RESEARCH ARTICLE OPEN ACCESS

Rock Mass Characteristics Modification Using Back Analysis Method for Isfahan Subway

Farzad Habibbeygi*
*(Former Sazbon Consult. Eng. Co Technical Manager, Tehran, IRAN)

ABSTRACT
In the recent decades, the construction of tunnels has been increased vastly in all over the world. Despite all the efforts have been put into the geotechnical investigation, determining the precise and reliable design parameters is a difficult task to accomplish. Therefore, using back analysis techniques to modify geotechnical parameters and optimize the design of initial and final supports is more effective and even less expensive nowadays. In this paper, the results of monitoring and instrumentation of Isfahan subway have been investigated. Moreover, two important parameters for stability analysis, the modulus of elasticity of rock mass and the coefficient of the lateral earth pressure, are modified based on the measured convergence by using the direct method. Results show the Young modulus of rock mass and the lateral earth coefficient are less than the initial values from geotechnical investigation.
Keywords-Lateral earth coefficient, Young modulus, tunnel, rock, back analysis

I. INTRODUCTION

The first step in designing underground projects is to determine geotechnical parameters for numerical modelling of the initial supporting system as well as final lining. However, defining geotechnical parameters with a reasonable accuracy is always a real challenge for geotechnical engineers especially in rock media. Rock mass strength parameters are hardly calculated precisely from laboratory tests by using small specimen in the light of joint sets, pore pressure, bedding and specimen size effect. Although various in-situ tests have been developed to measure geotechnical parameters, they are either expensive or impractical to perform especially in urban areas where there are traffic and environmental issues. Even by performing the in-situ tests, the results only represents the geomechanical parameters of rock mass in the specified region which the tests have been carried out. Since geological structures are heterogeneous and tunnels are too long, determination of the precise and reliable geomechanical parameters for all tunnel long is almost impractical. Therefore, back analysis methods are executed to calibrate the preliminary geotechnical parameters derived from geotechnical field investigation and laboratory tests. Back analysis method has been widely used as an indirect method for underground stability analysis all over the world to optimize either the design of supporting structures or to modify geotechnical parameters [1-10].

Back analysis methods can be divided into two major categories based on their methodology: direct and indirect approaches. Indirect back analysis approach also referred as either inverse approach or optimal back analysis [11, 12]. Sakurai and Takeuchi [13] were one of the first researchers used indirect back analysis method to compute the modulus of elasticity. Similarly, Gioda and Maier [14] used this method to define bulk modulus and shear modulus. In the indirect method, the measured displacement and stresses used as input parameters while unknown parameters such as earth pressure coefficient and elasticity modulus are the output parameters [15]. On the other hand, the direct method is based on trial and error technique. In this method, initial values for unknown parameters are chosen in stress analysis. Afterwards, the difference between the measured parameter and the computed parameter is calculated by an error function. The procedure continues until the error function gets minimized. Different methods have been suggested in the literature for minimizing the error function [14, 16]. Oreste [3] stated to choose a representative model, an error function and an algorithm when performing a back analysis. The algorithm is used to reduce the difference between calculated value from numerical analysis and monitored parameter at the field. In this experiment, the direct method is used for back analyses because of its ease of implying finite element program directly.

II. STUDY AREA

Isfahan has become a bustling city due to its historical background as well as its huge growth in the steel industry. Every year thousands of tourists from all over the world visit the city. In addition to tourist attractions, there is a variety of industries in Isfahan where lots of workers commute from suburbs to the central business district every day. Isfahan subway system has five major lines; Line 1 of Isfahan subway is a 12.5 km long twin tunnel that connects the north to the south of the city with 15 stations. The tunnels in this line are located in a distance varies from 11 m to 21 m (center to center of the tunnels). Tunnel section of a single tunnel of Isfahan subway line 1 is demonstrated in Fig. 1. As seen in this figure, the horse-shoe shape has been selected for ease of construction and providing enough standup time. The overburden depth of southern part varies slightly from 10 to 15 meters. Moreover, the overburden for studied region is approximately 10 m with a 5 m of alluvium atop of shale rock. However, tunnels go through different geological structures like shale, shaly sandstone, sandstone, and friable sandstone, geological structure is almost the same in the studied region that is grey to dark shale rock. Shale rock is fully jointed and the spacing of bedding is only a few centimeters that make it practically impossible to model the joints and beddings in computer software using distinct element methods. The geotechnical parameters including joint spacing, dip, dip direction, and roughness are listed in Table 1 for four main joints and bedding.

