Effect of engineering geological characteristics of Tehran's recent alluvia on ground settlement due to tunneling

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(*received: 25/05/2014 ; accepted: 08/12/2014*)

Abstract

Ground settlement due to the shallow tunneling in urban areas can have considerable implications for aboveground civil infrastructures. Engineering geological characteristics of the tunnel host ground including geotechnical parameters of surrounding soil, groundwater situation, and in situ stress condition are amongst the most important factors affecting settlement. In this research, ground settlement as a consequence of the excavation of the East-West lot of Tehran Metro line 7 (EWL7TM) has been investigated. This tunnel has been drilled into Tehran's recent alluvia composed of fine-grained and coarse-grained soils. Findings indicate that the fine-grained and coarse-grained soils do not have similar behavior during shallow tunnel excavation. In general, maximum settlements (S_{max}) occurred in the cohesion-less soil is greater than cohesive soil. In some sections of tunnel, measured settlements are lower than evaluated items, stemming from the lower volume loss (V_L). Performance of TBM and localized cementation of Tehran alluvia in the considered area have been regarded as significant reasons of occurred discrepancy. In other sections of tunnel, measured settlements are greater than evaluated figures. According to relative thickness constancy of the overburden, this antithesis can be attributed to the existence of old and obsolete underground spaces.

Keywords: *Ground Settlement, Tunneling, Tehran Alluvia, Engineering Geological Characteristics, Tehran Metro Line 7, Monitoring Data*

Introduction

The reaction of soil to the shallow tunnel excavation is widely influenced by its engineering geological characteristics. Therefore, it is important to recognize these characteristics of the soil. Two main clusters of factors control engineering geological properties of soils: first the physical properties and chemical composition of source rocks, and second, the geological, sedimentological, hydrogeological, and weathering processes. In other words, lithological composition and effective geological processes are dominant factors influencing engineering geological characteristics of the soil. Leca and New (2007) pointed out that engineering, geological characteristics of soil, including geological, hydrogeological, and geotechnical properties, in addition to tunnel geometry and depth, excavation methods, and the quality of workmanship and management are basic parameters affecting the ground settlement due to excavation of shallow tunnels.

Ground settlement due to tunneling has been studied by several researchers. Most of the studies are based on the seminal work of Peck (1969) who analyzed appropriate number of cases and illustrated that the transverse profile of surface settlements can be described by a Gaussian curve.

This issue has also been attractive for researchers during last 40 years and many notable review papers have been published. (e.g., Cording and Hansmire, 1975; Mair and Taylor, 1997; Attewell *et al*., 1986; Rankin, 1988; Franzius, 2003; Leca and New, 2007; Guglielmetti *et al*., 2008; Palmer and Mair, 2011; Marshall *et al*., 2012; Hasanpour *et al*., 2012; Fargnoli *et al*., 2013)

Shallow tunnel construction, however, particularly in urban areas, can cause ground movements, which can increase potential damage risk of surface buildings and subsurface structures and pipelines. Hence, an extensive site investigation should be performed to find out the physical and mechanical properties of the ground and underground water context, as well as deformation characteristics of soil.

Tunnel construction method is dependent first on the ground conditions (geological, geotechnical, and hydrogeological characteristics) and second on operational parameters such as time and cost constraints and construction requirements. Tunnels in soft ground (sands and clays) often are excavated using TBM-EPB to prevent the unexpected ground settlement. Moreover, in recent years, application of pressurized face tunneling techniques has had an efficient consequence in poor soil conditions (Golpasand *et al*., 2013).

In this study, the influence of engineering geological factors on ground settlement due to tunneling is investigated. For this purpose, required data has been obtained from the excavation of the East-West lot of Tehran Metro line 7 (EWL7TM).

The EWL7TM, approximate length of 12 km, has been drilled between the stations N7 (Navab-Quazvin bridge) and A7 (Amir-AL-Momennin town) in the South of Tehran. According to the proposed design, the tunnel is circular shaped, with excavation diameter of 9.164 m. In this research, because of the availability of instrumentation data of the ground settlement induced by excavation of EWL7TM from 0+000 to 4+500 kilometrage has been selected for the analysis. Location of the study area has been shown in Figure 1.

