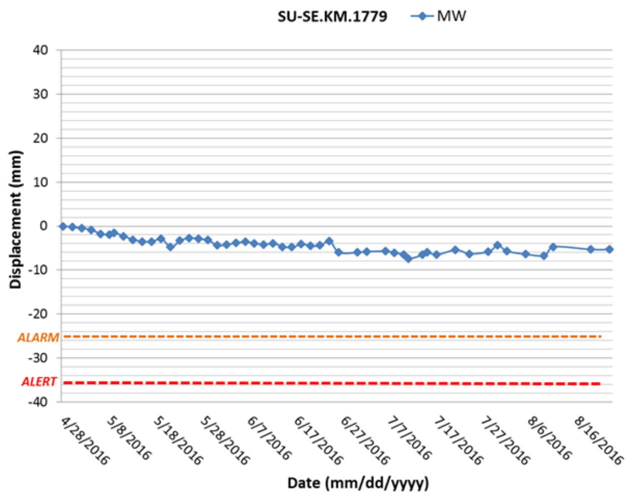


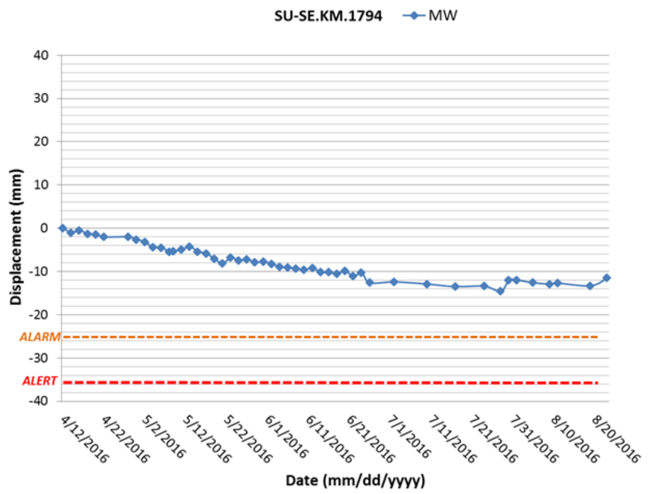
Fig. 8 Locations of monitoring stations Units



(a)



(b)



(c)

Fig. 9 Surface settlement of middle point based on the monitoring data at stages; (a) 1767m, (b) 1779m, (c) 1794m

IV. Numerical Back-Analysis Procedure

Identifying accurate soil specifications by back analysis, a numerical model relating measurements and estimations to the set of the geotechnical properties must be developed. Here, the field readings and measurement set include relative vertical displacement (surface settlement) checked by history.

As explained previously in Section II, part C, the geotechnical parameters were firstly estimated based on several field and experimental tests, in which the engineering judgments were involved. Thus, at the beginning, as it was required, the numerical model was analysed based on the primarily estimated soil parameters (see Table II). In this way, a prediction of the displacements, applying the primitive soil input parameters in numerical model, was firstly made. Then after, by a comparison between field settlement of ground surface and the obtained numerical results, the values of soil parameters were required to be updated appropriately. Here, as the excavation was carried out in several stages, the history of surface displacements along with its final value at the end of construction has been compared for both the numerical model and field data. Updating the soil properties, it was required to look back on the results of the soil tests. Accordingly, depending on the type of field soil test and its accuracy, the main uncertain parameters are considered to be revised, and the rests are fixed. In this study, the shear strength parameters were entirely estimated by laboratory tests, since there were spatial constraints in Africa Street to perform more efficient filed tests. Thus, the estimated ones are not enough reliable due to large disturbance in soil specimens. Altogether, due to existence of possible errors in test performance, too conservative estimations and limited numbers of field and laboratory tests in area around the intended tunnel, the mechanical parameter (i.e., modulus of elastic deformation "E") and the shear strength parameters (i.e., frictional angle "ϕ" and cohesion "C") have potentially the most uncertainty.

