

Advancement simulation of parallel tunnels and their interchange with two other subway lines using a new FEM approach, a case study

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ABSTRACT: Advancement of a tunnel face results in the disturbance and redistribution of the primary in situ stress field. Soil deformation, pore water pressure generation, and induced stress and strain are among engineer's interest. Depending on soil type, rate of TBM machine advancement, and also tunnelling method, soil may behave between the two extreme cases of drained or undrained condition. Understanding soil drainage condition often is not an easy task, however using numerical simulation programs, generated pore water pressure can be estimated and as a result effective and total stress can be obtained. In this paper, a simple procedure has been employed for drainage determination to be used in pre-coded Finite Element Analysis (FEA) program. The method is described and then an Earth Pressure Balance (EPB) tunnelling case study is analysed using proposed procedure. In this case study, two parallel tunnel lines which are being excavated by EPB method interchange with another two subway lines almost perpendicularly. Induced ground movement due to these underground constructions are measured in several locations and then a comparison is made with finite element analysis result.

1. INTRODUCTION

In congested and populated areas like Tokyo metropolitan, it is almost impossible to avoid performing underground construction close to various structures. Currently, underground construction is performed at larger area and deeper depth than before (Akagi, 2004). Starting of new underground activity, in these cases, could be challenging and many considerations should be taken into account. Limited knowledge of actual ground condition may lead to some assumptions and simplification to enable designer to do the predictions. Numerical simulations are mighty tool for prediction and investigation of soil behaviour due to tunnelling. Accessibility to monitoring field data made it possible to improve numerical prediction to match actual ground behaviour during underground construction. Advancement of a tunnel face results in the disturbance and redistribution of the primary in situ stress field

(Eberhardt, 2001). Soil deformation, pore water pressure generation, and induced stress and strain are among engineer's interest. Depending on soil type, rate of TBM machine advancement, and also tunnelling method, soil may behave between the two extreme cases of drained or undrained condition. Understanding soil drainage condition often is not an easy task, however using FEA programs, generated pore water pressure can be estimated and as a result effective and total stress can be obtained.

In this paper, a simple procedure has been employed for drainage determination to be used in pre-coded FEA program. In the first part, the method is described. In the second part an EPB tunnelling case study is analyzed using proposed procedure. In this case study, two parallel tunnel lines which are being excavated by EPB method interchange with another two subway lines almost perpendicularly. Induced ground movements due to

these underground constructions are measured in several locations and then a comparison is made with FEM output.

2. NUMERICAL APPROACH

2.1 Outlines of Used numerical simulation

Numerical 3D finite element analysis program used here is a Fortran coded FEM program which was developed by Komiya et al. (Komiya, 1999). Tunnel advancement simulation is carried out by dividing of whole excavation process into a number of stages. Tunnel face pressure in front of TBM machine changes linearly from top to the bottom of shield machine face. Tail void grouting is also applied perpendicular to tunnel perimeter through the length of one ring at the back of shield machine. Difference load between face pressure and initial earth pressure and also between grouting pressure and initial earth pressure are applied respectively to elements in front and at the back of shield machine.

2.2 Superposing method

In each step, same mesh is used for input loads to be applied. Applying these forces, nodes of elements are displaced, and stress and strain are developed throughout the mesh. Developed stresses in elements at each step are used as an initial stress for the next loading step.

Final displacement of any node at any step (any advancement step of tunnels) is obtained by adding

of all deformation of previous steps plus developed deformation of current step.

Same procedure is taken for stress and strain calculation.

2.3 Drained and Undrained criteria

Depending on soil type, rate of TBM machine advancement, and also tunneling method, soil may behave between the two extreme cases of drained or undrained condition. For low-permeability and high rate of tunnel advancement, undrained behavior is dominant. On the other hand, for high permeability soil, and slow rate of advancement, drained condition is expected. For other cases, soil condition varies between these two extreme cases. If the soil condition and TBM machine advancement rate do not vary too much, distance of tunnel face which is considered drainage boundary from monitoring location also play important role in drainage determination (Anagnostou, 1996). In other words, the areas too close and too far from drainage boundaries show drained and undrained behaviour respectively. Latter factor which is called “distance factor” was used for drainage determination in FEM calculation. It should be noted that during drained condition, earth pressure is calculated using effective stress of soil, while during undrained condition total stress of soil is used for earth pressure calculation.

