TUNNELING INDUCED GROUND MOVEMENT
AND
SOIL STRUCTURE INTERACTIONS

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ABSTRACT

Tunneling technique is normally a primary option that can't be avoided in building a Mass Rapid Transport System in a heavily populated metropolitan city. In this case, to avoid any unwanted effect to the surrounding structures and public utilities, the ground movement and the soil structure interaction analysis become important factors to consider. This paper elaborates the author experiences in analyzing such a case, both by employing available empirical formulas and by using a sophisticated finite element analysis.

KEYWORDS

Ground movement, Soil Structure Interaction, PLAXIS software

1. INTRODUCTION

Tunneling is the main technique that primarily employed in constructing an underground tunnel in an urban area. The construction of the bored tunnel inevitably causes ground movements. These movements may affect buildings and utilities located right above and in the vicinity of the tunnel construction. To avoid any adverse effects, it is important to predict the magnitude of the ground movement and its subsequent effects to the surrounding structures and utilities. It is quite often that a soil structure interaction analysis is required to assess the impact of tunneling to the surrounding structure or vise versa.

This paper elaborates the author experiences in adopting empirical formulas and finite element analysis in handling two underground Mass Rapid Transit Projects in Singapore.

2. EMPIRICAL APPROACH

One of the methods that commonly used to predict the ground movements due to a tunnel construction is the method developed by Mair et al (1996). Based on a considerable number of case records, Mair et al demonstrated that the resulting settlement trough immediately after a tunnel has been constructed is well described by a Gaussian distribution curve as shown in Figure 1 below.
The formula for estimating the ground settlement is:

\[ S_v = S_{\text{max}} e^{-\frac{y^2}{2i^2}} \]  

\[ S_{\text{max}} = \] maximum settlement above the tunnel center line
\[ y = \] horizontal distance from the tunnel center line
\[ i = \] horizontal distance from the tunnel center line to the point of inflection on the settlement trough; this \( i \) parameter is approximately a linear function of depth \( Z_0 \) and broadly independent of tunnel construction method. The relationship is:

\[ i = KZ_0 \]  

\( K = \) trough width parameter; on ground surface \( K=0.5 \) for clay and \( K=0.25 \) for sands or gravels

For a Circular tunnel, the maximum settlement above the tunnel center line is:

\[ S_{\text{max}} = \frac{0.31W_L D^2}{KZ_0} \]
where $D$ is the tunnel diameter and $V_L$ is volume loss. The volume loss, normally expressed as percentage, is the ratio of the area of the settlement trough to the excavated area of the tunnel. Its magnitude depends on the soil type and the tunneling method. Historical data shows that the volume loss is: 1-2% for Shield Tunneling in London clay; 1-1.5% for NATM; up to 3% for Earth Pressure Balance (EPB) method in soft marine clay of Singapore; about 0.2% for EPB machine in gravels below water table in Tokyo.

Apart from the vertical movement (settlement), the horizontal movement of the ground can also cause building or utility damage. The ground horizontal movement, $S_h$, is estimated by using a simple equation as follows:

$$S_h = \frac{y}{(Z_0 - Z_p)} S_v$$

................................. (4)

The settlement along the tunnel line, i.e. the longitudinal settlement trough (Fig. 2), directly above the tunnel head is estimated as follows:

$$SL_{max} = 0.5 S_{max}$$

................................. (5)

Fig. 2   Longitudinal Settlement Trough

When two or more tunnels are constructed it is generally assumed that the predicted ground movements for each tunnel acting independently can be superimposed. Where the clear separation of the tunnels is less than one tunnel diameter, the above assumption may be unconservative. Interaction can be taken into account by assuming a greater volume loss for the second tunnel and superimposing the resulting ground movement.

3. **FINITE ELEMENT ANALYSIS**

It can be seen that the above formulas do not take into account soil parameters. The K factor (see Equation 2) is the only factor that introduces the soil condition into the formula. However, the factor is a very crude factor. For example, it does not differentiate the K value for clay with different consistencies. No different values are given for soft, medium, stiff and very stiff clay. Apart from that, it is also indirectly assuming that the soil layers are uniform. The formulas also do not taken into account the ground water condition. Therefore, it can be said that the success in applying the above empirical formulas heavily depend on the pass experiences in similar soil condition with a particular tunneling method. Other than that, it can only be served as a preliminary estimate for the prediction of the ground movement.
When a more thorough analysis (which is involving a wide range of soil parameters, ground water condition and soil structure interaction) is needed, a complex finite element analysis will be required. The traditional objection in applying a finite element analysis is: It requires expensive computer, special and expensive software and costly computer run time. However, since mid of 1990s and especially as we enter this new millennium, advanced personal computer has become so powerful and affordable. The computer run time cost has also become relatively unimportance. And most importantly, finite element software, such as PLAXIS, CRISP, SIGMA, etc., which is specially developed to solve geotechnical problems has been available. The cost of the software has also become relatively affordable for most firms.