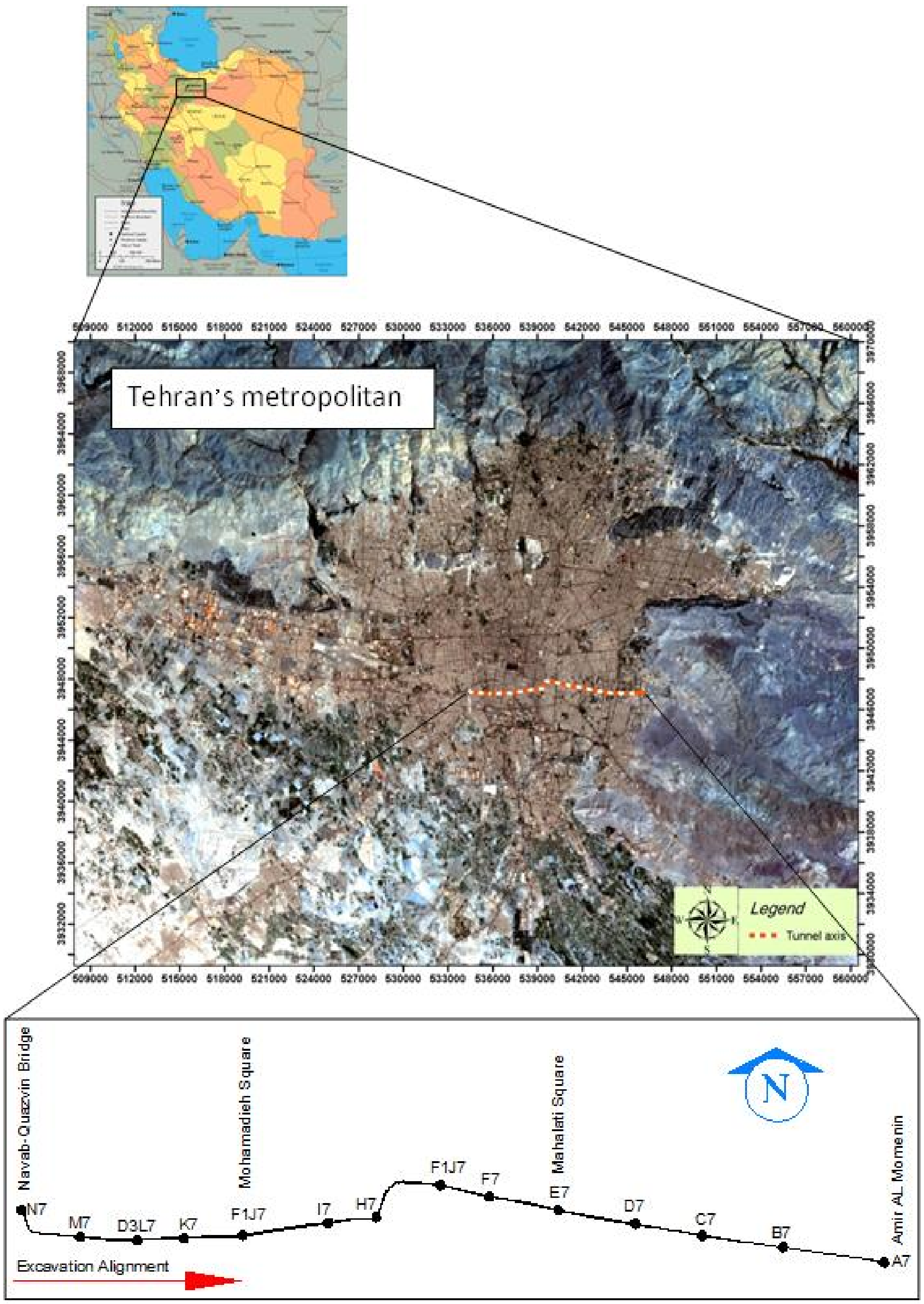


Figure 1. Location of study area

Geological Setting

Tehran metropolitan has developed on quaternary sediments originating from adjacent hills and mountains. Geological findings confirm that the quaternary alluvia and moraine deposits have developed in Tehran plain. In other words, Tehran plain (involving the Tehran city) mainly consists of alluvial materials, which are often the result of erosion and redeposition of former sediments. The plain extends to the South as a young fan, and generally composed of unsorted fluvial and river deposits that are called, in general, Tehran alluvia. Rieben (1955 & 1966) and Pedrami (1981) classified the Tehran plain into four formations identified as A, B (Bn and Bs), C, and D (from oldest to youngest).

The A formation is mainly composed of cemented, hard, and homogeneous conglomerate, with a maximum thickness of 1200 m. This formation is overlain by the B formation, which consists of a heterogeneous conglomerate with a maximum thickness of 60 m. The B category has variable cementation, but almost it is weakly cemented. Pedrami (1981) has divided the B category into two units: Bn in the North and Bs in the South. The C category is composed of variable grain sizes from clay to cobble, and has a thickness of <60 m. The cementation of the C formation is less than that of the A formation. The youngest formation in the Tehran alluvia is the D formation, which consists of materials with varying grain sizes from clay to boulder (Cheshomi, 2006a). A brief and useful comparison between these alluvia is presented in Table 1. From engineering geology point of view, this group of soil typically consists of alluvial deposits with an extensive range of grain sizes from cohesive fine-grained (clayey and silty clay) to cohesionless coarse-grained (sand or gravel) materials. Engineering geological observations of the Tehran alluvia have shown that grain size and shape, sedimentary age, cementation, grain contact type, faults, fractures, and weathering processes affect the geotechnical properties of these materials (Cheshomi *et al*. 2009). Asghari *et al*. (2003) have discussed effects of these factors.

According to the geological situation of Tehran plain, illustrated in Figure 2, the proposed tunnel route will apparently pass through the D formation; but according to the Table 1, the thickness of D formation is generally lower than 10 m. Thus, considering the tunnel depth in area being studied, and low thickness of the alluvia layers, it can be said that the tunnel encounters the older alluvia such as C formation. In other words, the route of EWL7TM is located mainly into C formation in the area being studied. This fact has largely affected the engineering geological properties of soils in tunnel route.

Geotechnical Studies

According to the tunnel layout in urban area of Tehran, the importance of the project and the high risk exposed to the existing buildings and other municipal utilities, the adequate geotechnical exploration practice has been conducted in this project. These studies include site investigations (borehole and test-pits drilling), insitu tests, and laboratory geotechnical practices (physical and mechanical tests). Exploratory drilling to identify subsurface soil layers continued to the depths lower than the tunnel floor.

Table 1. Comparison of Tehran alluvia based on the Rieben (1966) classification (after Cheshomi 2009)

Characteristic	Alluvium			
	A	B	C	D
Age	5 Ma	700 ka	50 ka	10 ka
Lithology	Homogeneous conglomerate	Heterogeneous conglomerate	Alluvial fan	Recent alluvial
Cementation	Strongly cemented	Variable, but usually weakly cemented	Moderately cemented	Non-cemented
Grain size	Clay to 100–250 mm	Very variable up to several meters	Clay to 100–200 mm	Clay up to several meters
Dip of layer (degrees)	0–90	0–15	0	0
Thickness	Maximum 1200 m	Maximum 60 m (thickness decreases toward south)	Maximum 60 m	<10 m
Sedimentary environment	Fluvial	Fluvioglacial and periglacial	Fluvial	Fluvial
Other name (local name)	Hezardareh alluvial formation	North Tehran heterogeneous alluvial formation	Tehran alluvial formation	Recent alluvial

Extensive in situ tests were performed during the field investigations to evaluate stratigraphy, strength, modulus properties, and hydraulic characteristics of host soil. Laboratory index and engineering tests were alsocarried out on collected field samples.

As a result, properties of soil layer and groundwater level were determined, and the engineering geological profile of the tunnel route was delineated. Based on these studies, soils along tunnel route have been categorized into four engineering geological types (soil types) that can be seen in Fig. 3. A layer of fill materials is detected in ground surface with various thickness and weak geotechnical properties.

Soil Classification

Surrounding soils were classified according to the grain size distributions (content of fine and coarse particles). The results of soil classification and the engineering geological characteristics of the categorized types are presented in subscript on Figure 3. Definitely, grain size distribution is one of the most visible engineering geological aspects of soil and has a profound effect on other geotechnical parameters. The cohesion is one of the geotechnical parameters of soil that is highly influenced by the grain size distributions. Guglielmetti *et al*., (2008) pointed to the effect of the ground type on its displacement in mechanized tunneling. Based on their studies, the cohesion of the soil has the main role in the classification criteria.

In general, two main groups of soils, cohesive and cohesionless, were categorized in their studies. Given the serious impact of the soil cohesion on tunneling settlement, this parameter has been considered in soil classification. In sum, the particle size distributions of soil (content of fine and coarse particles) as well as its cohesion are leading factors in soil classification. Other physical and mechanical properties of engineering geological types are shown in Table 2.

