Available online at www.CivileJournal.org

Civil Engineering Journal

Vol. 2, No. 6, June, 2016

Evaluation of Building Displacement Induced By EPB Tunneling Through GPS-GNSS Monitoring System and Back Analysis Technique (Tabriz Subway Twin Tunnels)

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Received 09 May 2016; Accepted 20 June 2016

Abstract

One of the main aspects of tunneling in urban areas is controlling the amount of settlement that might cause some damage to the structures and infrastructures. In this paper, the novel displacement monitoring system called Global Positioning System - Global Navigation Satellite System (GPS-GNSS) has been applied to monitor the building displacement .One of the most important features of this approach is that this system provides three dimensional displacement behavior of the building. Besides, in order to fulfill the purpose of accuracy, the amount of settlement induced by Earth Pressure Balance (EPB) tunneling was calculated by numerical, empirical and analytical methods. In order to achieve this purpose, the back analysis technique was considered. The order in which the geotechnical parameters are optimized depends on the amount of sensitivity function. That is, the parameter of high sensitivity function is optimized first. According to the calculations, the sensitivity analysis results show that the maximum amount of sensitivity function with the volume loss of more than 1% in respect to the internal friction angle is about 0.5, which is greater than other geotechnical properties. According to the results of back analysis technique, the optimized geotechnical properties were elastic modulus ($E = 22\ MPa$), internal friction angle ($\phi = 32$) and cohesion ($C = 8\ kPa$) found on the volume loss of 1.5% with less than 0.02% error. The maximum settlement of the building at the studied area**,** explored by the optimized numerical method**,** is about 4 mm**,** which is in the range of monitored data (3mm-13mm) obtained through GPS-GNSS procedure.

Keywords: Settlement; Geotechnical Properties; Tabriz Subway; GPS-GNSS Displacement Monitoring System; Back Analysis.

1. Introduction

Developing transportation system in populated cities needs constructing subsurface infrastructures such as tunnels. With a close attention to the populated urban areas, tunnels are more likely to cross near the adjacent structures and might lead to several difficulties on the surface structures. Thus, the observing of settlement that occurs due to tunneling in urban areas is an essential issue. The surface settlement can be estimated by using various approaches such as empirical or semi-empirical methods (Peck, 1969) [1], analytical methods (Loganatan–Poulos, 1998) [2] and numerical methods [3, 4]. These studies are based essentially on geotechnical soil features and working peculiarities (e.g., excavation geometry and methodology). Additionally, several monitoring systems have been developed to measure the building displacement and surface settlement. Among them, the Global Positioning System -

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Global Navigation Satellite System (GPS-GNSS) has been increasingly used to conduct the real time monitoring of the building displacement [5-7]. The system has the potential to measure ground and building displacement in various geotechnical projects. It is composed of three units including the measurement unit, the server unit and the user unit. The measurement unit consists of receivers which are placed at the monitoring site; the server unit consists of the communication device and a server computer, while the user unit is a standard PC equipped with a web browser. GPS-GNSS can be used in different ways. One of the most effective methods is monitoring through GPS-GNSS system via "static" mode. In "static" mode, one of the dual-frequency GPS receivers is placed at the base point and the others are placed on the building and receive the signals simultaneously from the same satellites. In order to process the raw data collected by receivers, GNSS (Thales Navigation) software is widely used. GNSS is the indispensable software tool for all surveyors who need to be efficiently and smoothly assisted in their surveys. GNSS offers high standards of performance, processing speed, compactness and flexibility. It includes all the necessary tools to prepare a real time job and upload it to your field unit. Once you have concluded your survey, GNSS provides the ability to accurately determine site location within the parameter you established. In addition, in order to grasp precise geotechnical properties, the back analysis technique was applied. Back analysis is generally defined as a technique which can provide the controlling system parameters by analyzing its output behavior. This technique has been constantly studied in geo-engineering during the last decade. The back analysis utilizes two different methods to solve the problems, defined as inverse and direct approaches (Ghorbani & Sharifzadeh 2009) [8]. The direct approach is based on an iterative procedure correcting the trial values of unknown parameters by minimizing error function. This method has the advantage that it can be applied to non-linear problems without having to rely on a complex mathematical background. In this research direct back analysis technique is used to determine the main soil properties consisting of elastic modulus (E), internal friction angle (ϕ) and cohesion (c). In order to verify the validity of the results, four steps have been taken into consideration. First of all, monitoring of the building displacement between Tabriz University square and Abrasan cross road was carried out by GPS-GNSS displacement monitoring system and building with high vulnerability index was selected. Secondly, in order to conduct further research and reach accurate geotechnical properties, the back analysis technique was applied. Then, according to the results of optimized back analysis technique, building displacement and surface settlement were assessed. Finally, the obtained settlement from numerical analysis method was compared with the results of novel monitoring system to optimize the results and verify the study [9].

2. Tabriz Subway

Tabriz, with 160 km^2 area and the population about 1,490,000, is one of the most crowded and important cities in the northwest of Iran. According to the city transportation studies, 4 subway lines with total length of 48 km and / extendable to 72 km / were considered. The case study of this paper is a part of Line 1 of Tabriz subway, which consists of a subway system with a length of 17.2 Km of twin 6.6 meter diameter tunnels and 18 stations. The line is routed through the downtown of a major metropolitan area and beneath the crowded city streets and adjacent to high raised and important buildings such as a Tabriz International Hotel. The studied area is located above water table level and generally in silty sand soil. Earth Pressure Balance (EPB) machines were used to excavate twin tunnels at shallow depth in the soft ground conditions beneath a developed part of Tabriz. Tunnels outer diameter is 6.88 m, inner diameter is 6 m and outer supported diameter with reinforced pre-fabricated concrete segments is 6.6m. The lining is essentially an assembly of concrete segments with 30 cm thickness and 1.4 m width. The void space of 14 cm between concrete segments and adjacent soil is filled by cement grout during earth pressure balance (EPB) machine advancing.

In the route of Line 1, twin tunnels are adjacent to heavy structures such as a Tabriz International Hotel. Crossing close to such a heavy surface structure dictates several constraints on the design and construction of Line 1 twin tunnels. Tabriz International Hotel, which is about 60 m in length and 16 m in width and 20 m in height, is a reinforced concrete building .The relative position of Tabriz International Hotel and twin tunnels is illustrated in Figure 1 The thicknesses of overburden from the center of tunnels at Tabriz International Hotel section is about 10.5 m [9].