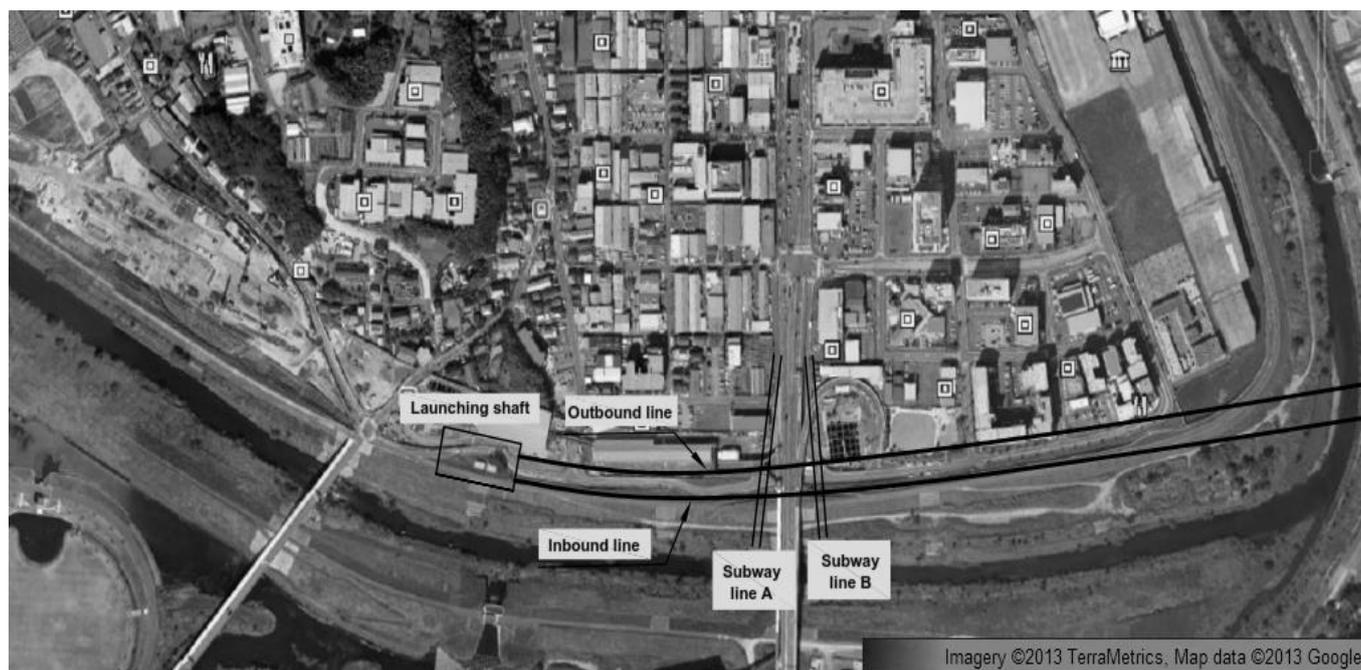


Figure 1. Yokohama site; launching shaft, Inbound and Outbound lines, Subway line A and B have been shown

3. CASE STUDY

3.1 Site description

A motorway of total length about 8.2 km is under construction in Yokohama, Japan by Tokyo Metropolitan Expressway Company to link between the Kohoku Interchange of the Third-Keihin Road and the Namamugi Junction of the Yokohama-Haneda Airport Line. About 5.9 km of this so called “Yokohama Circular Northern Route” is a side by side parallel tunnel, (Outbound and Inbound lines) each of them having diameter of about 12.5 m. Main part of this route is excavated using Earth Pressure Balanced (EPB) Shield tunneling method. Each of the lines is excavated separately and one of the lines is about 30 meter ahead of the other one. In order to check the safety of underground construction, three monitoring locations (ML1, ML2, ML3), were located along the two tunnel lines after of about 15, 50, and 180 meter respectively from launching shaft. Parallel tunnels in their path also interchange with another two subway lines (Subway line A, and Subway line B) almost perpendicularly. In addition to mentioned measurement locations (ML1, ML2, ML3), some measurement devices were also located on subway lines to measure their deformation during excavation of parallel tunnels. Subway lines were under operation during construction of new parallel tunnels, so it was uppermost important to minimize the ground deformation during new tunnels installation. Measurement points in monitoring locations and also in subway lines measure soil deformation before, during, and after of passing parallel tunnels. Fig. 1 shows an aerial image of Yokohama area in Japan, in which parallel tunnels (Outbound and Inbound lines), subway lines (Subway line A and subway line B) and also launching shaft have been highlighted. Newly parallel tunnels have a few degrees of downward inclination. These lines (Outbound and Inbound lines) pass below the subway lines (line A and Line B) by vertical clearance varying between 3.5 to 6.5 meters. The parallel shield tunnels’ lining composed of concrete rings with an outer diameter of 12.3 m and an inner diameter of 11.5 m, each having length of 2 m. Groundwater level was nearly close to surface, typically between 2.2 to 2.8 below ground surface.

3.2 Filed measurement data

Numbers of measurement devices in ML3 and also along subway lines have been shown in Table 1.

