Numerical and Analytical Investigation of Ground Surface Settlement Due to Subway Excavation

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Abstract   In the congested urban areas, tunnelling close to existing structures often occurs due to the lack of space. Consequently, tunnelling-induced ground movements may cause serious damage to the adjacent structures. Ground surface settlement caused by tunnel excavation varies in magnitude and trend depending on several factors such as tunnel geometry, ground conditions, etc. There are several empirical, semi-empirical equations and numerical methods available to estimate ground surface settlement associated with excavation activities, however among these methods most of empirical and semi-empirical based methods do not simultaneously take all the relevant factors in the complex condition into account. In complex conditions, numerical methods can be used for resulting accurate predictions. In this paper, ground movement in a highly populated region of Iran's capital city, Tehran, induced by excavation and construction of 7th line of Tehran subway has been investigated. It was aimed to analyse the ground displacements and surface settlement which has been induced by tunnelling using Earth Pressure Balance-Shield Tunnel Boring Machine. The analysis was performed by Finite difference Method (FDM) using FLAC[3D] FDM package. Results from performed numerical analysis show good agreement with obtained results from analytical and empirical methods. In addition, effect of some important factors such as tunnel geometry and ground properties on ground surface settlement has been investigated. Obtained results indicate that ground surface settlement is more sensitive to tunnel geometry rather than height of tunnel placement.

Keywords   Ground Surface Settlement, Tehran Subway, Finite Difference Method, FLAC[3D]

1. Introduction

Today one of the main issues of tunnelling activities in congested urban areas is interaction between tunnels and pre-existed urban structures. Ground surface settlement due to the subway excavation can simply affect sustainability and durability of pre-existed structures nearby the excavated area and hence it is one of the most important concerns in subway excavation process.

Surface settlement caused by shallow underground excavation creates a concave shape subsidence basin which is typically shown in Figure 1[1]. As shown in this figure the coordinate system is defined where y denotes the distance from the tunnel centre line in the transverse direction, x is the coordinate in the longitudinal direction and z is the depth below ground level (bgl).

Review of case history shows that the factors causing settlements can be categorised into four major categories (Figure 2): (1) tunnel geometry; (2) geological conditions;

(3) shield operation factors; and (4) loading conditions. Loading condition consists of two type of load; 1) dead load and 2) live load. Dead loads are such as weight of overhead structures and overburden materials and live load are dynamic stresses due to crossing cars. Though, the live load can be ignored to simplify analysis in a reasonable way.

Building damage due to settlements arising from tunnelling is predictable with a reasonable accuracy subject to the employed calculation technique. As a consequence the side-effects of settlement can be controlled through either variation of vertical/horizontal tunnel alignment, or partly by site specific measures such as underpinning, compensation grouting etc. Prediction of the magnitude and distribution of these movements is critically important in design process, as these predictions are used to estimate the tolerance of the neighbouring structures to the induced deformations. Several approaches have been presented by different researchers with the purpose of estimating maximum ground surface displacements along the tunnel axis and the surface area extension affected by deformation phenomena. The surface settlements can be estimated by using empirical or semi-empirical methods[2–8], analytical methods[9–15], and numerical methods[16–18]. Most of performed studies on this subject have been essentially

performed based on geotechnical soil features and working peculiarities (e.g., excavation geometry and methodology). Also there are a number of uncertainties associated with predictions based on available methods, such as soil propert ies variations, support system details and construction sequences. In practice, the observational method is commonly used to compare predicted performance with observed responses.

In this paper, in addition to numerical investigation of ground surface settlement due to the excavation of 7th line of Tehran subway and associated station (O_7 station), effect of neighbouring building weight on settlement has been evaluated.
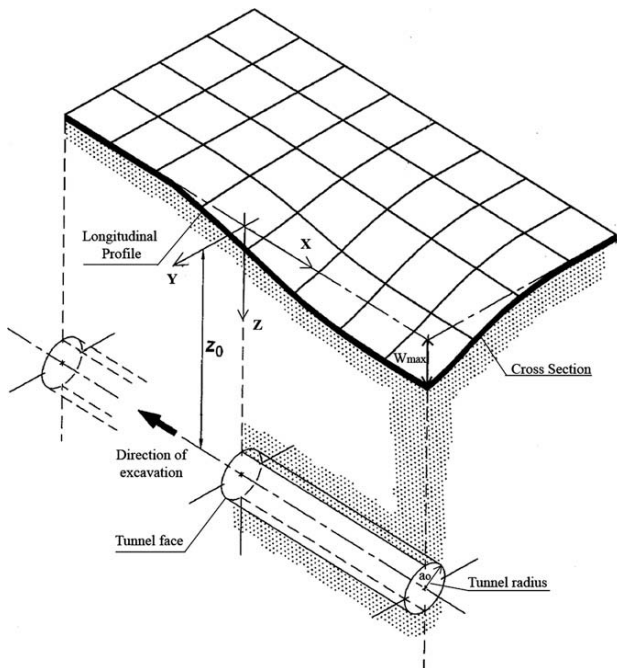