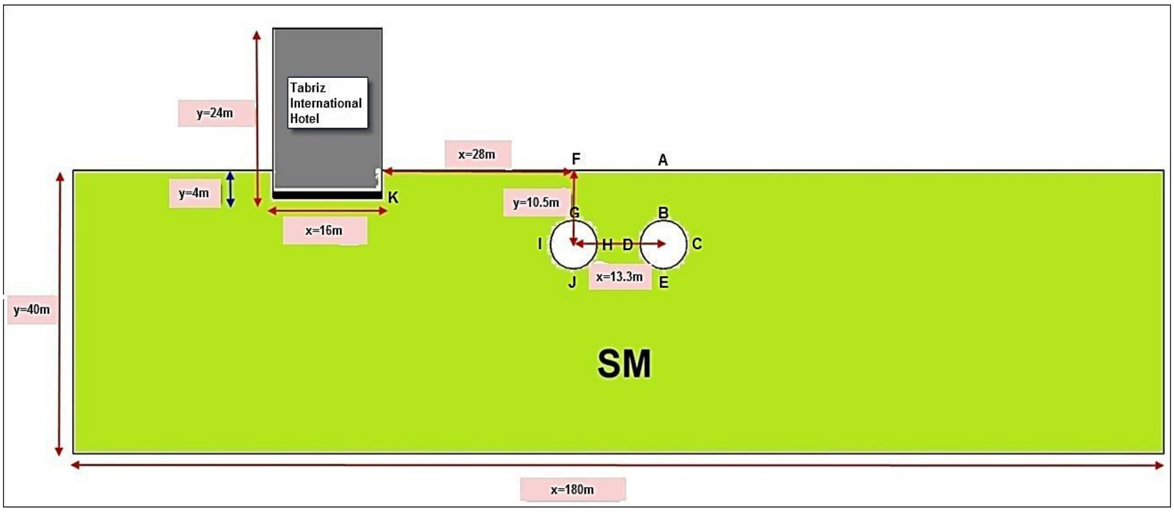


Figure 1. Relative position of twin subway tunnels to Tabriz International Hotel, soil layers and critical points

3. GPS-GNSS Monitoring System

3.1. Building Monitoring

Building displacement monitoring was performed to measure the displacement which occurred owing to full face tunnel excavation of Tabriz subway-Line1. For this purpose, at least three receivers were needed. In other words, three receivers are required in order to make triangle networks. That is, according to the standards of national cartographic center of the country and scientific principles of surveying activities, the triangle networks can help surveyors achieve more accurate coordinate by processing the networks and adjusting the amount of error caused by surveying. To accomplish this objective, first, receivers (MC1, MC2 and MC3) were installed on the building as fixed points to observe the building displacement. Then, in order to monitor the building displacement at least two phases of observation were required. In the first phase of observation, monitoring of the building displacement was performed in a "static" mode with receivers receiving the data for at least 48 hours before the tunnel excavation and the recorded data in "static" mode were processed by GNSS software to calculate the three dimensional displacement of the building in real time before the tunnel excavation. During the second phase of observation, the building displacement monitoring was performed in a "static" mode with receivers receiving the data for at least 48 hours after tunnel excavation and the obtained data were processed by GNSS software to obtain the three dimensional behavior of the building in real time after tunnel excavation. The position of GPS monitoring points on the building were measured relative to Tabriz station base point. One of the receivers was installed at this point while receiving and saving data from satellite every 48 hours. This point was considered as the base point to make a quadrilateral networks with three other GPS receivers installed on the building. Three dimensional displacement of the building was obtained after processing the measured data [9].

3.2. Monitoring Data Processing

After the surveys have been carried out, GNSS provides the ability to accurately determine site location within the established parameter. Figure 2 outlines the steps for processing the raw GNSS data. First, new project is created. Second, based on your country Time Zone default setting is modified; one of the primary advantages of GNSS solutions is the ability to work within your own coordinate system from the start of your project. Then, data are downloaded from receivers into your project. The data logged on the data card or receiver memory during your field surveys can be downloaded into a GNSS Solutions project. Next, downloaded data are processed. Raw data collected by a receiver must be processed to determine the differential relationship between the points occupied during data collection. The result of processing GPS raw data is a vector defining this relationship. Computation of these vectors is the role of the data processing module within GNSS Solutions. The data processing module automatically analyzes the quality of the raw data files and adjusts processing parameters to produce the best vector possible, transferring most of the processing effort from the user to processing software. GNSS data is processed through three steps:

i) Pre-process data analysis: point and observation properties such as Site coordinates, control point information and adding data file to a project;

ii) Processing: processing the created vectors from raw data;

iii) Post processing data analysis: processed GNSS vectors are analyzed to determine the quality of processed data.

Finally, network adjusting is carried out. The survey observation is one of the most important tasks to ensure accurate, reliable results. A network adjustment is performed to accomplish two results:

i) To test for blunders and errors in the observations (vectors between points in our case)

ii) To compute final coordinates for your survey points which are consistent with the used existing control points.

Adjustment takes place after the raw data have been processed and no unaccountable errors in the processed results have been found. Adjustment is typically featured by two stages:

i) The minimally constrained adjustment.

ii) The constrained adjustment.

The first stage is used to detect problems in the observations and control coordinates. You may have to iterate several times, using a number of different tools to check for blunders.

Once you are confident that no blunders remain, you can proceed to the second stage, where you hold fixed all the control points and readjust to obtain final site positions and accuracies [10].

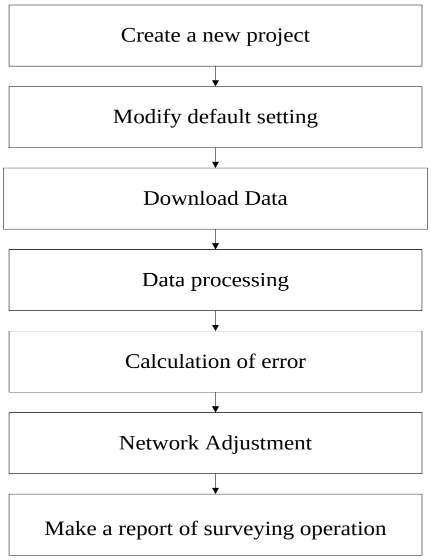


Figure 2. Monitoring data processing using GNSS [10]

3.3. Real Time Data Monitoring Before and After Excavation

In the first place, the real time data monitoring procedure was carried out on the building before excavation of the tunnels to measure and analyze the obtained data. In other words, all the steps discussed in Figure 2 were applied to achieve the three dimension displacement behavior of the building. In order to conduct a comprehensive monitoring activities, there should be an appropriate time overlap among the receivers which is the dominant factor in field surveying and also is required for accurate data processing through GNSS software. According to the calculations, the surveying time of MC1 is 14.5 hours, MC2 is 12.5 hours and MC3 is 24 hours, which shows a suitable time overlap among the receivers. Therefore, the amount of error that might arise due to lack of time overlap among the receivers decreases. The thorough information of processing stage such as the relative position of receivers, each vector length, vector components and the amount of error are illustrated in Table 1. As it can be seen, the amount of error for vector length is approximately 0.001 and amount of vector components error is about 0.001, which indicates the high accuracy of processing data. The final results of building displacement monitoring before the excavation of the tunnels can be seen in Table 2.