Concerning Table 2, it can be perceived that each soil type has distinctive geotechnical parameters compared to other types. According to the engineering geological profile of the tunnel route (Fig. 3), different soil types are located in various depths; so because of change in stress condition and thickness of overburden, it is expected that they

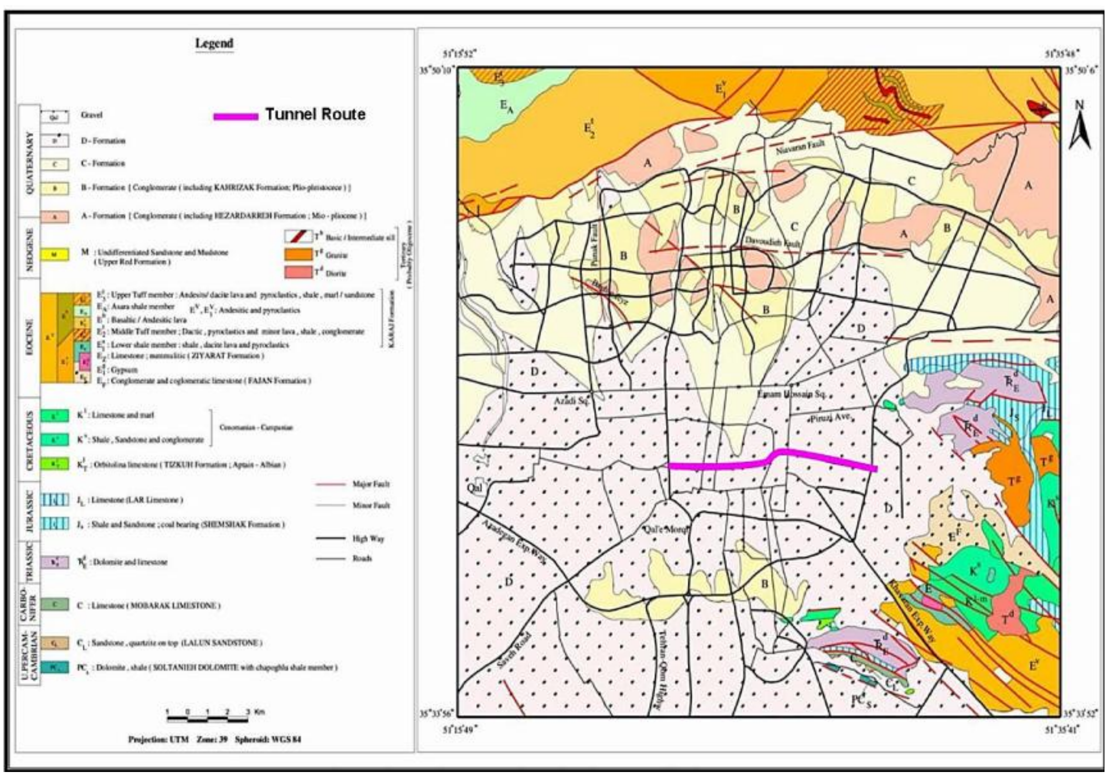
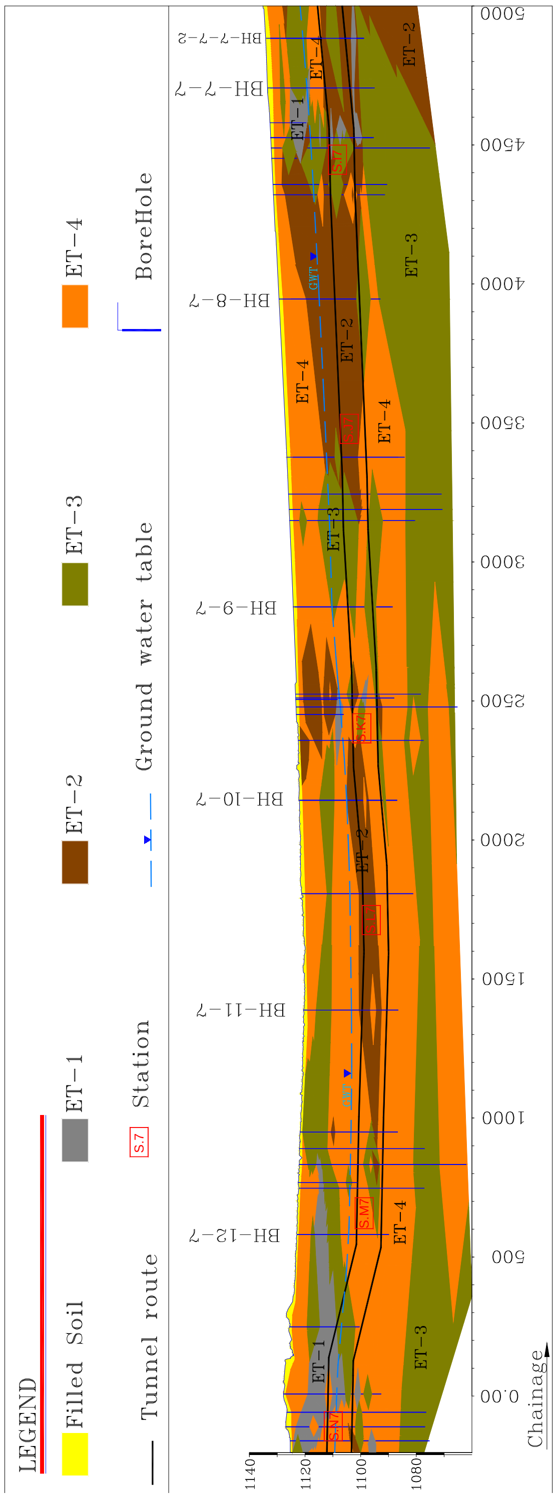


Figure 2. Geological setting of the tunnel route associated with Tehran alluvia (Jafari *et al*., 2007)

As cited previously, categorization of soil types was carried out mostly based on the particle size distribution and cohesion of the soil. Due to changes in the depth of soil types, other geotechnical parameters of soil types can be varied.



Engineering geological types	Description	Passing 200	Soil type
Fill	Very soft sandy CLAY or SILT (filled soil)	Various	-
ET-1	Well graded and poorly graded sandy GRAVEL and gravely SAND	Less than 12%	GW, GW-GM, GP-GC, SW, SP
ET-2	Gravely SAND with silt/clay	12 to 30%	SC, SC-SM, GC
ET-3	Verysilty/clayey SAND with gravel, very sandy CLAY/ SILT with gravel	30 to 60%	SC, SM, CL
ET-4	clayey SILT with sand, silty CLAY with sand, very sandy CLAY/SILT	More than 60%	CL, ML, CL-ML, rarely CH

Figure 3. Engineering geological profile of tunnel route (SCE, 2010)

Table 2. Geotechnical parameters of the engineering geological units (SCE, 2010)

Unit	C (kPa)	φ (degree)	C′ (kPa)	φ′ (degree)	E (MPa)	ν	γ_d (kN/m^3)
Fill	5~10	15~22	5~8	15~25	10~20	0.35	16~18
ET-1	11~24	29~30	9~20	33~35	50~80	0.32	17.5~19.5
ET-2	13~22	28~30	11~19	32~34	25~65	0.33	17.5~19.5
ET-3	32~48	22~27	24~36	28~38	20~50	0.34	18~20
ET-4	38~48	17~20	27~34	26~31	10~40	0.35	18~19

As shown in the engineering geological profile in Figure 3, EWL7TM has been driven mainly into ET-2 and ET-3 soil types. Geotechnical explorations indicate that these soil types contain diverse amounts of fine-grained and coarse-grained particles with various geotechnical parameters, which ultimately control the mechanical behavior of ground during shallow tunnel excavation. Admittedly, geological and geotechnical parameters of soil layers, in tunnel face and its overburden, are ruling determinants of ground settlement due to tunneling. This issue will be discussed in more detail in upcoming sections.

