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Analysis and Control of the Ground Displacement along the Drilling Way of Tehran Water Transmission Tunnel

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ABSTRACT

Geologic studies during project execution, installation of precision tools and monitoring of tunnel behavior can greatly help in careful analysis of the tunnel stability and modification of the initial temporary consolidation plan if necessary. In this study that has been undertaken on the Tehran water transmission tunnel while it was in drilling stage, and considering that the drilling was being carried out in shallow depths from the ground surface, development of settlement on the ground surface and occurrence of displacement are of important problems of the project. The tunnel maximum settlement on the upper tunnel axis level with overburdens of 40, 70 and 100m was calculated based on the empirical relationships. The displacement of the witness point at the tunnel crest after drilling was determined to be about 85m, and the displacement on the tunnel left and right cut walls after benching was calculated as equal to 28 and 32mm respectively. The final displacement rate after drilling of the tunnel middle section was less than the critical displacement and the settlement for the tunnel ceiling, left cut wall and right cut wall were 29, 23 and 24mm respectively. For the tunnel better stability, multi stage drilling was performed (one stage of the crest drilling and 3 stages of benching). Considering the existing tensile stress and extremely low strength of non-reinforced shotcrete, the triangular Lattice was designed through analytical method using three reinforcing bars of Ø22 at 60cm interval and the settlement rate was investigated afterwards. The results showed that this method is suitable for preventing from collapsing and meanwhile controlling the geotechnical features, execution of apron underneath the Lattice base and stapling them together as well as execution of consolidation operations immediately after drilling is required.
KEYWORDS: Tunnel drilling, geotechnical features, lattice design, stress.

1. INTRODUCTION

Creating the underground spaces for transportation, transmission of sewage, urban utilities, etc., considering the development of large cities is broadening. Since usually the cities have been located along the river banks and in alluvial lands, hence these spaces are bound to be constructed in loose grounds. Considering the low strength parameters, effect of the hydrostatic level and pore-water pressure role in alluviums, excavation and stabilization of these spaces is highly critical so that in case of inadequate design and execution, occurrence of possible collapses would cause casualties, financial losses, increase in the time of project execution etc[10].

In a general state, underground spaces create stresses in rock mass existing in ground depths called in-situ stress. The main factor playing role in creation of in-place stresses is the weights of the soil beds and the regional tectonic activities. Construction of tunnel in rock mass causes changing in the stress domain status around it in proportion to the state before construction. this change of status develops deformations on the tunnel cuts and the tunnel propping system. Obviously this deformation is not immediate and occurs gradually[13]. If after tunnel excavation in adverse grounds, the propping system it is not completed, the ceiling tunnel would collapse and there is the possibility that the collapse continues up to the ground surface[3].

In May 2000, a tunnel ceiling collapse was occurred during project execution in Afjeh workshop of Lavasan Water Supply Project (5m of the tunnel length with 7m of depth was collapsed). The tunnel section was of horseshoe type with an area of $17m^2$. The ground formation of tunnel project site was Hezardareh conglomerate; the presence of clay in this formation together with the change in the underground water conditions has caused decrease in the strength and collapse of the tunnel. After the collapse, shrinkage and twist was observed in the frames. The underground water problems in drilling of Lavasan water transmission tunnel, weakness of initial studies and the resulting collapse prolonged the project execution time to about two folds. [15] Also lack of enough information on the area where the tunnel is drilled might cause loss of life as well.Because the tensile and shear strengths in alluviums are by far less than the rocks, the stability time of the soft grounds and the soil is very short. The tunnel stability time which is the ceiling and tunnel cuts' stability duration without the propping system is up to several hours in sand grounds, reaching to several hours in compact clay soils [4]. Based on case study about Taloun pilot tunnel in Tehran-North Freeway is shown that we can use instrumentation data to obtain geomechanic parameters and horizontal stresses with direct back analysis. And FLAC software is used for back analysis. [2]

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One of the most important differences between rock and the soil from the underground excavation perspective, is the effect of hydrostatic level and the pore-water pressure role which causes soil bulking towards the drilled space and in case of non-provisioning suitable defensive works, the material collapse will occur under the bulking pressures [13,21].

