Approach to Numerical Simulation of Shield Tunneling and Its Evaluation by Comparison with Field Data

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ABSTRACT: Currently mechanized Earth Pressure Balance (EPB) shield tunneling method are widely used in excavating and advancing tunnels through any type of ground condition particularly below the ground water table. EPB tunneling can be simulated using finite element method by two or three dimensional model. Prediction of ground response due to tunneling, in this case EPB shield tunneling by numerical investigation could be valuable. In this work, an approach to numerical simulation of EPB tunneling which is using practical shield tunnel construction aspects such as face pressure, tail void grouting pressure and existing earth pressure was proposed. Furthermore, availability of a current case record of EPB tunneling made it possible to evaluate and verify the proposed approach by comparing its results with field measurement.

Keywords: Tunneling simulation, monitoring location, soil subsidence, twin tunnels

1 INTRODUCTION

Influences on the neighboring existing structures by changing stress-strain state of the soil due to tunneling are currently major concerns in urban areas within megacity. Currently mechanized Earth Pressure Balanced (EPB) shield tunneling method is widely used in excavating and advancing tunnels through any type of ground condition particularly below the ground water table.

EPB tunneling process can be simulated in a finite element analysis (FEA) using two or three-dimensional models. Field observations indicate that the ground response during tunneling is both three-dimensional and time-dependent (Finno .et al 1985).

In order to minimize the ground settlement, TBM face pressure is adjusted by controlling the advance speed or screw conveyor speed, which depends mainly on the experiences of operator (Koyama, 2003).

This paper presents a new EPB tunneling simulation approach using FEA. Furthermore, availability of a current case record of EPB tunneling in Japan made it possible to evaluate and verify the proposed approach by comparing its results with field measurement record. The field case record is obtained from a still ongoing tunnel construction site, named Yokohama Circular Northern motorway Route of the Metropolitan Expressway, Japan.

In the current paper, a proposed approach is described considering shield tunneling construction aspects such as face pressure, tail void grouting as well as existing earth pressure; then, an ongoing EPB shield tunneling construction site is introduced. Finally, comparing the field data of vertical displacement at studied site with output results of proposed FEA approach, the proposed method for predicting 3D ground displacement will be able to be evaluated and verified.

2 PROPOSED APPROACH OF FINITE ELEMENT ANALYSES

2.1 General

In order to simulate excavation process of shield tunneling based on new approach, a finite element program originally developed by Komiya et al (Komiya et al, 1999) was exploited. In this section, a new approach is described in order to simulate the EPB tunnel excavation process, taking into account face pressure, tail void grouting pressure and existing earth pressure.

2.2 Face pressure, grouting pressure, and initial earth pressure

In the case of shield tunneling, face pressure application and tail void grouting are commonly used to minimize ground surface subsidence during tunneling. Since the proposed approach is dealing with an EPB tunneling, it was attempted to consider face pressure, tail void grouting pressure and existing earth pressure in the presented method.

Methods for calculating earth pressure in the case of shield tunneling have been studied in earlier works. (Wang 2012; Koyama 2003). Here, existing earth pressure in a 3D element is assumed that has three normal components, one vertical and two horizontal. Vertical earth pressure is calculated based on soil overburden pressure. The horizontal earth pressure is assumed to increase with the depth and be obtained by multiplying the coefficient of horizontal earth pressure on each element.

Adjusting shield TBM advancement or screw conveyor speed can balance the face pressure leading to lower ground subsidence. In this case, face pressure is modeled to be distributed linearly and is assumed to increase by depth in front of the cutter face of TBM.

Tunneling through ground causes some void between lining and ground behind the TBM. This void is filled by grouting at the back of TBM immediately after excavation to counteract the settlement. Tail void grouting in this part is assumed to be active in a length of one ring exactly behind of TBM hydrostatically perpendicular to tunnel periphery.

