A parametrical study of tunnel-pile interaction using Numerical analysis

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1. Introduction

Increase in population needs new transportation routes in urban areas. Shallow urban tunnels are good alternate with compared to decreasing cost and congestion on surface routes. These shallow tunnels are usually in the range of soil or below in weathered rock. Subsurface utilities such as foundation of structures, bridge piles, sewer pipes, telephone and electric cables are very near to these driving tunnels causing adverse effects like settlement, tilt and in extreme case failure of structures. In order to secure the serviceability of these utilities protection is needed and in some cases a safe distance between driving tunnel and important structure is also important. The later alternate is usually not allowed in many cases. In this paper settlement of pile at lateral distance from crown has been studied with changing ground conditions of weathered rock.

Most of the research work has taken in soil and very limited in rocky ground. C.J. Lee and S.W Jacobsz (2006) studied in three dimensional parametric elasto-plastic numerical analysis for tunnel pile interaction. They concluded that when the pile is 0-0.6D of the tunnel then the settlement of pile do not follow the green field settlement distribution. Pile settlement is much more than green field settlement. They suggested a lateral distance of 1.0D safe for pile with driving tunnel. Yong-Joo Lee and Richard H. Bassett (2007)studied the effect of pile tip settlement using model test and numerical analysis (FEA). They proposed four influence zones associated with normalized pile settlement obtained both from model test and numerical analysis. There influence zones are relatively deeper and wider as compared to previous studies.

2. Ground behavior in shallow tunnels

2.1 Methods to analyze ground behavior due to shallow tunneling

The development in technology, research work and more increasing activities in Geo-technical engineering have emerged in a number of techniques to study the behavior of ground during underground activity. For example Laboratory Modeling techniques, Numerical analysis method and study of real ground behavior in field. In laboratory Modeling method more often centrifugal models are used. The model materials are specified and then the real stress condition of the ground is simulated by giving centrifugal acceleration to the model. During some activity like tunneling the ground behavior is studied and results are superimposed to real problem. M.Hisatake and S.Ohno 2007 studied the effect of ground deformation in soil ground using centrifugal model tests. They applied an acceleration of 35G(G is gravity which is 9.1m/s^2) and used a robot which was able to imitates the movement of road header for tunnel excavation. Using this centrifugal model they got good results for full face and ring cut with and without pipe roof supports. In numerical analysis the ground materials are defined and the response is studied numerically. Numerical analysis is one best technique used these days in Geo-technical problems. In most underground construction this is also one method for design. This method gives very good results if the material properties defined are same with actual problem. We can get the ground behavior in advance and can decide what steps must follow. Another method is the study of ground during underground work and to decide about excavation procedure and support estimation. This method today in Geo-technical engineering is known as NATM (New Austrian Tunneling Method). Today in underground construction this method is very common and most beneficial. One good aspect of this method is that it can be applied to changing ground.

2.2 Remedies to Improve Ground Condition

With tunneling the ground is deformed and stress condition is changed. This also change the stress condition near the ground utilities if they are near to driving tunnel. Stabilization of ground is the best remedy to control the adverse effects on these near utilities. Methods like pore piling, jet grouting ,sequential excavation, face stabilization, drainage control and other methods are used.

In fore piling pipe roof supports is applied before the face advance in weak ground. This is very effective auxiliary method as it controls the ground movement. M. Hisatake and S. Ohno 2007 studied the effect of pipe roof supports in front of the face advance in soil ground using centrifugal model test. They concluded quantatively that in full face excavation due to pipe roof support ground displacement is reduced to one fourth of the displacement without pipe roof support. A cement grout is applied in a sub horizontal way along with tunnel axis to form a protective umbrella. The grout is applied above the crown or also along the walls of the tunnel. This reduce the surface settlement. S.C Coulter and C.D. Martin applied pressurized jet grouting and made a sub horizontal arch umbrella with tunnel axis which reduced the surface settlement to a great extent. One method to secure the existing structures is separation method. In this method separation structures(piles or walls) between the underground space to be excavated and the existing structures in the vicinity of underground structures are installed.