From other important issues concerning the drilling of tunnels is the settlement resulting from tunnel excavation in urban areas. For example, the ground surface settlement due to the tunnel drilling in part of the line 2 of Tehran subway and the subsequent evaluation of its effects on the adjoining structures can be mentioned. Based on the geotechnical studies, the area soil consisted of different beds of GM-GC, GW, GC ,GM and lenz forms of SW sand beds with variable thickness. It must be mentioned that the resulting settlement has been less than the permissible level. Based on the studies, the permissible settlement level underneath the buildings and the streets' surfaces have been determined as equal to 10mm and 20mm respectively [13].

Considering the importance of the above mentioned details, suitable propping system can be designed through identifying the factors effective in tunnel instability and selecting proper methods of tunnel drilling, so that tunnel collapse and settlement of the ground above tunnel can be prevented. Consequently, the problems resulting from the tunnel collapse can be avoided and the project can be delivered on time with lower cost price.

The following items have been considered for the purpose of this study [13]:

1- Careful study of the geological and geotechnical settings of the project site;

2- Utmost consistency between the project and its execution and the ground conditions;

3-Minimum manipulation of the ground natural status;

4- Continuous record of the ground vibration around the created space considering the surrounding traffic texture;

Wide scope and diversity of the underground engineering projects has resulted in conspicuous achievements in design methods and drilling technology. The distinctive feature of the underground works is their great impressibility from the geologic settings in the desired construction site. Since the geologic and tectonics settings of different points are different from each other, presenting a common relationship for predicting the tunnel strength or the ground pressure imposed on the tunnel supports are almost impossible. For this reason attempts has been made by the researchers –through interpretation of the data acquired from the observations and considering the stability or instability of a large number of the tunnels– to define some definitive criteria based on which as well as considering the specific geomechanical conditions of the ground, the tunnels' stability and the related propping system can be specified with a conservative safety factor [5].

One of the important parameters in stability of the tunnels is proper selection of the drilling method. Selecting proper method of drilling depends on numerous factors, the most important of which to mention are such factors as the geologic settings of the area, hydrous or anhydrous beds, the project duration, the existing devices, the existing experts, the current budget for the project execution and the tunnel section. One of risks affecting the tunnel stability is the lack of control of the surrounding stress-strain field due to selecting an improper drilling method and propping. In construction of large scale underground spaces the above problems are aggravated causing proper selection of drilling a must. In line with aim, dividing the tunnel into smaller sections and consecutive drilling of them considerably helps in the stability of the excavated space [20].

Considering the previous literature, their deficiencies and drawbacks were analyzed and the geotechnical features of the Tehran alluviums were obtained. and the range and values of geotechnical parameters (Young module, cohesion, angle of internal friction, etc.(table 1) were estimated. Also different methods of tunnel drilling in soft grounds was discussed through determining the geotechnical (physical and mechanical) parameters of the project site. The factors effective in the tunnel stability in alluvial lands and the parameters effective in the stability of tunnel face in cohesive as well as the silt lands to determine the horizontal and vertical displacements were investigated and using empirical relationships, the seismic wave effect on the tunnel stability and the shrinkage phenomenon and other effective parameters were considered. And in the end, the effective measures for optimization of the tunnel specifications to avoid the tunnel instability have been proposed [14, 16].

2. General geologic setting of the Tehran alluviums

2.1. **Structure**: From the structure perspective, the Tehran field is divided into the Alborz mountain range and mountainous plain. The Tehran mountainous range with a height between 1300-1700m from the sea level is constituted by a series of eastern-western hills that are parted from each other by two valleys. The Tehran mountainous plain with a height of 1000 to 1300m from sea level starts at a downward gentle slope, covering the Abbas Abad, central and southern Tehran hills, ending at the Rey and Kahrizak valleys.

2.2. **Geologic setting:** One of the most obvious features of Tehran city alluviums is the gradual more or less orderly decrease in the size of the sediments' particle sizes from the north and northwestern parts to the south and southwestern regions. The Tehran expansive alluvium deposits during the past century and especially during the recent 5 decades have been studied by different researchers. Most of the researchers have studied the Tehran alluvial deposits from the geologic perspective and have proposed numerous classifications for this alluvium based on the formation time and their common features. Among these classifications that is used more frequently nowadays, the