2.4 Calculation process

Tunnel boring process was modeled using tunneling step by step loading. Using actual excavation time records, in each step the rings in which face pressure and tail void grouting are applying, were determined in each bound route. Knowing the value of face pressure and tail void grouting in each loading step, earth pressure was determined and difference between face pressure and earth pressure at the front of machine, and difference between grouting pressure and earth pressure at the back of machine were applied to soil elements. Applying these forces, nodes are deformed, and stress and strain are developed throughout the mesh. Developed stresses in elements at each step are used as an initial stress for next loading step. General procedure is briefly described as below:

Applied stress in front of machine (difference between face pressure and earth pressure):

Step 1: $P_1 = (FP_1 - EP_1)$

$$EP_1 = 1/3(\sigma_v + 2\sigma_h)$$

Step 2: $P_2 = FP_2 - (EP_2 + St_1)$

$$EP_2 = 1/3(\sigma_v + 2\sigma_h)$$

Step n: $$P_n = FP_n - (EP_n + St_{n-1}) \quad (1)$$

$$EP_n = 1/3(\sigma_v + 2\sigma_h) \quad (2)$$

Applied stress at the back of machine (difference between grouting pressure and earth pressure):

Step 1: $G_1 = (GP_1 - EP_1)$

$$EP_1 = 1/3(\sigma_v + 2\sigma_h)$$

Step 2: $G_2 = GP_2 - (EP_2 + St_1)$

$$EP_2 = 1/3(\sigma_v + 2\sigma_h)$$

Step n: $$G_n = GP_n - (EP_n + St_{n-1}) \quad (3)$$

$$EP_n = 1/3(\sigma_v + 2\sigma_h) \quad (4)$$

in which P_n and G_n are applied stress at loading step n to the elements in front and back of shield machine; FP_n is the face pressure at loading step n in elements in front of machine which is obtained from field data; GP_n is the grouting pressure in loading step n in elements around tunnel periphery at the back of machine which is obtained from field data; EP_n is the earth pressure at loading step n; and σ_v and σ_h are vertical and horizontal stresses of soil.

Existing earth pressure is calculated based on total and effective stress depending on soil type as well as water table level. According to Table 1, all of soil types except Ks (Sand and sandstone) are clayey impervious soils. In case of Ks, in which drainage can be done, earth pressure was calculated based on effective stresses. In other cases, total earth pressures were used.

In each step, undeformed mesh is used for applying forces and obtaining stress, strain, and deformation; because of this, final deformation of any node at any step (any advancement of tunnels) is obtained by adding of all deformation of previous steps plus developed deformation at current one.

Since major parts of soil layers are hard clayey formation, effect of consolidation and dissipation of pore water pressure have not been considered.

Lining of tunnels were also considered by applying fixity in x, y, and z direction to the nodes of tunnel periphery at the back of machine.

3 CASE RECORD

3.1 General description of site

This site which locates in Yokohama-Japan is a motorway of total length about 8.2 km, in which 5.9 km of it is a twin side by side tunnel of diameter about 12.5 m excavated using Earth Pressure Balanced shield tunneling machine, and cut and cover tunneling. The tunnel construction has started in 2010 and connects the Kohoku Interchange of the Third-Keihin Road to Namamugi Junction of the Yokohama-Haneda Airport Line of Metropolitan Expressway in the northern section of the Yokohama Circular Route Road.

After of about 15 and 50 m from the launching shaft, there are two monitoring locations, in which vertical displacement of soil is measured at various depths to secure the safety of construction.

Overburden of soil along the tunnel path varies from 16 m to 56 m. For about few hundred meters along tunnel axes from the launching shaft, boring direction in each of tunnels has about 2 degrees downward alignment. Lining in twin shield tunnels composed of RC rings with an outer diameter of 12.3 m and an inner diameter of 11.5 m, each having length of about 2 m. Clearance between two tunnels varies from 3 to 6.5 m.

Ground water table is assumed to be at 2 m below the ground surface with slight variation.

