The affect of shield operation parameters and overcut 
On existing tunnel in multi-level crossing tunnels


1-department of mining engineering, Isfahan university of technology
2-department of mining engineering, Isfahan university of technology:
3-department of mining engineering, Isfahan university of technology
4-sahel consultant society

torab.alem@gmail.com
bagherpour@cc.iut.ac.ir
smahdevari@cc.iut.ac.ir
fasihi_e@yahoo.com

Abstract
The development of transportation in large cities needs some new tunnels to be designed and constructed nearby existing tunnels. In this study a metro tunnel will be excavated by EBP shield at different level from the sewage tunnel and it’s beneath. Both the relative position of tunnels and the excavation procedure of new tunnel effect the soil displacement and existing tunnel lining. Hence, the effects of shield operation parameters (face pressure, grouting pressure), overcut between shield skin and surrounding soil are studied using a 3D finite difference analysis. The results showed that the largest interaction effects occur at the invert of existing tunnel. Analysis result showed the effect on existing support system in longitudinal section of tunnel is larger than cross section. Most effect on support occurs when ground moves into overcut space. However, these affects decrease significantly with increasing the face pressure, grouting pressures and bentonite flow into steering gap between shield skin and surrounding soil.

Keyword: Tunnel, EBP shield, Operation parameters, Overcut, 3D numerical model.

1. INTRODUCTION

Ground movements are an inevitable consequence of excavating and constructing a tunnel especially in soft ground. It is not possible to create a void immediately and provide an infinitely stiff lining to fill it exactly. In the time taken to excavate, the ground around the tunnel is able to displace inwards as the stress relief is taking place. Thus it will always be necessary to remove a larger volume of ground than the volume of the finished void. This extra volume excavated is termed the ‘volume loss’.

In the specific case of mechanized tunneling, the individual factors contributing to volume loss are:

- Face ground loss: It is caused by rotating of cutter head that remove material from tunnel face.
- Radial ground loss around shield: It is caused by moving ground to gap (overcut) between shield and ground and deformation of shield.
- Radial ground loss around lining: It is caused by moving ground to gap between lining and ground and deformation of lining.

Figure 1 shows ground loss around tunnel in mechanized tunneling. Settlements is caused by volume loss are mainly classified in short, medium and long-term settlement. Short-term settlement usually caused by tunnel excavation. Medium and long-term settlements are thought to be the result of creep and consolidation of ground.
Interaction between two or more tunnels in closely space is very important problem. In these cases, it is necessary to investigate the effects of tunneling on the existing structures such as support systems of adjacent tunnels. There are many people who work in these cases, such as Kimmance et al (1996), Kim (1996), Yamaguchi et al. (1998), Mroueh & Shahrouj (2003), Ng et al. (2004), Liu et al. (2008), Liu et al. (2009) and etc. The data of field observation are often incomplete or incorrect. Physical modeling tests are limited to some cases and empirical methods ignore the presence of existing tunnel in two or more tunnel analysis and therefore are not realistic and 2D numerical analysis can only be used to study interactions between parallel tunnels. Hence, It seems 3D numerical modeling is the most appropriate way to investigate the interaction between two or more none-parallel tunnels.

2- Geometric and ground condition

The line -7 metro Tehran is constructed using an EBP Shield and crosses the sewage tunnel at different level. Geometry of sewage and metro tunnels in cross section are shown in fig 2. The overburden of sewage tunnel is 11 meter and pillar width between two tunnels is 2 m. The outer and final diameter of metro tunnel is 9.164 and 8.85m, respectively. Detail of cross section sewage tunnel is given in fig 2b, c. As shown in figure 2b, tunnel has 3.72 m high with final 2.73m high, and width 3 m with final 2m. The support system of sewage tunnel consists of 20 cm shotcrete with 3bar lattice girders and 30 cm lining segment (fig 2c). Geotechnical parameters are given around cross section two tunnels are presented in Table 1.

<table>
<thead>
<tr>
<th>Geological engineering unit</th>
<th>Cohesion (kg/cm²)</th>
<th>Elastic modulus (kg/cm²)</th>
<th>Poisson ratio</th>
<th>Dry density (kg/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ET-2</td>
<td>0.18</td>
<td>750</td>
<td>0.3</td>
<td>1840</td>
</tr>
</tbody>
</table>

3- Numerical model

The model developed for the analysis is shown in Fig 3a. The model is 96m long and wide and 44.2m deep. The model width was considered sufficient to minimize boundary effects. In Fig 3b, cross section of up tunnel to measurement is shown. Numerical modeling of the tunnel construction using TBM constitutes a hard task, because it requires consideration of complex aspects such as the soil excavation, the overcut, the application of the face pressure, the installation lining rings and the grouting of the annular space. To avoid
the problem caused by displacement of ground around TBM, usually tunnel is excavated larger than design plan that is called “overcut”. Certainly, the overcut around Shield is not symmetric, but it does consider oval shape and the gap value in top of shield is larger than bottom of shield. Loganathan (1999) described the ratio 3/1 for gap value (top/ bottom) around shield or lining. In this paper, the traditional method of estimating ground loss is redefined based on the “gap parameter” introduced by Rowe and Kack (1983), and it is called equivalent ground loss. In this paper excavation procedure is considered sequential. The parameters are used in model is given in Table 2.

