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Evaluation of movements and structural forces resulting from bored tunnelling in sand

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ABSTRACT: Case histories of bored tunnels in sand in southern Taiwan are selected as a research background in this paper. Details of these tunnels and observations of ground movement induced by tunnel construction are briefly described here. By using numerical analyses, this paper aims to interpret possible ground loss rate induced by tunnel construction. Further, structural forces on tunnel segments are further evaluated associated with different approaches and output are compared and discussed. It is concluded that less than 0.5% of ground loss rate is reached during tunnel construction. Larger displacement was predicted from numerical analyses and the possible reason shall be mainly related to larger contraction ratio used. In addition, from exercises of the use of two different constitutive models, the analyses using Hardening Soil model could give a better description of settlement trough since said model has an additional consideration regarding soil stiffness under unload-reload conditions. Second, various approaches are adopted to explore structural forces, such as shear force, axial force and bending moment during tunnel construction. Under the same load condition and soil profile, it is initially found that the analyses using the beam-spring method to predict the smallest bending moment, shear force and axial forces rather than others. It is aware that ground loss rate and twin tunnel interaction would not affect any result from analyses using elastic equation method and beam-spring method since only loads and details of ring are considered in these two approaches. Considering analytical results, it is understood that the middle of ring (90 degree from top of tunnel) have to take the maximum bending moment.

1 INTRODUCTION

Due to the increasing need for underground space in urban areas, bored tunnelling methods are commonly adopted. However, construction of such tunnels in soft ground may induce significant displacement on the surface so attention must be paid to tunnels located in crowded urban areas. By using an example of a tunnel project in Kaohsiung, Taiwan, as the background, displacements simulated from various approaches are compared with field observations. Further discussions are delivered also.

In addition, structural forces on tunnel lining are considered in engineering practice for bored tunnel design as such issues could determine the reinforcement used for lining. Based on tunnels on the Hanoi Metro, Do et al. (2013) delivered analyses by using several methods, such as Einstein and Schwartz (1979), elastic solution method (JSCE, 1996) and two-dimensional analysis by using FLAC commercial software. Such works indicated some differences in terms of the bending moment and normal forces on tunnel lining structures.

In fact, structural forces stated above may possibly be affected by several factors, such as deformation and loading on the structures, ground conditions as well as details of tunnel lining adopted and influences on forces from these factors are essential to be explored. This has resulted in the motivation to deliver this study associated with different analytical schemes with a bored tunnel case in Kaohsiung to examine such impact.

2 THE SITE AND THE OBSERVATIONS

A twin-parallel-bored tunnel located in Package CO2 on Orange Line of Kaohsiung MRT has been taken as background for the study. The tunnel was connected between O6 and O7 Station and its internal diameter is 5.6 m. The track length of said tunnel is 851.8 m. The tunnel was constructed using a 6.23 m diameter earth
pressure balance (EPB) machine and the main ground consisted of a very deep and thick sandy material with occasional clay content. As indicated in Figure 1, the surface level above tunnel is quite uniform, in the range of elevation of 104 to 105 m. It also shows the tunnel was mainly bored in sand. The twin tunnel remains parallel at the same depth and the centre to centre distance of the two tunnels is approximately 12.0 m at one end but becomes slightly wider (14.0 m) at the other. Overburden depth varies from 15 to 17 m. Table 1 presents some basic information on the soil at the project site, such as description, depth, total unit weight ($\gamma_t$) and SPT-N value. Please refer to Hsiung (2011) for further details.

An intensive instrumentation system was installed which mainly aims to explore surface settlements induced by tunnel construction in both longitudinal and transverse directions. Figure 2 shows the location of settlement points above the tunnel and it is seen that the monitoring array starts at a central point between two tunnels and further expands to two sides for measurement of surface settlement in transverse directions. The further most point is located at 19 m from the central point between two tunnels. Similarly, settlement measurement points are installed every 5 to 40 m on surface level along the tunnel axis so settlement induced by tunnel construction in longitudinal tunnel could thus be monitored.

Hsiung (2011) shows settlement measured on surface level from the project. As indicated in Figure 1, the ground starts to settle once the shield machine approaches and this is mainly related to loss of ground caused by shield advance and tail void. After 20–40 days of passing of the machine, the ground becomes stable since no consolidation settlement is expected. The ground may settle dramatically again due to the approaching of the 2nd shield machine for the other parallel tunnel nearby. It is aware that surface settlements varying from 6 to 10 mm during the 1st drive but can reach up to 8 to 12 mm at the end after the passing of the 2nd machine.