Table 1. Vector components before excavation of tunnels

Vector identifier	Vector length(m)	Error%	Vector components(m)		Error%
MC1-MC3	27.376	0	X	-15.325	0
			Y	21.715	0
			Z	-6.561	0
MC2-MC1	20.195	0	X	2.311	0
			Y	-17.495	0
			Z	9.819	0
MC2-MC3	14.063	0	X	-13.014	0
			Y	4.220	0
			Z	3.258	0
TAB-MC1	1836.082	0.001	X	628.332	0.001
			Y	-1527.231	0.001
			Z	802.474	0.001
TAB-MC2	1816.454	0.001	X	626.021	0.001
			Y	-1509.515	0.001
			Z	792.655	0.001
TAB-MC3	1809.926	0.001	X	613.007	0.001
			Y	-1505.516	0.001
			Z	795.913	0.001

Table 2. Final coordinates of Tabriz International Hotel based on (UTM/WGS84 zone map)

GPS position		Tabriz International Hotel	
Station position		Surveying station	
Beginning date		2009/9/14	
End date		2009/9/16	
receivers	MC1	MC2	MC3
Beginning time	7.00	7.00	7.00
End time	21.30	7.00	19.00
Time(hours)	14.5	24	12
X(m)	616324.938	616338.891	616351.122
Y(m)	4213864.62	4213850.276	4213856.672
Z(m)	1478.559	1481.212	1478.548

In the second place, the real time data monitoring approach was performed after excavation of the tunnels (Figure 3. a) represents the duration of surveying time of the receivers. The surveying time of MC1 is 17.5 hours, MC2 is 17.5 hours and MC3 is 17.5 hours, which indicates suitable time overlap among the receivers. Therefore, like the process of monitoring activities before excavation of the tunnels, the amount of error that might arise due to lack of time overlap among the receivers decrease. (Figure 3. b) illustrates the position of the monitoring points installed on Tabriz International Hotel. The detailed information of processed vectors such as **(**the relative position of receivers, time duration, each vector length, vector components and the amount of error**)** are illustrated in Table 3. As it can be seen**,** the amount of error for vector length is about 0.001 and amount of vector components error is about 0.001 which indicates the high accuracy of data monitoring processing. Final results of the building displacement monitoring after the tunnels excavation are shown in Table 4. Total displacement of the building can be obtained by comparing the monitoring results before and after the tunnels excavation which are shown in Table 5.

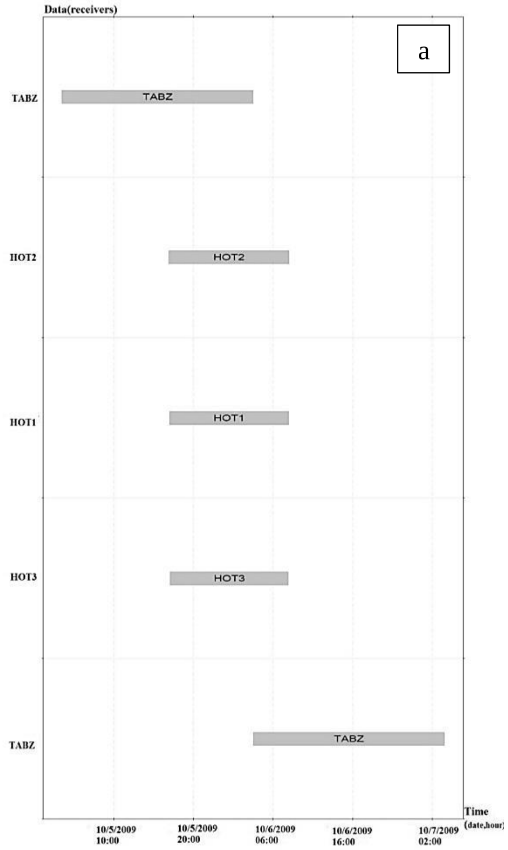


Figure 3. GPS-GNSS monitoring setup after tunnel excavation: (a) duration of surveying time of the receivers; (b) position of the monitoring points on the building after excavation of twin tunnels

Table 3. Vector components after excavation of tunnels

Vector identifier	Vector length(m)	Error%	Vector components(m)		Error%
MC1-MC3	27.373	0	X	15.322	0
			Y	-21.716	0
			Z	6.553	0
MC2-MC1	14.067	0	X	-13.017	0
			Y	4.223	0
			Z	3.259	0
MC2-MC3	20.189	0	X	2.306	0
			Y	-17.493	0
			Z	9.811	0
TAB-MC1	1809.923	0.001	X	613.005	0.001
			Y	-1505.514	0.001
			Z	795.910	0.001
TAB-MC2	1816.455	0.001	X	626.021	0.001
			Y	-1509.738	0.001
			Z	792.652	0.001
TAB-MC3	1836.076	0.001	X	628.327	0.001
			Y	-1527.230	0.001
			Z	802.463	0.001

Table 4. Final coordinates of Tabriz international hotel based on (UTM/WGS84 zone map)

GPS position		Tabriz International Hotel	
Station position		Surveying station	
Beginning date		2009/10/05	
End date		2009/10/06	
receivers	MC1	MC2	MC3
Beginning time	17.30	17.30	17.30
End time	7.30	7.30	7.30
Time(hours)	14	14	14
X(m)	616324.941	616338.888	616351.123
Y(m)	4213864.615	4213850.275	4213856.672
Z(m)	1478.551	1481.21	1478.547

Table 5. Total displacement of Tabriz international hotel

Receiver	X(mm)	Y(mm)	Z(mm)
MC1	3	5	8
MC2	3	1	2
MC3	1	0	1

4. Determination of Geotechnical Properties Using Numerical Back Analysis

As shown in Figure 4, the geology of soil crossed by Line 1 Tunnel mainly consists of alluvium formations with different particle size: i) sand with silt; ii) gravel with silt and loam admixtures; iii) finer soil, being composed by pure silt and loam. Both shallow and deep geotechnical investigation, in-situ and laboratory tests were performed to know soil physical and mechanical properties. The ground water level was observed approximately at 5 m below the tunnels floor. Table 6 provides a description of physical and geotechnical properties of soil layers: in particular, soil type has been classified using Unified Soil Classification System. According to Figure 4, the top soil is rather homogeneous in terms of thickness and geotechnical properties.

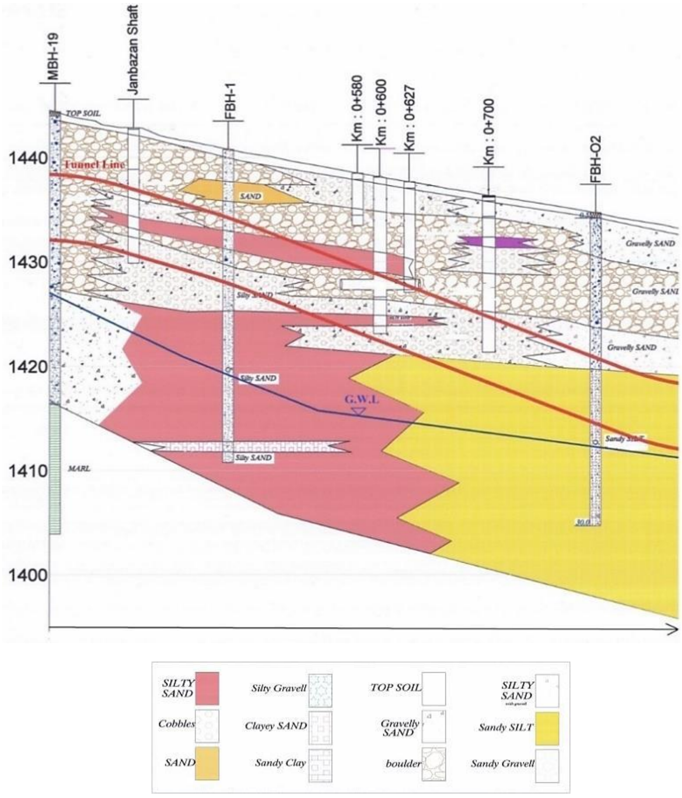


Figure 4. Geological profile of studied area

Table 6. Physical and geotechnical properties of soil layers.