3. Interaction between shallow soft ground tunnels and surface utilities

Ground along tunnel parameter is deformed. In shallow and soft ground surface settlement trough is produced and the ground moves toward the tunnel cavity. This situation is very dangerous for near foundations as differential settlement occurs and in some cases failure of whole structure on surface.

3.1 Interaction in soil ground

When tunneling in soil ground the behavior and ground loss is very clear. In many studies the influence zone boundary have been assumed at an angle of $\beta = 45 - \frac{\phi}{2}$ from vertical of the tunnel boundary to ground surface where ϕ is the friction angle of soil. The width of the surface can be defined at $2.5i$ at the ground surface with its source at the tunnel lining. Where i is the distance from tunnel centerline to point of infection (Young-Joo Lee, Richard. H. Bassett , 2007).

3.2 Interaction in weak and weathered rock

In weathered and weak rock the adverse effect is movement of ground toward cavity. The influence zone and width of ground settlement trough depends on rock condition, overburden and cavity dimensions. The pile near the tunnel is effected by deformation of ground around them. Change of axial force and shear force along the pile parameter takes place. The main adverse effect is the settlement when the ground below the pile tip moves toward cavity.

In weathered and weak rock tunneling combination of support system such as shotcrete, rock bolts, wire mesh and steel sets to stabilize the ground is required (Suleyman Dalgic, 2000). (Continued)

4. Numerical Analysis

Numerical Analysis is one technique to study the effect of ground due to tunneling. Finite Element, Finite difference, UDEC, Boundary condition method etc are different numerical techniques used in Geo-technical engineering these days. For continuum model Finite Element, Finite Difference like FLAC 2D, FLAC 3D and Boundary Element methods are used. For dis continuum model UDEC program is used.

Numerical analysis solve equation of equilibrium and compatibility by giving the mechanical properties and boundary conditions in Numerical Model. These programs divide the rock mass into small elements and then force and displacement is calculated.

Numerical analysis is a very quick and easy method to estimate the amount of support used or to study the effect of ground due to tunneling. Numerical analysis is perimetrical technique and very commonly used in Geo-technical engineering. The good results depend on the input data, so a little error in input data will give wrong results.

Shallow ground tunneling is 3-D numerical problem. At tunnel face there is already some deformation and so this effect can not be shown in 2-D numerical analysis. In 3-D numerical analysis lateral and longitudinal settlement trough can be studied while in 2-D only lateral settlement trough is possible to examine. S. Coulter and Martin 2006 writes in his paper displacement analysis due to tunneling is a 3 dimension problem as the face advance, while it is recognized that a 3-D may be required to completely analyze the complete stress strain response 2-D can be successfully used to back analyze the tunnel induced settlement trough. Panet and Guenot (1982) reported pre-convergence values of about 27% of the final displacement using 3-D simulation and an elastic model. Higher values up to 50% have been observed in field measurements and computational analysis in soft ground tunneling e, g (Moraes, 1999). Load transfer in the longitudinal direction due to soft ground arch can not be represented in 2-D analysis (Marcio Muniz de Farias, Alvaro Henrique Moraes Junior, Andre Pacheco de Assis, 2004).

M. Karakus 2007 suggested methods in which 3-D response can be applied to 2-D tunneling method in numerical analysis. He made remarks that tunnel construction cause volume loss and this is completely 3-D response. This response can not be simulated in 2-D analysis so when tunneling is analyzed in 2-D some assumptions like volume loss, percentage of theoretical volume and deformation prior to structural elements applied is needed. He suggested the methods such as convergence confinement method, disk calculation method, progressive softening method, volume loss control method and hypothetical modules of elasticity (HME). Some details about these methods can be find in his paper.