3 MOVEMENT ANALYSES

Lake et al. (1996) commented that the displacement occurred at a place ahead of the tunnel face but is difficult to be verified by observations shown on this project (Hsiung, 2011) as the settlements here only presents data immediately after passing the shield head, though it is very unlikely since only very limited settlement seen during the passing of shield head.

Figure 1. Longitudinal and ground profile of the project.

Figure 2. Location of layout of monitoring of settlement.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Description of ground</th>
<th>Depth</th>
<th>$\gamma_t$ (kN/m$^3$)</th>
<th>SPT-N value</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Grey silty sand</td>
<td>Surface to 6.5 m below; groundwater level observed at 2.9 m below ground level</td>
<td>19.6</td>
<td>5 to 6</td>
</tr>
<tr>
<td>II</td>
<td>Grey silt with sandy clay</td>
<td>6.5 m to 8.5 m below ground level</td>
<td>18.9</td>
<td>6</td>
</tr>
<tr>
<td>III</td>
<td>Grey silty sand occasionally with sandy silt</td>
<td>8.5 m to 23.0 m below ground level</td>
<td>20.0</td>
<td>6 to 15</td>
</tr>
<tr>
<td>IV</td>
<td>Grey silty clay with sandy silt</td>
<td>23.0 m to 25.0 m below ground level</td>
<td>19.3</td>
<td>12</td>
</tr>
<tr>
<td>V</td>
<td>Grey silty sand with sandy silt</td>
<td>Beneath 25.9 m below the surface level</td>
<td>19.8</td>
<td>12 to 19</td>
</tr>
</tbody>
</table>
The difference may be connected with ground conditions, workmanship and innovation of the machine but further exploration will be necessary.

Considering observations measured in the transverse direction at three sections on the ground surface above tunnel named LUO08-1, LUO08-2 and LUO08-3 and details of the project, Hsiung (2011) carried out analyses associated with empirical approaches suggested by Peck (1969) and O’Reilly and New (1982) to verify possible ground loss rate caused by tunnel construction. It is interpreted that ground loss rate induced by tunnel construction is in a wide range of 0.12% to 0.75% but it is 0.27% to 0.57% for up-line tunnel and 0.30% to 0.76% for down-line tunnel at Section LUO08-1. In such method, surface settlement measured has to be applied as a base of interpretation and the settlement trough from empirical approaches has to fit such measurement data. The ground loss rate can thus be determined.

Alternatively, similar works have been done by using numerical tools. Similar to works accompanied with empirical approaches, measured settlement trough in transverse direction shall be fit with settlement trough from numerical simulation by varying ground loss rate (contraction ratio) as well as fixed ground and structural details. The commercial software PLAXIS was adopted for such works. Section LUO08-1 was selected as the background of this study and Table 2 presents soil parameters used for all numerical analyses. In Table 2, $\phi'$ is effective friction angle of soil and $\gamma_1$ is total unit weight of soil.

It is noted that elastic modulus of soil ($E$) is determined by:

$$E = 2000N \text{ (unit: kPa) for sand; and}$$

$$E = 500Su \text{ for clay}$$

in which $Su$ is undrained shear strength of clay and $N$ is SPT-N value.

Except soil parameters, structural parameters have to be given based on actual tunnel lining properties. An elastic plate element is selected for simulation of behavior of tunnel lining. Pre-cast reinforcement concrete was adopted for tunnel lining here and width and thickness of lining is 120 cm and 25 cm, respectively. Elastic modulus of reinforcement concrete lining is assumed to be 31,400,000 kPa associated with its strength. All parameters used for tunnel lining are listed in Table 3.

In PLAXIS, the software provides a built-in element named “tunnel element”. In such element, it gives a function called “contraction ratio”. By assuming contraction ratio, it indicates convergence of circular tunnel so the ground consequently settles accompanied with said convergence.

The constitutive model named Mohr-Coulomb (MC) is adopted for analyses. MC model is an elastic perfect plastic model and parameters of cohesion force and effective friction angle of soils are adopted for determining yield surface of the soil. A single constant elastic modulus has to be provided to define stress-strain behavior of soil.