The shield machine advances through the ground mainly composed of mudstone (Km), sandy mudstone (Kms) and sand and sandstone (Ks), all of which have an N-value of 50 or higher. Soil layers and their material parameters have been shown in Table1.

Table 1. Soil layers description and their material parameters

Soil type	description	γ kN/m^3	c kN/m^2	φ	E Mpa	ν	K_0
B	Fill material	14.0	30	0	1.2	0.45	0.80[1]
Ac	Cohesive soil	15.5	35	3	3.3	0.45	0.80[1]
Ks	Sand and sandstone	19.5	60	42	289	0.3	0.33[2]
Kms	Sandy mudstone	19.0	1840	10	492	0.35	0.16[3]
Km	Mudstone	18.5	2020	7	430	0.35	0.16[3]

[1]Based on Standard Specifications for Tunneling, Shield tunnel, Japan Society of Civil Engineers, 2006.
[2]Based on Jacky's formula
[3]A value of experience obtained during operation of shield machine

3.2 Monitoring locations

Two monitoring locations were laid out 15 and 50 m consecutively from the launching shaft along the tunnel path. In each of monitoring locations, a several numbers of apparatus have been installed exactly above the tunnel axes to measure ground subsidence at various depths including ground surface. Table 2 shows the number of apparatus at each monitoring location (ML).

Table 2. Number of apparatus at each Monitoring Locations (ML)

Line	ML1	ML2
Westbound route	5	6
Eastbound route	6	7

In each of these locations, vertical ground displacement has been measured continuously.

3.3 Excavation scheduling record

Excavation process of twin tunnels do not advance with the same rate and one of them (westbound route) is proceeding about 30 m ahead of the other one. Stop time, void grouting and face pressure location in each bound route is different from the other one. In Figure 1, excavation time, ring (R) numbers, as well as monitoring location has been displayed in each bound route on plane map.

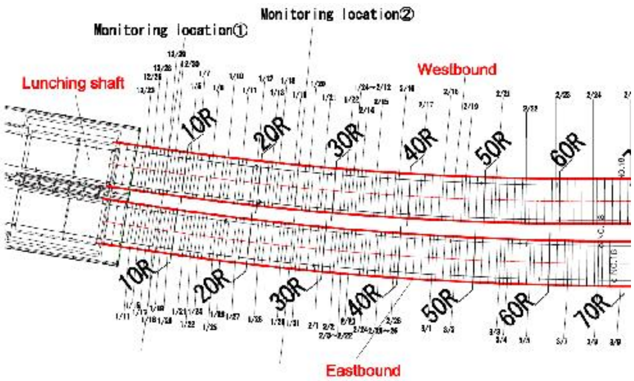


Figure 1. Excavation time, ring numbers, and monitoring location in each bound

As the length of shield machine is known, so, based on data from figure 1, the rings in which face pressure, and void grouting pressure are applied in each bound was determined to use in loading steps of excavating process simulation of finite element analyses.

3.4 Field data record

In this site, data of face pressure at the top and center of cutter face, void grouting pressure, and jacking forces at the back of machine were gathered in each ring along each of bounds. Furthermore, vertical ground displacement at monitoring location in various depths has been measured by time. For instance, figure 2 shows vertical ground displacement at apparatus of monitoring location 1 in westbound.

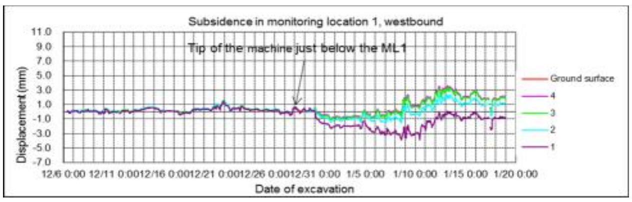


Figure 2. Subsidence in sensors located at monitoring location 1, westbound by date of boring