Analyzing tunnel in 2D plain strain condition is widely used method to calculate tunneling induced settlement and soil structure interaction (M. Karakus, R. J. Fowell, 2005). Using 2D FDM they back calculated settlement, stress redistribution and ground movement adopting the excavation method of NATM. (Continued)

5. Settlement trough due to tunnel excavation

In vicinity of excavated tunnel ground is deformed which reaches ground surface in case of shallow overburden. Tunnel-foundation interaction occurs if foundation is in the influence settlement zone.

H.Mroueh and I.Shahrour used U_{dec} and L_{dec} to model TBM excavation in 3-D numerical analysis, where U_{dec} and L_{dec} represents partial stress release before line installation and unlined excavated length. In their results they got softening factor coefficient $R_{dif} = \frac{W_{surface}}{W_{crown}}$ equal to 0.4. They concluded that 40% of the settlement reaches the surface to that of the crown. The R_{dif} value depends on material properties, tunnel dimensions, overburden value and values of U_{dec} and L_{dec}. They used the value of Ü and L_{dec} as 0.5 and 1D respectively. (Continued)

6. Numerical model of tunnel and pile

6.1 Problem definition

In this study we examine the pile settlement results with changing ground (rock) parameters. Piles already exists in the ground with applying load. The results are checked for changing elastic modules, cohesion, friction angle and tensile strength of rock.

Problem grid size is 84*56. The tunnel is two arc tunnel of 12m base length and 8m height having its crown 27m below the ground surface. The top 8m of grid is soil and the remaining is considered weathered rock. Six piles of 9m length and 0.3m radius are erected at increasing successive lateral distance of 0,6, 12, 18, 24 and 30 meter from tunnel center line. Each pile tip is at 18 meter distance from tunnel crown 1 meter deep in weathered rock. The model is fixed in x direction in walls and in y direction at bottom.

6.2 Constitutive Model used and input data

In this problem we use Mohr-Coulomb constitutive model. This model can give the required elastic plastic response of ground. The Mohr-Coulomb failure criteria is based on the following formula.

$$\tau = C + \sigma_n \tan\phi \qquad (6.1)$$

C is cohesion σ_n is normal stress on plain and ϕ is friction angle.

6.3 Model Simulation Chart

Initial Simulation of ground before pile installation

Pile installation with applied load and model simulation

Initialization of ground to zero displacement and zero velocity

Full face tunnel excavation and simulation of model

Settlement results for each pile

Fig 1 Model Simulation Chart

6.4 working Load Condition

When the pile is embedded in rock the load is not completely taken by foundation base but a part of it is taken as skin friction along the length of pile embedded in rock. In contrast to it the foundation passing through soil medium a great portion is taken by the base of foundation and very less or no by the length of pile passing through soil medium. The bearing capacity of a pile is not only determined by uniaxial compressive strength but by the factor such as the ratio of pile diameter to the computed depth in rock, the ratio of elastic modules of rock to elastic modules of concrete pile and condition between the rock surface and pile (M.G. Zertsalov and D.S. Konyukhov 2007).

In this work we examine only the end bearing piles and ignore the portion of load taken as friction resistance along the length of the pile embedded in the rock portion.

According to M.G. Zertsalov and D. S. Konyukhov 2007 the bearing capacity of piles supported on rock is determined from the formula.

$$F(MN) = \gamma_p RA \qquad (6.2)$$

γ_p is the working factor and can be assigned as γ_p=1

A is the bearing area of pile which is cross sectional area in square meter.

R is the end resistance of the rock to bearing pile.

For piles enclosed in undisturbed rock for no less than 0.5 meter.

$$R = \frac{R_{cn}}{\gamma_g}\left(\frac{l_d}{d_f} + 1.5\right) \qquad (6.3)$$

R_{cn} is the uniaxial compressive strength of rock.

γ_g is the reliability index of soil and in this case its value is 1.

l_d is the computed depth of pile in rock portion.

d_f is the outside diameter of pile.

