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Modeling, stress distribution and the change of ground behavior during excavation of Tehran city underground tunnel

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Abstract
From these factors the tunnel form, dimensions, depth, the tunnel drilling method, presence of water, etc. can be mentioned. Achieving an optimized design requires identification of effective parameters and the relationships among them. Effective design parameters include stability of the tunnel face, horizontal and vertical displacement, seismic wave, and shrinkage phenomenon, etc. Also considering the importance of the structural stability on the ground surface above the tunnel, especially in the urban tunnels, the settlement level of the ground surface must be considered. In this study the stability of an 8m width and height horseshoe sectioned tunnel located in Tehran alluvium having been projected for water transmission has been technically analyzed using the FLAC 2D software and the comparison wad made using empirical and analytical methods. The results showed that the maximum settlement resulting from design step using software is 14.9 mm which is almost consistent with the value obtained from the empirical relationship, and the difference estimation lies within the acceptable range (0.7%). Also the obtained horizontal displacement shows that the maximum displacement within the 25m distance from the tunnel axis (on both sides) is 2mm and the values obtained from the empirical relationship showed that the maximum horizontal displacement occurs on the 25m distance from the tunnel axis.

Key words: FLAC.2D software, urban tunnel, displacement, Tehran city.

1. Introduction

The trend towards creating underground spaces is broadening considering the development of the large cities. The stability of such spaces is of special importance. When underground excavation is in progress, the stresses previously existing in the rocks is changed, resulting in new stresses in the rocks adjacent to the excavated spaces. The project designer must estimate the stability of the constructed building and give careful consideration for erecting the required propping if necessary. Wrong estimations result in collapse. The collapse of the tunnel can cause material damage and sometimes loss of life if precautionary measures are not adopted (Farooqhusaini 1997). The tunnel collapse and consequently the delay in project implementation can be prevented through identifying the factors effective in the tunnel instability and adopting the required solutions. This would cause increase in performance (manpower, machinery, etc.)

and decrease in total cost of the project (Sadeghi 2011and Komiya et al. 2013).

Analysis of the tunnel stability can be carried out in three empirical, analytical and numerical methods. In initial stages of the project where enough information is not accessible, designing through using empirical method would provide for more simplicity and speed. In analytical methods, the propping system is designed based on the formulation and implementation of some of the theoretical models. The numerical analysis methods are advanced methods introduced to the mining engineers' community during the last decades (Delaram et al. 2006). The numerical methods are based on conversion of a medium with infinite degree of freedom to a medium with limited degree of freedom in a certain points of that medium. Through consideration of the force and loading effect in these points and determining their deformation using interpolation, the deformation of other points can be obtained (Qasempor et al. 2009). The classification respectively from the empirical method toward the numerical method entails increase in the precision and reliability of the design. During the recent years, the numerical methods have widely been employed in modeling of the underground spaces (Madani 2007). In initial stages of the project where enough information is not accessible, designing through using empirical method would provide for more simplicity and speed. However using the empirical methods has serious limitations in designing of the underground spaces and they can only be used in the initial stages of the design work (Delaram et al. 2006). Employing the empirical methods is dedicated to the conditions where the geometry and the boundary conditions ruling the problem are simple (Delaram et al. 2006). Frequently the instability mechanisms ruling the problem are relatively complicated and the rock's behavior cannot be described in a soluble differential equation. For example the underground waters' role, discontinuities, in-place stresses, non-linear nature of the rock's behavior, variability of the rocks' strength properties after breakage in different points and non-linear complex loadings are among the factors complicating the behavioral equations of an instability. The role of all the above mentioned factors cannot be considered in empirical methods of designing (Delaram et al. 2006).

In recent years, different computerize analytical methods have been introduced, having been based on the numerical methods and using them, the stress status can be specified in different cases (Sharifzadeh et al. 2006). One of these software is the FLAC.2D which is two-dimensional software, having been built based on the finite differential method. This software was first introduced by Pitter Kondal in 1986 (Madani 2007). In this paper, the revision 2000 FLAC.2D software has been used for tunnel stability analysis. The rigid frame and Lattice design was performed using the analytical method. The ground surface settlement was carried out using two numerical and empirical methods and the results were compared with each other.

2. Geologic setting of the under study area

Alluvial deposits of Tehran have been classified into 4 categories of A, B, C and D based on the oldest to the newest sediments. The size and concentration of each one of these formations distinguishes between them to some extent. The maximum thickness of D formation is estimated to be about 10m. The aggregate constituent size of this formation is decreased from the north to the south. The particle size of the aggregates amounts to 40cm in northern regions, while in central and Sothern regions, their particle size reduces below 20cm. The unit D deposits includes younger alluviums aged bellow 10,000 years that cover the late Quaternary deposits in form of a thin layer with maximum 10m thick (Asghari et al. 2004).

In relatively eastern and northern parts, the C formation sedimentary depositions exist from the main features of which the high concentration and relatively high sorting of their sedimentation components can be mentioned. The maximum size of the deposit pieces of the unit reaches to 20cm. A sample of the C unit deposits has been spotted in construction site of Shahid Zeinedin highway. The unit deposits are significantly uniform in central regions. The major difference of the deposits in northern and southern parts is decreasing their constituents' particle sizes as well as decreasing the clay and silt content of their gravel and cobble deposits.

The clayey alluvium of northern Tehran (B) is unhardened clayey sediment which is a combination of cobblestone, muck, and silt with stone or rectangular distribution. Based on these geologic descriptions, this deposition is the result of accumulation of strong flows during a short period. Depositions of the B unit show the most faciese changes along the way. These changes due to their origin and formation are grossly natural. While in northern regions the unit deposits are gravel coarse-grained accompanied with large pieces of cobble and rocks, in southern parts the unit deposition changes into fine-grained clay and silt. In fact these depositions have glacial origin; hence large variety is observed concerning the nature and concentration of their constituents.

In northern regions the size of stones amounts to about 1m, but in central parts the size of sediments are decreased, so that toward the south of Abbas Abad hills the maximum size of the particle reach to 30cm or less. A sample of the B unit deposits has been observed around Shahid Beheshti Street. Also in southern parts and along the northern Rey fault the unit deposits can be observed which mainly consist of silt and clay depositions with low stiffness (SPT lower than 15). In fact the deposition of conglomerate has been homogeneous and includes the gravel, the sand and gravel-stone whose pores have been filled with the silt and clay. These depositions locally include the lentoid clayey silt with limited thickness of several centimeters to 1 meter. The depositions specifications are as follows:

- Bedding: high slope (80° in some points), folded bedding, high mechanical strength, high cementation, low porosity.