4 Comparisons of field data with numerical simulation results

4.1 Finite element mesh generation

In order to perform numerical investigation, a 3D mesh was generated based on longitudinal profile of soil resulted from borehole logs. In this mesh, diameter of each tunnel route is about 12.3 m. Length, width, and height of the whole mesh are about 120, 127.5, and 54 m, respectively, in which it's boundary are far enough from tunnel boundary and are assumed to not have any significant impact on each loading step result. All of the elements in mesh are 8 nodded cubic. The material constitutive model is assumed to be defined by elastic parameters of each soil layer and a hard mudstone is extended to the depth of 70 m. Figure 3 shows the overall mesh, tunnels, boundaries and its dimensions.

Two monitoring locations (ML1 and ML2) are located at ring numbers of 8, and 25 consecutively in both bounds.

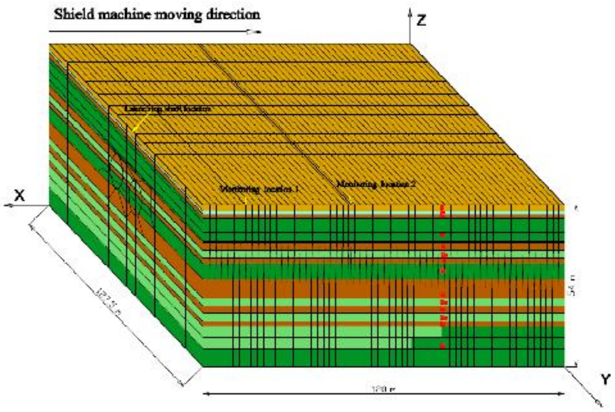


Figure 3. Overall mesh, tunnels, and monitoring locations

4.2 Comparison results

In order to evaluate and verify accuracy of the proposed approach, it was attempted to compare the calculation results of proposed approach with field data measurements. Vertical ground displacements in tunneling site were recorded continuously. As time effect of data such as stop period of shield machine and consolidation effect of soil have not been considered in analytical procedure, field data were set to be shown by distance from a base point like monitoring locations to enable compare with analytical results. So, in this section output results of finite element analyses were also shown based on the distance from the same monitoring locations.

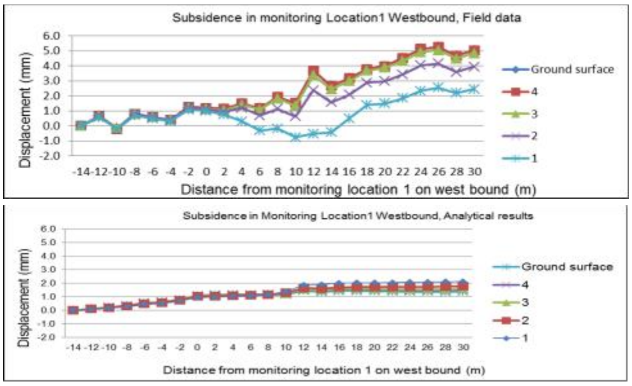
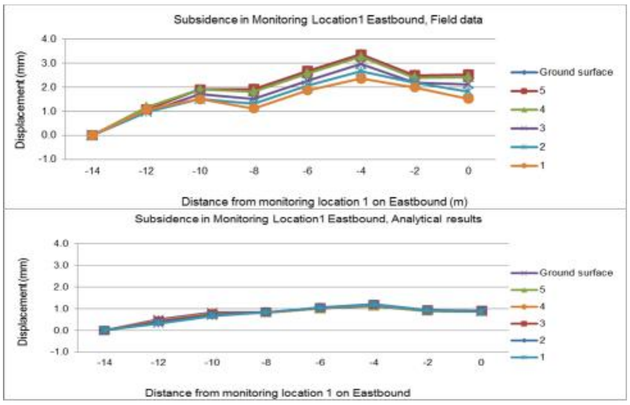