Ground Settlement Induced by Tunnelin

The topic of ground settlement induced by tunneling was defined in previous sections and the main operative factors were mentioned. Evaluation of ground settlement is done based on a classical and conventional semi-theoretical method proposed by Peck (1969) who analyzed several case studies and illustrated that the transverse profile of these surface settlements can be described by a Gaussian curve. Other researchers have studied on this subject and introduced physical and geometrical parameters of settlement trough that are presented in Figure 4. This theory is based on the following equation:

$$S = S_{v,max} \exp\left(-x^2/2i_x^2\right) \tag{1}$$

where, S is ground surface settlement at distance x from centerline of tunnel (mm); S_{max} is the maximum settlement at tunnel line (mm); x is the cross-sectional distance from the centerline of the tunnel (m) and i is the transverse distance from the centerline of the tunnel to the point of inflexion (m).

There are several suggested methods for prediction of the point of inflexion (i). O'Reilly and New (1982) also proposed a linear relationship between i and z_0, and suggested the simple relationship between these two parameters:

$$i = kz_0 \tag{2}$$

where k is known as the trough width parameter and is believed to be largely independent of the construction method (Mair & Taylor, 1997). They collected wide range of field data and concluded that $0.4 < k < 0.6$ for clays and $0.25 < k < 0.45$ for sands and gravels. Other values were proposed by Chapman, *et al*., 2010; Leca and New, 2007, and Guglielmetti *et al*., 2008.

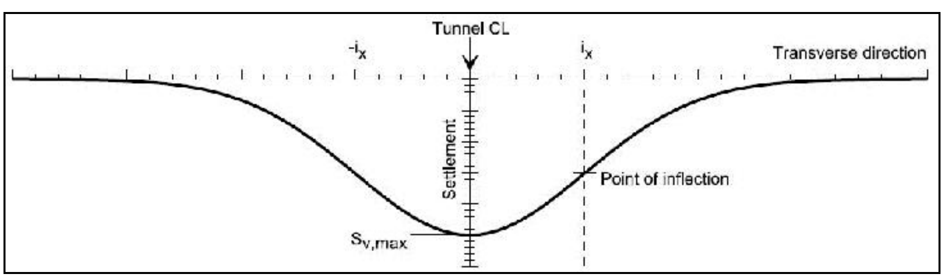


Figure 4. Transverse aspect of ground settlement due to tunneling (Franzius, J. N. 2003)

Maximum Ground Settlement (S_{max})

There are several suggested empirical methods for the evaluation of the maximum surface settlement (S_{max}). A simple and practical method to calculate maximum settlement (S_{max}) was proposed by O'Reilly and New (1982):

$$S_{max} = 0.313 V_L (D^2/i) \tag{3}$$

where D is tunnel diameter, V_L is volume loss and has been defined in the equation (2).

V_L is the ratio of the deference between volume of excavated soil and tunnel volume (defined by the tunnel's outer diameter) over the tunnel volume. The V_L mainly depends on geological and geotechnical characteristics of soil and the method of tunnel excavation. Many studies have been conducted into this parameter and several values (or ranges) have been proposed based on soil types. In this section, concerning available information such as geotechnical and geometrical data, the V_L has been estimated and the resultant ground displacement induced by the EWL7TM tunnel excavation is evaluated.

Evaluation of Ground Settlement Resulting From Excavation of EWL7TM Tunnel

EWL7TM tunnel has been excavated by an EPB-TBM in Tehran young alluvia that are composed of cohesive and cohesionless soils. In order to predict ground settlement due to excavation of this tunnel, based on engineering geological conditions governing the tunnel route and considering recommendations of other researchers, the rational values of *k* and V_L have been estimated (Table 3). It is apparent that according to the engineering geological type of soil, different values have been considered for *k* and V_L. Cohesive soils have relatively high *k* and low V_L, and cohesionless soils have relatively low *k* and high V_L. Many researchers (e.g. Guglielmetti *et al*., 2008) have emphasized this issue. With regard to sufficient amount of information available about the geometry of the tunnel and estimation of the semi-empirical parameter, the maximum ground settlement (S_{max}) can be assessed. The results of this evaluation are presented in Table 3 and are illustrated as a column diagram in Fig. 5. It is demonstrated that S_{max} due to tunneling in sandy cohesionless materials are relatively higher than clay bearing cohesive soils. This can be justified according to plasticity and deformability properties of cohesive soils, which have been emphasized by Guglielmetti *et al*., 2008. According to Figure 5, assessed settlements in ET-2 types in 2+500 and 4+000 kilometrage are higher than other points. This has happened due to high values of V_L in these soil types that consist of sandy materials with lower cohesion.

Table 3. Predicted ground settlements

Point Number	kilometrage	Engineering Geological Types	Parameters required to predict settlement				Maximum Settlement Smax (Cm)
			V_L(%)	*k*	Z_0 *(m)*	*i (m)*	
P1	1+000	ET-3, ET-4, rarely ET-2	0.7	0.46	25.2	11.592	-1.81
P2	1+500	ET-4, ET-3 and ET-2	0.8	0.44	25.6	11.264	-1.63
P3	2+000	ET-4, ET-3 and ET-2	0.8	0.44	25.7	11.308	-1.63
P4	2+500	Mostly ET-2	0.9	0.35	24.7	8.645	-2.73
P5	3+000	ET-3 and ET-4	0.6	0.47	23.7	11.139	-1.41
P6	3+500	ET-2 and ET-4	0.75	0.42	24.1	10.122	-1.95
P7	4+000	Mostly ET-2	0.9	0.35	24.5	8.575	-2.76

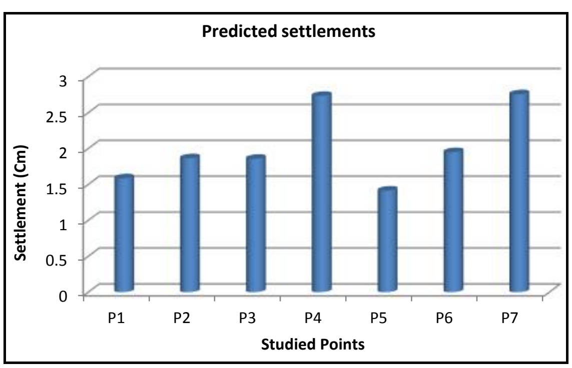


Figure 5. predicted ground settlements in selected points

Actual Settlement (Measured Settlement)

Tunneling in urban areas causes to ground settlement and consequently displacement and deformation of buildings and pipelines. Considering sustainable significance of existing structures along the tunnel route, advanced instrumentation and precise monitoring procedures are necessary. The objectives of instrumentation during tunneling would change depending on configuration of construction, geotechnical conditions, and project schedule (Ghorbani *et al*., 2012). In this research, leveling methods were carried out in order to survey the surface settlements induced by the EWL7TM excavation. Some control points have been considered along tunnel route on the ground surface and measuring equipment has been installed. According to the aim of this study, seven points were selected for comparison with the results of assessments. Locations of measuring points relative to the tunnel route are indicated in Figure 6.