Although the sediments of the unit include a broad spectrum of fine-grained and coarse-grained depositions, the field inspections however showed that the unit deposits are to a large extent similar to the C unit and has been constituted from coarse-grained gravel depositions with high concentration. An example of the unit deposits can be observed around the Ghoochak mountain pass.

3. Discussion: Evaluation of the ground displacement due to the tunnel excavation operations

Settlement: Constructing the tunnel structure causes a series of deformations in the ground above the tunnel, and if the excavation is carried out in shallow depths, it can impose settlements in the ground surface (Fig.1), (Leca et al. 2002).

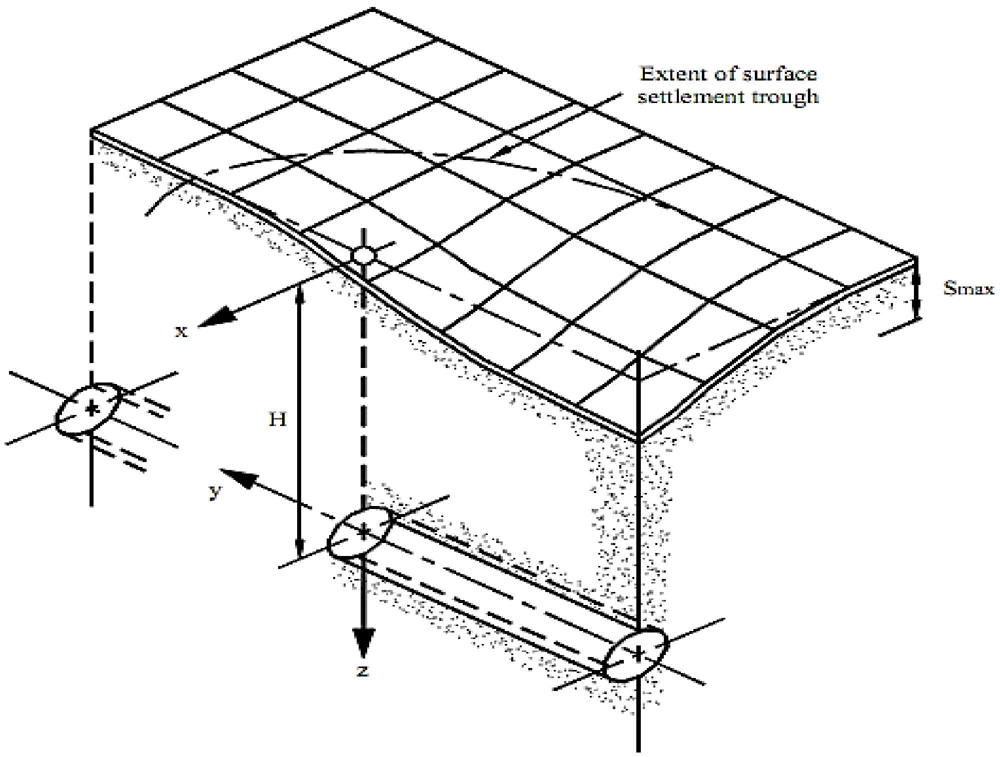

Fig. 1 Three-dimensional distribution of the settlement due to the tunnel excavation (Leca et al. 2002)

Peck (1969) considering a number of tunnel projects affected by surface settlement, showed that there is an inflection relationship as shown in Figure 2. And accordingly the ground surface settlement S(X) is a function of distance from the tunnel axis as shown in the following Figure, (Kashuan et al. 2008):

$$s(x) = S\max \times \exp\left(-\frac{x^2}{2i^2}\right) \qquad (1)$$

Where the Smax is the maximum ground settlement occurring on above of the tunnel crest .

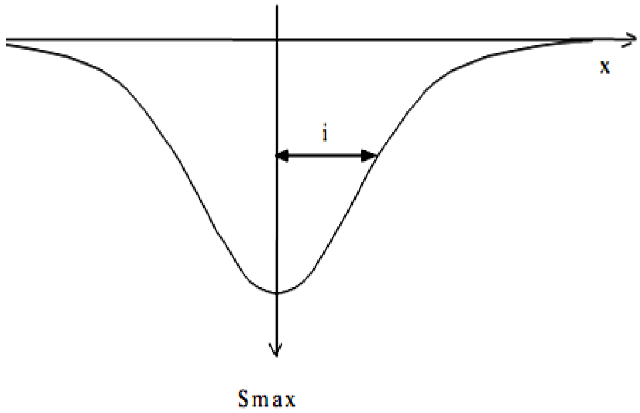

Fig. 2 Surface settlement profile (Leca et al. 2002)

The equation used for approximating the Smax is follows (Safavy et al. 2006):

$$Smax = 0.785 \times (\gamma \times H + Ps) \times \left(\frac{D^2}{(i \times E)}\right) \quad (2)$$

$$i = \frac{\mathrm{i}_1 + \mathrm{i}_2 + \mathrm{i}_3}{3} \quad (3)$$

$$\mathrm{i}_1 = 0.386 \times \mathrm{h} + 2.84 \quad (4)$$

$$\mathrm{i}_2 = 0.5 \times \mathrm{h} \quad (5)$$

$$\mathrm{i}_3 = 1.392 \times \left(\frac{\mathrm{D}}{2}\right) \times \left(\frac{h}{D}\right)^{0.704} \quad (6)$$

where S_{max}: Maximum settlement as per the meter.
i: Center of the curve as per the meter.
D: Tunnel diameter as per the meter.
g: The specific gravity of the soil (ton/m^3)
E: Modulus of elasticity (ton/m^2)
H: Depth from the center of tunnel as per the meter

The maximum settlement values obtained using the above relationships have been shown in Table 1: asTable1. The maximum settlement values in ground surface above the tunnel axis with 70m overburden

Table1. The maximum settlement values in ground surface above the tunnel axis with 70m overburden

Value	Unit	Parameter
70	m	H
22	ton/m^3	**g**
17000	ton/m^2	E
0.015	m	Smax

Based on the equation (1), the ground surface settlement curve above the tunnel axis can be represented as the graph of Figure 3 bellow