6.5 Load applied on each pile in problem for elastic modules change

The load applied on each pile in the problem for elastic change is according to the above prescribed formula 6.2 and the compressive strength calculated has taken from changing elastic modules according to Hook formula

$$\sigma = E\epsilon \qquad (6.4)$$

Load on each pile has taken according to the following changing elastic modules and compressive strength Table 1.

l_d is equal to 1 meter and each pile diameter is 0.6 meter.

Compressive strength of rock were calculated based on equation 6.4 with ϵ=0.003 and then load were taken for different values of elastic constant based on equation 6.2 and the values given in Table 1.

Table 1 Load calculation based on compressive strength for Elastic values

Elastic Modules E	Compressive Strength R_{cn}	$R = \frac{R_{cn}}{\gamma_g}\left(\frac{l_d}{d_f} + 1.5\right)$	$F(MN) = \gamma_p RA$	Load (MN) applied with a FOS=3
1E7	3E5	9.5E5	2.6E5	8.6E4
2E7	6E5	1.8E6	5E5	1.6E5
3E7	9E5	2.8E6	7.8E5	2.6E5
4E7	1E6	3.1E6	8.6E5	2.8E5
5E7	1.5E6	4.6E6	1.3E6	4E5
1E8	3E6	9E6	2.5E6	8E5
5E8	1.5E7	4.6E7	1.3E7	4E6
1E9	3E7	9E7	2.5E7	8E6

6.5.1 Material input data for rock elastic/cohesion/tensile and friction change

Table 2 Ground properties used in Numerical Simulation

Material	Density Kg/m^3	Elasticity E(Pa)	Poisson Ratio v	Cohesion C(Pa)	Tension T(Pa)	Friction Angle ϕ
Soil	2000	1E05	0.25	1E03	0	25
Rock	2500	Changing for elasticity	0.25	1E06	1E03	35

Table 3 Pile mechanical and Geometrical properties used in Numerical Simulation

Elasticity E (Pa)	Normal Stiffness	Normal Tension	Normal Friction Angle	Shear Stiffness	Shear Cohesion	Shear Friction Angle	Radius of Pile
1.2E10	1.2E09	1E02	0	1E10	1E09	30	0.3

6.5.2 Results obtained from Numerical Analysis

According to M. G. Zertsalov and D. S. Konyukhov 2007 the bearing capacity and settlement of piles depends on rock mass strength around and below the lower end of piles.

With Changing the elastic constant of rock the settlement results showed decreasing tendency with increasing distance from center line of crown but still the effect was significant. For cohesion change/tensile strength change these parameters were changed while the elastic value was chosen 1E8 and 5E7 for cohesion and tensile/friction value respectively while other properties same as given in table 2 and the load for each case were calculated according to Elastic change. The results for each case are given in Fig 2, 3, 4 and 5.