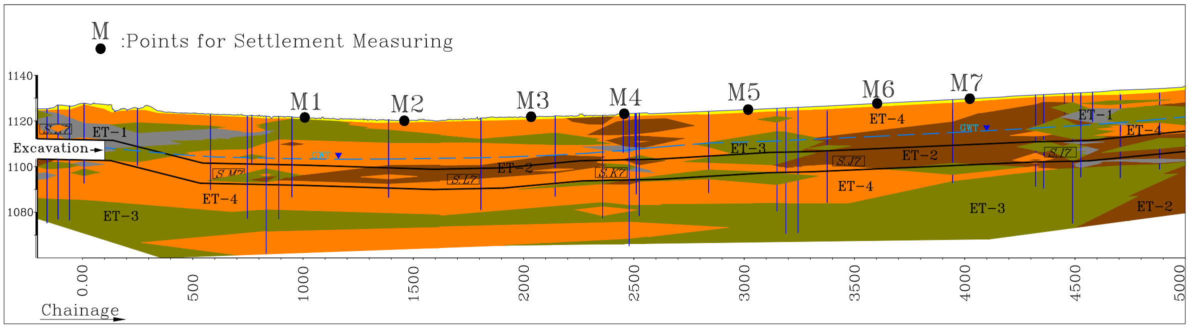


Figure 6. location of instrumentation and levelling points

Installation of equipment into the benchmarks and leveling of points were accomplished based on principals recommended by Dunnicliff (1993).

The ground surface is often covered by asphalt or pavement in urban areas; therefore, the measuring equipment must be bolted in depths lower than the level of the asphalt or pavement and the upper part of the equipment's rod (approximately 20 cm) must be free from the ground. Considering limitations of measuring equipment installation in urban areas, it could be difficult to install leveling points exactly in object kilometrage; therefore, the tolerance of ±50 m would be acceptable with the position of the leveling points. Schematic feature of measuring equipment is shown in Figure 7.

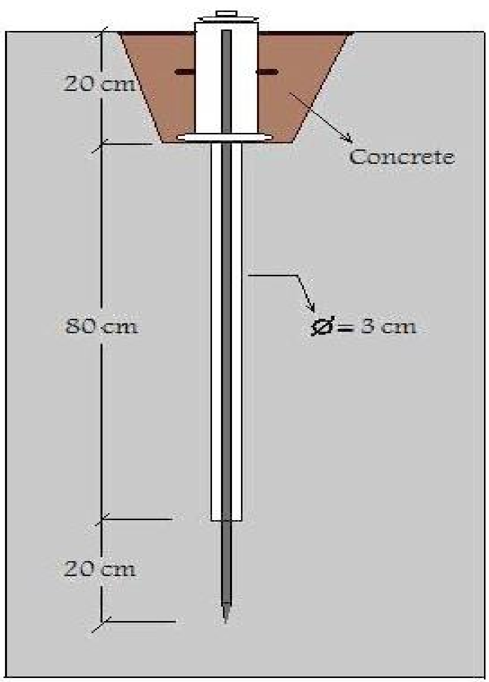


Figure 7: Schematic feature of a measuring equipment

The steps of the installation of a measuring tool and ground settlement leveling are shown in Figure 8.

Ground settlement was measured using leveling techniques. Required precision to measure ground displacement is 0.5 mm. Leveling tools and measuring methods should be in such a way that minimizes human error. Many field data was achieved using this method. Regarding main purpose of this study, the S_{max} has been selected for more discussion.

Measurement of displacements of the benchmarks was started before the passing of shield from the point and continued until the level of points reached a constant value, so the S_{max} was recorded. The process of the settlement recording lasted nearly from 15 to 30 days in a normal situation. Figure 9 shows a sample of settlement measuring diagram, which has been drawn during TBM passing of 2+456 kilometrage through tunnel.

From this figure, in a 21-day period, vertical displacement in this point was 43.9 mm. Leveling points were approximately coinciding with the predicted points. Values of the maximum settlement of leveling points are presented in Table 4. Figure 10 shows the bar chart of the measured settlements. Concise observation of the settlement data shows that it has a proportionate upward trend to tunnel advance, from 1+000 to 4+000 kilometrage. According to the relative constancy of overburden thickness, and drilling and operational parameters, it seems that the variation of ground settlement, in the study area, correlate virtually with engineering geological factors. This issue will be more discussed in the next section.

Discussion and Analysis of Findings

Leca and New, 2007 emphasized the influence of the engineering geological properties of tunnel route on the ground settlement due to tunneling.

Concepts of evaluated and measured settlements were discussed in previous chapters. As cited previously, settlement prediction was conducted based on the geometrical and geotechnical properties of EWL7TM tunnel. On the recommendations of Mair and Taylor, 1997, Guglielmetti *et al*., 2008, and Chapman, *et al*., 2010, values of the k and V_L were determined based on the geotechnical properties of ET-2, ET-3, and ET-4 soil types and finally, ground settlement caused by excavation of EWL7TM tunnel was predicted. In the next stage, measured settlements, derived from instrumentation and leveling methods were utilized to validate the predicted settlements. Statistics of the two groups of data are shown in Figure 11.

Table 4. Measured ground settlements

Point Number	kilometrage	Engineering Geological Types	Measured S_{max} (Cm)
M1	1+000	ET-3, ET-4, rarely ET-2	-0.86
M2	1+500	ET-4, ET-3 and ET-2	-1.24
M3	2+000	ET-4, ET-3 and ET-2	-1.11
M4	2+500	Mostly ET-2	-2.23
M5	3+000	ET-3 and ET-4	-3.91
M6	3+500	ET-2 and ET-4	-4.39
M7	4+000	Mostly ET-2	-5.11