Similar to Chen (1997) previously the vertical boundary of the analysis is set at 50 m away from centerline between two tunnels, approximately 8 times of tunnel diameter. Similarly, the horizontal boundary has to be ensured to be set at the depth which not affected by any activity of construction so it was set at 45 m below surface level.

For comparison purposes, the other constitutive model called the Hardening Soil (HS) model is selected for analyses too. Main differences between these two models (MC & HS) are elastic modulus of soil has to remain the same all the time in the analysis of MC model but different elastic modulus of soil can be assigned for soil under unloaded condition in HS model.

As soil stiffness at small strain level is considered for ground behavior induced by deep excavations and as Hsiung (2002) commented that soil stiffness interpreted from shear wave velocity measured on site is more suitable to be adopted for simulation of deep excavations in sand, said value thus was used for definition of reference soil modulus ($E_{50}$) in HS model in order to explore impact of selections of constitutive model and also soil stiffness. Again, reference soil modulus under unload-reload condition ($E_{ur}$) is assumed to be three times of $E_{50}$.

Figure 3 presents numerical analyses by the use of PLAXIS and the ground loss rate interpreted (0.7%) from analyses using empirical method were taken to be contraction ratio in numerical analyses. It is seen from Figures 3 that no matter analyses using MC or HS model, the settlement from numerical simulation seems to be greater than observed settlements and reasons are further explored. First, a higher value for contraction ratio is selected and this could possibly

### Table 2. Soil parameters used for analyses.

<table>
<thead>
<tr>
<th>Soil classification</th>
<th>Depth at bottom of layer (m)</th>
<th>$\gamma_1$ kN/m³</th>
<th>$\phi'$°</th>
<th>$E$(kPa)</th>
<th>Poisson ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>SM</td>
<td>6.5</td>
<td>19.6</td>
<td>31</td>
<td>12000</td>
<td>0.30</td>
</tr>
<tr>
<td>CL</td>
<td>8.5</td>
<td>18.9</td>
<td>30</td>
<td>21000</td>
<td>0.45</td>
</tr>
<tr>
<td>SM</td>
<td>23.0</td>
<td>20.0</td>
<td>32</td>
<td>22000</td>
<td>0.30</td>
</tr>
<tr>
<td>CL</td>
<td>25.0</td>
<td>19.3</td>
<td>32</td>
<td>45500</td>
<td>0.45</td>
</tr>
<tr>
<td>SM</td>
<td>45.0</td>
<td>19.8</td>
<td>33</td>
<td>54000</td>
<td>0.30</td>
</tr>
</tbody>
</table>

### Table 3. Structural parameters used for tunnel lining.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Internal radius (m)</td>
<td>2.8</td>
</tr>
<tr>
<td>Thickness (m)</td>
<td>0.25</td>
</tr>
<tr>
<td>Axial stiffness (kN/m)</td>
<td>$7.85 \times 10^6$</td>
</tr>
<tr>
<td>Bending stiffness (kN/m)</td>
<td>$4.09 \times 10^4$</td>
</tr>
<tr>
<td>Equivalent thickness (m)</td>
<td>0.25</td>
</tr>
<tr>
<td>Equivalent unit weight (kN/m/m)</td>
<td>6.0</td>
</tr>
<tr>
<td>Poisson Ratio</td>
<td>0.2</td>
</tr>
</tbody>
</table>
give a larger displacement from numerical analyses. Second, as ground loss rate interpreted from empirical approach is directly based on settlement trough on surface but contraction ratio is taken from convergence of tunnel in deeper ground for numerical analyses, these two numbers could be different.

Settlement troughs from predictions using MC and HS model were compared and it was found that settlement trough from HS model has a larger settlement at the same location of tunnel centre and then becomes much smaller in the place away from the tunnel. In contrast, the settlement trough from MC model becomes much flat and extends further in the place away from the tunnel. As tunnel construction is a stress-relief activity and HS model does have an additional description in soil stiffness during unloaded condition, this is expected to be the reason leading for such difference. In addition, a greater surface settlement at centre of tunnel predicted from analyses of HS model might be connected with lower estimated value of $E_{ur}$. However, neither of them could have good performance at far end of the excavation and this is expected to be related to limit of both models in definition of soil stiffness of small strain behavior.