Ribben classification (1995) can be mentioned, based on which the Tehran alluvial deposits have been classified into 4 categories of A, B, C and D in accordance with the deposition age (oldest to the newest deposits)[6]. The size and concentration of each one of these formations to some extent distinguishes them from one another. 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 southern 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. These deposits are in the form of wide and expansive alluvial cones that their ingredients' particle sizes decrease distancing from the heights, so that in lowland areas they are changed into the silt and clay, often classifiable among the GP, SP and GC soils according to the unified classification criteria. In northern Tehran, the deposits' constituents are coarse-grained gravel and sand. Also the grading of samples collected from the coarse-grained part of this deposition unit shows that these deposits are of weak grading with maximum thickness of 5m located in upper part of the unit. It seems like that the atmospheric and physical factors effect has caused degradation of its strength and compaction. These deposits have considerable strength and compaction in depths more than 5m. It's worth mentioning that in a number of the soil borings drilled in these areas, beds and Lenz forms of fine-grained deposits (mostly silt) with a maximum thickness of 1m are seen locally among the coarse-grained deposit beds. Such a state is observed among the existing soil borings around the Azadi Square and Blvd. Lateral expansion of these beds is limited and is often exclusive to lentoid deposits.

Although the detailed geologic and engineering description of Tehran alluvium largely encompasses their specific features, introducing the geotechnical parameters of these alluviums however can provide for a more clarified image of their situation [1]. For this purpose, some of the geotechnical features of the alluviums extracted from the investigation of soil borings have been introduced in table 1.

Table 1- Some of the geotechnical parameters of Tehran alluvial field

Alluvium material	Soil type classification system in USCS	Dry specific weight (Kg/m^3)	The ground reaction Module (Mpa)	Internal friction angle- deg.	Young module (Mpa)	Cohesion (Mpa) cohesion- (Mpa)
Coarse-grained gravel relating to the a formation in northern Tehran	GC-GP-GM	1900-2200	3	37-30	120-80	0.02-0.1
Coarse-grained gravel relating to the B formation in northern Tehran	GC-GP	2000-1700	2	34-25	50-20	0.005-0.01
Coarse-grained gravel relating to the C formation in central and northern Tehran	GW-GC	2000-1800	1-2.5	40-30	80-40	0.01-0.03
Coarse-grained silt relating to the central and southern areas	SP-SC	1900-1650	0.5-1	30-25	20-15	----
Fine-grained silt in southern areas	ML	1800-1600	0.4-0.8	20-15	30-20	0.015-0.02
Fine-grained clay in southern areas	ML-CL	1700-1500	0.2-.04	10>	25-10	0.01-0.04

3. Geotechnical parameters of the project site

3.1. **Physical parameter:** The physical parameters include: Water percentage (W), Relative density, D_{10}: The screens diameter through which % 10 of total soil particles pass. D_{30}: The screen diameter through which %30 of total soil particles pass.

D_{60}: The screen diameter through which %60 of total soil particles pass. Coefficient of uniformity, $C_U= D_{60}/ D_{10}$
Compaction index: $C_C=(D_{30})^2/ (D_{60}\times D_{10})$

Table 2- Part of the physical parameters of the project site

Av.Sample (1,2,3)	Av.Sample (4,5,6)	parameter
0.17	0.35	D_{10}
0.84	1.59	D_{30}
4.71	6.13	D_{60}
26.26	17.75	C_U
0.94	1.19	C_C
2200	2200	G (Kg/m^2)
11	8.2	W%

3.2. **Mechanical parameters:** Mechanical parameters include**:** Internal friction angle, Cohesion Young module, uniaxial compressive strength (Mpa), Poisson's ratio

The angle of internal friction and cohesion are measured using three methods:

-The laboratorial method, (triple axis test and direct shear test).

-Field method , (large scale shear test).

-The empirical formula based on the laboratorial and field methods.

Results of the direct shear test performed on six samples are represented in the table 3

Table 3- Part of the mechanical parameters of the project sit (direct shear test)

Av.Sample (4.5.6)			Av.Sample (1,2,3)			Unit	parameter
sp			Sw			Unified classification	Soil type
1.5	1	0.5	1.5	1	0.5	Kg/cm^2	Vertical stress
1.06	0.8	0.45	1.1	0.7	0.55	Kg/cm^2	Shear stress
11			8.2			Percentage	Humidity
31			28			deg.	Internal friction angle
0.016			0.023			Mpa	Cohesion

The results obtained from the point load test have been represented in table 4 . Considering the above test results, the angle of internal friction and cohesion have been obtained as 51° and 0.23 respectively.