Table 1. Number of measurement devices

Name	Number
ML3	6 on Outbound line and 6 on Inbound line according to Fig. 2.
Subway line A	19 along the subway springline, according to Fig. 4 (AV1 to AV19)
Subway line B	20 along the subway springline, according to Fig 4 (BV1 to BV20)

As previously mentioned, in order to check the safety of tunneling construction, three monitoring location (ML1, ML2, and ML3) and also some measurement point on subway lines were installed. Vertical and horizontal displacements of soil were measured in these locations. As in this paper only part of the route dealing with ML3, and interchange by subway lines were taken into account, therefore, only field data regarding these location have been presented. Fig. 2 shows the cross section of Outbound and Inbound lines in monitoring location 3 (ML3).

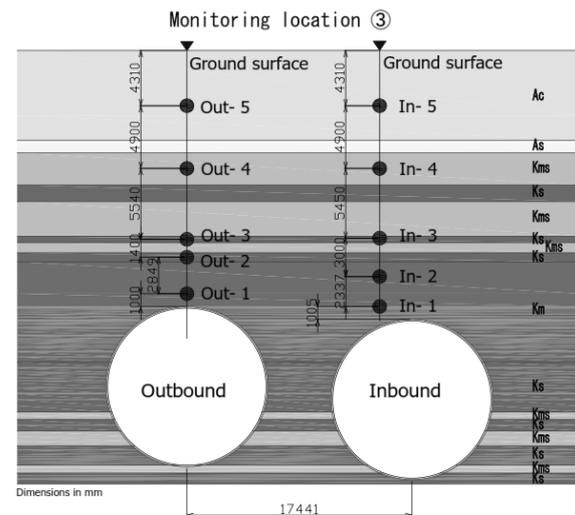


Figure 2. Cross section of newly constructed parallel tunnels in the ML3; Outbound and Inbound lines as well as measurement devices location are shown.

3.3 Numerical model and material property

In Yokohama site, part of the route including ML3 and interchange of parallel tunnels with subway lines were taken into account and then a mesh was created for this part to be used for FEM analysis. Width, length, and height of created 3D mesh are 128 m, 136 m, and 55 m respectively. Fig. 3 shows the 3D mesh. Furthermore, orientation of parallel tunnels and their interchange with subway lines of A and B have been illustrated in Fig. 4. The shield tunnels advance through the ground mainly composed of mudstone (Km), sandy mudstone (Kms) and sand and sandstone (Ks), all of which

have an N_{spt} value of 50 or higher. The unconfined compressive strength is 1000 kN/m^2 or higher for Km and Kms. The ground is hard. Orientation of parallel tunnels interchanging with subway lines have been illustrated in Fig 2. Soil layers and related soil properties of these layers are summarized in Table 2. In Table 2, γ is unit weight of soil, c is cohesion, ϕ is internal friction angel, E is elastic modulus, ν is poisson ratio, K_0 coefficient of earth pressure at rest, and OCR os overconsolidation ratio. As it can be seen, the major part of the tunnel instalation is carried out in hard soil (Km, Kms, and Km). Reference to Table 2, because of high value of soil Elastic modulus, and OCR, and due to the large and complex mesh, linear elstic was selcted as a constitutive relationship between stress and strain for all soil type. By comparison of result of FEA with measurement data, later it was found that in this kind of hard soil, this assumption is not far from reality. Because induced strains in Earth Pressure Balance method in which input forces are nearly equall to initial soil condition, are quiet small. In this model, whole of numerical simulation includes 115 loading step. Used values of face pressure, and tail void grouting pressure in numerical simulation were exact the same as field values.

3.3.1 Monitoring location 3

After passing about 180 meter (90 rings) from launching shaft along the parallel tunnel route, a

Table 2. Soil layer's properties

Depth (m)	Symbol	Soil type	γ (kN/m ³)	c (kN/m ²)	ϕ (degree)	E (MPa)	ν	K_0	OCR
0-1.5	B	Fill material	14.0	30	0	1.2	0.45	0.8^1	-
1.5-10	Ac	Cohesive soil	15.5	35	3	3.3	0.45	0.8^1	2.9
10-12.3	As	Sand	18.0	20	33	6.0	0.40	0.46	13.3
12.3-14.5	Ks	Sand and sandstone	19.5	60	42	289	0.30	0.33	>10
14.5-17.5	Kms	Sandy mudstone	18.0	1840	10	492	0.35	0.16^2	>10
17.5-19	Ks	Sand and sandstone	same as Ks						
19-24	Km	Mudstone	18.5	2020	7	430	0.35	0.16^2	>10
24-27	Ks	Sand and sandstone	same as Ks						
27-28.5	Kms	Sandy mudstone	same as Kms						
28.5-30	Ks	Sand and sandstone	same as Ks						
30-31	Kms	Sandy mudstone	same as Kms						
31-33.5	Ks	Sand and sandstone	same as Ks						
33.5-55	Km	Mudstone	same as Km						

¹ Based on 'STANDARD SPECIFICATIONS FOR SHIELD TUNNELING', Japan Society of Civil Engineers

² A value of experience.

monitoring section namely ML3 was placed to measure ground horizontal and vertical deformation.