V. Results

As previously mentioned in Section IV, a numerical model developed by the first estimated soil parameters. According to the obtained results of that numerical model, the surface settlement of about 6.8 cm and 7.2 cm occurred at the ground surface and top of the tunnel section, respectively (Figs. 10 and 11). Contours of the vertical displacements of the ground mass and the total value at the ground are presented in Figs. 10 and 11, respectively. As shown in Fig. 10, a large settlement occurred at the top area of the tunnel. While, with regard to the field observations and monitoring data (as shown previously in Fig. 9), low and relatively uniform displacements recorded at three stages rather than each other (see on every diagram in Fig. 9, Section III). Again in Fig. 12, the field settlement diagram was compared with the obtained numerical result of the corresponding point at stage No. 1794. Based on this figure, the displacement ratios by the numerical result are in big difference with real ones, in spite of similar ground behavior at different phases (the phases are marked with dotted lines in Fig. 12). In the other words, a non-uniform distribution of settlement occurred at different phases of tunnel construction and this is unlike with the field observations and monitoring data which are low and relatively uniform at those phases (Fig. 12). In total, it indicates that the primary considered soil properties are uniformly too weak rather than real geological observation and geotechnical characteristics.

According to Fig. 13 (a), the results of soil tests shows that the primarily considered values of internal friction angle "ϕ" are adequately well-estimated which are near to the upper limits of field and laboratory results. While for the cohesion "C" in Fig. 13 (b), the values are low-estimated in comparison with field tests. Despite low estimation for cohesion values in laboratory rests, only the field results are reliable due to remolding effects for soil sample. Also, as shown in Fig. 13 (c), the primitive considered values for modulus of elastic deformation, are conservatively low-estimated. Finally, with a view to the applied constitutive model (HS model) and considering fixed values for more reliable parameters including friction angle "ϕ" and P_{ref} , a sensitivity analysis performed by revising the rest of soil parameters like elastic modulus and cohesion. Therefore, the values of "Elastic modulus E" and "Cohesion C" parameters are updated in two categories (Table IV), as much as, the final surface settlement matched with the monitoring data.

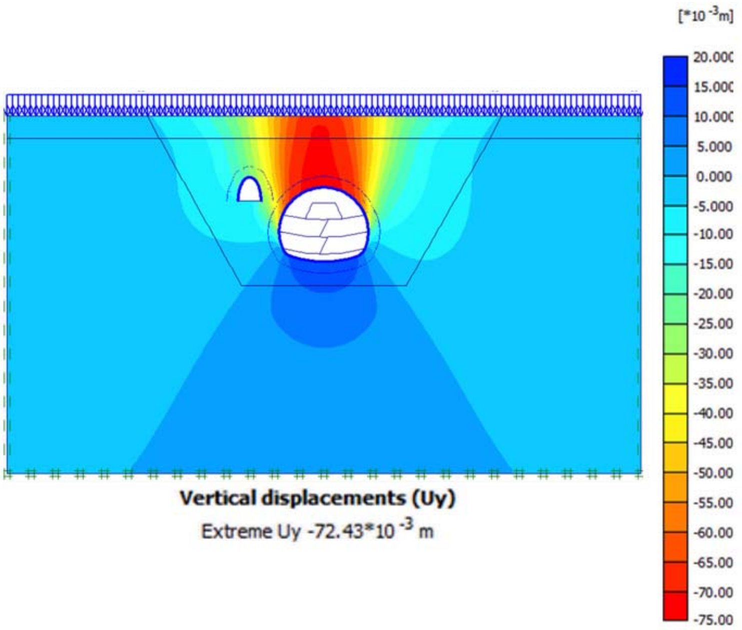


Fig. 10 Induced vertical displacement at the end of construction (based on primitive properties)

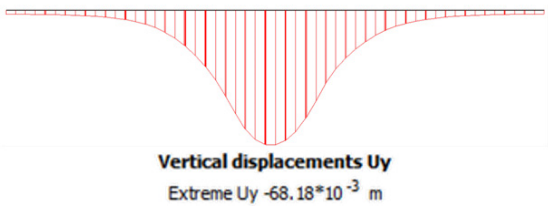


Fig. 11 Induced vertical displacement at the ground surface (based on primitive properties)