Table 4-The point load test results

No.	Width (mm)	Diameter (mm)	Load (N)	Ucs (Mpa)
1	70	45	300	1.02
2	65	35	700	0.88
3	75	68	1000	1.55
4	60	35	500	0.82
5	60	40	600	0.96
6	70	48	800	1.31

In general, the geotechnical features of the A and B formations have been represented in the table 5.

Table 5- The geotechnical parameters of the tunnel site

Formation	Specific weight Kg/m^3	Shear module Mpa	Bulk module Mpa	Friction angle deg.	dilation angle deg.	Young module	Cohesion Mpa
A	2200	68	113	43	12	170	0.15
B	1960	30	50	35	10	75	0.012

4. Drilling stage

The above tunnel drilling will be executed in 1 stage of tunnel crest excavation and 3 stages of benching according to the Figure 1.

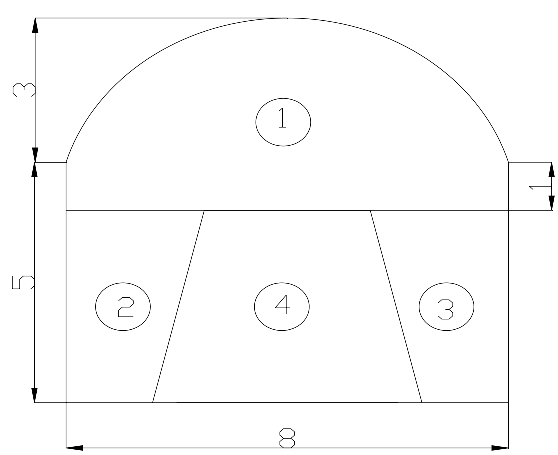

Fig 1- Tunnel section drilling sequence

4.1. Tunnel crest excavation: Figure of 3 represents the displacement of the witness point at the tunnel ceiling after drilling of the tunnel crest. As can be seen from the above curve, the vertical displacement amounts to about 85mm, taking into account the obtained displacement value, temporary propping system is needed for stabilization of the section. The modeling procedure of the propping system includes the following steps: after tunnel excavation, the model solution will continue to the point where about 40% of the ground surface displacement occurs in case the propping system is not installed. Then the model solution is halted and the propping system is applied. Again the solution of model will continue up to the equilibrium point [11,12]. After consolidation in accordance with the Fig (3), the witness point displacement in tunnel ceiling will decrease to 24mm.

4.2. Benching of the tunnel left cut bank: The Figure 2 shows the ceiling witness point displacement after benching of the tunnel left cut bank (before consolidation). Also the left cut wall witness point displacement after benching of the left cut bank (after consolidation) can be seen from the Figure 3. Considering the above figures, after consolidation of the tunnel left cut wall, the ceiling and left cut wall witness points' displacements will decrease from 30 to 27mm and from 28 to 22mm respectively.

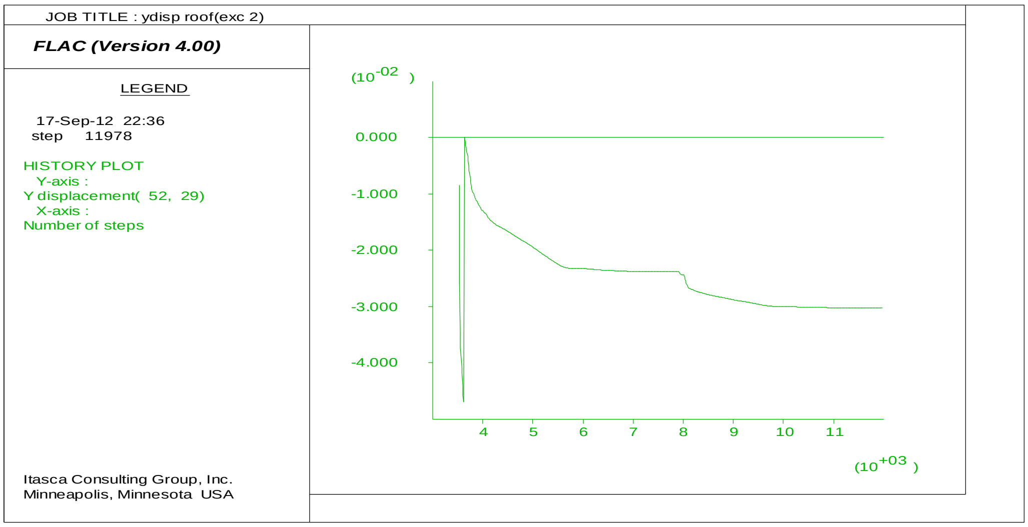

Fig 2- The ceiling witness point displacement; the tunnel left cut wall drilling (before consolidation)