The software, PLAXIS for example, is capable in solving many geotechnical and soil structure interaction problems. It is capable to model and solve: diaphragm wall, tunneling, groundwater flow, consolidation, ground anchor and struts, geosynthetic wall, piling, etc. It provides beam element and also slip/interface element, which is very useful in modeling the structural element and the relation of soil-structure interfaces. It also supports various soil models to simulate the behavior of soil continua. A short discussion of the available models is:

- **Linear elastic model**: This model represents Hooke's law of isotropic linear elasticity. The model involves two elastic stiffness parameters, namely Young's modulus, $E$, and Poisson's ratio, $\nu$. The linear elastic model is very limited for the simulation of soil behavior. It is primarily used for stiff massive structures in the soil.

- **Mohr-Coulomb model**: This well-known model is used as a first approximation of soil behavior in general. The model involves five parameters, namely Young's modulus, $E$, Poisson's ratio, $\nu$, the cohesion, $c$, the friction angle, $\phi$, and the dilatancy angle, $\psi$.

- **Hardening Soil model**: This is an elastoplastic type of hyperbolic model, formulated in the framework of friction hardening plasticity. This second-order model can be used to simulate the behavior of sands, gravel and overconsolidated clays.

- **Soft Soil model**: This is a Cam-Clay type model, which can be used to simulate the behavior of soft soils like normally consolidated clays and peat. The model performs best in situations of primary compression.

- **Soft Soil creep model**: This is a second order model formulated in the framework of viscoplasticity. The model can be used to simulate the time-dependent behavior of soft soils.

The PLAXIS software is also capable to carry out dynamic analysis, such as: earthquake load, the vibration effect of piling and machinery. The newest version of the plaxis, i.e. PLAXIS 3 D TUNNEL, is even capable of handling three dimensional analysis. It is clear that a suitable soil model as well as the structural model can be chosen for handling a specific problem.

The section below presents two case studies. The first one is a comparison of ground movement prediction by using the above empirical formula and finite element approach. The second case is solving a soil structure interaction problem with the help of PLAXIS software.
4. CASE STUDY - GROUND MOVEMENT PREDICTION

The case study is extracted from a particular area between Hougang and Kovan stations along the NEL Project of the Singapore Mass Rapid Transit system. At this particular area, twin tunnels of 6.3 m diameter spanning 15.2 m center to center were excavated by using the Earth Pressure Balance (EPB) shield method.

The subsurface condition generally consists of fill layer followed by the Old Alluvium (OA) formation. The Old Alluvium is divided into four sub-layers according to their SPT N-values, consistency and shear strength. The soil type is predominantly silty and clayey sand. Thickness of the soil layers and the initial design parameters are shown in Figure 3 below. The depth of the center axis of the tunnels in the entire route vary between 12.7 to 24.2 m below ground level and this makes the tunneling works totally carried out in Old Alluvium formation.

![Fig. 3 The Subsurface Soil Condition](image)

<table>
<thead>
<tr>
<th>Identification</th>
<th>$\gamma_{\text{dry}}$ [kN/m$^3$]</th>
<th>$\gamma_{\text{wet}}$ [kN/m$^3$]</th>
<th>$\nu$</th>
<th>$E_{\text{ref}}$ [kN/m$^2$]</th>
<th>$c_{\text{ref}}$ [kN/m$^2$]</th>
<th>$\varphi$</th>
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<tr>
<td>Fill</td>
<td>16.0</td>
<td>18.3</td>
<td>0.35</td>
<td>10000.0</td>
<td>80.0</td>
<td>30.0</td>
</tr>
<tr>
<td>OA1</td>
<td>18.0</td>
<td>19.5</td>
<td>0.35</td>
<td>10000.0</td>
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<td>30.0</td>
</tr>
<tr>
<td>OA2</td>
<td>18.0</td>
<td>19.5</td>
<td>0.35</td>
<td>40000.0</td>
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<td>35.0</td>
</tr>
<tr>
<td>OA3</td>
<td>18.0</td>
<td>19.5</td>
<td>0.35</td>
<td>60000.0</td>
<td>120.0</td>
<td>35.0</td>
</tr>
</tbody>
</table>