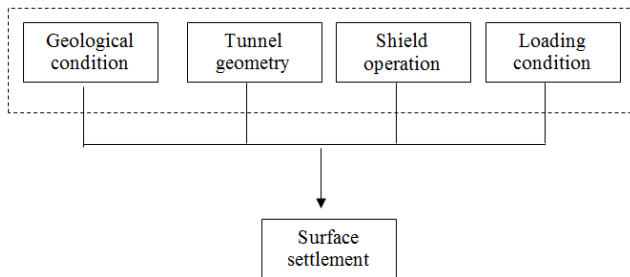**Figure 1.** Subsidence basin induced by a shallow tunnel excavation[1]

Figure 2. Main factors which cause ground surface settlement(Modified from Suwansawat and Einstein, 2006)

2. Technical Background

The 7th subway line is extended from North-West of Tehran towards the South-East of Tehran. Associated tunnels for this line are fully designed and planned to be excavated using Earth Pressure Balance Tunnel Boring Machine (EPB-TBM). The outer diameter of tunnel is 9.16m. The subsurface material in site area is formed from the quaternary aged alluvial fan deposits, which known as the Tehran Alluvial Formation. The thickness of these

deposits has been reported as 60 m in the project area. The particle size of the alluvial deposits in the site area varies from clay to cobbles, with variable distribution, both in depth and horizontal extents. In this formation, there are alternated layers of coarse and fine grained soils, which could influence the design and construction of the proposed project. According to performed geotechnical and hydrogeological field investigations, the groundwater level is under the designed tunnel and the groundwater level variation has no considerable role in design and construction of the proposed subway line and associated station structures. The O_7 station is one of the 7th line important underground stations which are located under a complex urban area. There are several buildings and slums in this region. Therefore, considering the underground works, environmental impact is an important issue in this excavation.

To construct the subway station, a 60 m long narrow trench is vertically excavated to 18m bgl. This trench is used to transfer the workers, portable machinery and excavation devices to underground. From the base of the excavated trench, a 19.7 m length horizontal worker access tunnel is excavated normal to the trench axis and extended toward both sides (see Figure 3).Vertical boreholes are excavated from the end of horizontal access tunnel and lateral concrete bored piles are constructed inside the excavated boreholes, finally reinforced concrete pads connect each two bored pile together. Having both reinforced lateral concrete piles (as the wall) and a concrete pad (as the roof) will prepare a safe area for mass excavation which is required to construct the subway station. The mass excavation is performed layer by layer and total excavated depth consists of four layers of different materials as shown in Figure 3. Properties of these materials are presented in Table 1.
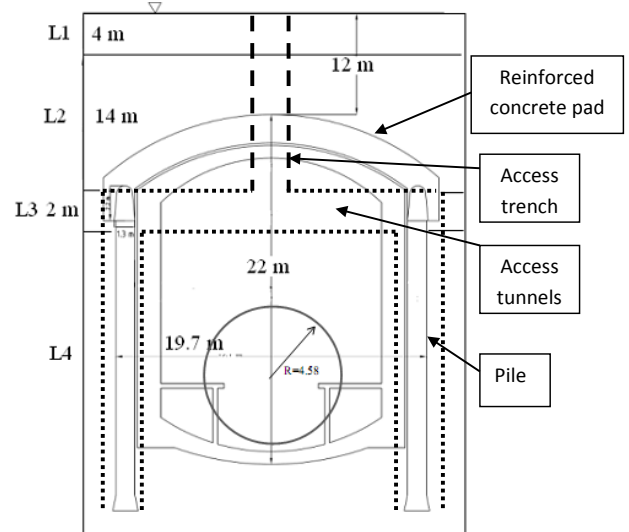

Figure 3. Geometry of O_7 station and tunnel

The tunnel will be supported by concrete segments. Mechanical characteristic of segment and shied is shown in Table 2.