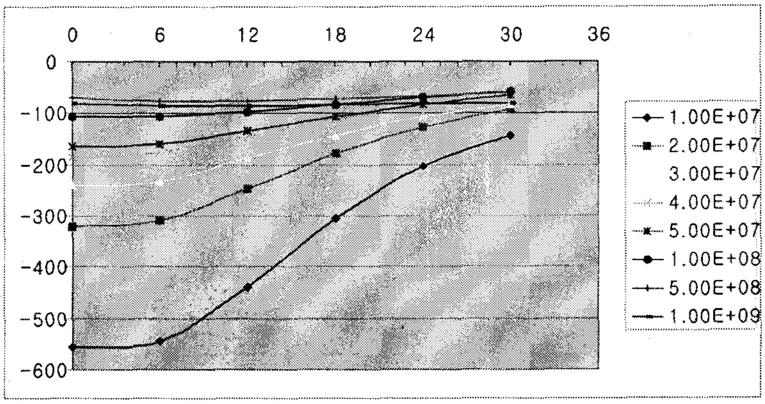

Fig. 3 Pile lateral distance V/S settlement results for rock cohesion change

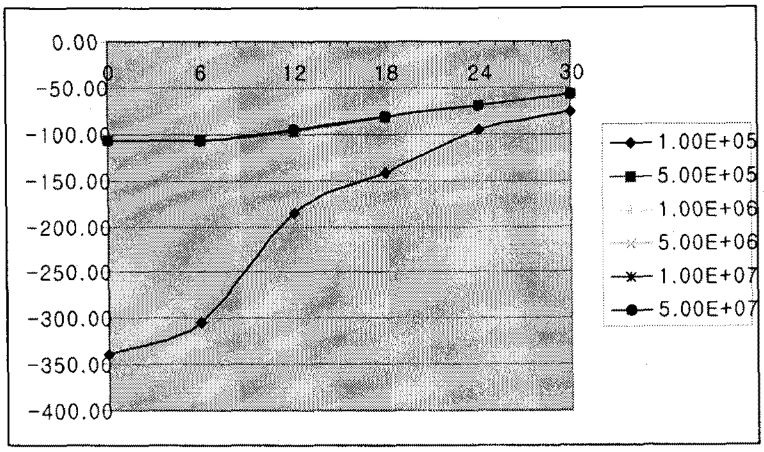

Fig. 4 Pile lateral distance V/S settlement values for tensile strength change

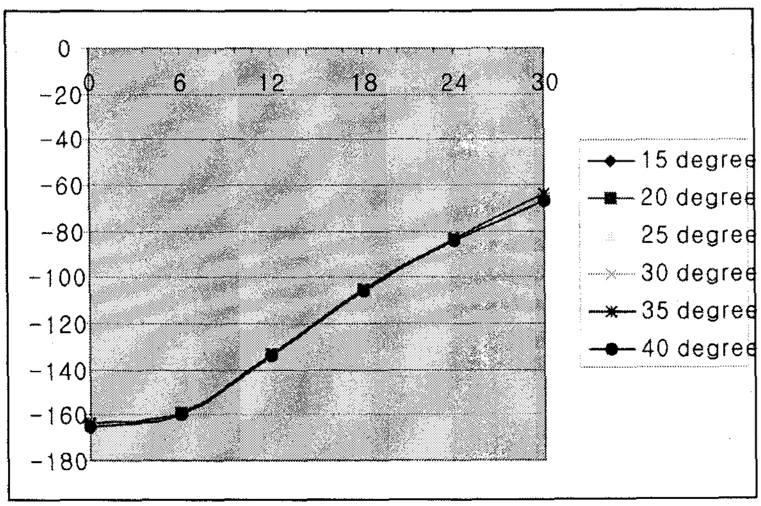

Fig. 5 Pile lateral distance V/S settlement values for friction angle change

7. Conclusions

The settlement results for end bearing piles were studied by changing the parameters of bed rock. The conclusions from these results can be summarized as follow.

i) The elastic constant of bed rock which is in direct relation to compressive strength decides about the settlement results.

ii) For end bearing piles up to a certain value the settlement not depends on cohesion but for value lower than that specific cohesion the settlement changes into progressive failure as the rock in the vicinity fails in cohesion.

iii) Tensile strength and friction angle change while other parameters of rock remain unchanged showed no change in settlement results for all six piles.

iv) For a fix vertical distance from tunnel crown as the lateral distance increased the settlement also decreased. It can be concluded that for a fix vertical distance there is a horizontal distance from tunnel for which the pile is not affected by tunnel excavation.

The numerical results obtained in this study are based on full face excavation with no support applied. Also the problem has solved in 2-dimension plain-strain condition. Actual tunnels are driven these days on NATM principle in sequential excavation. The tunnel settlement produced is basically 3-D problem. These shortcomings are expected for the present problem to overestimate the results. However the conclusions drawn can be remained fixed for real problems.

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