Depth(m)	Soil type	Density (saturated) $\frac{gr}{cm^3}$	Density $\frac{gr}{cm^3}$	C(kPa)	ϕ(deg)
0-4	SP-SM	2.2	2.04	16	40
4-8	SP-SM	2.2	2.04	16	40
8-14	SP-SM	2.1	1.93	18	37
14-18	SM	2.1	1.93	11	36
18-20	SM	2.1	1.92	8	33
20-21	SM	2.1	1.89	6	39
21-24	SM	2.1	1.88	11	39
24-26	SM	2.03	1.85	7	30
26-27.7	SC	2.03	1.84	3	39
27.7-29	SC	1.98	1.77	2	36
29-30	SM	2.03	1.84	3	37

4.1. Description of Modelling

In this study, numerical simulations have been performed using the finite difference code FLAC2D, which has special features to analyse the geotechnical problems. The whole numerical model consists of 35878 rectangular elements, featured by different sizes: the smallest elements have been assigned around tunnels and ground surface in order to better model the effects induced by tunnelling. The behaviour of grout and soil layers is assumed linear elastic and elasto-plastic Mohr-Coulomb, respectively. The mechanical and physical properties of material used in modelling are presented in Table 7 for the grout (based on ASTM D 2166) and segments. The 70 kPa loading on the building foundation have been applied for modelling the Tabriz International Hotel (a7storey building). The tunnel construction process has been modelled by using the back analysis approach [11-15].

Table7. Pre-fabricated segments and grout properties

Parameters	Model	Density ($Kg/m3$)	E (MPa)	Poison ratio (ν)
Pre-fabricated segments	Elastic	2400	23000	0.15
grout	Elastic	1880	365	0.3

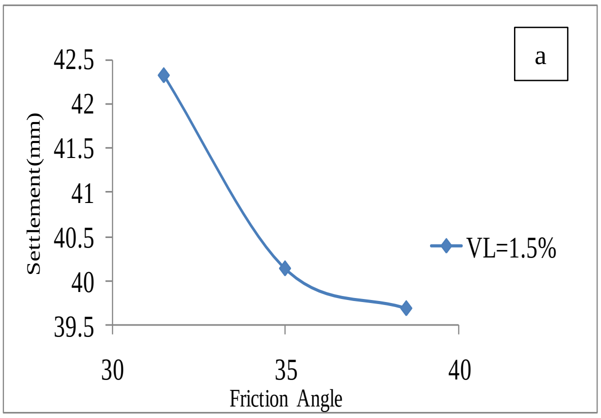
4.2. Sensitivity Analysis

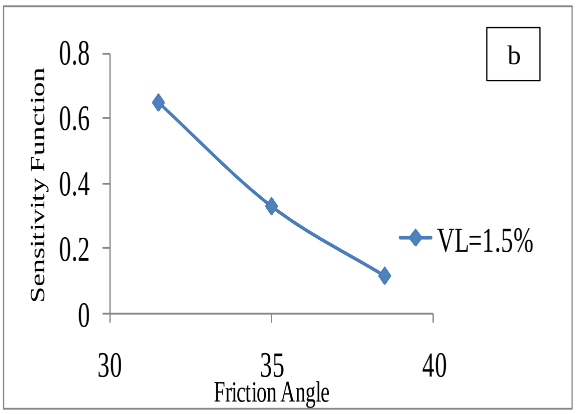
Sensitivity analysis is the method used to analyze the stability of a system. Given a system whose character, P, is governed mainly by n factors of $\alpha = \{\alpha_1, \alpha_2, \dots, \alpha_n\}$, $P = f\{\alpha_1, \alpha_2, \dots, \alpha_n\}$ under a reference state of $\alpha^* = \{\alpha_1^*, \alpha_2^*, \dots, \alpha_n^*\}$ the character is described by p^*. Once the basic parameter set is determined, the sensitivity analysis can be performed on each parameter. The sensitivity analysis is to, let the above individual factors vary within respective possible range and then analyze both tendency and extent to which the character of system, P, departs from the base state due to the variation of factors. The first step of sensitivity analysis is to establish the system model, i.e., functional relation between the system's character and the factors; $P = f\{\alpha_1, \alpha_2, \dots, \alpha_n\}$. When analyzing the effect of α_k on the characteristic of the system, p, the parameter (α_k) in the set is varied within a possible range while the remaining parameters are kept constant. In this case the system's character displays the following relation:

$$P = f(\alpha_1^*, \alpha_2^*, \dots, \alpha_n^*) = \varphi_k^{(\alpha_k)} \tag{1}$$

The above analysis only gives the sensitivity behavior of the system character P to a single factor. The character of the real system is generally governed by many factors of different physical quantities and units. Therefore it is difficult to compare the sensitivity of the various factors by the above method. To solve this problem, dimensionless treatment and analysis can be applied. In dimensionless analysis, the sensitivity function and sensitivity factor are defined in dimensionless term. The ratio of the relative error (δ_p), of the system character P (($S_P = \frac{|\Delta P|}{P}$)) to the relative error of parameter α_k $\left(S_{\alpha_k} = \frac{|\Delta \alpha_k|}{\alpha_k}\right)$ is defined as the sensitivity function. It is defined by Equation (1). In this study the system character, namely, assessment of the excavation on settlement, the parameter used for sensitivity analysis include, elastic modulus (E), cohesion (C), internal friction angle (∅). Accurate soil properties can only be obtained from large in situ tests. Such tests are seldom carried out as they are very expensive and time consuming. Sensitivity analysis of parameters can be applied for the optimization of testing schemes. All the parameters have been analyzed one by one using the method stated earlier. It should be noted that the above conclusion have been drawn on the basis that each basic parameter varies independently. The interactions between parameters have not been considered. A more comprehensive study of sensitivity can be conducted by coupling the present method with a parameter interaction study. Nevertheless, the present method is usually sufficient to quantify the sensitivity of various parameters. In order to determine the sensitivity of geotechnical parameters, sensitivity analysis was performed on each parameter for different volume loss (0.8%, 1.1%, 1.3% and 1.5%). The basic parameters for sensitivity analysis are given in Table8.The character of U-ϕ is plotted from Equation (1) which describes the sensitivity of U (surface settlement) to disturbance of (∅). The rapid change of the curve around $\emptyset = 30$ shows a high sensitivity of U to ∅: slight change of ∅ causes a great change of U. On the contrary, the curve is gently around $\emptyset = 37$, the system character is less sensitive to the parameter (∅): U varies slightly with a large change of ∅. In other words, when the parameter is near ($\emptyset = 30$), it is a parameter of high sensitivity; it can be concluded that such high sensitivity is likely due to some plastic failure induced by failure that probably occur for friction angle lower that 37; when it is near $\emptyset = 37$, it is of low sensitivity. (Figure 5.a). Curves representing U-E, U-C are plotted from computing results are shown in (Figures 6.c and e). From Equation (2) the sensitivity function curve of parameters can be obtained which is shown in (Figures 5.b, d and f). As shown in (Figure 5.b), S_ϕ-∅ is a decreasing function and the sensitivity is high when ∅ is low and decrease with increase ∅. Similar analysis can be performed on other parameters (Figure 5.d and f). Table 8 summarizes the sensitivity factor of other parameters. Table 9 indicates that when the settlement of the ground surface is used to judge the settlement, the most sensitive factor that affects the surface settlement is (∅) [16].