Table 2- physical – mechanical properties of support

<table>
<thead>
<tr>
<th>Tunnel</th>
<th>Support type</th>
<th>Thickness (cm)</th>
<th>E (GPa)</th>
<th>Poisson’s ratio</th>
<th>Unit weight (kN/m³)</th>
<th>Unconfined pressure strength(Mpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sewage</td>
<td>Shotcrete</td>
<td>20</td>
<td>25</td>
<td>0.2</td>
<td>2300</td>
<td>27</td>
</tr>
<tr>
<td></td>
<td>Lining</td>
<td>30</td>
<td>32</td>
<td>0.21</td>
<td>2400</td>
<td>37</td>
</tr>
<tr>
<td>Metro tunnel</td>
<td>Segment</td>
<td>35</td>
<td>35</td>
<td>0.2</td>
<td>2500</td>
<td>40</td>
</tr>
</tbody>
</table>

4. Effect of new TBM tunneling on existing support system

This section presents the results of the interaction behavior between two perpendicularly crossing tunnels observed when a new deep tunnel is driven beneath an existing shallow tunnel. If the bending moment tends to put the side of the lining facing towards the tunnel opening into tension and the side facing the rock mass into compression, it is regarded as positive, Otherwise, it is negative. Positive and negative values of axial force refer to tension and compression, respectively.

2-2: The effect of face pressure and grout pressure on internal forces, moment and displacement of existing support system

During modelling, several locations, as marked in Fig. 3b, are monitored to quantify the effects of tunnelling on the existing support system. Analysis was performed in two cases: with and without bentonite pressure behind shield skin. It also was performed for different face and grouting pressure. Fig 4 depicts the variations of circumferential bending moments monitored at the right (leading) side, the crown, the left (far) side, and the invert of the lining of the shallow tunnel during the driving of new tunnel at different face and grouting pressures. Each figure consists of two parts: One before the shadow line (6m after tunnel face) that shows the effects of face pressure on existing lining support and another one to show the effects of grouting pressure on existing lining. For each face pressure, the effect of grouting pressure is investigated and in this paper result is only shown for 2 bar of face pressure. During the driving of the deep tunnel different trends are observed
at the different measuring points. As the face of the deep tunnel advances, the change of bending moments on the existing lining is observed first at the right (leading) side, then at the invert, then at the crown, and finally at the left (far) side of the shallow tunnel. In first parts of Fig 4, As the face of the deep tunnel advances (i.e. from -16.5 to -3m) the negative bending moment in the existing lining gradually increases in high face pressure at left and right side but amount of increases in left side is more larger than right side. After the new tunnel face passes the monitored points, the negative moments significantly decrease to their maximum and have positive values around 5.5m behind the new tunnel face due to the overcut of TBM. At the crown of the shallow tunnel, the positive moments in the lining increase a little in high face pressure as the face of the deep tunnel approaches. After new tunnel face passes the monitored points, the positive moments significantly decrease until around 5.5 m behind the tunnel face. At the invert, the positive moments increases in high face pressures as the tunnel face approaches and passes tunnel face change in moment is more large than other marked points. After this point left side of tunnel has big influence due to new tunneling construction. In second part of Fig 4, the effect of grouting pressure shown. It can be seen, at left and right side of existing lining, as the tunnel face passing the half of shield skin the positive bending moment gradually decrease but at tail void of shield, decrease is more significant and have negative value, then increase and according to grouting pressure it is stable with negative value smaller, same or larger than their initial values. At crown and invert marked point, around the end of shield skin, for high grouting pressure changes in bending moment is larger. Generally, in Fig 4 it can be observed the effect of face pressure at invert and left side of tunnel is larger than crown and right side and the effect of grouting pressure at invert and right side of lining larger and high pressure has larger effect on exiting lining.

*Face Pressure   ** Grout Pressure   *** Legend for four picture is same

**Figure 4: variation of circumference bending moment at marked locations**

Fig 5 shows the circumferential axial force variation at monitoring point. As shown in first part of Fig 5, it can be seen that, as approaching new tunnel face, negative axial force at left and right side increases then significant decrease occurs for negative axial forces with tunnel face passing and continue to around half of
shield skin. At crown it is not importing effect on lining due to face pressure but after passing new tunnel face, significant increase in negative axial force is observed. At invert there is a different process, as the tunnel face approaches the marked point, in high face pressures the negative axial forces decrease then after passing new tunnel face it increases up to 1.5 m behind the tunnel face and then it decreases. In the second part of Fig 5, the effect grouting pressure for 2 bar tunnel face pressure is given. At left and right side of lining negative axial force increase with increase of grouting pressure around the end of shield skin. It is stable according to grouting pressure. At crown around the end of shield skin that the grouting pressure applied, decrease in negative axial force can be observed. At invert grouting pressure cause increase in negative axial force.