Furthermore, in tunnel construction by EPB machines the front pressure, injection pressure behind segments and pressure of jacks which pushes the shield forward are the main factors. Value of this pressure is shown in Table 3.

Table 1. Mechanical properties of soil

Formation	Dry density	Cohesion	Friction angle	Elasticity modulus	Poisson's ratio
	g/cm^3	kg/cm^2	Degree	MPa	-
L1	1.9	0.2	38	100	0.27
L2	1.9	0.3	35	80	0.27
L3	1.9	0.3	30	50	0.3
L4	1.9	0.4	27	30	0.35

Table 2. Mechanical characteristics of segment

	Density	Elasticity modulus	Poisson's ratio
	kg/m^3	GPa	-
Concrete segment	2500	20	0.2

Table 3. Different amounts of pressures witch effect excavation induced displacements

Parameters	truest force	Face pressure	Grouting pressure
Unit	kN	kN/m^2	kN/m^2
Average pressure	9000	55	150

In this paper, the ground surface settlement due to the construction of the subway tunnel and station, also effect of settlement on neighbouring building has been investigated using both empirical methods and numerical analysis.

3. Empirical and Analytical Methods to Estimate the Settlements

Several empirical, semi-empirical and analytical methods are developed to estimate the ground surface movements due to the underground excavation activities. These empirical and analytical methods are briefly described below.

3.1. The Peck Method

One of the most well-known empirical methods in this field is Peck method. This method is developed by Peck[2] and Schmidt[20] and well-being used in engineering practices. Using this method the vertical ground surface settlement along the tunnel axis can be estimated using a Gaussian distribution function shown as Eq. 1.

$$w = w_{max} e^{\frac{-y^2}{2i^2}} \tag{1}$$

Where w is magnitude of settlement at distance y from the tunnel centreline, w_{max} is maximum value of w over the tunnel centreline and i is distance from tunnel centreline to the point of inflexion on the settlement trough. The value of w_{max} can be calculated using volume of settlement trough

per unit length of tunnel (V_s) where $w_{max}=V_s/i \sqrt{(2\pi)}$[3,5,21–23].

3.2. The Sagaseta Method

Sagaseta developed a closed form solution based method that evaluates the strain field in an initially isotropic and homogeneous incompressible soil[9,10,24]. In this solution the virtual-image technique is used to consider the presence of the top free surface. In this case, using the following equations, the vertical soil movement in the orthogonal plane to the tunnel axis $w(y)$ and in the longitudinal plan $w(x)$ are computable.

$$w(y) = \frac{V_s Z_0}{\pi \times (y^2 + Z_0^2)} \tag{2-1}$$

$$w(x) = \frac{V_s}{2\pi Z_0}(1 + \frac{x}{\sqrt{x^2 + Z_0^2}}) \tag{2-2}$$

Where V_S is the volume loss and Z_0 is the tunnel-axis depth.

This method simultaneously considers the effect from the ground loss, ε_0, the ovalisation (ratio of the maximum value of the radial displacement of the tunnel wall, u, and the tunnel radius), ρ, and the volumetric compressibility, α[25]. The general expression of the vertical surface displacement field is given by:

$$w(y) = 2\varepsilon a_0 (\frac{a_0}{Z_0})^{2\alpha-1} \frac{1}{(1+y'^2)^\alpha}(1+\rho\frac{1-y'^2}{1+y'^2}) \tag{3}$$

Where ε is the radial contraction equal to u_0/a_0, Z_0 the tunnel depth, u_0 the uniform radial displacement, a_0 the tunnel radius, y' the distance from the tunnel centre scaled by the tunnel depth, α the exponent for volumetric compressibility and ρ the relative ovalisation that is equal to δ/ε, where δ is the ovalisation.