Since there is no direct precedent for tunneling in the Old Alluvium formation in Singapore, no typical values of K factors and volume loss (see Eqs. 2 and 3) are available. Since the soil mainly consists of sandy materials the K factor of 0.25 was adopted. And a volume loss of 1 to 2% was assumed. This value assumes good workmanship which involves: prevention of the material into the face of the shield, no adverse steering problems or severe alignment corrections by the shield, timely and effective grouting of the tail void skin. Figure 4 shows a typical prediction of the transverse settlement trough, for a tunnel depth of 15 m, $K = 0.25$ and a volume loss of 1%.
The actual transverse ground settlement profile was measured by set of settlement markers installed prior to the tunneling work. The monitoring results are also presented in Fig. 4. It can be seen that the maximum predicted settlement is in the order of 34 mm while the actual settlement was about 6 to 7 mm. Back analysis shows that only with K value of 0.5 and a volume loss of 0.25% the actual settlement would be matched.

![Figure 4: Typical Predicted and Observed Transverse Settlement Trough](image)

Fig. 4 Typical Predicted and Observed Transverse Settlement Trough

![Figure 5: Predicted Settlement by Using PLAXIS Finite Element Software (Volume Loss = 1 %)](image)

Fig. 5 Predicted Settlement by Using PLAXIS Finite Element Software (Volume Loss = 1 %)
Using the soil parameters presented in Figure 3 and with a tunnel thickness of 0.25 m, a finite element analysis was also performed by using Plaxis software. With a volume loss of 1%, the maximum predicted settlement was in the order of 30 mm (see Fig. 5).

Using the same parameters, when the volume loss of 0.20% is adopted, the results show comparable values with the actual settlement. Figure 6 shows the results of the analysis. It can be seen that the maximum predicted settlement is in the order of 8 mm, which is only in the order of 1 to 2 mm different with the observed settlement.

The finite element analysis shows better results. The shape of the settlement trough is very similar to the observed settlement. With a volume loss of 0.20%, without changing any of the soil parameters - which it should really be -, the predicted settlement yields a much closer value to the observed settlement.
5. CASE STUDY - SOIL STRUCTURE INTERACTION

In a densely populated city, it is not uncommon that a subway tunnel must be constructed underneath an existing building foundation or the reverse, that is to construct a building on top of an existing tunnels. In 1998, the author had a chance to evaluate such a problem. At that time a twin tunnel subway project was on its way. These 6.3 m diameter twin tunnels shall cross some 35 m underneath a land where a condominium building was planned. The landlord was wondering when to construct his building, before or after the tunneling? Figure 7 shows the cross section of the proposed construction.

If the building was constructed before the tunnels passed the area, he had no responsibility on the tunnel construction and it would be the tunnel contractor responsibility to take precaution not to induce any negative impact to the building. However, at that time the macro economy situation was not favorable for the sales of the condominium. On the other hand, if the building was constructed later, the impact of the building construction to the twin tunnels had to be studied. And this might lead to a more costly foundation, as there is a requirement that any pile foundation from the ground surface to the spring-lines of a subway tunnel must not bear any friction resistance. The other option available is to strengthen the tunnel lining to anticipate the future additional stresses that come from the building foundation. And the building owner would have to contribute on its construction cost.

Figure 8 shows the initial condition of the site and the subsequent soil parameters. The center of the tunnel lines is 35 m below the ground surface. Landscaping of the site required a 1.5 m excavation and this was done before the tunneling. The base of the raft foundation would be around 3.5 m from the ground surface. The groundwater level was found at about 3.75 m below the ground surface. Table 1 shows the soil data. Mohr-Coulomb soil model was adopted to perform the analysis.

Many possible construction sequences were analyzed. One of the construction sequence that was analyzed is presented below:

- Overall excavation up to 1.5 m deep.
- Bored piles construction
- Tunneling (followed by volume loss of 0.25%)
- 2.0 m excavation for raft construction
- Raft construction
- Building Construction and Load Application

The results of the final stage construction are presented in Figs. 9 - 15.
Table 1  The Soil Data