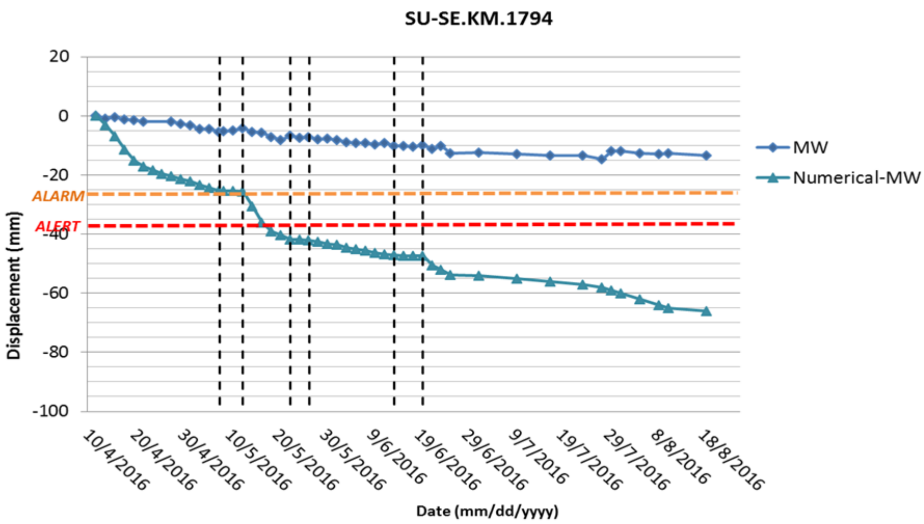


Fig. 12 Surface settlement; the numerical results vs. field data at stage No. 1794 m; (black dotted lines refer to similar behavior of the soil at different phase of construction)

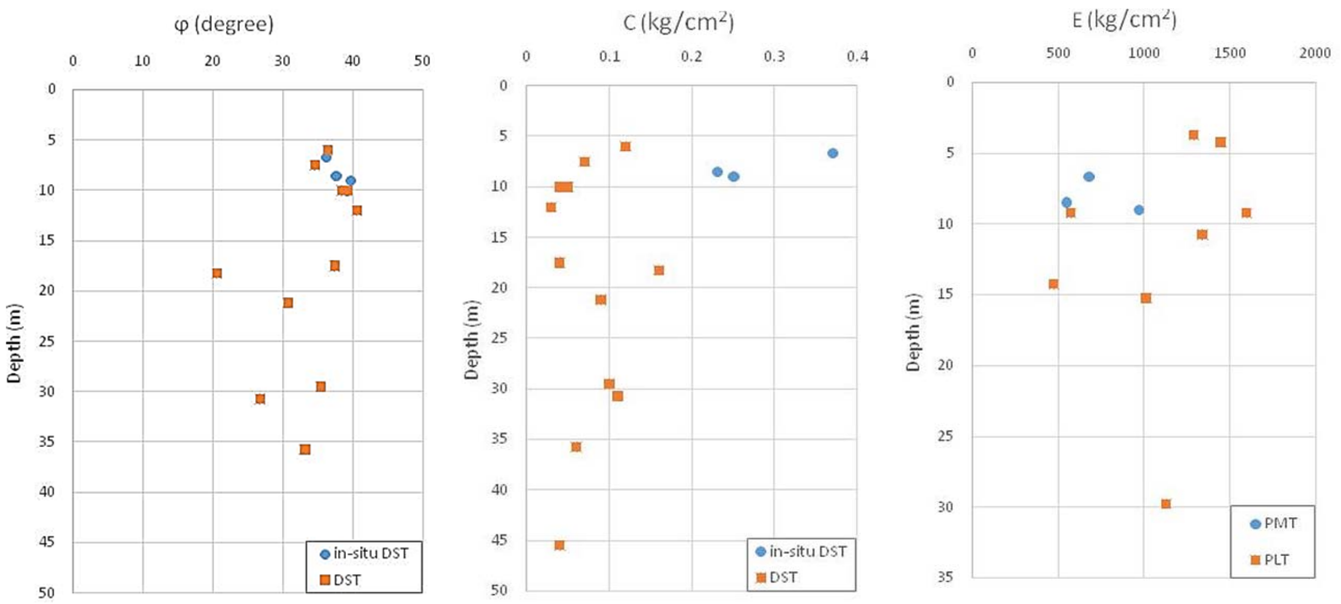
According to the different values of updated soil parameters, six more numerical analysis were developed in two categories, through which the maximum responses of the ground surface were diagrammed (Fig. 14). It should be noted that the cohesion "*C*" and elastic modulus "*E*" parameters in Fig. 14, are weighted average in a soil layer with specific thickness (i.e., considered a thickness of 3.5 m and 54 m for the first and the second soil layers, respectively). With regard to the results diagrammed in Fig. 14, an exponential trend made for the both two categories (i.e., changed a specific parameter cohesion "*C*" or elastic modulus "*E*" for each one). As discussed in the previous section, the criterion is to reach near to the intended field settlement as high as 1.5 cm. The obtained results show that it is possible to derive proper values for the parameters. Although, there is low experimental information about the first 3.5m soil layer, engineering judgment made by field observations and borehole samples (see the Appendix), and consequently, it could be possible to rise the cohesion "*C*" and elastic modulus "*E*" values up to 0.15 and 500 kg/cm^2, respectively. Considering these values and using the trend lines, the parameters of the second layer were calculated. In the other words, the calculated values indicate the proper upper limits, based on which, the elastic modulus "*E*" and cohesion "*C*" parameters for the second layer have an upper limit to 0.39 and 1870 kg/cm^2, respectively. On the other hand, it is required to consider the interaction of both two parameters in final displacement obtained by numerical analysis. Finally, by making an interpolation between the soil parameters at intended settlement and again developing numerical analysis by one more additional set of interpolated parameters, the final set of values was obtained. The final updated parameters are summarized in Table V. Eventually, as shown in Figs. 15 and 16, by the revised values of soil properties, the final vertical displacement at the ground surface reached to 2.3 cm which is close to monitoring data (1.5 cm). It is noteworthy to say that the parameters changed up to 40%, which emphasize that the field and laboratory tests data are not enough reliable to be applied.