$$S_K(\alpha_K) \triangleq \frac{\left(\frac{|\Delta P|}{P}\right)}{\left(\frac{|\Delta \alpha_k|}{\alpha_k}\right)}, k = 1,2, \dots, n \tag{2}$$





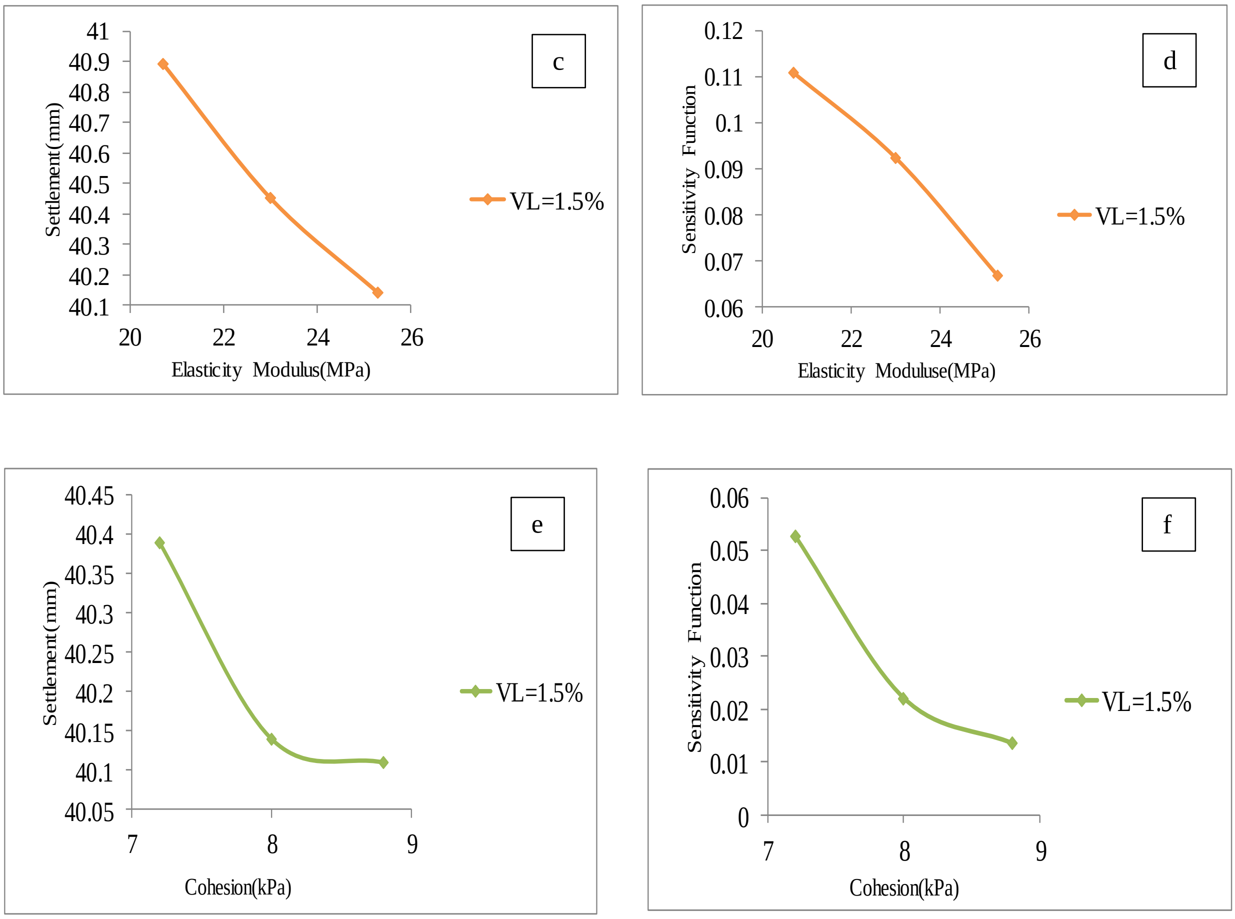


Figure 5. Sensitivity analysis results for different parameters with 1.5% volume loss. : (a) characteristic curve of system U-ϕ; (b) curve of sensitivity function S_{ϕ}-ϕ; (c) characteristic curve of system U-E ; (d) curve of sensitivity function S_E-E; (e) characteristic curve of system U-C; (f) curve of sensitivity function S_c-c

Table 8. Basic parameters set for sensitivity analysis

E (MPa)	C(kPa)	ϕ
22	7	30

Table 9. Total Results of sensitivity analysis

Sensitivity	VL (0.8%)	VL (1%)	VL (1.3%)	VL (1.5%)
S(ϕ)	0.22	0.23	0.3	**0.6**
S(E)	0.27	0.21	0.13	0.11
S(c)	0.026	0.028	0.04	0.05

4.3. Back Analysis

The equivalent ground loss model is adopted for back-analysis based on optimization theory, aimed to obtain the optimized parameters which will result in good agreement of predicted ground movement with field measurement. In order to perform back analysis technique, first, the average amount of displacement at tunnel boundaries after excavation of tunnel and prior to lining, considering different volume loss, was determined through Equation (3), where V_L is volume loss (m^3 per meter advance), V_s is ground loss (m^3 per meter advance), D is the excavation diameter (m), D_X is the tunnel diameter after specific amount of convergence based on the applied volume loss at tunnel boundaries. Therefore, according to volume losses previously determined from sensitivity analysis results (which were in the acceptable and valid ranges for full face excavation), the average amount of convergence at tunnel boundaries was obtained through Equation (3).

Then, the error function defined by Equation (4) was calculated and checked whether it is converged or not. If the error function is converging into a permitted limit, the iteration step will be stopped, or else, after modifying the parameters using optimization technique, step will be repeated. (u_k ,u_k^*) are computed and measured displacements at the measuring point I, respectively. N is the number of measuring points. In this paper direct back analysis technique was used to determine the main parameters for tunnel design that affect the surface settlements, such as elastic modulus (E), internal friction angle (ϕ) and cohesion(c) [17].