<table>
<thead>
<tr>
<th>Mohr-Coulomb Number</th>
<th>Identification</th>
<th>Type</th>
<th>$\gamma_{dry}$ [kN/m$^2$]</th>
<th>$\gamma_{wet}$ [kN/m$^2$]</th>
<th>$k_x$ [m/day]</th>
<th>$k_y$ [m/day]</th>
<th>$\nu$ [-]</th>
<th>$E_{ref}$ [kN/m$^2$]</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>stiff silty clay</td>
<td>Drained</td>
<td>16.0</td>
<td>18.0</td>
<td>1.3000E-3</td>
<td>9.0000E-4</td>
<td>0.33</td>
<td>300000.0</td>
</tr>
<tr>
<td>2</td>
<td>hard silty clay</td>
<td>Drained</td>
<td>16.0</td>
<td>20.0</td>
<td>1.3000E-3</td>
<td>9.0000E-4</td>
<td>0.33</td>
<td>600000.0</td>
</tr>
<tr>
<td>3</td>
<td>sandstone</td>
<td>Drained</td>
<td>15.0</td>
<td>21.0</td>
<td>0.0130</td>
<td>0.0086</td>
<td>0.33</td>
<td>1E5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Number</th>
<th>Identification</th>
<th>$c_{ref}$ [kN/m$^2$]</th>
<th>$\varphi$ [$^\circ$]</th>
<th>$\psi$ [$^\circ$]</th>
<th>$R_{inter}$ [-]</th>
<th>Interface Permeability</th>
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<tbody>
<tr>
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<td>stiff silty clay</td>
<td>5.0</td>
<td>22.0</td>
<td>0.0</td>
<td>0.70</td>
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<tr>
<td>2</td>
<td>hard silty clay</td>
<td>10.0</td>
<td>30.0</td>
<td>0.0</td>
<td>0.70</td>
<td>Neutral</td>
</tr>
<tr>
<td>3</td>
<td>sandstone</td>
<td>15.0</td>
<td>42.0</td>
<td>5.0</td>
<td>1.80</td>
<td>Neutral</td>
</tr>
</tbody>
</table>

Fig. 8 - The Finite Element Model of The Initial Condition
Fig. 9 - Deformed Mesh

Fig. 10 - Pile Raft and Tunnels Total Displacement
Fig. 11 - Pile Raft and Tunnels Axial Force

Fig. 12 - Pile Raft and Tunnels Shear Force

Fig. 13 - Pile Raft and Tunnels Bending Moment
Fig. 14 Changes of Displacement right above Tunnel Crown

Fig. 15 Changes of Effective Stresses right above Tunnel Crown
With the said construction sequence, the result of the analysis shows that the maximum pile raft settlement would be in the order of 25 mm. The analysis also predicted that the building would exert additional vertical stress of 66 kN/m², with a corresponding 10 mm additional vertical displacement, to the tunnel crown. The above example was one of the input for project evaluation.

The data was submitted to the resident engineer of the tunnel construction to study the impact to the tunnel structure. It was later reported that the option of strengthening the tunnel lining was adopted.

6. CLOSURES

The above discussions and case studies show the importance of predicting the ground movement due to tunneling work. It also shows the importance of soil structure interaction between the tunnel and the building on top or around the vicinity of the tunnel construction area.

The prediction of the ground movement shall determine the safety of the surrounding utilities and structure. An excessive movement of the ground surface may cause utility and building damage. A new building constructed above a tunnel may also cause adverse additional movement and stresses to the tunnel underneath the building.

A simple empirical formula in predicting the tunneling induced ground movement can be used for a preliminary assessment and when there is relatively no important structure or utility around the construction area. However, in a more complex situation, it is suggested to carry out finite element analysis and perform a study on the soil structure interaction.

The paper demonstrates that the geotechnical finite element software is capable to handle complex soil structure interaction problem. Which cannot be solved by a simple empirical or other simple analytical formula. Many soil models have been incorporated into the software. The PLAXIS software even comes with dynamic module, which is capable to evaluate soil structure interaction due to dynamic load and earthquake loading. Recently, its newest version, PLAXIS 3D TUNNEL is capable to perform three dimensional analysis.

Despite the availability of a sophisticated and a relatively cheap software, do remember that soil is not manmade materials. Therefore, strong theoretical knowledge and sophisticated engineering software alone is not adequate. A geotechnical engineer must gain plenty of practical experiences in order to come out with a sound engineering judgment in determining the relevant soil parameters for a particular soil model. It does not matter how sophisticated computer software is, the adage “Garbage in Garbage out” always prevails.

REFERENCES


Mair, R.J., 1997 (unpublished), Building and Utility Damage Assessment in Relation to Tunneling and Deep Excavation, Lecture Notes, Singapore.