The Southern part of the line is a 3.4 km twin tunnel is constructed by NATM (New Austrian Tunneling Method) in two stage. In the first stage, the head of the tunnel is excavated by roadheaders and the initial supporting system includes reinforced shotcrete and lattice girders will be installed immediately. After the head stage is completed the excavation of bench stage will commence; the same initial supporting system has been used for this stage. The whole thickness of supporting system after installation is about 20 cm (steel lattice girder and reinforced shotcrete combined).

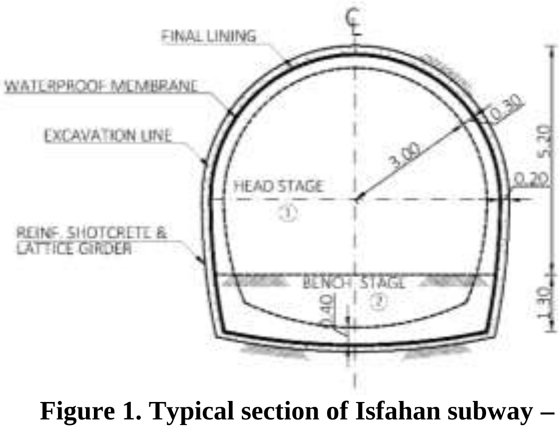

Figure 1. Typical section of Isfahan subway – Single tunnel [17]

Even with having the most elaborated computer programs to analyze the stability of underground excavation and design the initial supporting system, the most important issue is having the accurate input data for geotechnical parameters. One of the most effective methods for this purpose is certainly Hoek-Brown failure criterion[20]. This criterion is highly accepted in engineering projects worldwide to derive equivalent friction angle and cohesion strength for rock mass. Geotechnical parameters for various geological structures of the studied area are tabulated in Table 2. These equivalent parameters can be used both in limit equilibrium and numerical programs. Parameters can be derived simply by linking empirical classification indices like either Rock Mass Rating (RMR) or Geological Strength Index (GSI). However, using GSI is much more reliable in weak rock to find input data [20]. Based on the geological investigation, RMR of the studied region is about 30 to 40 [18].

III. BACK ANALYSIS PROCEDURE

The monitoring program of the southern part of Isfahan subway comprised of convergence pins, surface settlement meters and multi-point borehole extensometers (MPBX). More than forty seven convergence stations and fourteen MPBX installed in this part [17]. Typical section of convergence pins installation is illustrated in Fig. 2. As shown in this figure, five pins have been installed immediately after excavation to measure the initial deformation. Monitoring has been performed by inspecting and evaluating the measurements from five pins: one in the crust of tunnel (C), two in the walls in top stage (R, L) and two in the walls in the bench stage (R_1, L_1).

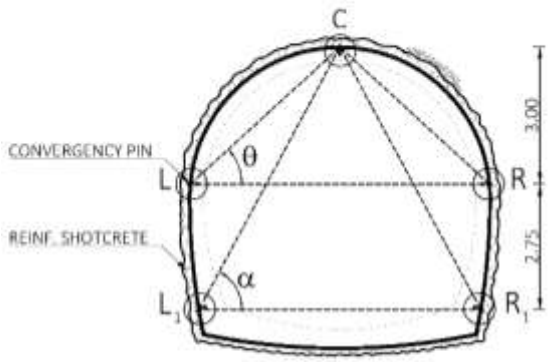

Figure 2. Typical section of convergence pins installation [17]

Since in the time of monitoring of the west tunnel, the second tunnel (east tunnel) had not been excavated, the effect of this tunnel was ignored. Fig. 3 illustrates the model used in back analyses. PLAXIS® is used for numerical analysis in this study. First, the gravity analysis has been done to model the initial in-situ stress. In this stage, 20 KN/m has been applied to model the traffic loading of the street that tunnel is excavated beneath it. After this stage, the top excavation is modeled and initial supporting system consists of steel lattice girder and 20 cm reinforced shotcrete has been activated.