Table10 illustrates the results of back analysis for different volume losses. According to Table8, for 0.8% volume loss, the amount of error function is about 125% which means that for this amount of volume loss optimized geotechnical parameters are found with 125% error. In other words, this trend shows high amount of uncertainty because of considering low amount of volume loss than real practical one. When the amount of volume loss soars to 1%, the optimized geotechnical parameters are found with 33%, amount of error function which means that the amount of error function plunges to 33% since the amount of determined volume loss is approaching its actual value. Therefore, as the amount of volume loss increases from 0.8 to 1.5%, the amount of error function plummets from 152 to 0.02%. Thus, the most optimized geotechnical parameters are on volume loss 1.5% and the amount of error function is 0.02%. The optimized geotechnical parameters will be used as input parameters in numerical analysis.

$$V_L = \frac{V_s}{\mathrm{V_{excavation}}} \tag{3}$$

$$EROR = \frac{\sum_{k=1}^{N}[\mathrm{u_k} - \mathrm{u_k^*}]^2}{\sum_{k=1}^{N}\mathrm{u_k^*}} \tag{4}$$

Table10. Total results of back analysis

Volume loss%	ϕ	E (MPa)	C (kPa)	Error %
0.8	30	20	4	152
1	30	21	4	33
1.3	30	20	4	0.19
1.5	32	22	8	0.02

5. Numerical Analysis of the Effect of Twin Tunnel Excavation on Surface and Building Settlement

5.1. Surface and Building Settlement Due To Excavation of First Tunnel (RT)

In this part of research, the influence of excavation of first tunnel on surface and building settlement are evaluated. In this study, the FDM numerical model is adopted to simulate the process of tunneling in respect to 1.5% volume loss. Since, this is the more accurate amount of volume loss in regard to the results of sensitivity analysis and back analysis technique and is caused by the convergence of the ground into the tunnel after excavation. The vertical ground settlement profile after the first tunnel excavation is given in Figure 6. As it is clear, the maximum surface settlement is about 41 mm. maximum settlement at the building foundation is about 1mm.

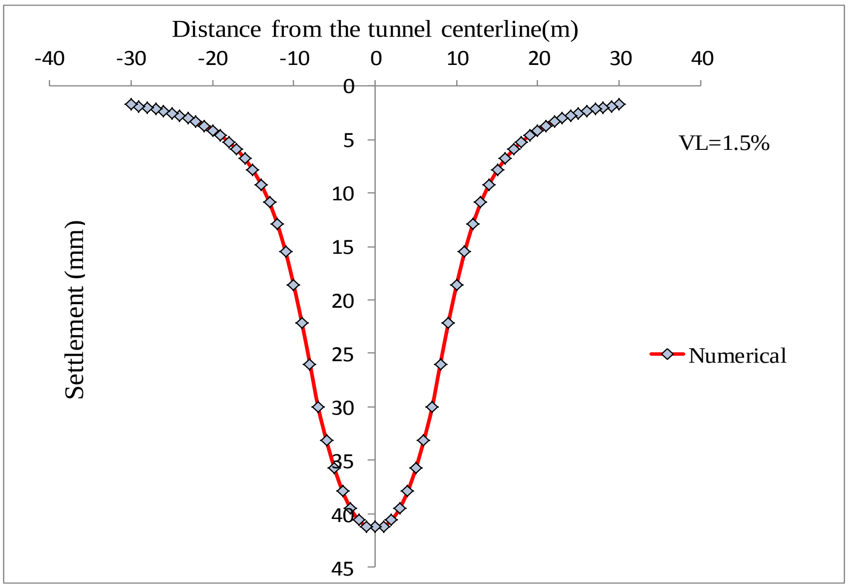


Figure 6. Surface settlement profile after excavation of first tunnel (RT)

5.2. Influence of Excavation of the Second Tunnel (LT) on Surface and Building Settlement

In this part, in order to analyze the influence of second tunnel excavation crossing parallel to the first one, surface and building settlement during the second tunnel excavation are evaluated. Higher settlement is expected in the left tunnel excavation since the previous TBM excavation activities on the right tunnel overlaps the previous deformation. The effect of left tunnel excavation on settlements is presented. As seen, after the left tunnel excavation maximum surface settlement reaches around 50 mm (Figure 7). It can be seen that there is a significant increase on surface settlement after the second tunnel excavation with the maximum surface settlement occurring on the centerline and the transversal trough shifting toward the centerline of this tunnel. Maximum settlement at the building foundation is greater than 4 mm.

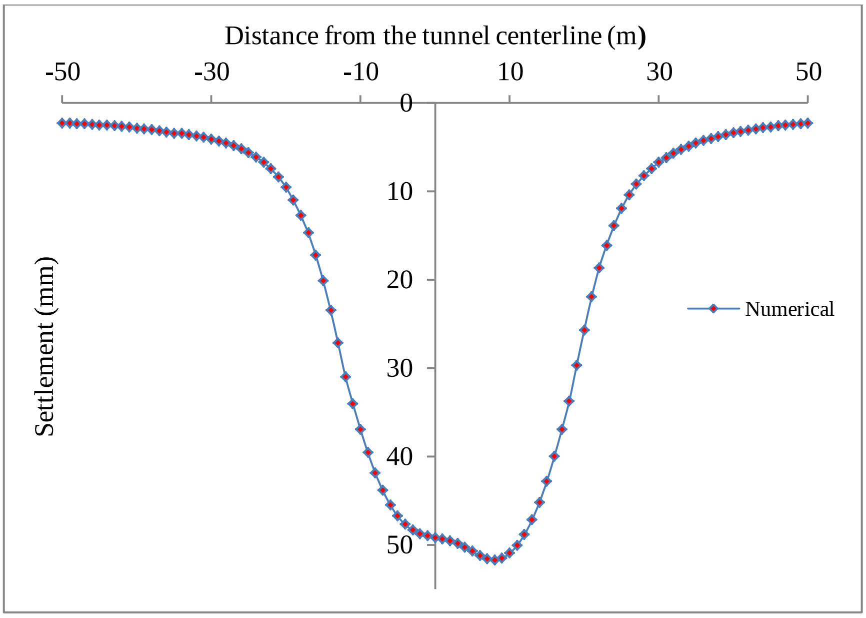


Figure 7. Surface settlement profile after excavation of second tunnel (LT)

5.3. Verification of Prediction by Field Measurements and Discussion

5.3.1. Field Measurements

The monitoring data recorded during the excavation phase have been compared to the cross-section settlement curves determined by applying empirical, analytical and numerical methods. Figure 8 shows the plan view of tunnel site which depicts the surface measurement section and building monitoring points. The results of measurements performed at monitoring point on ($Km\ 6 + 960.463$) for the first tunnel (RT), by Tabriz Urban Railway Organization is shown in Figure 9. It should be noted that the first tunnel excavation started about one year before the second tunnel construction. Settlement observation on the center line above a 6.88 diameter EPB shield in silty sand shows that most of the construction settlements occurred ahead of the tunnel face. This range is in accordance with the recent case histories: the Porto Metro, the Turin Metro, the Bologna High Speed railway link (Minguez et al., 2005) [18]. The pre-settlement ahead of the tunnel face reached an average 30% of the final settlement in these three cases.

As it can be seen from Figure 9, field measurement activities were carried out regularly for about 7 months at determined section for specific points which were located above the centre line of first tunnel. With a close attention to the trend of settlement, it can be concluded that the most of the construction settlements are associated with the tail void. That is, the pre-settlement ahead of the tunnel face reached an average 33% of the final settlement for considered section in this case which is approximately equal to 15 mm and the rest of settlement, about 26 mm, occurs due to factors which are mostly attributed to the tail void influence on settlement. In other words, the severity of settlement variation after cutter head passes the monitoring section becomes more noticeable. The final surface settlement for this case at monitoring point on ($Km\ 6 + 960.463$) is about 41mm.