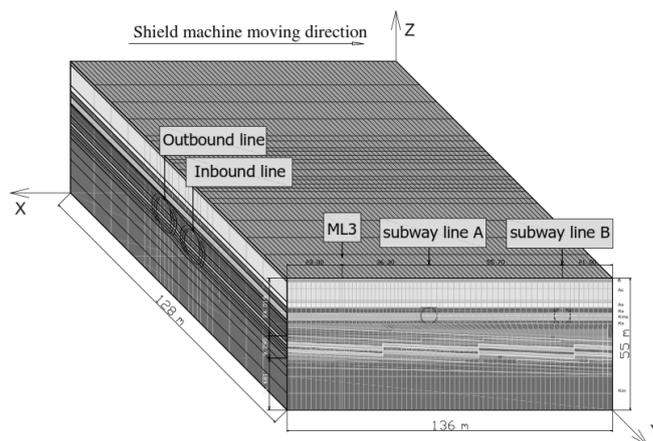


Figure 3. Finite Element Model (Dimension in m)

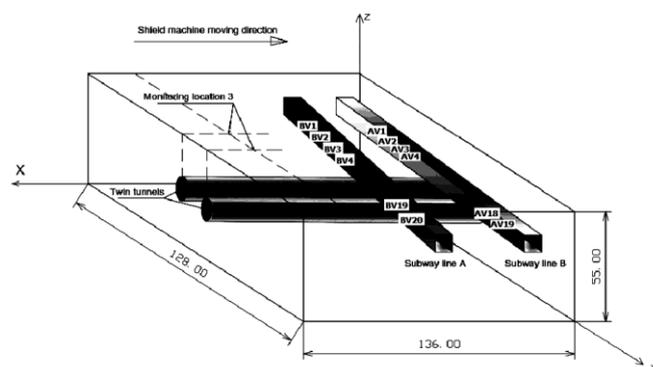


Figure 4. Orientation of new tunnels and existeing subway lines

Measured field data of vertical displacement at this location on Outbound and Inbound line are shown in Fig. 5.

3.3.2 Subway lines of A and B

Along the parallel tunnels, after about 228, and 270 meter respectively (114 and 135 lining rings) from launching shaft, subway line A and B are under operation. Depth of existing subway lines are about 15.5 meter below the ground surface. According to Fig. 4, subway line B is almost parallel to Y axes and perpendicular to newly excavating tunnels, but subway line A is somehow diverging from Y axes. On the springline of each of subway line, some measurement devices have been installed (Refer to Table 1, and Fig. 4) for horizontal and vertical movement control. Vertical displacement recorded at field has been shown in Fig. 6 for both Subway line A, and B. Displacement of some points along subway lines far away from tunneling route has not been shown (only AV5 to AV16, and BV6 to BV17 were illustrated in Fig. 6). Reference to Fig. 6, it can be understood that displacement of subway line were less than 1 mm. Because the subway lines were under operation, their movements were strictly controlled probably using ground improvement agents like grouting in soil

enclosed between parallel tunnel, and subway lines. Referring to Fig. 7 which shows the vertical displacement along subway lines, it can be also seen that at the points where parallel tunnels are passing (In Fig. 7, this locations have been shown by arrow), vertical displacement are nearly equal to zero. According to field data, parallel tunnel excavation rate (TBM machine advancement rate) on both lines averagely was between 5-10 m/day, while this rate increased to 14 m/day during passing under the subway lines. Fast passing of TBM machine also dictates undrained condition which leads to small amount of ground deformation.

4. RESULTS

After creating of the mesh, excavation steps for parallel tunnels were performed using FEM program. Whole of simulation carried out in 115 loading steps in which each of the tunnel lines moved 68 rings length. After performing of simulation, vertical displacement of points in the same location of field measurement points in ML3, and also along subway lines were plotted. In each of loading steps same mesh was used with the aforementioned mentioned method as section 2.2.