3.3. The Verruijt–Booker Method

Verruijt and Booker[11] develop an analytical solution for evaluating the ground movements due to the tunnel excavation in a compressible soil. In fact, their method is a general form of Sagaseta's solution and it computes the deep and surface vertical displacements and the horizontal displacements along a cross-section to the excavation direction. The Eq. (4) presents the closed form solution to estimate the vertical displacements:

$$w(y,z) = -\varepsilon \times a_0^2 (\frac{z_1}{r_1^2} + \frac{z_2}{r_2^2}) + \delta \times a_0^2 \left[\frac{z_1(ky^2-z_1^2)}{r_1^4} + \frac{z_2(ky^2-z_2^2)}{r_2^4}\right] +$$
$$\frac{2\varepsilon \times a_0^2}{m}\left[\frac{(m+1)z_2}{r_2^2} + \frac{mz(y^2-z_2^2)}{r_2^4}\right] - 2\delta \times$$
$$a_0^2 Z_0 \left[\frac{y^2-z_2^2}{r_2^4} + \frac{m\times 2z\times z_2(3y^2-z_2^2)}{(m+1)\times r_2^6}\right] \tag{4}$$

Where ε and δ are parameters indicating the relative displacement of the tunnel surface, for uniform radial displacement case (ε) and the ovalisation case (δ), $z_1 = z - z_0$, $z_2 = z + z_0$, $r_1^2 = y^2 + z_1^2$, $r_2^2 = y^2 + z_2^2$, $m = (1 - 2\nu)^{-1}$, $k =$

$v(1 - v)$. Also, v is the soil Poisson's ratio; a_0 and z_0 are the tunnel radius and depth, respectively.

The vertical field displacement at the ground surface estimates using the Eq. (5)

$$w(y) = 4\varepsilon \times a_0^2 (1-v) \times \frac{Z_0}{y^2 + Z_0^2} - 2\delta \times a_0^2 Z_0 \times \frac{y^2 - Z_0^2}{(y^2 + Z_0^2)^2} \quad (5)$$

By integrating Eq. (5) from $-\infty$ to $+\infty$, the uniform radial ground-loss value can be estimated. Also, total area of the settlement is obtained from Eq. (6):

$$A = 4(1-v) \times \varepsilon \pi R^2 \quad (6)$$

From which;

$$\varepsilon = \frac{V_s}{4(1-v)} \quad (7)$$

Where, V_S is the volume loss defined as the ratio of the settlement volume trough per meter and the excavated volume per unit advance.

3.4. The Loganathan–Poulos Method

This method is developed by improving the Verruijt and Booker method[12]. The improved method can takes the ground loss into account, considering it uniformly distributed along the tunnel wall.

To consider the soil parameters, tunnelling method and support system, the authors introduced the gap parameter, g, which estimates the ground-loss value[26].

In particular, the equivalent un-drained ground loss, ε_0, is defined as:

$$\varepsilon_0 = \frac{\pi(a_0 + \frac{g}{2})^2 - \pi a_0^2}{\pi a_0^2} \times 100\% = \frac{4ga_0 + g^2}{4a_0^2} \times 100\% \quad (8)$$

Where g (the gap parameter) is the equivalent of a two-dimensional (2D) void formed around the tunnel. The parameter, g, can be estimated as the sum of three components[27]:

$$g = G_p + U_{3D}^* + \omega \quad (9)$$

Where, G_p is a physical gap representing the geometric clearance between the shield and the lining. For a circular tunnel, $G_p = 2\lambda + \delta$ where λ is the thickness of the tailpiece and δ is the clearance for erecting the lining[28]. The authors developed Eq. (10) to estimation the vertical surface displacement. The proposed equation considers only a short-term, un-drained condition and the ground deformation resulting from the long-term ovalisation of the tunnel lining is neglected.

$$w(y) = 4(1-v)a_0^2 \frac{Z_0 \times (4ga_0 + g^2)}{(Z_0^2 + y^2) \times 4a_0^2} \exp\left[-\frac{1.38y^2}{(Z_0 + a_0)^2}\right] \quad (10)$$

3.5. The Oteo Method

A semi-empirical method suggested by Oteo and et al. to estimation of the vertical surface displacements along the cross section of the tunnel axis. In this method the displacement is calculated by applying a modified Peck's error curve to consider some additional tunnel and ground parameters:

$$w(y) = \psi \frac{\gamma(2a_0)^2}{E}(0.85-v)e^{(-\frac{y^2}{2i^2})} \quad (11)$$

Where, i is the position of the inflexion point that estimated as $i = a_0\eta(1.05z_0/2a_0 - 0.42)$, ψ is an empirical parameter, E is the soil Young's modulus, a_0 is the tunnel radius, and v is the Poisson's ratio