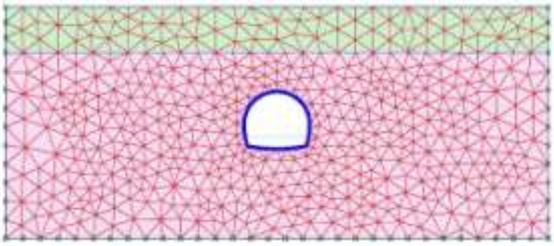

Figure 3. Mesh generation used for numerical analyses

Then bench excavation as the final stage is done. Normal stiffness (EA) and flexural rigidity (EI) of initial support (20 cm shotcrete) are considered $2.9\text{x}10^6$ (kN/m) and 9865 (kNm^2/m) respectively in numerical analysis. Relative displacement (convergence) monitored during a 2-year period at Km.9+308 (the studied section) is shown in Fig. 4. The difference between numerical results and monitored displacement is computed by error function as below [10, 21]:

$$Error\ \% = \frac{\sum_{i=1}^{N}(u_i - u_i^*)^2}{\sum_{i=1}^{N} u_i^*} \times 100 \qquad (1)$$

whereu_i and u_i^* are measured and calculated value of point i, and N is the number of measured points. According to Eq. 1, error target can be derived from Eq.2 to Eq.8 for Isfahan subway. In these equations, α=42° and θ=62°.

$$R_1L_1 = \sqrt{\left(x_{L_1} - x_{R_1}\right)^2 + \left(y_{L_1} - y_{R_1}\right)^2} \qquad (2)$$
$$RL = \sqrt{(x_L - x_R)^2 + (y_L - y_R)^2} \quad (3)$$
$$CR = (x_c - x_R)\cos\theta + (y_c - y_R)\sin\theta (4)$$
$$CL = (x_c - x_L)\cos\theta + (y_c - y_L)\sin\theta (5)$$
$$CR_1 = (x_c - x_{R_1})\cos\alpha + (y_c - y_{R_1})\sin\alpha (6)$$
$$CL_1 = (x_c - x_{L_1})\cos\alpha + (y_c - y_{L_1})\sin\alpha (7)$$

(8)

$$Error\ \% = \frac{(RL - RL^*)^2 + (R_1L_1 - R_1L_1^*)^2 + (CL - CL^*)^2 + (CR - CR^*)^2 + (CL_1 - CL_1^*)^2 + (CR_1 - CR_1^*)^2}{RL^* + R_1L_1^* + CL^* + CR^* + CL_1^* + CR_1^*}$$

where$R_1L_1, RL, CR, CL, CR_1, CL_1$ are the convergence values according to Fig.2.

Target function has been computed for earth pressure coefficient and young modulus of elasticity of rock mass herein. Fig. 5 shows the error function for young modulus. As it seen in this figure, the error is minimum for the value about 1.5 GPa. The error value between elasticity modulus of 1 GPa to 2 GPa is remained almost constant. According to this analysis, the real Young modulus is less than that considered in the geotechnical investigation. The main reason for less stiffness for rock mass is the existence of multiple joint sets that caused the shale rock completely crushed. Furthermore, back analysis has been performed by the modified Young modulus to compute the lateral earth pressure. The error function for this computation is illustrated in Fig. 6 for the modified young modulus and lateral earth pressure coefficient. As it can be seen in this figure, the error is minimum for lateral earth pressure of 1.5 which is higher the initial value.

IV. CONCLUSIONS

Back analyses for determining the modified values of Young modulus and lateral earth pressure have been performed for Isfahan subway, Line 1 in this study with the observed displacement of monitoring and instrumentation. Six different relative displacements have been monitored for a 2-year period and utilized in numerical modeling to modify the geotechnical parameters of the rock mass. According to this study, the Young modulus of the rock mass is less than the initial value from laboratory test results because the rock mass is completely jointed and fully crashed. However, the error value remains almost constant in a range of Young modulus from 1 GPa to 2 GPa. Back analysis performed by modified Young modulus shows the lateral earth pressure is higher the initial value and reaches 1.5.