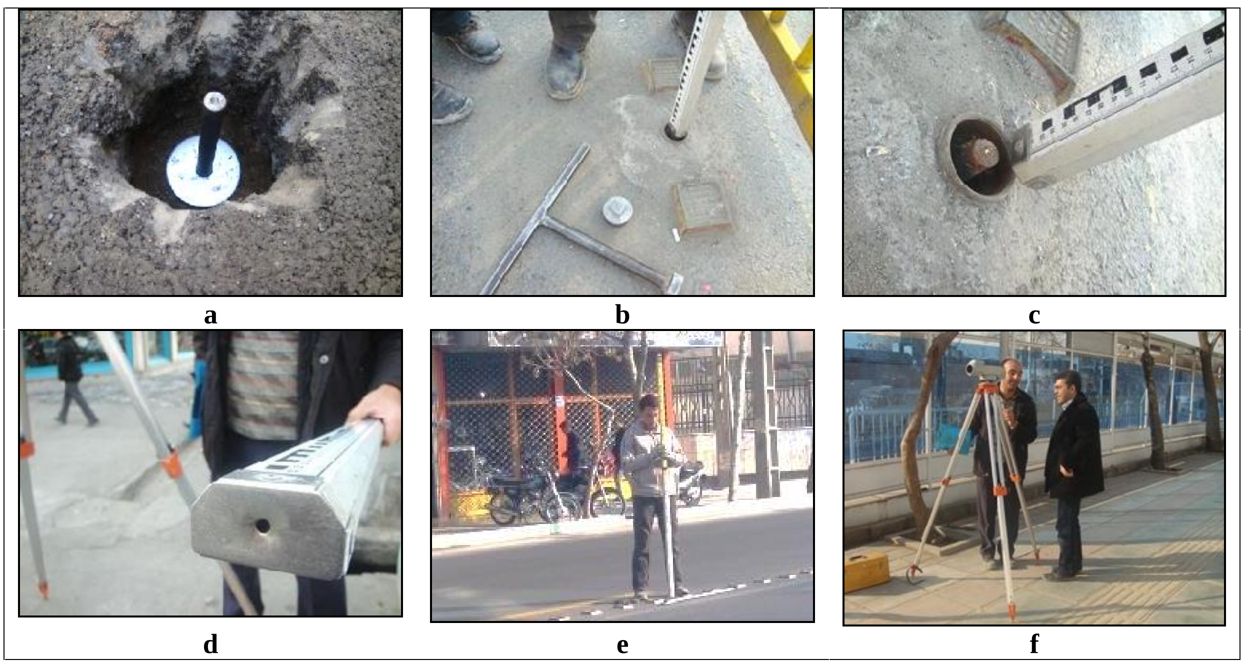


Figure 8. Steps of the installation of measuring equipments and ground settlement leveling

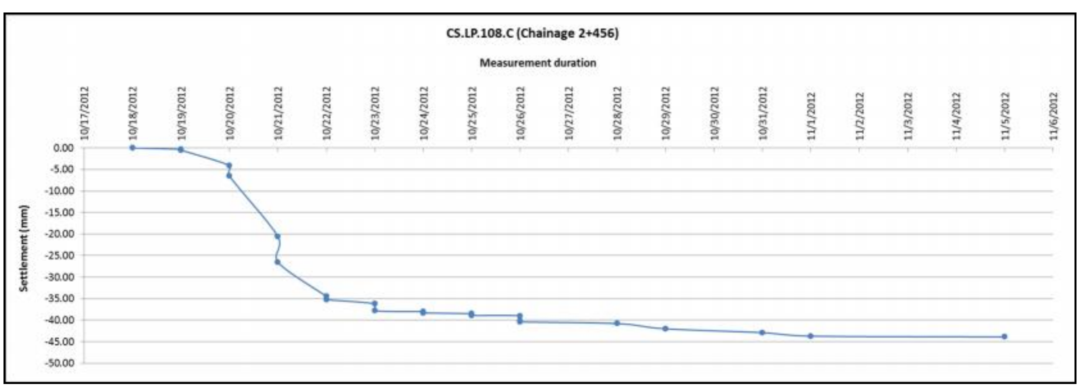


Figure 9. A sample of settlement measuring diagram

According to Figure 11, predicted and measured ground settlements have distinctive discrepancies in all of study points from M1 to M7. The tunnel overburden is almost constant through chosen study area; therefore, it seems that the variation in settlement values has been resulted due to change in geological and geotechnical parameters. In other words, variation in the engineering geological characteristics of the soil types is the main factor of variations in ground settlement due to excavation of EWL7TM tunnel.

Points M1, M2, and M3 are located in ET-4, ET-3, and ET-2 soil types that are mostly composed of fine-grained and rarely coarse-grained soils; therefore, based on the selected values of k and V_L parameters for these soil types, S_{max} values are relatively low. M4 is mostly located in ET-2 soil type composed of coarse-grained soil. As it can be seen in Figures 10 and 11, ground settlements of M4 are clearly higher than M1, M2, and M3. According to the proposed values of the k and V_L, parameters for cohesionless soils, higher values of S_{max} in M4 seem to be acceptable.

In M5 to M7 with the increase of sandy soils content (ET-2 soil type) in tunnel route from 3+000 to 4+000 kilometrage, the values of S_{max} have been risen. Based on clarified reasons, these changes appear to be rational.

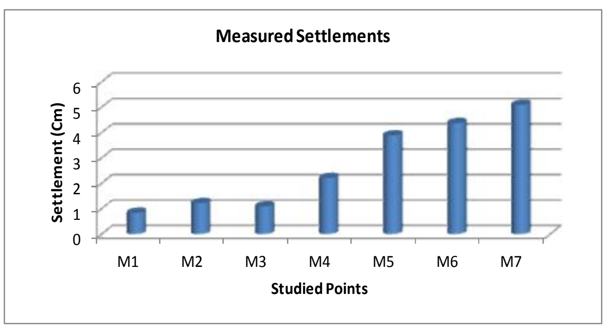


Figure 10. Measured ground settlements in selected points

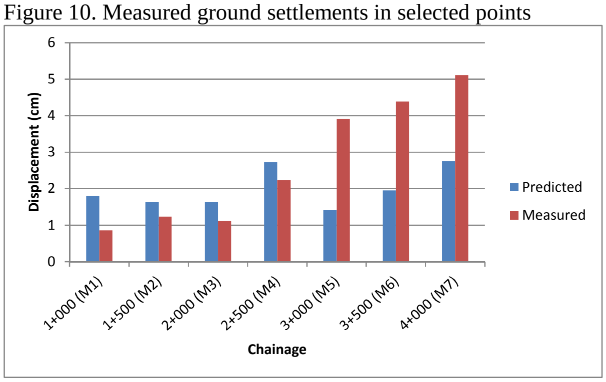


Figure 11. Comparison of predicted and measured settlements

Analysis of Findings

Analysis of the ground settlement through the points M1 to M7 has proved that cohesionless sandy soils caused higher S_{max} in ground surface along the tunnel route. Precise examinations of measured settlements (Fig. 11), in addition to comparison with predicted settlements for each of seven points, have revealed new results. It is apparent that predicted settlements and measured settlements do not perfectly coincide. Considering comparison of two groups of results for points M1 to M7 in Figure 11, the differences between predicted and measured settlements are illustrated in Figure 12.