Figure 5: variation of circumference axial forces at marked locations

Variation of bending moment and axial forces are given in Fig 6, 7. Similar discussion of Fig 4,5 is performed here for longitudinal bending moment and axial forces. In first part of Fig 6, at left side face pressure cause decrease in negative bending moment and increase it at right side. At crown and invert, face pressure causes increase in positive bending moment. In the second part of Fig 6, grouting pressure causes significant influence on longitudinal bending moment at left side and invert. In first part of Fig 7, it can be seen that face pressure have little effect on longitudinal axial force at crown an invert but at left side it cause small reduction in axial force in high face pressure rather than low pressure and vice versa at right side. Positive axial force increases at left side and decreases at right side and invert of existing lining due to high grouting pressure. Also, negative axial force decreases with increasing grout pressure.

2-2: The effect of bentonite pressure on internal forces, moment and displacement of existing support system

According to the above discussion, it can be understood that the gap between shield skin (overcut) and ground has significant undesirable effect on existing lining. Therefore it is important to prevent the ground displacement into void space behind shield skin. Some of new tunneling machines enable to pomp the bentonite or grout behind the shield skin. Usually bentonite is used to fill overcut space and grout for gap between lining and ground. Often the bentonite pressure is considered as the face pressure or 0.5 bar larger than face pressure. Hence in this paper, the face pressure applied to face and around the shield skin of new
tunnel. Here bentonite pressure value along the shield skin is consider constant. Fig 8 shows the effect of bentonite pressure on bending moments and axial forces on existing lining at invert.

Figure 6: variation of longitudinal bending moment at marked locations

Figure 7: variation of longitudinal axial forces at marked location
Changes of bending moment and axial forces at invert of existing tunnel are high due to driving new tunnel beneath it. Hence this point is chosen to monitoring. As shown in Fig 8, with increasing bentonite pressure negative Circumferential and longitudinal bending moment decreases and convert to positive bending moment in high bentonite pressure. Circumferential axial force decreases with increasing the bentonite pressure and positive longitudinal axial force decreases. The relationship between bending moments and axial forces is shown in Fig 9. According to Figure, in circumferential and longitudinal direction are within the limit of both compressive failure and tensile cracking. In longitudinal direction, there are both relatively big bending moments and axial forces at left side and invert of existing lining and it is likely the tension cracking occur at invert in large displacement.

Figure 8: effect of bentonite pressure on bending moment and axial forces of existing lining

Figure 9: Relationship between bending moments and axial forces and typical capacity of composite support.
5. Conclusions

It is generally recognized that the interactions between tunnels are complex, especially for perpendicularly crossing tunnels, which necessary to be investigate using 3D analysis methods. The interactions between closely-spaced perpendicularly crossing tunnels line-7 metro Tehran are investigated using a full three-dimensional (3D) finite difference analysis coupled with elasto-plastic material models. Special attention is paid to the effect of face pressure, grouting pressure and overcut in EBP tunneling on the support system of the existing tunnel. Crown, invert, left and right side chosen to investigate. The following conclusions can be drawn:

1. The existing support system in the crossing area is affected first at the leading side, then at the invert, after that at the crown, and finally at the far side as the underlying tunnel face advances, but relatively far from the crossing area remains almost unchanged during new tunnelling.

2. The overcut have significant undesirable effect on existing support system and cause tension or compressive state of existing support system has been changed.

3. Lining of existing tunnel affected Due to face pressure applied on underlying tunnel face. Variation bending moments and axial forces shown in Fig 4-7 indicate that the effect of face pressure on circumferential bending moments and axial forces at left and invert is larger than crown and right side. The effect of face pressure is first on left side and it begins from about 15 meter before the center of existing tunnel. The effect on face pressure in longitudinal direction on bending moments and axial forces at left and right side is larger and high face pressure have significant effect on bending moment at right side.

4. The grouting pressure at tail void of EBP shield has significant effect on existing system support as advancing new tunnel. It cause the positive bending moment convert to negative bending moment and support in tension state convert to compressive (initial) state and vice versa. In high grouting pressure, at invert and right side the circumferential bending moment have larger value rather than initial relatively to positive or negative value. Variation of axial force at right and left side is large but in circumferential direction is larger. Changes in longitudinal bending moments value at right and are larger than two other points.

5. Bentonite pressure behind shield skin prevent the surrounding soil displacement into overcut space and then causes influence on existing support to be low.

6. When a new tunnel is driven by EBP shield beneath an existing perpendicularly crossing tunnel, large changes occur at invert, left, right and crown of existing tunnel respectively, due to shield operation parameters and overcut.

7. For safe excavation using EBP machine especially close to sensitive structure such as existing tunnel, it is necessary that the machine work at proper face and grouting pressure with adequate advance rate.

6. REFERENCES