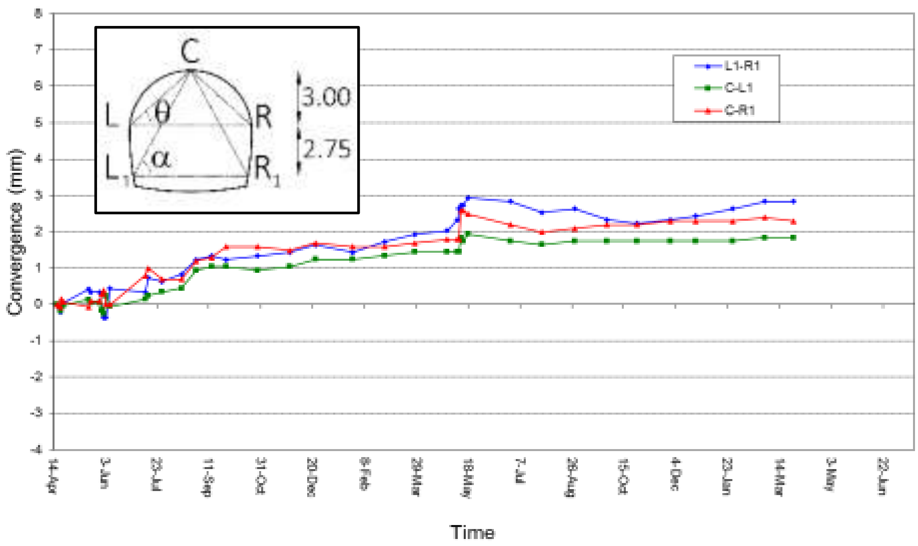

Figure 4. Relative displacement at Km.9+308 [17]

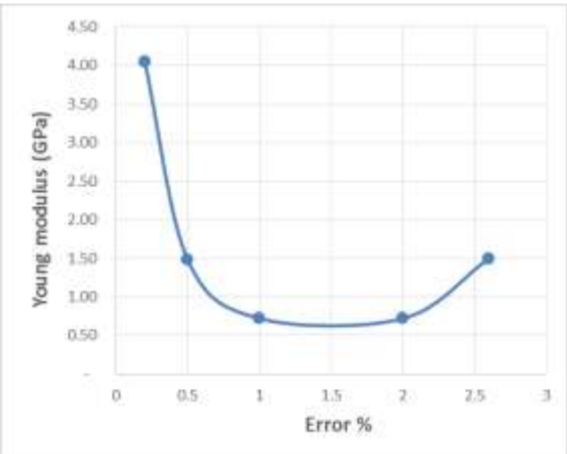

Figure 5. Target function minimization graph for young modulus of rock mass media

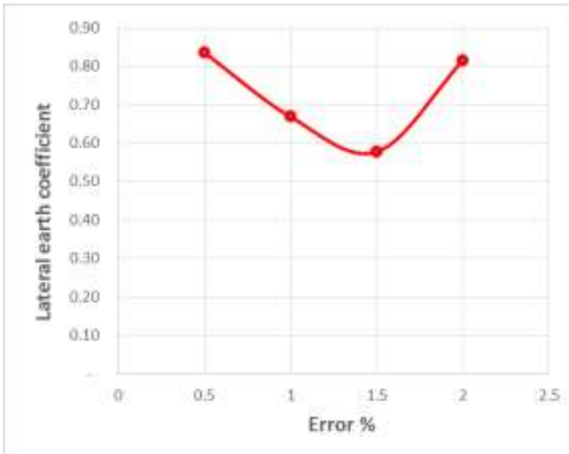

Figure 6. Target function minimization graph for lateral earth coefficient

Table 1. Joints and bedding properties [18]

Discontinuity	Dip°	Dip Dir.°	Length (m)	Spacing (m)	Aperture (mm)	JRC*	Infilling
Joint set 1	*83*	*90*	*1.5 to 6*	*0.25 to 0.5*	*2*	*2 to 4*	*None - Soft filling*
Joint set 2	*75*	*236*	*1 to 3*	*0.1 to 0.3*	*1.5*	*2 to 4*	*None - Soft filling*
Joint set 3	*75*	*286*	*0.5 to 2*	*0.2 to 0.65*	*2*	*4 to 6*	*None*
Bedding	*55*	*10*	*0.15*	-	-	-	-

* JRC: roughness profile [19]

Table 2. Geotechnical parameters of rock mass [18]

Rock Type	γ_{dry} (kN/m^3)	γ (kN/m^3)	n	E_m (GPa)	C (kPa)	f °
Alluvium	*17*	*18*	*0.4*	*0.5*	*60*	*25*
Shale	*26*	*27*	*0.32*	*2.6*	*170*	*42*
Sandstone	*26*	*27*	*0.28*	*7.2*	*380*	*48*
Weak Sandstone	*26*	*27*	*0.28*	*2.5*	*140*	*46*

γ_{dry} : Dry rock unit weight , γ: Unit weight of rock, **n :** Poisson ratio,
E_m: Young modulus of the rock mass, C: Cohesion of the rock mass,
f : Rock mass friction angle

REFERENCES

[1] Guan, Z., et al., Back analysis technique for mountain tunneling based on the complex variable solution. International Journal of Rock Mechanics and Mining Sciences, 2013. 59: p. 15-21.