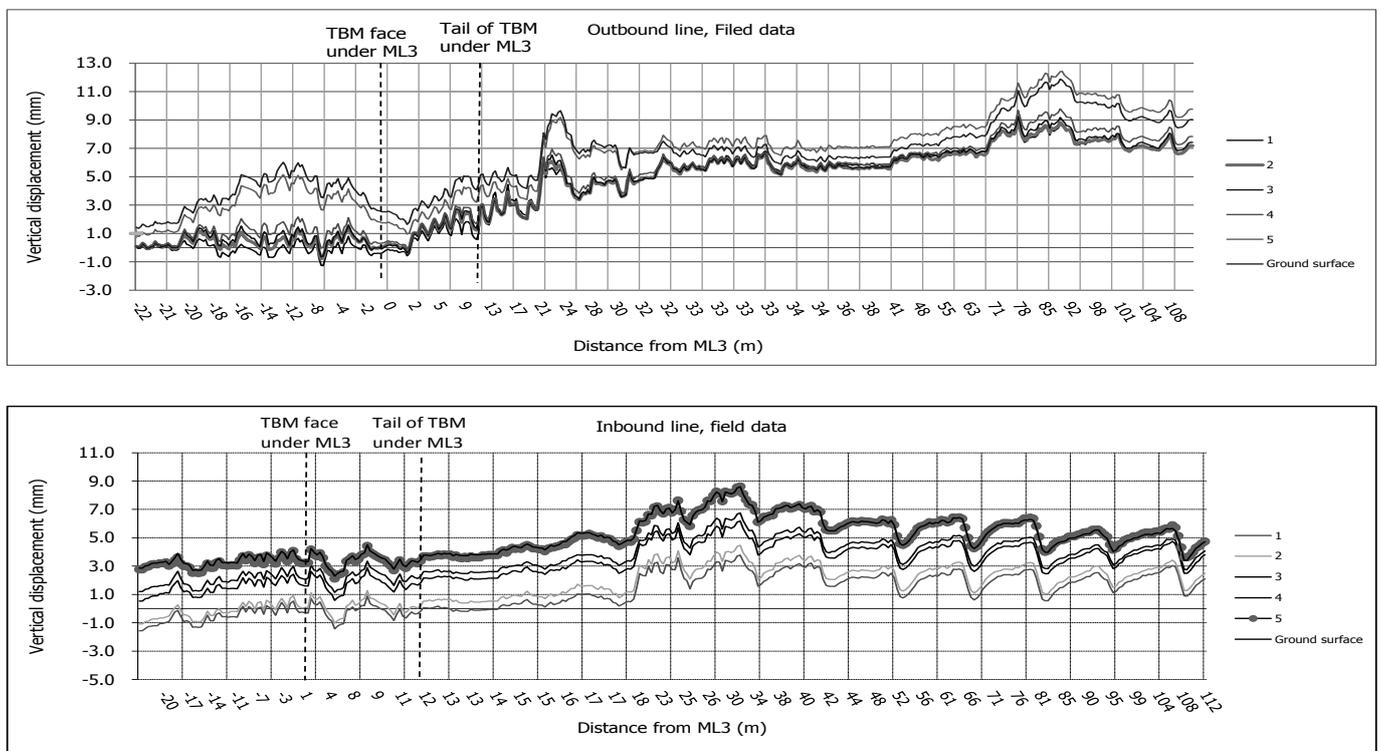


Figure 5. Recorded data in field; vertical displacement of soil at ML3, Inbound and Outbound line (Refer to Fig. 2 for measurement points' location)