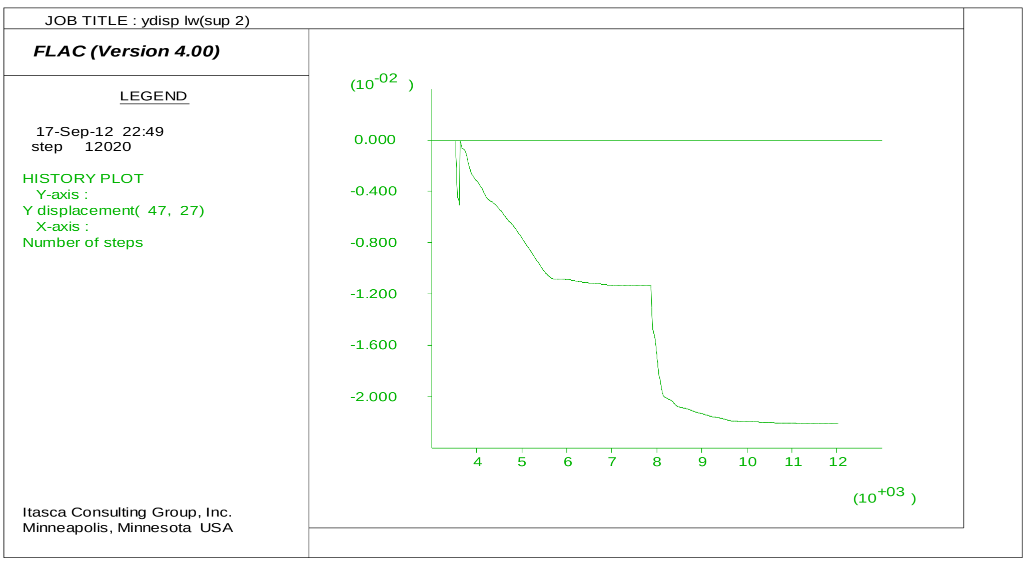

Fig 3- The left cut wall witness point displacement; the tunnel left cut wall drilling (after consolidation).

4.3. Benching of the tunnel Right cut bank : The Figure 4 shows the right cut bank witness point displacement after benching of the right cut bank (before consolidation). Considering the above curves, after consolidation of the tunnel right cut bank, the right cut bank witness point displacement will decrease from 32 to 24mm.

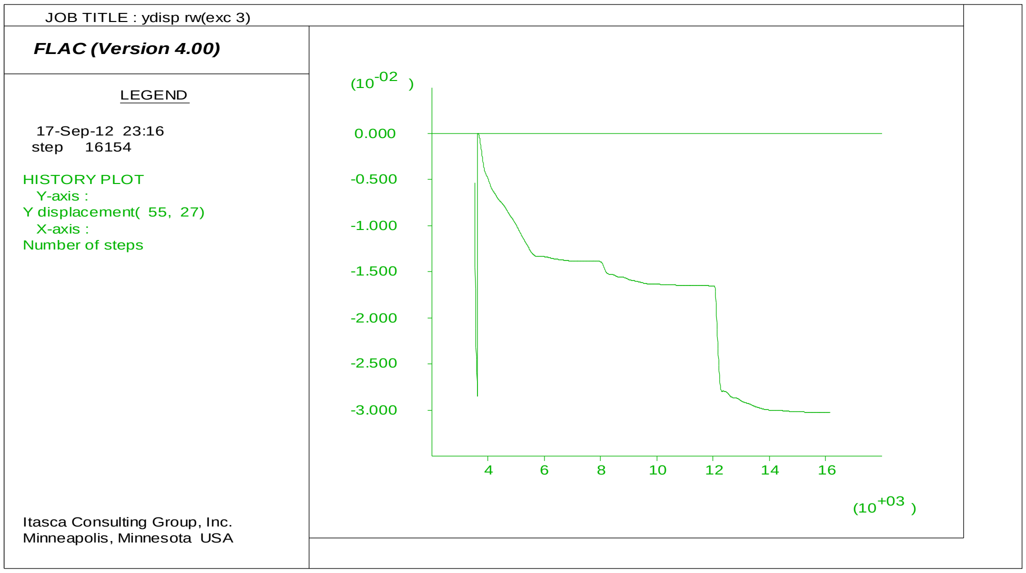

Fig 4- The right cut bank witness point displacement; the tunnel right cut bank drilling (before consolidation).