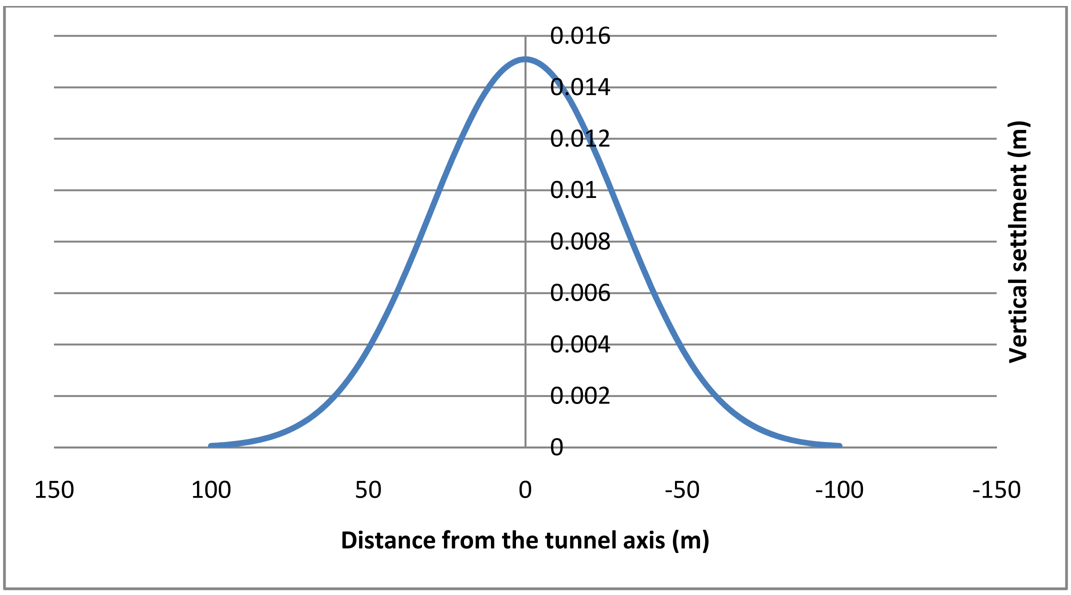

Fig. 3 The settlement graph

3.1.2. Calculation of critical displacement

One of the important parameters that usually are considered in tunnel stability analysis is the displacement rate and the strain ratio around the underground structure (Dufour et al. 2012). Through estimating the displacement rate around the underground space, the stability or instability of a desired structure can be evaluated (Safavy et al. 2006). If the displacement rate and the strain ratio around underground excavations are lower or equal to the critical value, then it could be said that the desired structure is stable, otherwise it is evaluated as unstable. But the question that comes to mind is that how the displacement and critical strain values can be specified (Qasempor et al. 2009).

Calculation of the critical displacement using the Sakurai and et al. (1999) method:

Sakurai and et al. have introduced the calculation method of the critical strain (σ_{cm}) based on the relationship (7) (Qasempor et al. 2009).

σC: Uniaxial compressive strength- Mpa

E: Modulus of elasticity- Mpa

m/n: A coefficient depending on the rock or soil specifications and the joint features which is variable between 1 and 3. □cm: Critical strain

For calculation of the critical displacement, first we calculate the uniaxial compressive strength from the relationship (8).

$$\epsilon_{cm}=\left(\frac{m}{n}\right)\times\left(\frac{\sigma_c}{E}\right) \qquad (7)$$

In the above relationships:

$$\sigma_C=\frac{(2\times c\times \cos\varphi)}{(1-\sin\varphi)}=\frac{(2\times 0.15\times \cos 43)}{(1-\sin 43)} \quad (8)=0.6\ \text{mpa} \quad (8)$$

Considering the relationship (7) we have:

$$\left(\frac{m}{m}\right)=2 \Rightarrow \varepsilon_{cm}=\left(\frac{m}{n}\right)\times\left(\frac{\sigma_c}{E}\right)=2\times\left(\frac{0.6}{170}\right)=0.0071$$

$$u_{cm}=\varepsilon_{cm}\times r_0 \ =0.0071\times 400=29\ \text{mm} \qquad (9)$$

σ_{cm} : Rock mass uniaxial compressive strength- Mpa

C: Cohesion- Mpa

φ: Internal friction angle,deg. U_{cm}: Deformation or critical displacement, mm. r_o: tunnel radius, mm

Considering the relationship (9) the critical displacement will be equal to 29 mm.

3.1.3. full- face drilling

At first of the ceiling witness point as well as considering the expansion of the plastic zone, practically the full-face drilling is impossible. Therefore the section excavation will be executed through multi stage drilling method. Figure 5 shows the state of stress in place before drilling. And Figure 6 represents the ceiling witness point displacement the model excavation has been executed using full-face drilling.

3. 1.4. Investigating the horizontal and vertical displacements

The Figures 8 and 9 show the (horizontal and vertical) displacements' effects developed on the ground surface