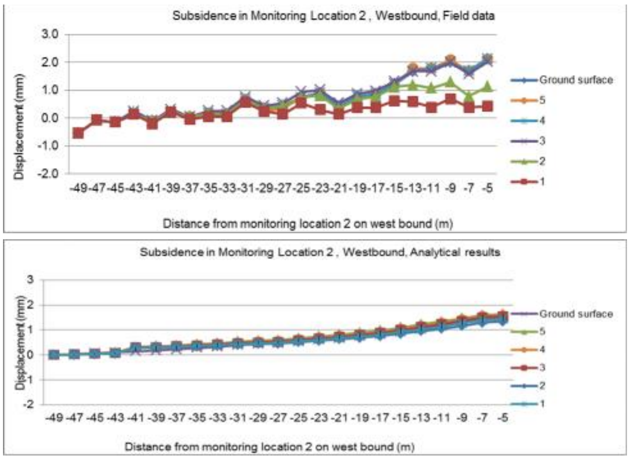
Figure 4. Vertical displacement in monitoring location 1, westbound from field data and analytical results

Vertical ground displacements of between field data and by analytical results were compared for 23 loading steps.

During these 23 loading steps, shield machines in tunnels have advanced for about 23 rings length in westbound route and 8 rings length in eastbound route. Figure 4 compares the vertical displacement in monitoring location 1 west bound between field data and analytical result.



Figures 5. Vertical displacement in monitoring location 1, eastbound of field data and analytical results

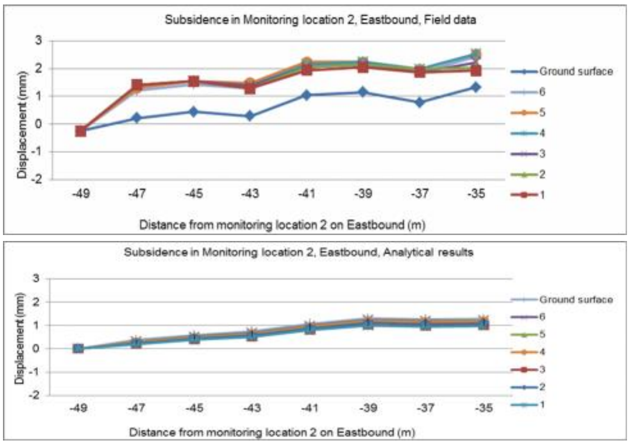


Figures 6. Vertical displacement in monitoring location 2, westbound of field data and analytical results

Figures 5, 6, and 7 present similar graphs of monitoring location 1 eastbound, as well as monitoring location 2 westbound and eastbound.

By comparing the results, it can be seen that proposed approach is able to predict the same trend of field data with good proximity. Furthermore, investigating both field data and analytical results, there seems to be some correlation between vertical displacement and difference of face pressure and lateral earth pressure. Similarly, ground heaves were observed and simulated numerically when tail void grouting is approaching the monitoring location.

But it also can be observed that value of vertical displacements from filed data is slightly greater than the analytical results. One possible reason is that values of face pressure at eastbound were lower than westbound in a way that in some cases earth pressure value was greater than face pressure leading to ground settlement in the entire of mesh; In fact, this difference could be about some soft ground layer in field with lower elastic modulus values than it was assumed in Table 1.



Figures 7. Vertical displacement in monitoring location 2, westbound of field data and analytical results

5 CONCLUSIONS

In this paper, an approach to simulate excavation process of EPB tunnels is proposed. Presented results of this paper are summarized as follows:

1) New presented approach with previously coded FEM program is well established to estimate the 3D deformation of soil due to tunneling.

2) This approach takes into account EPB tunneling aspects such as face pressure, tail void grouting pressure, as well as existing earth pressure and can be used to simulate excavation process of any shield tunnel.

3) Field data of newly tunneling site were gathered. Using these data and comparing the vertical displacement of this site with analytical result enable to evaluate the accuracy of proposed approach. Comparing the results show that there is a good conformity between field data and analytical results.

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