3.6. The Romo–Diaz Method

Using the finite element analysis Romo and Diaz[30] proposed an empirical equation to estimate the settlements along the tunnel axis as Eq. (12).

$$\lambda_b(\zeta) = (0.0083 - 0.0014\frac{Z_0}{H})(\sigma_h - p)F(\zeta)(Z_0 + D)\frac{\varepsilon_f}{\sigma_f} \quad (12)$$
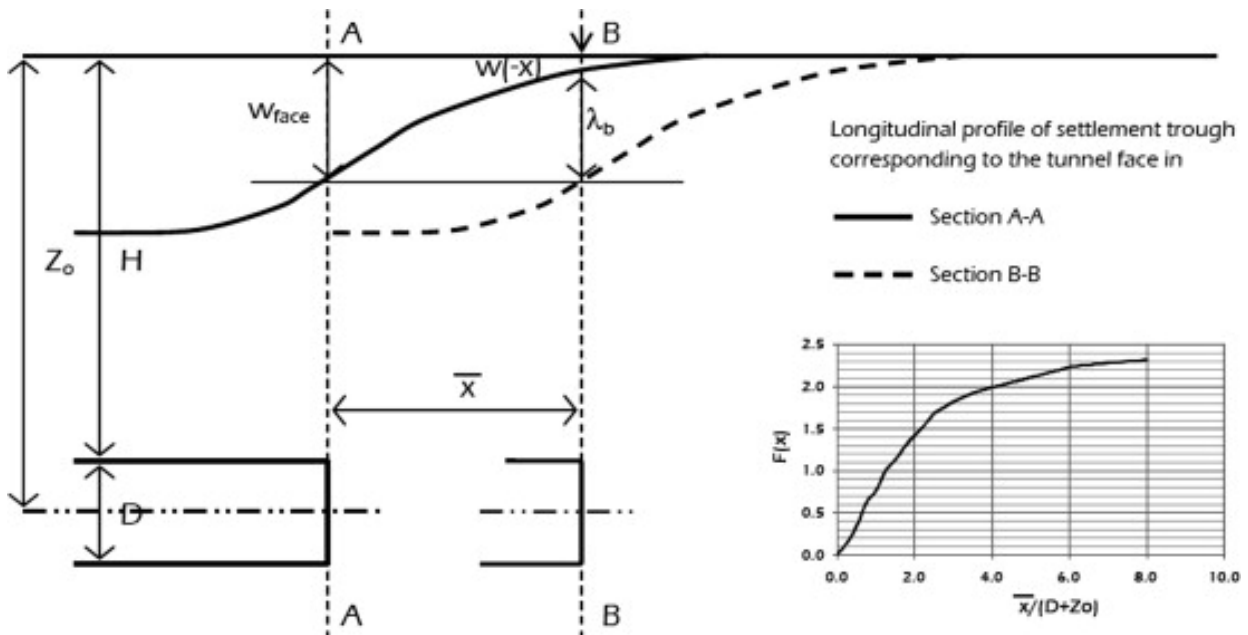

Figure 4. Geometrical condition and values of the function F used in Romo–Diaz method[1]

In this equation λ_b is the settlement at the section B–B (Figure 4) due to the passage of the tunnel face from the section A–A to the section B–B, Z_0 is the tunnel-axis depth, H and D are tunnel overburden and tunnel diameter, respectively, σ_h is the horizontal stress at the tunnel axis, p is the pressure acting at the excavation face, σ_f is the strength average value from the ground surface and the tunnel invert, ε_f is the average value of strain at the failure from the ground surface and the tunnel invert and ζ a function related to the distance between the sections A–A and B–B (Figure 4). The value of w(-x) represents the settlements at a distance x from the tunnel face and it can be estimated as Eq. (13).

$$w(-x) = w_{face} - \lambda_b(x) = \lambda_b(\infty) - \overline{\lambda_b(\overline{x})} \qquad (13)$$