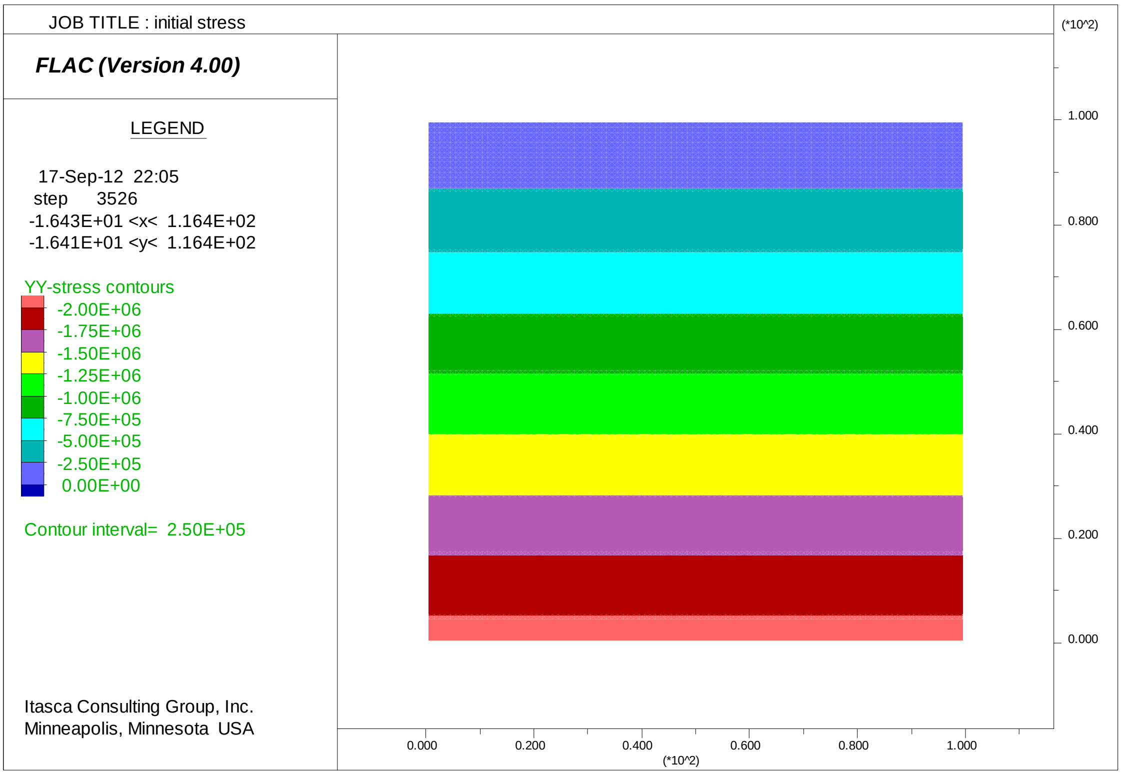

Fig. 5 The in-situ stress before tunnel drilling

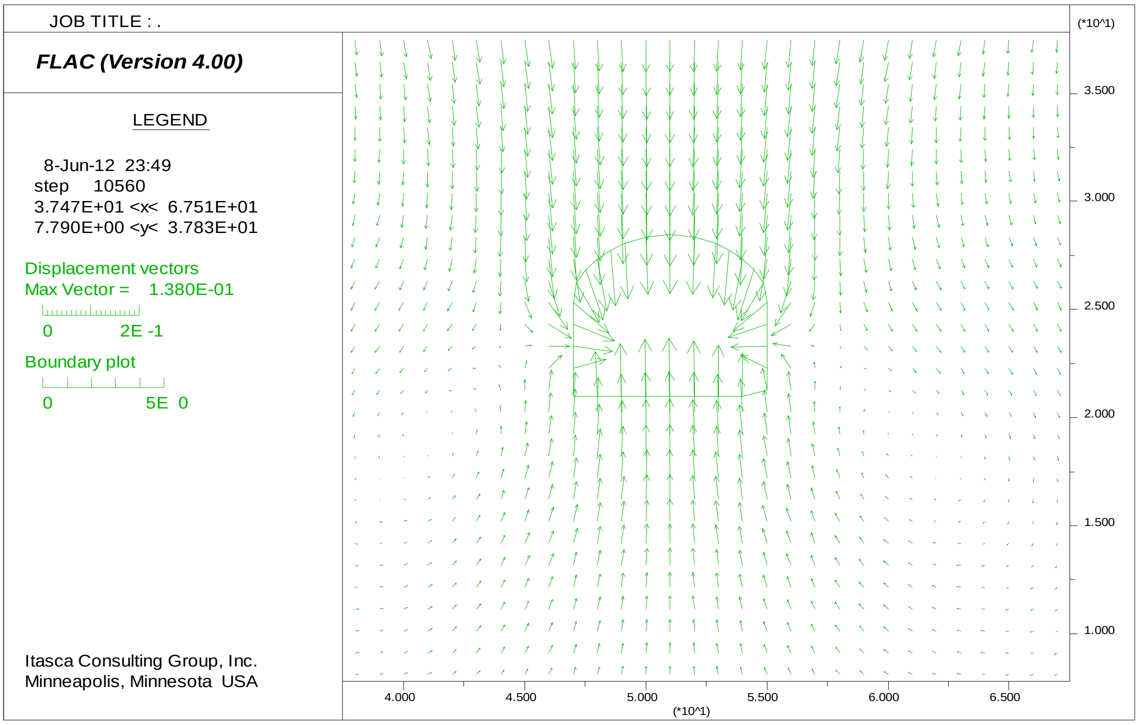

Fig. 6 The tunnel displacement contour; single stage

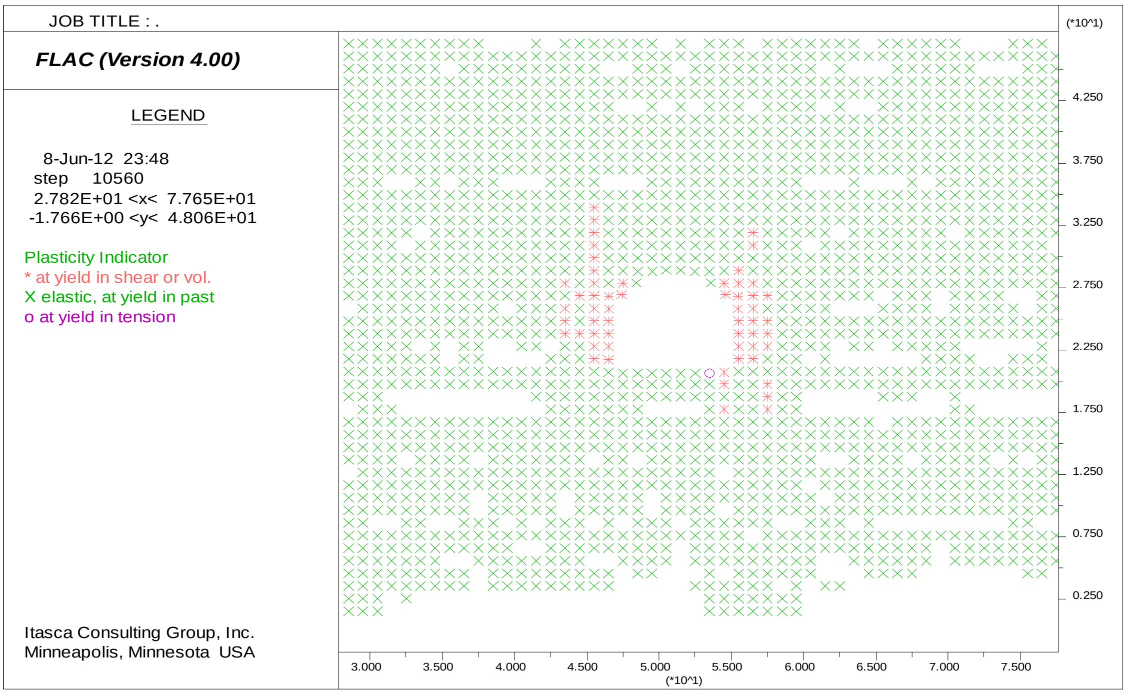

Fig. 7 Expansion of the plastic zone around the section

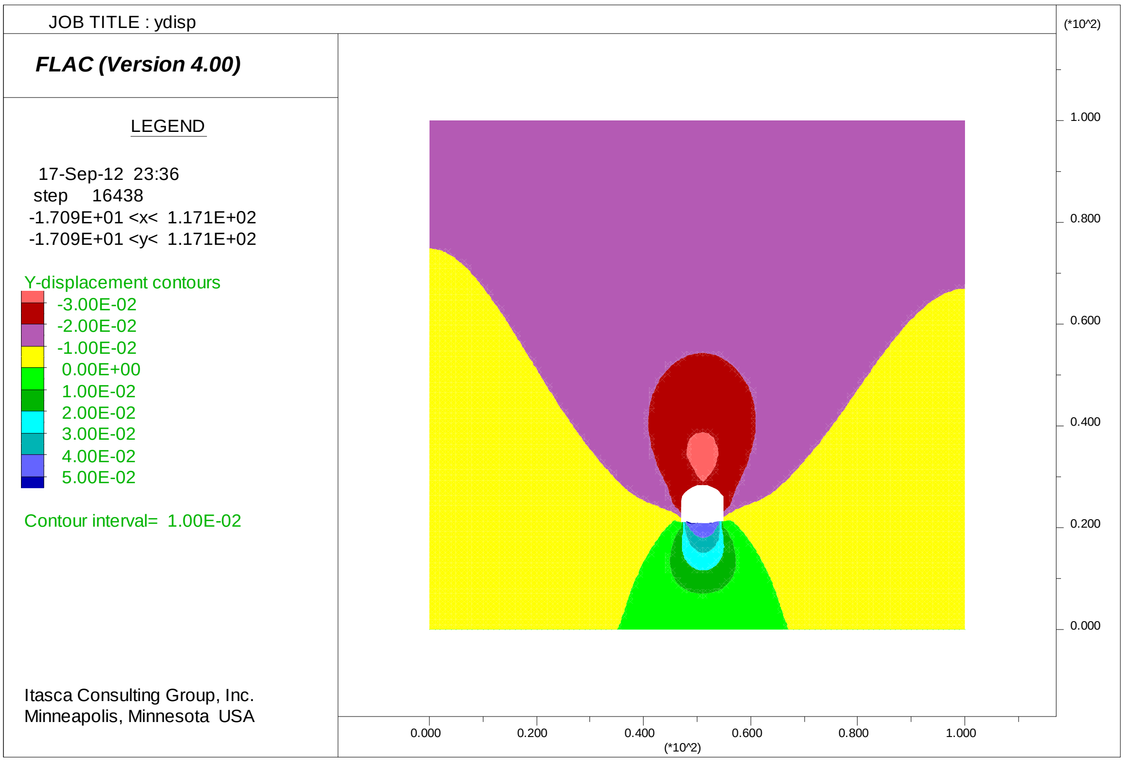

Fig. 8 The section vertical displacement

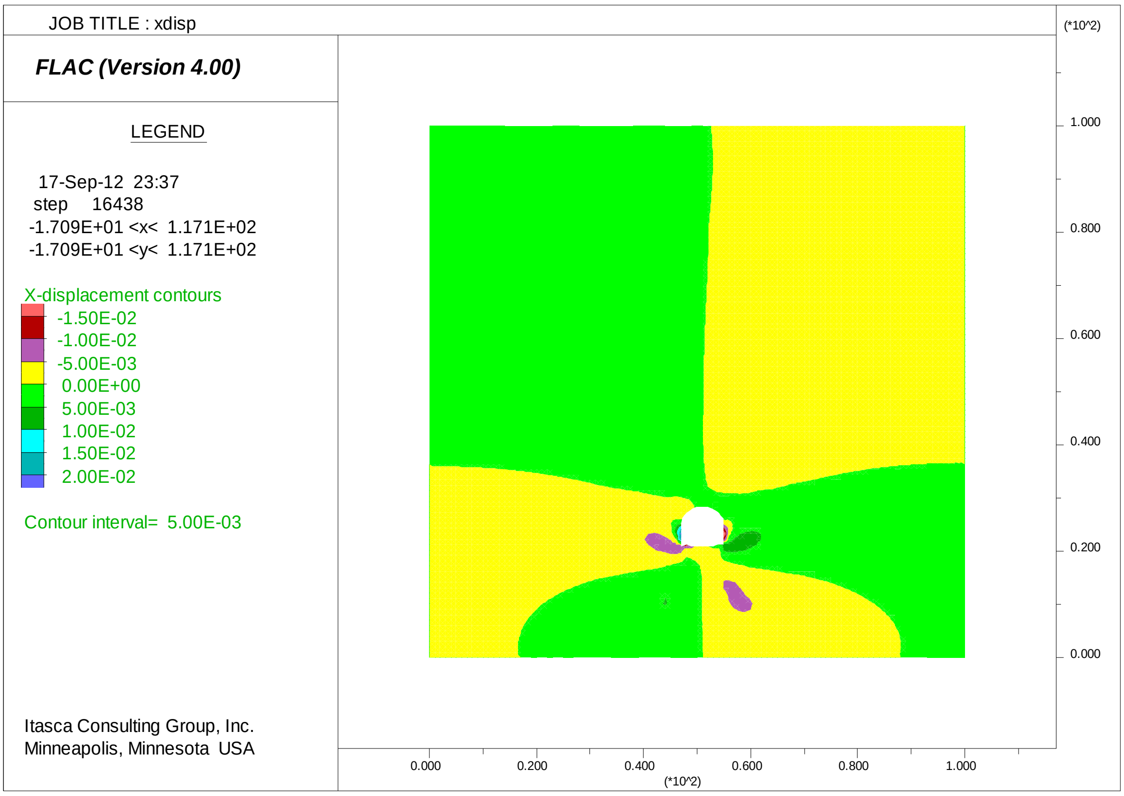

Fig. 9 The horizontal displacement around the tunnel section

Considering the Figure 10 the maximum settlement is 14.9 mm which is almost consistent with the value obtained from the empirical relationship (15 mm) as well as the Figure 3 (with 0.7% difference). Also the horizontal displacement obtained from the Figure 11 shows that the maximum horizontal displacement at 25m distance from the tunnel axis (both sides) is 2mm, having been represented in Figure 11.