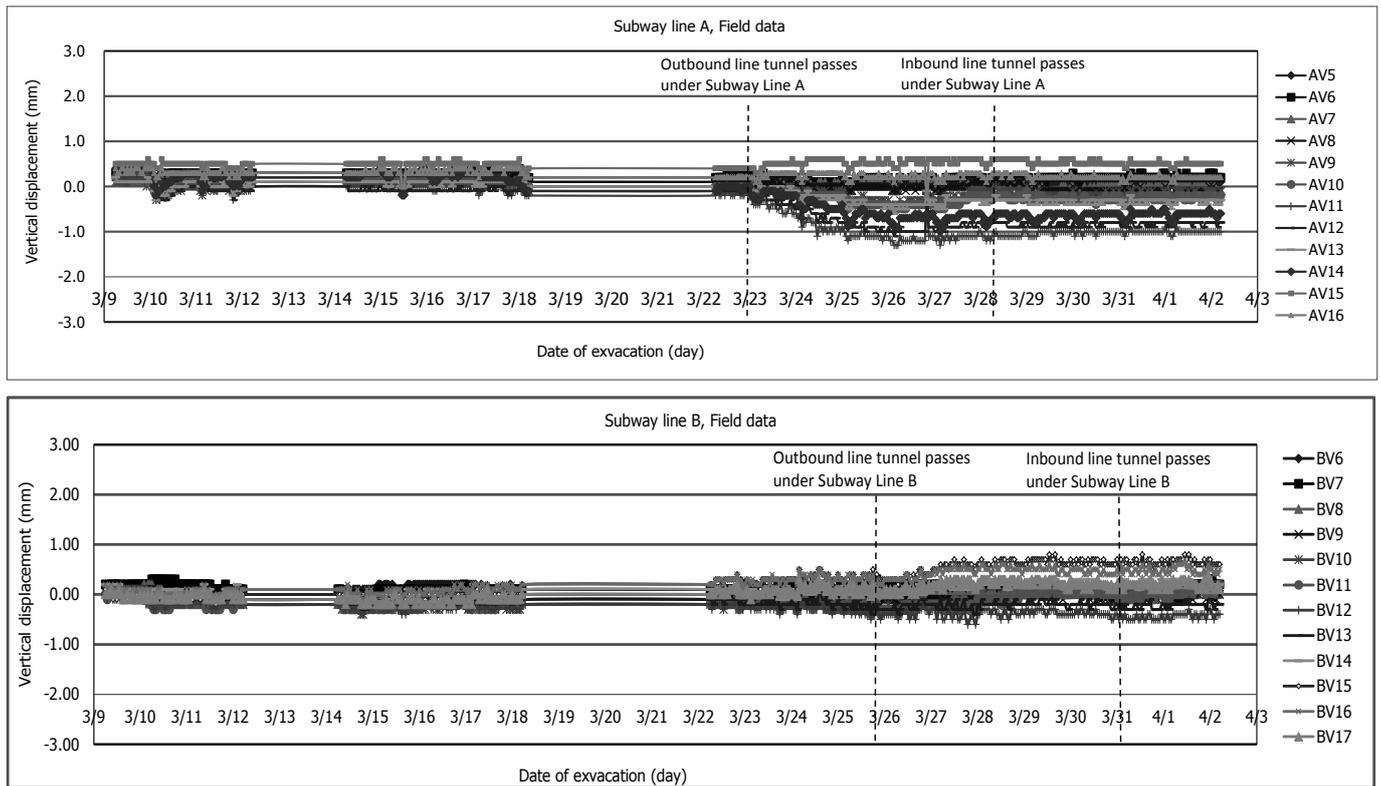


Figure 6. Recorded data in field; vertical displacement of measurement points on subway line A, and B.

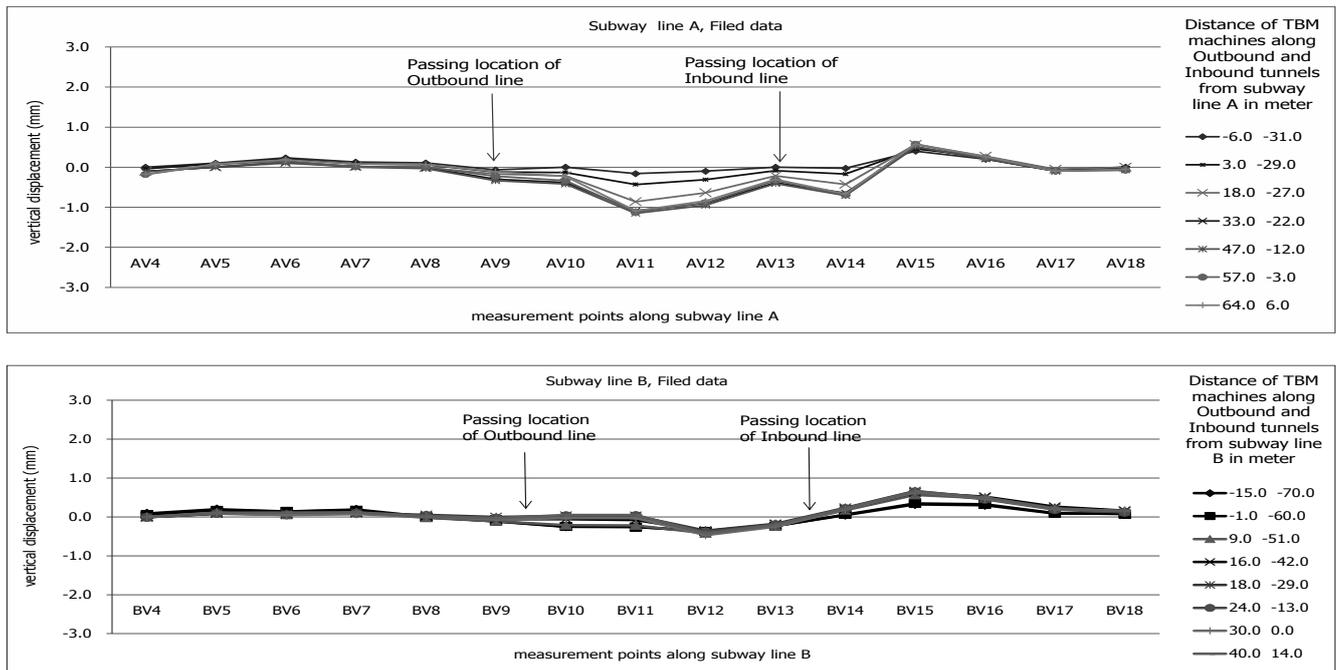


Figure 7. Recorded vertical displacement along subway line A, and B.