As indicated above, the contraction ratio may not be the same with ground loss rate interpreted from surface settlement trough. Therefore, a different approach is delivered by using TUNNEL 3D. Such software is similar to PLAXIS, a finite element package but allows the user to generate 3-dimension mesh of tunnel to evaluate deformation and stability of tunnel instead of 2-dimensional one. For each new 3-dimension project, a 2-dimensional cross section has to be first developed and then further define 3-dimension mesh by given details in longitudinal direction. Necessary nodes and elements for analyses are thus defined herein.

By using different contraction ratio directly, the settlement troughs are compared with the one from field measurement so possible ground loss rate can be defined. Therefore, ground loss rate from interpreted from empirical approach aren’t relied upon anymore and the settlement trough in both longitudinal and transverse directions can be explored.

First of all, boundaries and mesh of analytical models of TUNNEL 3D have to be defined. The same set-up of vertical and horizontal boundaries used for analysis of PLAXIS is adopted but the boundary in longitudinal direction has to be additional determined here. Sensitive analyses by the use of 1 m, 10 m and 100 m were carried out and results indicate no significant difference is seen from analyses having difference length of model in longitudinal direction. Considering results of sensitivity analysis of boundaries conditions, 100 m long boundary in longitudinal direction was adopted. Again, elastic-perfect plastic Mohr-Coulomb model is selected for analyses using TUNNEL 3D. Soil parameters listed in Table 2 are used for analyses.

By varying contraction ratio in the range of 0.5 to 2.0%, surface settlement troughs from numerical analysis are presented and compared with field observations. It is found that measured data in general fit the settlement trough predicted using 0.5% or even less. Although additional conditions in longitudinal direction can be defined in TUNNEL 3D but it is very likely such function does not provide any advantage in comparison with analyses of PLAXIS as predicted settlement troughs from analyses by use of both software are almost the same once the same contraction ratio is used, though TUNNEL 3D is claimed as a 3-dimensional analytical software.

4 STRUCTURAL FORCES ANALYSIS

As indicated above, except displacement induced by tunnel construction, stresses on tunnel lining, such as shear forces, bending moment and axial forces are important features and have to be explored so details of reinforcement can thus be designed. Previous studies related evaluation of structural forces on tunnel segments and related analytical approaches include Muir-Wood (1975), Curtis (1976), and ITA (2000) etc.

In order to examine structural forces on tunnel lining, several approaches, such as updated elastic equation method proposed by JSCE in Japan Standard for Shield Tunneling (JSCE, 2006), beam-spring method (Mitsuho Information & Research Institute, 2006) and Finite Element Method (FEM) by using TUNNEL 3D. Details of each approach are briefly described, as follows.

First of all, similarly to Do et al. (2013) described, the elastic equation method is a simple method allowed to calculate forces, such as bending moment, shear force and axial force of a circular tunnel associated with vertical load, horizontal load, horizontal triangular load, soil reaction and dead load. Structural forces are interpreted first individually and sum up together afterwards.

Secondly, structural forces are defined by a built-in beam-spring model in a commercial software named “MOLEMAN-I” which is widely used and accepted in Japan and other countries in Asia. The tunnels are built with staggering joints in the model and up to 3 rings are allowed to be assumed in the model. Since the dimension of the tunnel in longitudinal direction is comparatively larger than the dimension in the other two directions, it can be expected that the tunnel will act as plane-strain behavior. Therefore, the analysis can be delivered by assuming a model having two half-ring and one full ring to represent the tunnel.
structure. Each ring is assumed as a beam and soils are assumed to act as springs to connect with the beam with various stiffness which defined by the user.

By giving necessary input parameters, such as loads, details of number of segments, joint conditions and imposed displacements on tunnel, displacement and structural forces, such as bending moment, share force and axial forces of tunnel segment.

5 DISCUSSIONS

In order to properly express structural forces on segments, an index system was designed and adopted. In said system, the direction of 12 o’clock is designated as “0 degree” and the direction of 6 o’clock is designated as “180 degree”. As the structure of tunnel is a symmetric structure, only structural forces on half of the structure from different approaches are presented.