Based on the first phase studies of Tehran subway by Soferto Co (French Corporation), the authorized settlement under the buildings and on the streets' surfaces have been determined about 10 and 20 mm respectively (Sadeghi 2011). The maximum vertical settlement and horizontal displacement resulting from the FLAC 2D software is equal to 15mm and 2mm respectively.

Considering that there is a street above the tunnel, the settlement lies within the authorized range.

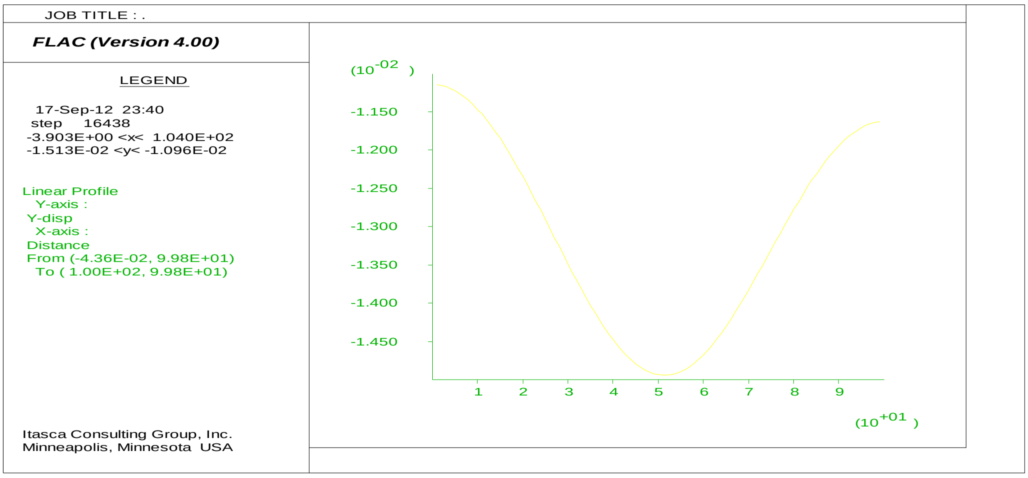

Fig. 10 Settlement in the section

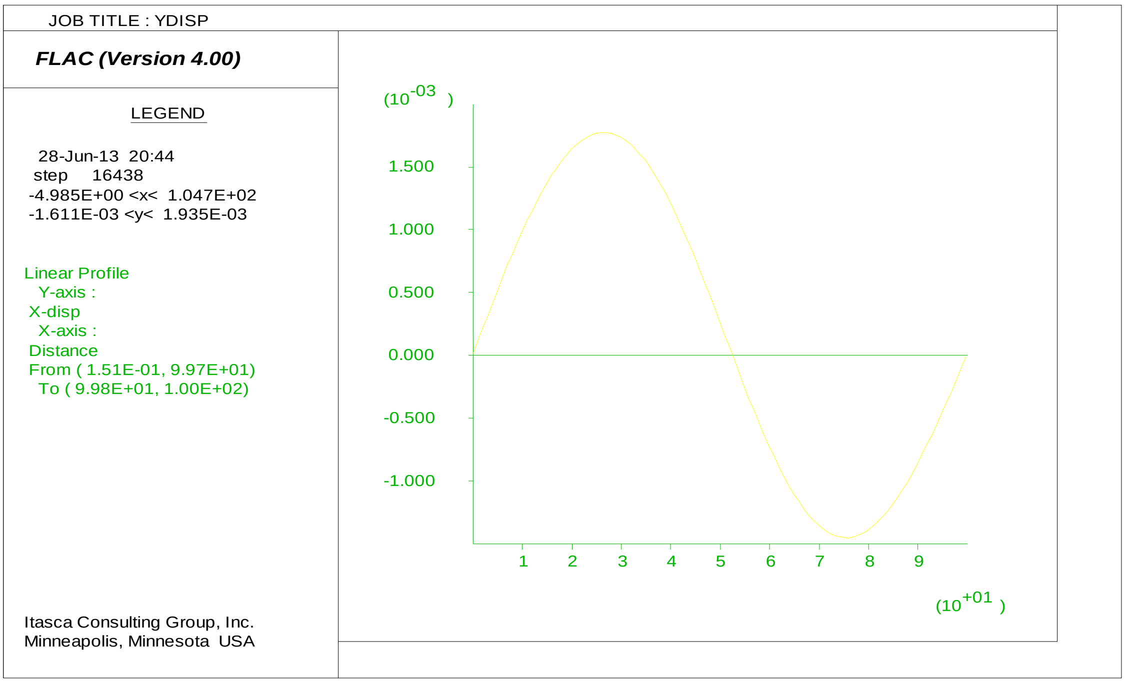

Fig. 11 Horizontal displacement in the section

3.2. Analysis of the tunnel collapse factors and designing the tunnel after collapse

3.2.1. Identifying the tunnel collapse factors

The tunnel collapse has occurred after excavation of the left cut wall. After modeling, the displacement of the Lattice base is nearly 40 mm considering the Figure 12. Considering the unconsolidated status of the tunnel cut bottom, underneath the Lattice base in left cut wall will remain hollow. The stresses imposed on the shot Crete resulting from improper shot Crete cohesion to the tunnel cut, have caused separation of shot Crete from the tunnel frame. Also because of the gravitational force resulting from the shot Crete and Lattice weight, the created displacement is increased and practically the supplied consolidation will be ineffective. Consequently, considering the Figure 13, the developed displacement is more than the critical limit; one of the important factors of collapse is the shear force developed at the springer up to the tunnel floor which has been represented in Figures 14 and 15 (Kaboli. 2012).