Comparison results of measured data and calculated output have been shown in Figs. 8, 9, and 10. Fig. 8 illustrates vertical displacement of the points in measured data and calculated output at ML3. Similarly, Figs. 9, and 10 respectively show measured data and calculated output of measurement points along subway line A, and B. In Fig. 8, calculated results show few millimeter larger values than measured data. Points close to tunnel in depth have larger deformation.

Majority of this deformation is about tail void grouting, when TBM machine tail passing the monitoring section. Part of the difference between measured and calculated data arise from the point that overburden due to surface irregularity in field is changing, but in FEM calculation, ground surface is considered to be uniform and flat and overburden value does not change as it does in field.

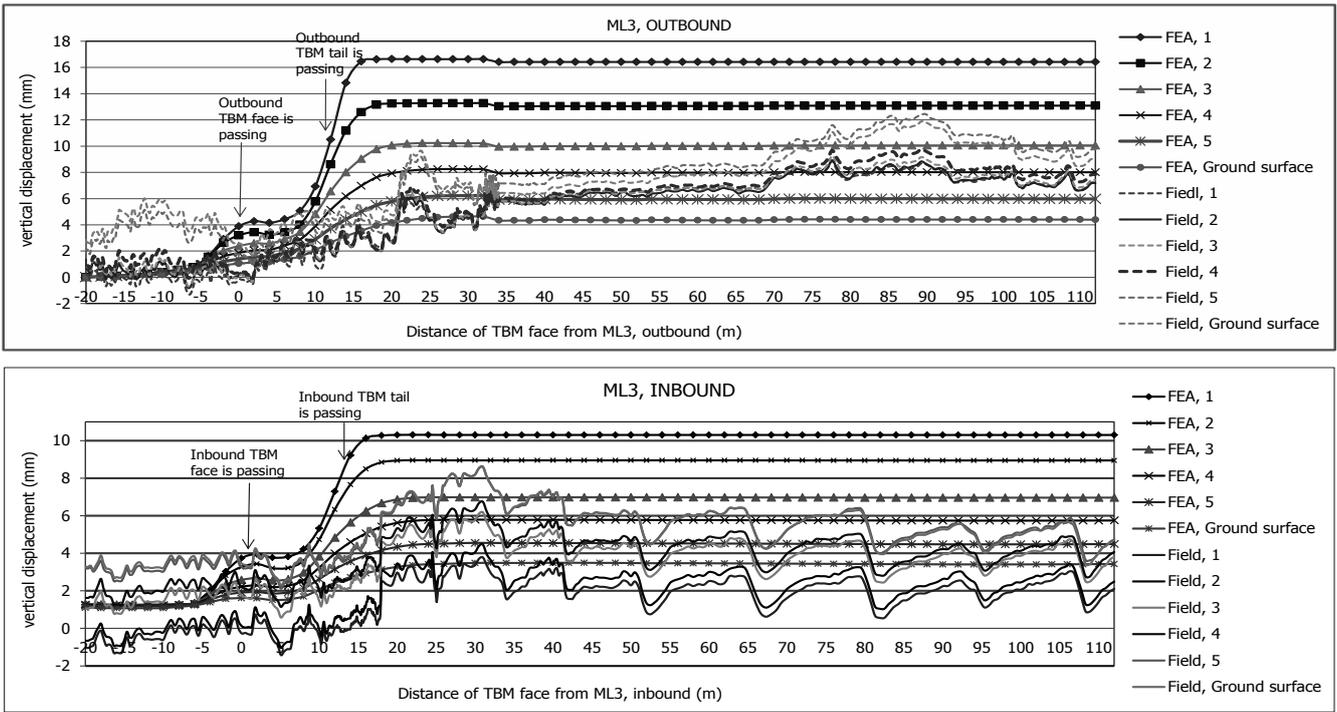


Figure 8. Comparison result of field data with FEA output in ML3 (refer to Fig. 2 for points' location).