[2] Hisatake, M. and Y. Hieda, Three-dimensional back-analysis method for the mechanical parameters of the new ground ahead of a tunnel face. Tunnelling and Underground Space Technology, 2008. 23(4): p. 373-380.

[3] Oreste, P., Back-analysis techniques for the improvement of the understanding of rock in underground constructions. Tunnelling and Underground Space Technology, 2005. 20(1): p. 7-21.

[4] Russo, G., et al., Artificial Ground Freezing to excavate a tunnel in sandy soil. Measurements and back analysis. Tunnelling and Underground Space Technology, 2015. 50: p. 226-238.

[5] Shang, Y.J., et al., Intelligent back analysis of displacements using precedent type analysis for tunneling. Tunnelling and Underground Space Technology, 2002. 17(4): p. 381-389.

[6] Shen, W.S., Optimization on Highway Tunnel Structure. Applied Mechanics and Materials, 2013. 312: p. 37-41.

[7] Xu, Z., et al., Back-analysis approach for the determination of hydraulic conductivity in rock caverns. Tunnelling and Underground Space Technology, 2015. 47: p. 233-238.

[8] Yang, C.-X., Y.H. Wu, and T. Hon, A no-tension elastic–plastic model and optimized back-analysis technique for modeling nonlinear mechanical behavior of rock mass in tunneling. Tunnelling and Underground Space Technology, 2010. 25(3): p. 279-289.

[9] Zhao, H., Z. Ru, and S. Yin, A Practical Indirect Back Analysis Approach for Geomechanical Parameters Identification. Marine Georesources&Geotechnology, 2014. 33(3): p. 212-221.

[10] Dadashi, E., et al., Support system suggestion based on back-analysis results case study: Babolak water conveyance tunnel. Arabian Journal of Geosciences, 2011. 5(6): p. 1297-1306.

[11] Feng, Z. and R. Lewis, Optimal estimation of in-situ ground stresses from displacement measurements. International journal for numerical and analytical methods in geomechanics, 1987. 11(4): p. 391-408.

[12] Cai, M., et al., Back-analysis of rock mass strength parameters using AE monitoring data. International Journal of Rock Mechanics and Mining Sciences, 2007. 44(4): p. 538-549.

[13] Sakurai, S. and K. Takeuchi, Back analysis of measured displacements of tunnels. Rock mechanics and rock engineering, 1983. 16(3): p. 173-180.

[14] Gioda, G. and G. Maier, Direct search solution of an inverse problem in elastoplasticity: identification of cohesion, friction angle and in situ stress by pressure tunnel tests. International Journal for Numerical Methods in Engineering, 1980. 15(12): p. 1823-1848.

[15] Swoboda, G., et al., Back analysis of large geotechnical models. International Journal

for Numerical and Analytical Methods in Geomechanics, 1999. 23(13): p. 1455-1472.
[16] Cividini, A., L. Jurina, and G. Gioda. Some aspects of 'characterization'problems in geomechanics. in International Journal of Rock Mechanics and Mining Sciences &Geomechanics Abstracts. 1981. Elsevier.
[17] Alamoot Bridge & Building Engineering Co. and Sazbon Consult. Co, Monitoring and instrumentation report of southern part of line 1, Isfahan subway. 2006: Iran.
[18] Alamoot Bridge & Building Engineering Co. and Sazbon Consult. Co, Geotechnical and geological report of southern part of line 1, Isfahan subway. 2004: Iran.
[19] Barton, N. and V. Choubey, The shear strength of rock joints in theory and practice. Rock mechanics, 1977. 10(1-2): p. 1-54.
[20] Hoek, E., C. Carranza-Torres, and B. Corkum, Hoek-Brown failure criterion-2002 edition. Proceedings of NARMS-Tac, 2002. 1: p. 267-273.
[21] Jeon, Y. and H. Yang, Development of a back analysis algorithm using FLAC. International Journal of Rock Mechanics and Mining Sciences, 2004. 41: p. 447-453.