Finally, after drilling of the middle part, the tunnel drilling will come to the end. Figures of 5, 6 and 7 represent the witness point final displacement trend of the ceiling, left cut wall and the right cut bank respectively. Table 6 shows the displacement rate after the drilling and consolidation operations. The displacement developed is less than the critical displacement lower. Consequently the treatments controlled the displacements within the permissible range.

Table 6-The tunnel final displacement rate

Location	The displacement rate (mm)
Tunnel ceiling	92
Tunnel left cut wall	23
Tunnel right cut bank	24

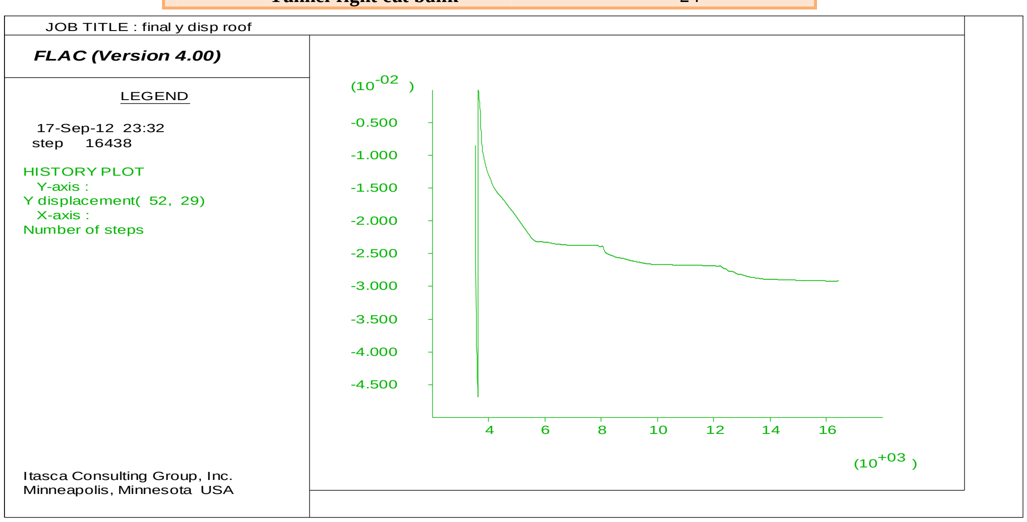

Fig 5- Final displacement trend of the ceiling witness point (29mm)

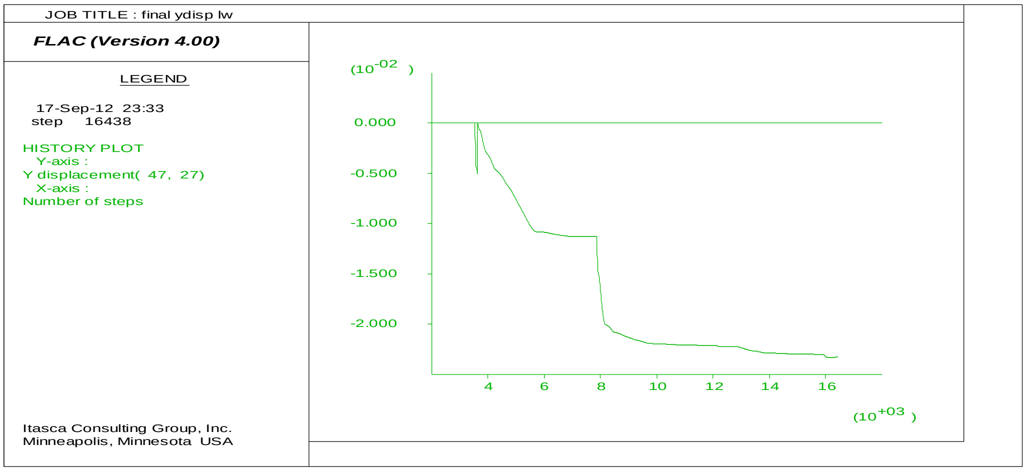

Fig 6- Final displacement trend of the left cut wall witness point (23mm)