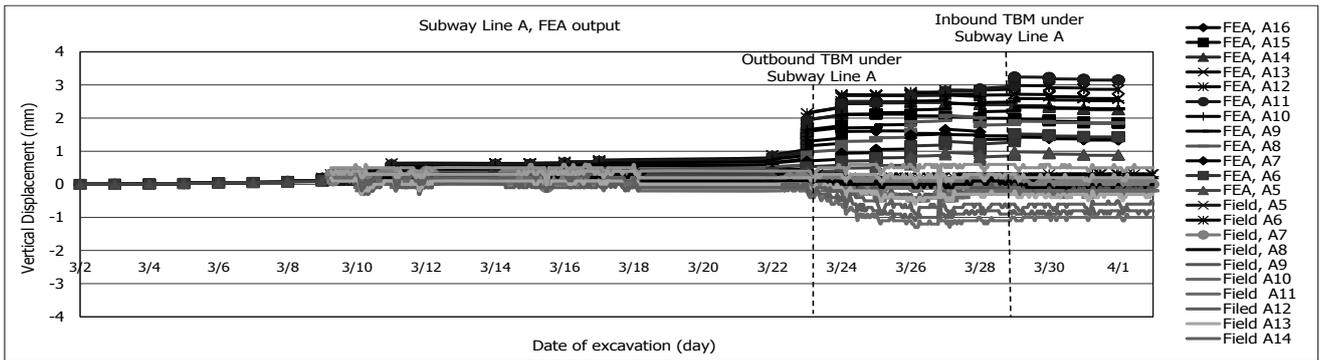


Figure 9. Comparison result of field data with FEA output along subway line A.

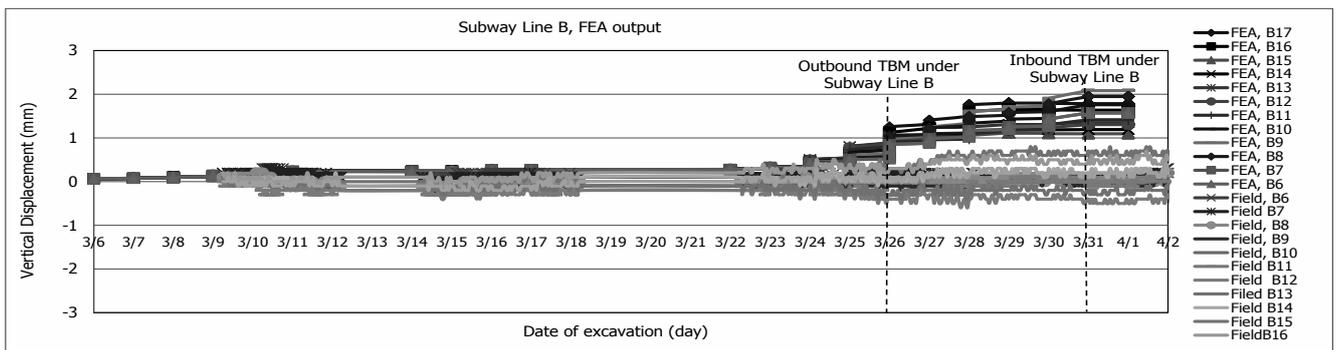


Figure 10. Comparison result of field data with FEA output along subway line B.

These changes in overburden depth affect earth pressure value which leads to not uniform displacement during tunnel advancement. According to Figs. 9 and 10, it can be seen that field measured vertical displacement is less than 1 mm. Referring to the same figures, calculated value have value less than 3 mm in line A and less than 2 mm in Line B.

It should be noted that due to fact that subway lines were under operation during excavation of parallel tunnels, for sure some sort of ground improvement like large extent grouting has been used to lower the ground deformation around subway line to less than allowable value in which in this case is 1 mm. In FEM calculation effect of such soil improvement has not been taken into

account. However, trend and value of vertical displacement between measured and calculated data were in good harmony.

5. CONCLUSIONS

This paper dealt with EPB tunnelling advancement simulation. In the first part of this paper, a used finite element method is described in brevity. In second part, an EPB tunnel case was studied. In the presented case study, two parallel tunnels were passing under previously constructed subway lines. Because most of the lands above the tunnel routes were private property, and also subway lines were under operation, some monitoring points in both lines (12 measurement points in so called ML3) as well as along each of subway lines (19, and 20 measurement points in subway A, and B respectively) were considered to check the safety of underground construction by measuring ground displacement.

Using previously coded finite element analysis program, tunnelling construction of above mentioned case study was simulated. In this way, pertinent mesh was created, and displacements of same measurement points were obtained. Investigating measured data, and calculated results, and also making a comparison between them, following results is highlighted:

- A finite element analysis used for tunnelling simulation used a so called “superposing method” to calculated ground displacement, stress and strains. Output results of FEA displayed that this method is reliable way to calculate ground displacement. Using this method for simulation of mentioned case study showed good harmony between calculated results and measured data.
- Vertical displacement along subway lines in measured data were less than 1 mm, while this value in calculated results were less than 3mm for line A, and less than 2mm for line B. The difference arises from the point that exerted extensive ground improvement in field to decrease the subway settlement and heave to less than allowable value has not been taken into account in finite element calculation.

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