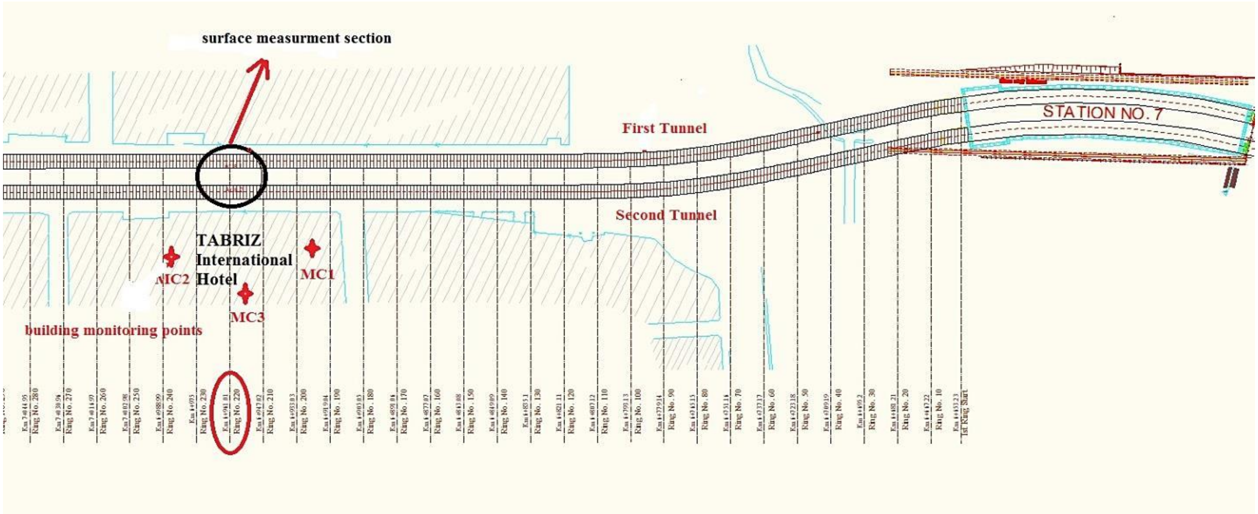


Figure 8. Plan view of tunnel site

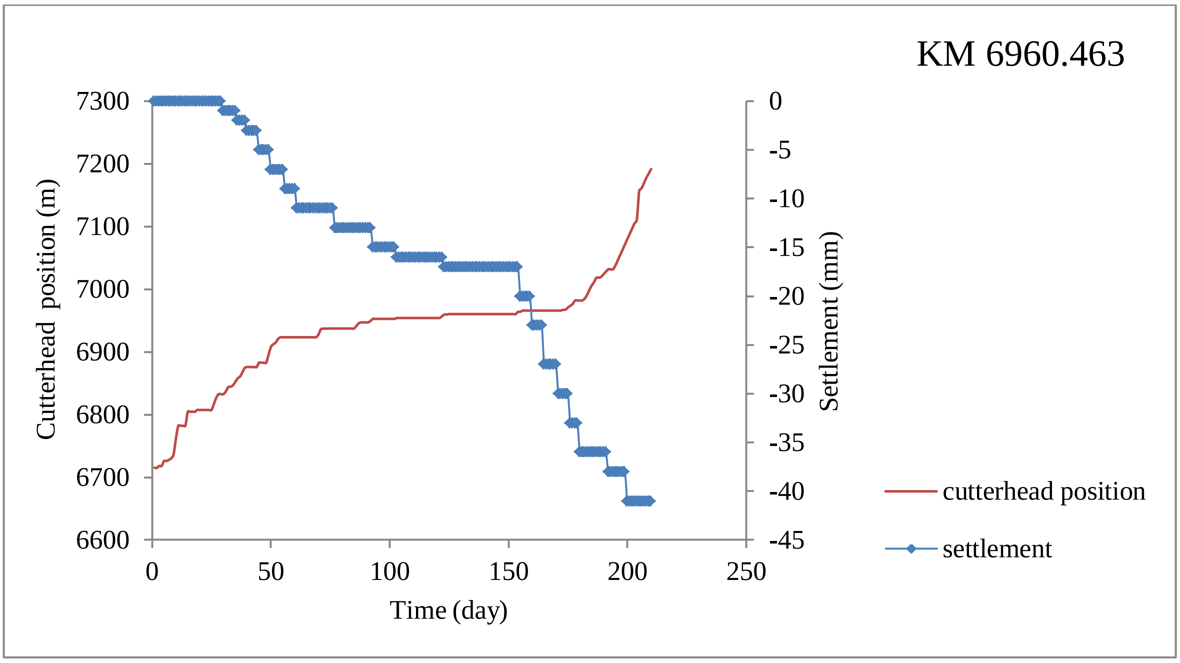


Figure 9. The variation of surface settlement by cutter head advances at monitoring point

5.3.2. Empirical Methods

The empirical formulation most utilized in engineering practice is that developed by Peck (1996) defined by Equation (5) which estimates the vertical surface settlement profile along tunnel axis cross section as having a normal probability distribution function.

$$S = S_{max} exp\left(\frac{-y^2}{2i^2}\right) = \frac{V_S}{i\sqrt{2\pi}} exp\left(\frac{-y^2}{2i^2}\right) \tag{5}$$

where S is surface vertical settlement (m), y distance of the considered point from the tunnel axis (m), V_S volume of the settlement trough per meter of tunnel advance ($\frac{m^3}{m}$), defined as a percentage (V_L) of the unit volume V of the tunnel and (i) is the trough width parameter, expressed as: ($i = k.z$), where k is a dimensionless constant depending on soil type and z is the depth of tunnel axis below surface. There are several suggested methods for prediction of the point of inflection (i). Estimation of (i) in this study is based on average of some empirical approaches given in Equation (6).

$$i = \frac{\left(\mathrm{R}.\left(\frac{Z_0}{2R}\right)^n\right) + \left(0.9R\left(\frac{Z_0}{2R}\right)^{0.88}\right) + (0.28z_0 - 0.1) + (0.25(1.5Z_0 + 0.5R))}{4} \tag{6}$$

Where n is a dimensionless constant parameter, which is in the range of (0.8-1). Measured values of settlement corresponding to total vertical displacement were interpolated with Peck curves for different volume loss [19, 20]. The influence of volume loss values on the surface settlement profiles is shown in Figure 10. The maximum surface settlement measured and computed with empirical and numerical methods along a tunnel cross-section for different amount of volume loss. The observation in Figure 10 allows us to note that for the volume loss of more than 1%, the results obtained from empirical and numerical methods are in good agreement. As seen from Figure 10. d, surface settlement profile for volume loss of 1.5% is the most compatible with the results of optimized numerical modeling.