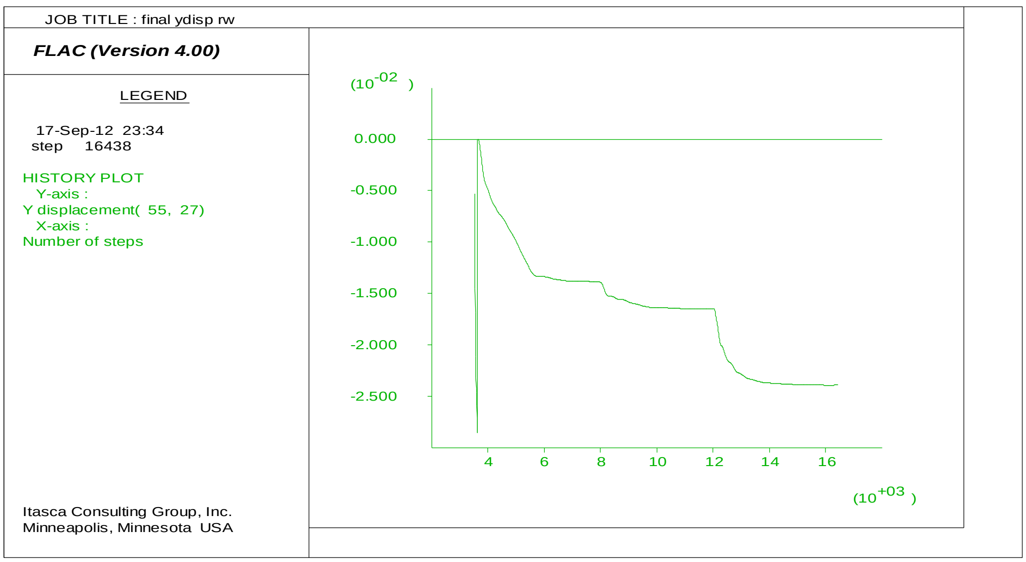

Fig 7- Final displacement trend of the right cut bank witness point (24mm)

After drilling and temporary consolidation (300mm shotcrete) the plastic zone expansion is as shown in the Figure 8.

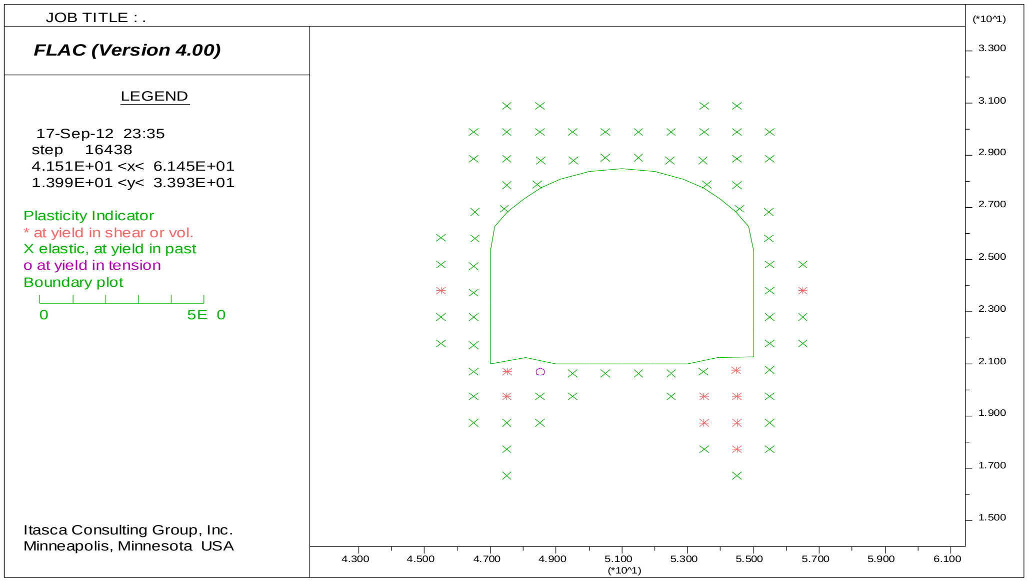

Fig 8- The plastic zone expansion after termination of the consolidation operations

4.4. Investigating the shot Crete strength: It can be observed that the maximum compressive stress imposed on the 100mm shotcrete is 10934 t/m^2 which is more than the final allowed shotcrete value (2000 t/m^2). Therefore the execution of another 100 mm shotcrete is required.