4. Finite Difference Method

In this section, ground surface settlement has been simulated using FDM, with the use of Flac^{3D} package. The FDM is perhaps the oldest numerical technique used to solve sets of differential equations, given initial values and/or boundary conditions. In FDM, every derivative in the set of governing equations is replaced directly by an algebraic expression written in terms of the field variables (e.g., stress or displacement) at discrete points in space; these variables are undefined within elements[31].This program simulates the behaviour of structures built of soil, rock or other materials that may undergo plastic flow when their yield limits are reached. Materials are represented by elements, or zones, which form a grid that is adjusted by the user to fit the shape of the object to be modelled.[32].
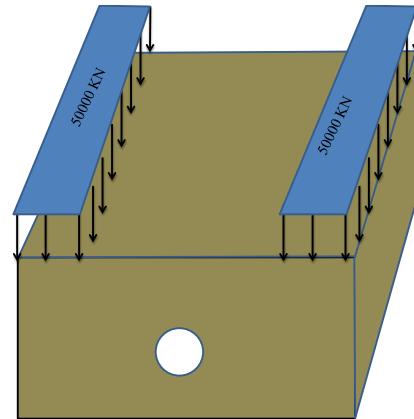

Figure 5. Applied uniform pressure in model

In this paper, the model geometry dimensions of 100 m × 90× 70 m were chosen. This model is constituted of 220950 zones and 227194 grid points, and it is confined by appropriate boundaries along the x, y and z, whereas the upper surface in z direction is free to move, as it corresponds to the effective ground surface and other side in x and y directions are fixed to prevention of any movement. Also, bottom boundary in z direction is fixed too. To model the weight of over-ground neighbouring structures, two 50000 KN uniform pressures have been applied on the two side of geometry surface (see Figure 5). Also, face and grouting pressures are imported to model. Figure 6 shows the designed FDM model.

The mechanical behaviour of the soils is adopted to be elastic-plastic according to the Mohr-Coulomb yield criterion. The used mechanical properties are given in Table1. Also, the material properties that are used for modelling the shield and segments, as the elastic materials, are given in Table 2.
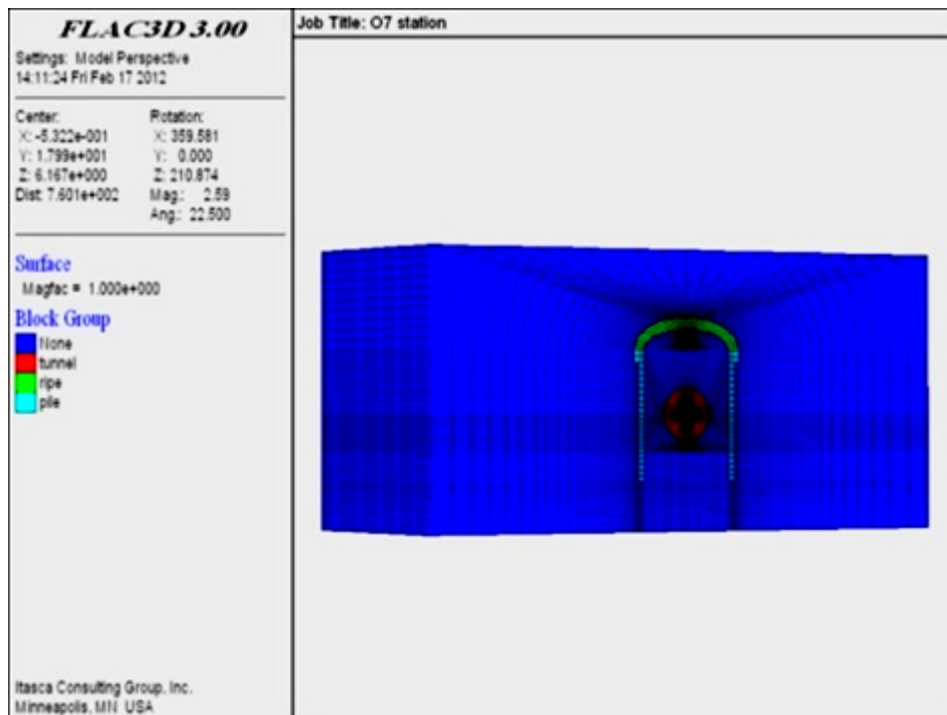

Figure 6. Finite difference model of O$_7$ station and tunnel

5. Results and Discussion

As explained before, the objective of presented research work in this paper is simulation of ground surface settlement due to the construction of underground subway tunnel and station. Hence, first settlement due to the tunnelling is assessed using the numerical, empirical and analytical methods, then simulation of underground station construction is performed and associated ground settlement is calculated.