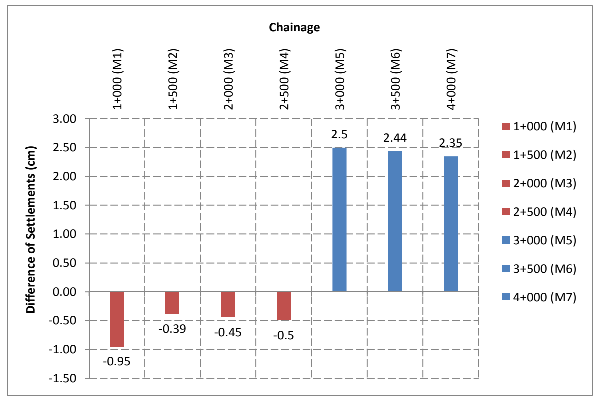


Figure 12. Differences between predicted and measured settlements

Combined review of Figures 11 and 12 indicates that measured settlements have increased significantly through points M5 to M7. In these points, unlike points M1 to M4, measured settlements are higher than predicted cases.

As mentioned previously, tunnel thickness overburden is approximately constant over the study area. According to the engineering geological profile of the tunnel route, there are no phenomenal conditions in M5 to M7 sites. Thus, the abnormal increase in ground settlement in this area should be induced because of other factors.

Preceding studies indicate that old and obsolete underground spaces such as qanats could have a contribution to this issue; therefore, it is suitable to express briefly on this subject.

Ancient and obsolete qanat chains are the most important aspects of engineering geology in Tehran. The qanat chains have been constructed and used to exploit the groundwater in the past few decades. It is evident that existence of these underground spaces can influence the geotechnical properties of soil, such as soil compaction, and the engineering behaviors of it. Many studies have been done to determine the length and location of the qanat chains. Some of them have been led to design maps showing the approximate length and location of qanat chains (e.g. the maps produced by TDMMO 2009). Rayhani and El Naggar (2007) considered probable qanats in Tehran area, studied about collapses induced by qanat using numerical methods, and concluded that weak soil and the erosion of qanat tunnels due to water flow are the most important factors that could increase the displacement of the tunnel walls and cause its collapse. Their studies finally resulted to introduce Zonation maps of Tehran city for collapse hazard. Cheshomi, 2006b studied on engineering geology of Tehran alluviums at the 3rd and 7th metro lines and pointed out that qanats existing in parts of tunnel route (Fig. 13) can act as a hazardous factor for tunnel collapse.

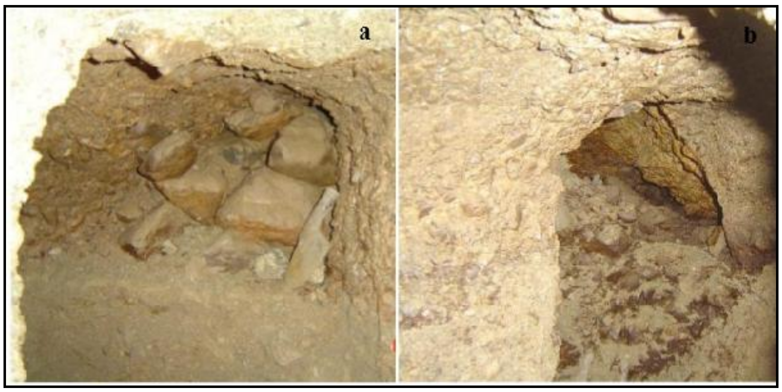


Figure 13. samples of qanat holes in studying area (Cheshomi, 2006b)

In this study, the map of qanat chains along the route of EWL7TM tunnel, developed through the historical documents and geophysical studies of the tunnel have been used. Parts of this map have been shown in Figure 14. It is seen that the qanat chains are gathered dominantly after the chain age 3+000 (especially between stations I7 and J7), coincide with part of the tunnel that measured settlements are higher than predicted ones. However, it can be said that preexisting qanats at the tunnel route are caused to increase ground settlement due to excavation of EWL7TM tunnel. It is worth noting that the qanat chains were introduced in Fig. 14, probably have minor branches; therefore, it is difficult to exactly determine the junction of qanats and tunnel. Finally, it should be noted that although many details of the Tehran's qanats are available, there is a necessity for further studies.

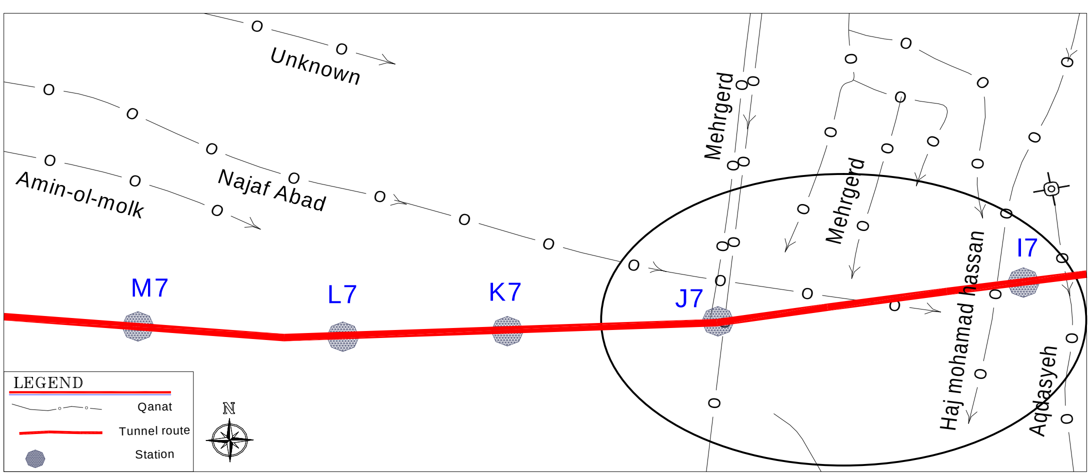


Figure 14. approximate location of qanat chains relative to the tunnel (SCE, 2009)

Through the points M1 to M4, measured ground settlements are generally lower than the predicted ones. Given the engineering geological profile of the tunnel route, it is obvious that both of cohesive and cohesionless soils exist in this part of the tunnel, and rational and expected results, relevant with ground settlement were achieved. Lower measured settlements were probably obtained due to low value of ground loss (V_L), which took place during tunnel excavation. In other words, real value of V_L must be lower than the values previously estimated in the prediction stage. It seems that two factors have main effects:

- Effective and operative performance of the mechanized tunneling processes such as grouting and face pressure. These pressures are applied to the internal surface of the tunnel to prevent further displacement toward the tunnel face and walls.
- Localized cementation of soils created because of of geological factors, probably causes to increase in strength parameters of the soil in tunnel route. In this case, with reducing the soil displacement into the tunnel, lower V_L and consequently lower ground settlement is observed.

Cementation of Tehran alluvia is one of the most important subjects in relation with the engineering geological features. As pointed in geological setting, tunnel route is located into the C alluvia. Cementation of C alluvia is the main difference with the D alluvia. In other words, according to Table 1, in the studied area the tunnel route is located into C alluvia that are naturally cemented materials.