TABLE IV
CONSIDERED SOIL PARAMETERS FOR BACK-ANALYSIS PROCEDURE

Category	Quantity			
	First Layer		Second Layer	
	E (kg/cm^2)	C (kg/cm^2)	E (kg/cm^2)	C (kg/cm^2)
1	400	0.10	900	0.25
	500		1000	
	500		1200	
2	400	0.15	700	0.30
		0.15		0.35
		0.20		0.4

TABLE V
NEW PARAMETERS OF SOIL MATERIALS (BACK-ANALYZED PARAMETERS)

Symbol	Quantity	First Layer	Second Layer
ϕ	Internal friction angle (degree)	30	37
C	Cohesion (kg/cm^2)	0.15	0.35
γ_m	Natural density (gr/cm^3)	17	18
P_{ref}	Reference vertical effective stress (kN/m^2)	10	33
ν_{ur}	Poisson ratio of unloading/reloading	0.2	0.2
E_{50}	Secant deformation modulus (kg/cm^2)	500	1000
E_{ur}	Unloading stiffness (kg/cm^2)	1500	3000
ψ	Dilatancy angle (degree)	0	7



(a) (b) (c)

Fig. 13 Variations of soil parameters vs. depth of soil sample based on in-situ and laboratory tests; (a) internal friction angle, (b) cohesion, (c) modulus of elastic deformation

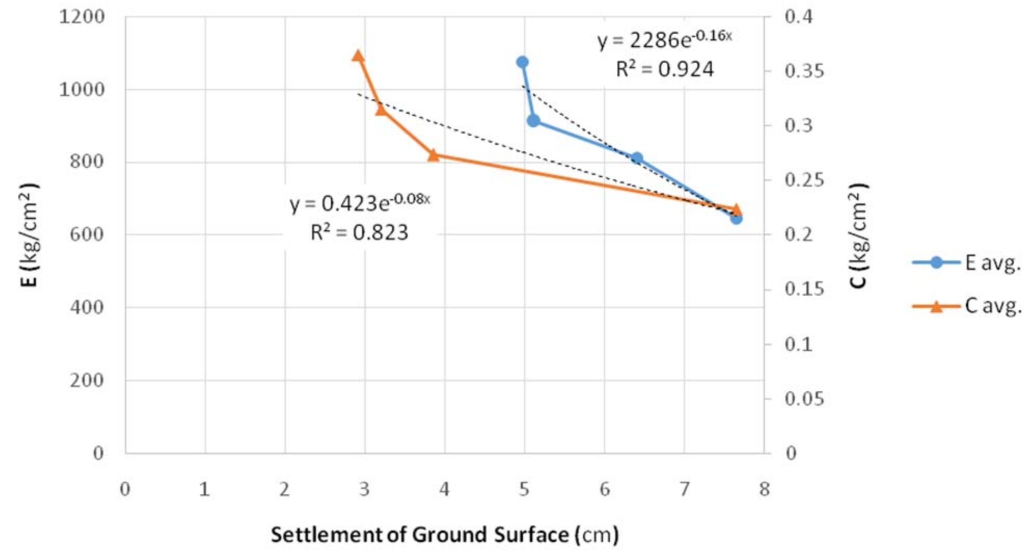


Fig. 14 Variation diagrams of E_{avg} and C_{avg} values vs. maximum surface settlement (E_{avg} and C_{avg} are weighted average in soil layer)