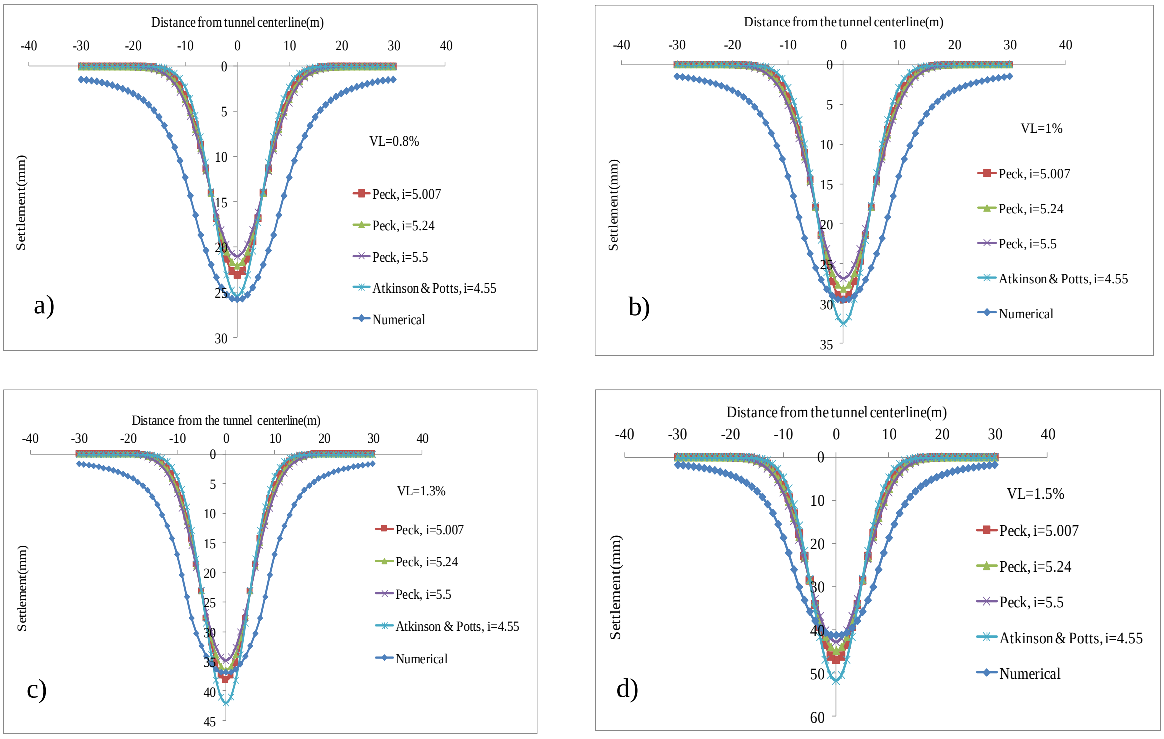


Figure 10. Comparison surface settlement profiles with numerical, analytical and empirical models at volume loss of (a) 0.8%, (b) 1%, (c) 1.3% and (d) 1.5%

5.3.3. Analytical Methods

There are several analytical models for the prediction of maximum surface settlement for shielded tunneling operations. The method suggested by (Loganatan–Poulos, 1998) has been used in this study.

This method is applied to evaluate the transversal settlement profile by considering different volume loss. In this method a theoretical gap parameter (g) is estimated by Equation (7).

$$g = G_P + U^*{}_{3D} + \omega \tag{7}$$

Where (G_P) is the physical gap, representing the geometric clearance between the outer skin of the shield and the liner, ($U^*{}_{3D}$) is the equivalent 3D elasto-plastic deformation at tunnel face, and (w) is a value that takes into the quality of workmanship. Maximum surface settlement is predicted by theoretical Equation (8):

$$w(y) = 4(1-\upsilon)a_0{}^2 \frac{Z_0}{Z_0{}^2 + y^2} \frac{4ga_0}{4a_0{}^2} exp\left[-\frac{1.38y^2}{(Z_0+a_0)^2}\right] \tag{8}$$

Where, (υ) is Poisson's ratio, g is the gap parameter (m), (a_0) and (Z_0) are the tunnel radius and depth, respectively. Figure 11 shows the surface settlement for different amount of gap parameter [21]. As it can be seen the amount of error is minimum for $g = 46.2\ mm$ and $volume\ loss\ = 1.5\%$. Figure 12. depicts the results of empirical method, analytical method and numerical method at volume loss of 1.5%. As it is clear from figure there is a good compatibility between the results of analytical method and optimized numerical modeling at volume loss of 1.5%.

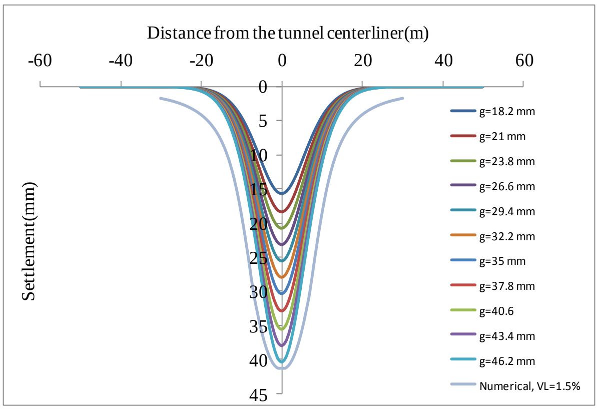


Figure 11. Surface settlement profiles at different volume loss using analytical and numerical modelling

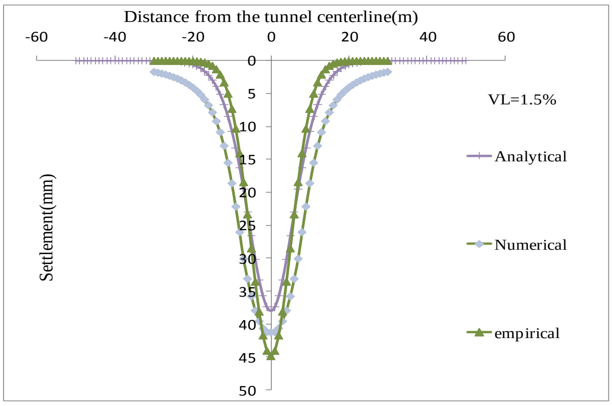


Figure 12. Comparison of surface settlement profiles using analytical, numerical and empirical methods

6. Conclusions

In this study, the building displacement and surface settlement due to EPB tunneling of Tabriz subway tunnels has been analyzed by numerical, analytical and empirical methods and the obtained data are compared with the measured displacement. The new GPS-GNSS displacement monitoring system has been applied to observe the building displacement. According to the results of optimized numerical procedure, the maximum settlement of the building after first tunnel excavation was about 1 mm and the maximum surface settlement was about 41 mm. Maximum amount of building settlement after excavation of the second tunnel was about 4 mm and the maximum surface settlement was about 50 mm. According to the final monitoring results of Tabriz International Hotel displacement via GPS-GNSS approach, the maximum settlement of the building was in the range of (3-13) mm which was compared with the maximum settlement obtained by numerical modeling which was about 4 mm. Although the field data do not cover the complete settlement trough, the maximum settlement were well-predicted by the two dimensional analysis. The results show that applied optimization technique through back analysis approach and GPS-GNSS procedure can lead to accurate results in predicting building displacement, which is of high importance in urban tunneling design.

7. Acknowledgements

The authors gratefully acknowledge the support of this work by Tabriz Urban Railway Organization and Tabriz Municipality.

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