Estimated maximum surface settlement due to the excavation of tunnel is shown in Table 4. Also the calculated settlement profile is show in Figure 7. As shown in this figure, the maximum settlement occurs just over the tunnel axis and vertical displacement decreases by increasing the horizontal distance from the tunnel axis. Moreover this figure shows that the Peck method predicts the upper value of ground surface settlement compare to other methods. Results of FDM analysis show excellent agreement with results from empirical methods especially Loganathan–Poulos method which shows only 9.5% difference.

Table 4. Maximum surface settlement assessment using different methods

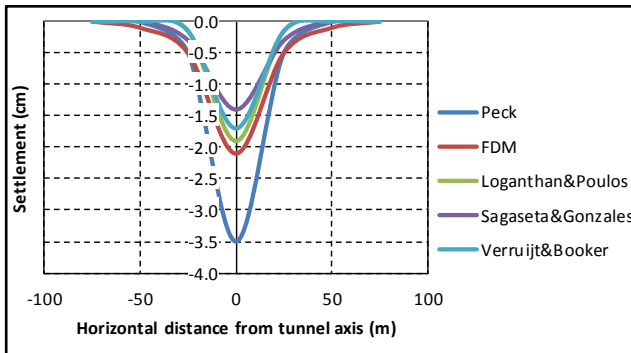| Method	FDM	Peck	Loganthan &Poulos	Verruijt& Booker	Sagaseta& Gonzales
Settlement (cm)	2.1	3.5	1.9	1.7	1.4

Figure 7. Predicted ground settlement due to tunnelling using various methods
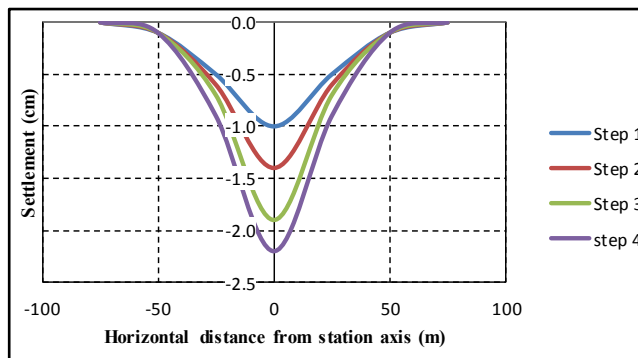

Figure 8. Numerically measured ground settlement due to O7 station excavation

In the next step, ground settlement due to station construction is investigated using numerical method. As

explained in the prior section, the O_7 station is constructed using underground construction method. The excavation process is modelled layer by layer downward. Figure 8 shows the ground settlement due to the excavation steps of the station. In step 1 the access trench is excavated, in step 2 the pile and concrete pad is located, in step 3 top head of station is excavated and finally in step 4 total remaining soil volume is excavated. As shown in Figure 7, the maximum settlement due to station excavation is 22 mm.

Considering the threshold of building damage that has been developed by Rankin and shown in Table 5, the underground construction of the O_7 station has no considerable effect on the existing over-ground buildings.

Table 5. Building damage category due to settlement

Maximum building settlement (cm)	Damage type	Damage level
1	Shallow damage is not probable	Neglect able
1-5	Shallow damage has not structure effect	Low
5-7.5	Shallow damage to building	Medium
7.5<	Structural damage to building	High

6. Conclusions

Numerical and analytical modelling of Tehran 7th line subway tunnel has been performed using distinct element method in a commercial FDM package and four different empirical methods. Results of analysis revealed that predictions using both numerical and empirical solutions are in excellent agreement. Considering the marginal difference (9.5%) between calculated settlement using FDM and Loganthan-Poulos method and also serious difficulties and required skills which is associated with FDM analysis, empirical methods such as Loganthan-Poulos method is preferred than numerical analysis since they are cheaper and easier to use.

To simulate the excavation of subway station, there is no available empirical method which can be used to model such a complicated geometry as designed for Tehran O_7 subway station. In this case FDM is a great tool which can be used to simulate complicated underground excavation as presented in this paper. Also using FDM, it is possible to take the overburden stress due to the weight of over-ground neighbouring structures into account.

Result of simulation of underground O_7 station, confirmed that ground settlement due to excavation of the station, do not impose structural damage of over-ground buildings since the maximum ground settlement is less than 5 cm which is allowable range of ground displacement based on Rankin's theory.

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