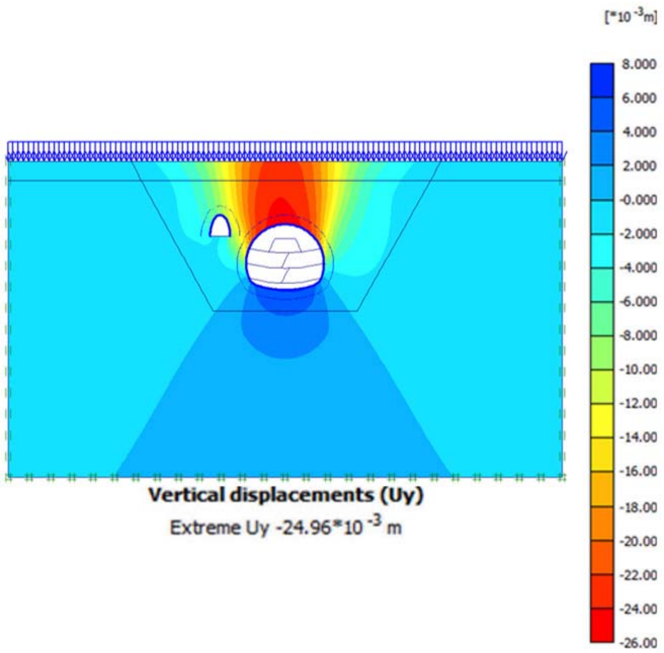


Fig. 15 Induced vertical displacement at the end of construction (based on the on the revised soil properties)

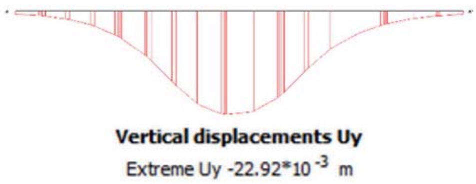


Fig. 16 Induced vertical displacement at the ground surface (based on the revised soil properties)

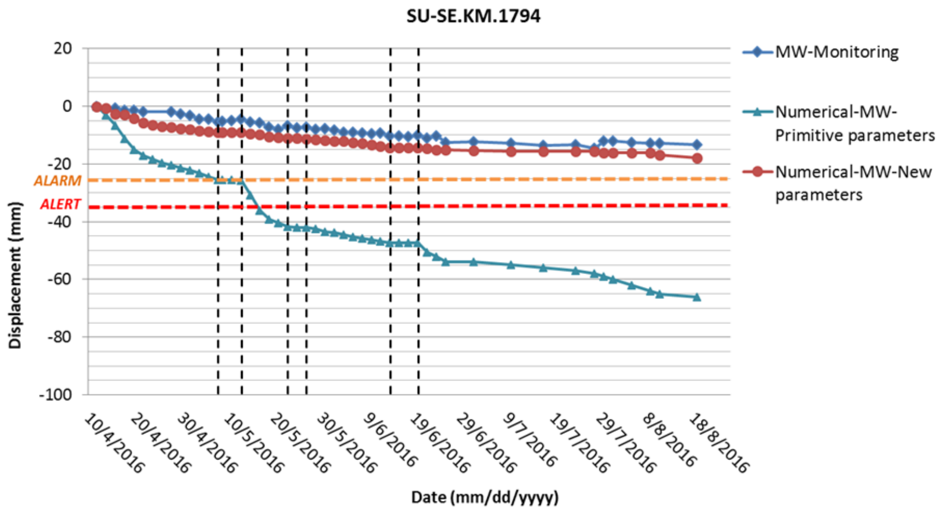


Fig. 17 Surface settlement; the numerical back-analyzed results vs. field data at stage No. 1794 m; (the back dotted lines are referred to the similar behavior of the soil at different phases)

APPENDIX

In this project, different boreholes were excavated in three different places along the tunnel axis, and down to a depth of about 30 m. The borehole locations are presented in Fig. 7, Sections II and III. As shown in this figure, the intended boreholes are numbered BH-A1, A2, A3. Here, some images from the soil samples to the depth of 18 m are presented.

TABLE VI
CORE SAMPLES OF FIRST 18 METERS SOIL LAYER AT THREE DIFFERENT BOREHOLE, LOCATION NUMBERS: A1, A2, A3

Box No.	BH-A1	BH-A2	BH-A3
0 to 4 m			
4 m to 8 m			
8 m to 13 m			
13 m to 18 m			

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