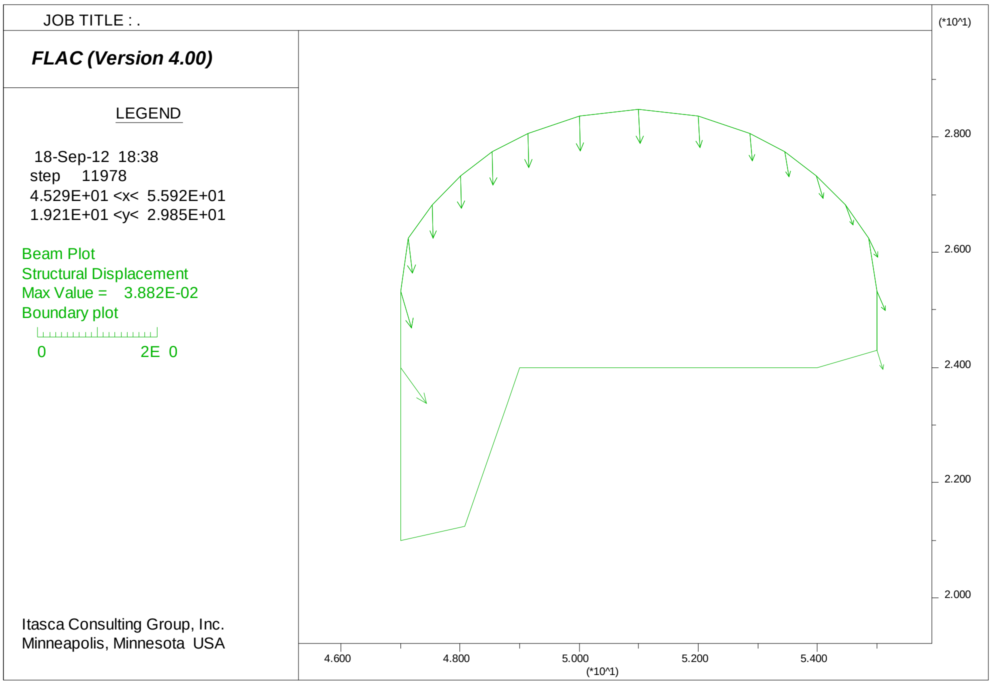

Fig. 12 Shot Crete displacement after excavation of the left cut bank

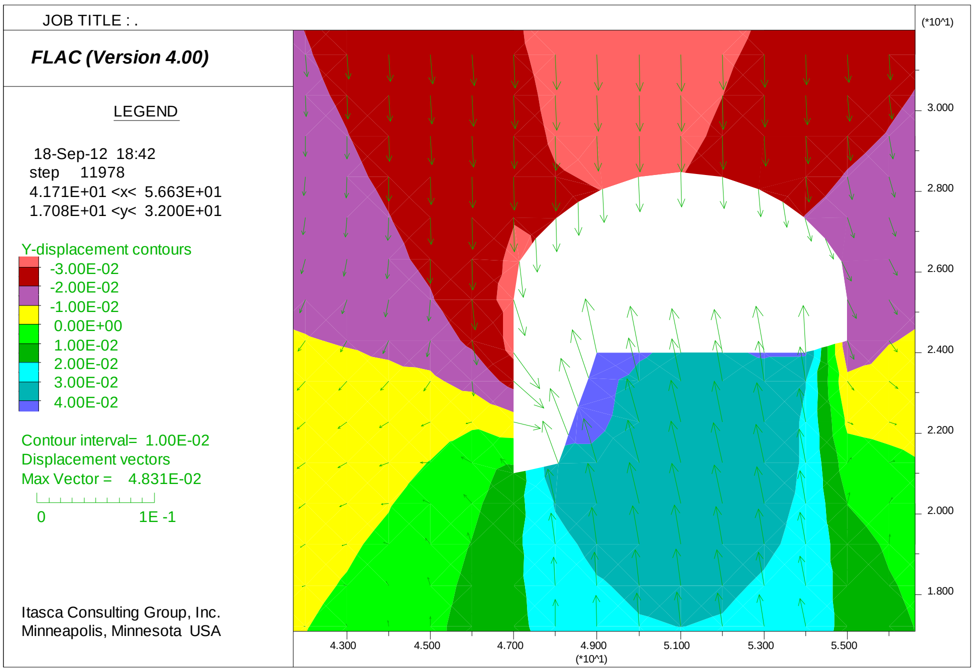

Fig. 13 Vertical displacement after excavation of the left cut wall

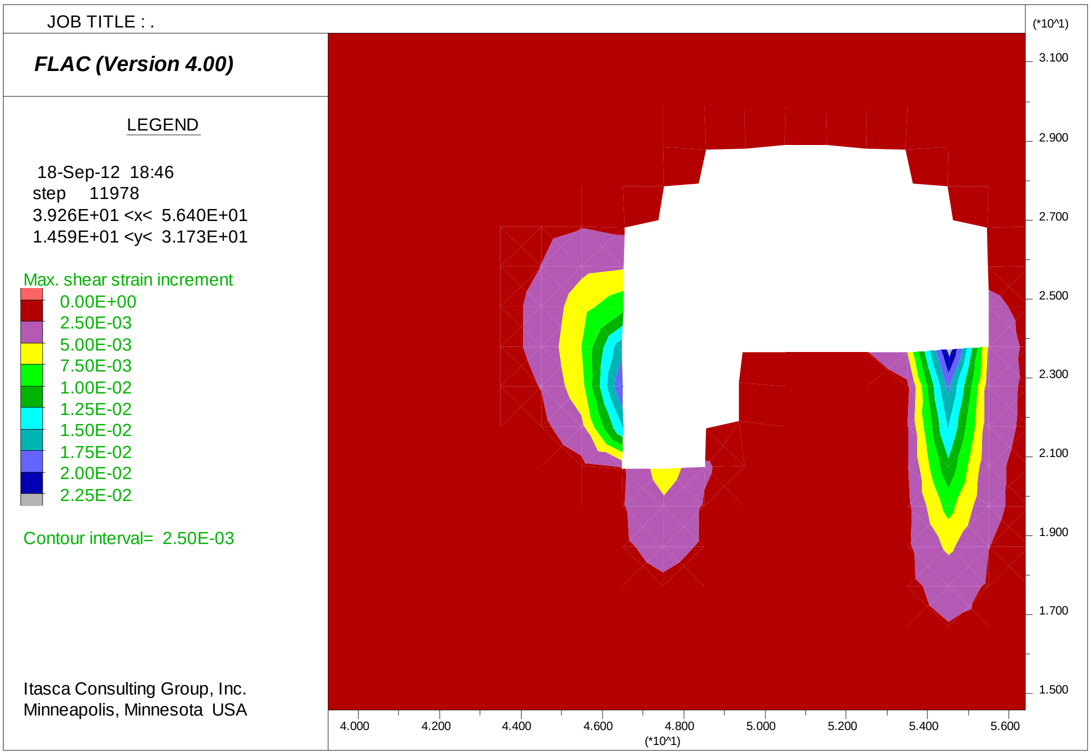

Fig. 14 Shear force developed after excavation of the left cut wall

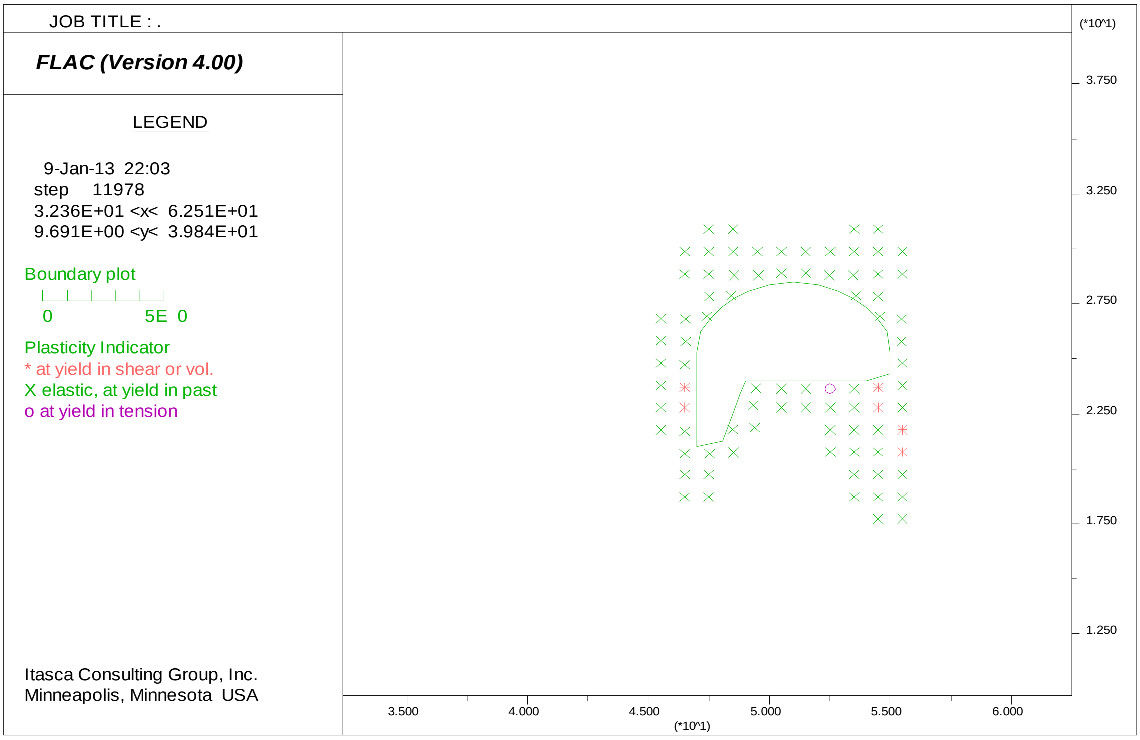

Fig. 15 Expansion of the plastic zone after excavation of the left cut wall

3.2.2. Designing the propping system after collapse: applied load

Determining the load applied on the propping system according to the under study tunnel conditions with previously collapsed parts is complicated and problematic. Considering the Terzaghi load calculation method applicable to tunnels with metal frame propping, the applied load estimation amounts to around 12 ton/m^2 ; the shot Crete and the metal frame bear this amount of load and the applied load on the frame amounts to 10 ton/m^2(Kaboli 2012).

3.2.3. Designing the rigid frame

The stress imposed on the frame is calculated through the relationships (10) and (11).
(Hemmatian 1997 and Sadeghi 2011).

$$\sigma = \frac{p}{F} + \frac{M_{max}}{w} \qquad (10)$$

$$\sigma = \frac{r \times q_t}{F} + \frac{A_Y(h' + 0.5\frac{A_Y}{q_t})}{w} \leq \sigma_{sf} \qquad (11)$$

σ: Stress absolute value (ton/m^2)

F: Section profile area (m^2)
W: Profile section modulus (m^3)
σ_{sf}: Steel authorized stress

$$F = 0.149W + 9.78 \qquad (12)$$

$$A_Y = \frac{(0.785h' + 0.666) \times q_t \times r^3)}{0.666h'^3 + \pi r h' + 4h' r^2 + 1.57r^3} \qquad (13)$$

For the studied section we have:
r: frame radius= 4m
h′= 5m q_t = 10 ton/m^2

The authorized stress for the St-37 steel is 240 Mpa. By replacing the above values in the relationship, the Ay value is obtained to be equal to 5.186, by replacing of which in the relationship, the W is gained to be about 1170 cm^3; accordingly the installation distance of 400mm is obtained for the IPE=240 metal frame.

4. Conclusion

The stability analysis of the ground surface settlement due to excavation of water transmission tunnel in Tehran alluvium has been performed using the FLAC 2D software and the following results have been obtained:

1- From the most important discussions concerning designing of the underground urban tunnels, the ground surface settlement during the project execution can be mentioned that is of utmost importance due to the presence of the surface structures. And collapse has occurred in a section of the above tunnel. Considering the investigation results shows that one of the main factors effective in the tunnel collapse has been the inadequate cohesion of the shot Crete to the tunnel cut. And by the decrease in the drilling depth, the collapsing possibility is intensified due to the fine size of the alluvium particles.

2-Where the excavation is carried out through full-face (single stage) drilling, the maximum displacement amounts to 138mm. Considering that this amount of displacement is higher than the authorized displacement rate, the tunnel excavation was performed in a four stage drilling operation (1 stage for crest drilling and 3 stages for benching).

3-The maximum settlement resulting from the software is equal to 14.9 mm which is consistent with the values obtained from the empirical relationship (with 0.7% difference). Based on the first phase studies of Tehran subway by Soferto Co. (French corporation)، the settlement level lies within the authorized range.

4- Considering the factors effective in the above tunnel collapse, it seems like that in the upper part consolidation stage, a number of preventive measures including the execution of apron, installation of the rolled beam under the Lattice base and stapling them together and nailing can be taken to prevent from separation of the temporary consolidation from the tunnel cut. Another important point to consider is the execution of consolidation operations immediately after drilling.

5- Using the analytical method, the execution with rame withmetal f 400 mm distance from eachIPE 240 other is necessary for the purpose of stabilization of the collapsed space.

Acknowledgement

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