Predicted shear force is an important index of design of shear reinforcement and Figures 4 presents shear force on segment from up-line tunnel tunnel of LU008. As shown in Figure 6, it is indicated that the predicted shear force which segment has to take is in the range of zero to 60 kN/m from analyses by the use of TUNNEL 3D for the case having 0.5% of ground loss rate. Further, it seems that there is no definitely relationship between axial force and ground loss rate. Since axial force of tunnel segment from analyses of TUNNEL 3D is defined by relative displacement between two points on the segment, ground loss rate thus does not have direct impact on axial force of segments. Similar results of axial force are reached from analyses using elastic equation method but such results are larger than results from beam-spring method (in the range of zero to 30 kN/m). However, the locations with the maximum shear force is different from these three approaches.

Moreover, prediction of shear force of down-line tunnel presents that maximum value reaches up to 40 kN/m from analyses by the use of TUNNEL 3D for the case having 0.5% of ground loss rate. However, it is not consistent with results from analyses for up-line tunnel since shear force increases once the ground loss rate increases and the reason for such inconsistence has to be further explored. The predicted shear forces from both elastic equation method and beam-spring method are exactly the same for down-line tunnel with up-line tunnel. As only loads on tunnel are considered for analyses and factors of ground movements and interaction between twin tunnels are ignored in both methods, it is anticipated to be the reasons that predicted shear forces for up-line and down-line tunnel remain the same.

Secondly, predicted axial force on tunnel segment is further demonstrated and it is indicated that the maximum axial force can reach up more than 800 kN/m from analyses using TUNNEL 3D for the case having 0.5% of ground loss rate for up-line tunnel and axial force can be further reduced once the ground loss rate decreases. The maximum axial force is only 700 kN/m for the case having 2.0% of ground loss rate. The maximum axial force predicted from analyses using elastic equation method and beam-spring method is approximately 730 kN/m and 600 kN/m, respectively. Similar to results of shear force, analyses using beam-spring method give the smallest value rather than others. Attention is paid to the location having the maximum axial force and it looks like analyses associated with elastic equation method and beam-spring method indicate the place having 90 degree from top of tunnel shall take the maximum axial force during tunnel construction, though theory of interpretation is different. It is found that very similar results are reached for down-line tunnel, even if a different approach is used.

At last, Figure 5 expresses the bending moment from predictions using different approaches for up-line tunnel. It is seen that predicted maximum bending moment by the use of TUNNEL 3D is in the cluster of 50 to 75 kN-m, no matter what ground loss rate is. The place having the maximum bending moment is seen in the place having 90 degree from top of tunnel. Moreover, the trend of prediction of bending moment from analyses using elastic equation method and beam-spring method are similar to results from analyses using TUNNEL 3D but the magnitude seems to be smaller, up to 50 kN-m only. Simulations also show similar results for bending moment of down-line tunnel. It is herein concluded that impacts from ground loss rate (soil displacement) and twin-tunnel interaction on magnitude of bending moments are limited since both factors don’t cause differences in both tunnels.
The following conclusions are made associated with the output of this study. Firstly, a wide range (0.12% to 0.76%) of ground loss rate was interpreted from empirical approach for a bored tunnel constructed in sand based on observed surface trough. In contrast, predictions from numerical analyses indicate less than 0.5% of ground loss rate can be reached for said tunnel construction. Secondly, a larger displacement was predicted from the analyses using PLAXIS and the possible reason shall be mainly related to larger contraction ratio in analyses used. In addition, from the exercise of the use of two different constitutive models, the analyses using HS model can give a better description of predicted settlement trough since said model has an additional consideration regarding soil stiffness under unload-reload conditions. However, surface settlements at locations far away from the tunnel can’t be properly predicted by any model used in this study due to limits in description of soil stiffness at small strain level.

Apart from displacement, structural forces on tunnel segments are important features in tunnel design. Therefore, various approaches were adopted to explore structural forces, such as shear force, axial force and bending moment during tunnel construction. Under the same load condition and soil profile, it is first found that the analyses using beam-spring method predict the smallest bending moment, shear force and axial forces rather than others. It is noticeable that ground loss rate and twin tunnel interaction would not affect any result from analyses using elastic equation method and beam-spring method since only loads and details of ring are considered in these two approaches. Analyses using TUNNEL 3D for prediction of shear force and axial forces are not recommended since there are still some unknown to be solved. Considering analytical results, it is understood that the middle of ring (90 degree from top of tunnel) have to take the maximum axial force and bending moment. At the end, it is also concluded that limited impacts can be generated from ground loss rate and twin-tunnel interaction on magnitude of predicted bending moment, no matter which method is adopted.

REFERENCES