In addition, the results of geotechnical investigations in the studied area can be employed to prove that cementation of soil causes to decrease ground settlement due to tunneling.

Two groups of geotechnical tests have been used for this reason:

1. Triaxial and shear test involving in situ and laboratory tests: The results of these tests are presented in Table 5.

Table 5. Triaxial and shear tests results (involving in situ and laboratory)

No.	Triaxial test		Laboratory shear test		in-situ shear test		Level	chainage
	C′ (kPa)	φ′ (degree)	C′ (kPa)	φ′ (degree)	C′ (kPa)	φ′ (degree)		
1	15	19					1110	2511
2					36	26.6	1108	2505
3					36	36.1	1113	2505
4	8	30					1100	2478
5			8	31			1094	2358
6			7	32			1108	2358
7			9	17			1098	2143
8	28	27					1103	2143
9	14	26					1102	1388
10	12	29.4					1100	889
11			9	33			1097	889
12			17	28			1092	889
13			8	34			1109	832
14	11	30.6					1099	832
15					38	29.3	1107	768
16			4	35			1112	768
17			13	22.9			1100	747
18			13	22.9			1099	747
19			16	27.9			1092	747
20	6	38					1098	949
21	24	36					1100	949
22			6	30			1099	581
23			4	35			1095	832

The results of in situ and laboratory direct shear tests that have been performed between chain age 0+000 to 3+000 are presented and compared with each other. It is seen that the cohesion of soil obtained from in situ tests are obviously higher than laboratory tests. The cohesions obtained from in situ shear tests are approximately 0.37 kg/cm^2, while cohesions from laboratory tests are in the range of 0.06 kg/cm^2 to 0.28 kg/cm^2 in triaxial tests and from 0.04 kg/cm^2 to 0.17 kg/cm^2 in laboratory direct shear tests, so the cohesions obtained from in situ tests are higher than laboratory tests. In order to interpret this issue, cementation of C alluvia and the disturbance of samples of laboratory tests should be considered. As a matter of principle, in situ tests are performed on the undisturbed soil with natural cement, then high cohesion of soil is derived from the test. In contrast, during laboratory tests disturbance of sample, due to several factors such as sampling tools, transportation, laboratory practices, and other factors, cause to decay natural cement and lead to decreased soil cohesion. So it can be said that natural cementation of soil (belonging to C alluvia) cause to increase soil cohesion.

2. SPT test: SPT is one of geotechnical tests that have been performed in studying area. The results of this test are illustrated in Figure 15. It is clear that SPT numbers are considerably increased, mostly at depths greater than 10 m. This can interpret according to low thickness of D alluvia and cementation of C alluvia. In other words, cemented materials of C alluvia are located below the D alluvia. As SPT reaches to the C alluvia, the soil becomes denser as a result of natural cementation and consequently the numbers of SPT increase with depth.

Finally, based on the aforementioned items, and comparison of the values of predicted and measured settlements in each of points, M1 to M7, the engineering behavior of the ground against the excavation of EWL7TM tunnel can be summarized in Table 6.

Table 6. Engineering geological behavior of tunnel host ground

No	Engineering Geological Types	Comparison of measured and predicted settlements	Engineering geological features
M1	ET-3, ET-4, rarely ET-2	Generally measured settlements are lower than predicted settlements	Low value of ground loss (V_L) has been occurred due to two reasons: 1) Efficient performance of TBM. 2) localized cementation of Tehran alluvia in this area
M2	ET-4, ET-3, and ET-2		
M3	ET-4, ET-3, and ET-2		
M4	Mostly ET-2		
M5	ET-3 and ET-4	Generally measured settlements are greater than predicted settlements	1) Increase in measured and predicted settlements due to the rising percentage of cohesionless soils context. 2) Abnormal increase in measured settlements due to existence of old and obsolete underground spaces such as qanats
M6	ET-2 and ET-4		
M7	Mostly ET-2		

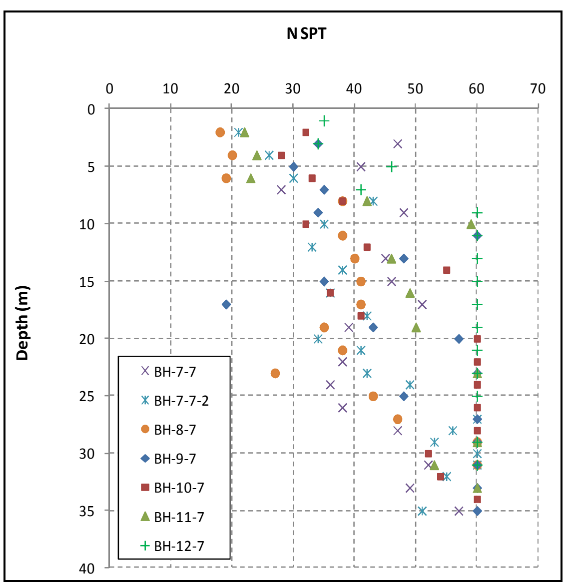


Figure 15. The results of SPT tes

Conclusion

Effects of engineering geological characteristics of tunnel route on ground settlement induced by excavation of EWL7TM tunnel were examined in this research. Based on geological profile of the study area, tunnel route is situated into the soil layers composed of fine-grained and coarse-grained materials. Materials were categorized into four engineering geological soil types. According to geotechnical properties of soil types and with respect to the geometry of tunnel, ground settlement was predicted in specific points using Peck's approach. In the next phase, actual settlements of ground, happened during excavation of EWL7TM tunnel was measured in approximately same positions of predicted points utilizing instrumentation and levelling approaches. Then, measured settlements were compared with predicted settlements. According to the relative constancy of the overburden thickness and considering the ground settlements data, following results were obtained:

- In general, greater displacements occur in cohesionless soils as it is obvious in settlement increase through M1 to M4. In this area, the amount of sandy soils increases with advancing tunnel. The plasticity characteristics of the cohesive soils can be introduced as the prime reason. High values of V_L occurred in cohesionless soils; thus, greater ground settlements happened in these materials.

- In distance of M1 to M4, measured settlements settlements are lower than predicted settlements. It can be explained that lower V_L has occurred during excavation of this area. Two major reasons can be described here: 1. satisfactory performance of several sectors of TBM system such as grouting pressure and face pressure. 2. Localized cementation of Tehran's alluvia in this part. These factors have prevented further displacement of tunnel face toward the tunnel; so lower settlement is is observed in this part of the tunnel.

- Both measured settlements and predicted settlements increase through the points M5 to M7. According to the increase of sandy soils content in this part, high values of predicted settlements appears to be logical. Abnormal increases in measured settlements are probably attributed to the existence of old and obsolete underground spaces such as qanats. Further study into this field is recommended.

Acknowledgment

Authors would like to express their sincere appreciation to SCE Institute for providing access to the instrumentation database of the EWL7TM project.

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