It can be observed that the maximum compressive stress imposed on the 200 mm shotcrete is 3045 t/m^2 which is more than the permissible compressive strength value of the shotcrete (2000 t/m^2). Therefore the execution of another 100 mm shotcrete is necessary [7,8and 9]. Table 7 shows the stress imposed on the shotcrete. It can be observed that the maximum compressive stress imposed on the 300 mm shotcrete is1491 t/m^2 which is less than the permissible compressive strength value of the shot Crete (2000 ton/m^2).

Table (7) -The stresses existing in the 300mm shot crete

Element	Shear force (N)	Axial force (N)	Moment-1 (N/m)	Moment-2 (N/m)	Stress-1 (ton/m^2)	Stress-2 (ton/m^2)
1	-7.73E+04	1.21E+06	-5.07E+00	-1.02E+05	402	-277
2	-6.33E+04	1.33E+06	1.02E+05	-1.62E+05	1121	-635
3	1.61E+05	1.24E+06	1.62E+05	-4.91E+04	1491	87
4	5.07E+04	1.17E+06	4.91E+04	-1.38E+04	717	298
5	2.59E+04	1.06E+06	1.38E+04	3.60E+03	444	376
6	1.11E+04	8.94E+05	-3.60E+03	1.11E+04	274	372
7	1.04E+04	7.72E+05	-1.11E+04	2.06E+04	184	395
8	1.05E+04	5.96E+05	-2.06E+04	3.12E+04	61	406
9	-1.55E+04	5.86E+05	-3.11E+04	1.55E+04	-12	299
10	-8.12E+03	7.27E+05	-1.55E+04	8.08E+03	139	296
11	-7.76E+03	8.11E+05	-8.13E+03	2.91E+03	216	290
12	-2.28E+04	9.27E+05	-2.92E+03	-1.24E+04	289	226
13	-6.04E+04	1.02E+06	1.24E+04	-5.45E+04	424	-22
14	-1.44E+05	1.02E+06	5.45E+04	-1.55E+05	704	-691
15	8.60E+04	1.04E+06	1.55E+05	-7.36E+04	1379	-144
16	1.01E+05	9.00E+05	7.36E+04	2.89E+04	791	493
17	7.77E+03	1.05E+06	5.97E-01	9.90E+03	349	415
18	2.24E+04	1.23E+06	-9.89E+03	3.25E+04	345	628
19	-4.59E+04	1.25E+06	-3.25E+04	-1.01E+01	200	417
20	-7.69E+03	8.51E+05	1.36E+01	-7.80E+03	284	232
21	-2.02E+02	-3.63E+04	-2.04E+02	-6.98E-01	-13	-12
22	-1.03E+04	1.00E+06	8.01E+03	-2.89E+04	387	-140

5. Designing the Lattice

Considering the Table (7) it is evident that many of its elements are under tension and even the tensile stress has been increased to -691 ton/m^2; consequently considering that the shotcrete is unable to bear tensile stress, using the Lattice as reinforcement would be necessary. For the calculations of Lattice, the reinforcement amount required to resist the tensile stress must be calculated. Hence we have[17, 18]:

M_e=M+(P×e)=15.5+(102×0.15)=30.8 Ton.m (1)
M=15.5 Ton.m , p=102 Ton , h =30 cm , e=15 c m , b=1 m , K_c=0.63
$A_s = K_s . (M_e/h) + (p/\alpha_c) = 13.68^2$ (2)
⇨ 2 Ø 22 @ 600 mm

P: Axial force (t), M: flexural moment (ton/m^2), M_e: Maximum moment effective on the section (ton/m), h: width of section (m), e: half of the section width (m), b: The initial distance considered for the Lattice (m), A_s: Area of the required reinforcement section (m).

Therefore the triangular Lattice has been designed using 3 Ø22 reinforcing bars (1 for the compressive and 2 for the tensile part). The Lattices are 600mm apart from each other.

6. Conclusion

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 shotcrete 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- For the tunnel stability, multi stage drilling (one stage of tunnel crest and three stages of benching) declines the displacement rate and settlement and involves lower collapsing risks.

3- Using the analytical method, the execution of triangular Lattice with 3 reinforcing bars of 22mm diameter and the Lattice distances of 60cm together with IPE= 240metal frame with 400mm distance from each other is necessary for the purpose of stabilization of the collapsed space.

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- We propose that for a more precise analysis of the tunnel stability, the numerical methods' results are controlled through using the empirical and analytical methods[14]. Meanwhile the appropriate geologic studies during the execution stage can be used for inspection of the designed treatments.

7. Acknowledgement

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