Modelling ground response to TBM tunnelling with active face support

T. Xu, W. H. Zhou
State Key Laboratory of Internet of Things for Smart City & Department of Civil and Environmental Engineering, University of Macau, Macao, China

A. Bezuijen
Department of Civil Engineering, Ghent University, Ghent, Belgium & Deltares, Delft, The Netherlands

ABSTRACT: This article presents a simple numerical investigation of ground responses to tunnel-boring machine (TBM) tunnelling with active face support. Unlike other numerical modelling in which a flow inward the excavation chamber was assumed, in this modelling a flow inward the ground surrounding the face was assumed. The later flow is caused by the penetration of slurry (for slurry shield) or foam (for EPB). The effect of soil layering (semi-confined and unconfined aquifers) was also taken into account. The numerical results show that the ground response to TBM tunnelling with active face support can be significantly influenced by soil layering. In a semi-confined aquifer, at a distance larger than approximately three times of tunnel diameter from the face, the excess piezometric heads are the same at any depths. This finding is helpful to improve the solution for the excess pore pressure caused by TBM tunnelling in a semi-confined aquifer proposed by Xu et al. (2019). The existing analytical solution of excess pore water pressure distribution for the case of semi-confined aquifer matches well with the numerical result. It appears that for an unconfined aquifer, contour of the excess pore water pressures is like a radial distribution. The steady state model of piezometric head for the homogeneous soil results in excellent matches with the values derived from the numerical modelling. For a semi-confined aquifer, the ground surface heave was observed. Though the face collapse potential was found for the unconfined aquifer, the displacement around the face was very small.

1 INTRODUCTION

Improper face support probably leads to face instability during tunnel-boring machine (TBM) tunnelling, especially in aquifers. Insufficient support may lead to face collapse and too high may lead to face blowout. Active face support with pressurised slurry or foam can successfully stabilise the face (Stack, 1992) and hence it has been widely used.

With active face support, since the pressure in the excavation chamber is higher than that in the soil, slurry or foam or water from the foam will penetrate into the surrounding soil and thus generates excess pore water pressures. In this condition, the effective face support pressure is reduced. Many analyses have been conducted to investigate the face stability. Some authors assumed that there would be an ideally impermeable layer formed at the face and thus set an uniform (Kasper & Meschke, 2004; Lambrughi et al., 2012; Ukritchon et al., 2017) or a linear face pressure in their analyses (Kim & Tonon, 2010; Zhao et al.2015). Some others considered that the effective face support pressure decreases with the distance of slurry or foam penetration into the soil in front of the face and proposed some formulas to describe the reduced effective support pressure (Anagnostou & Kovári, 1994; Broere, 2001). However, only for standstill, there can be a linear pressure over the thickness of the penetrated zone, going from the pressure at the tunnel face to the hydrostatic pressure in the soil. For drilling the penetration of slurry or foam will generate excess pore water pressure in the soil mass.

In reality, the cutter head will cut off the soil and carry it into the gap between the TBM and the ground and thus soil-slurry-mix or soil-foam mix rather than ‘clean’ slurry or foam presents at the face. For pure slurry or foam there will be a low permeable layer and the pressure drop will be at the boundary between the slurry or foam and the soil. In this condition, there will be no low permeable layer formed at the face, but a continuous penetration (Xu & Bezuijen, 2019). Since the penetration velocity decreases with the penetration distance, after some time the penetration is very slow and hardly influences the pore pressures in the soil. This has significant influence in the pressure transfer at the tunnel face. For this situation the traditional calculation method is valid. The question is then: how is the ground response to TBM tunnelling under such a penetration.

This article therefore aims to convince readers that the groundwater flow has to be taken into account and that concepts for the face pressure described in previous research (references) have to be adapted. A simple numerical modelling using a finite element method (FEM) was carried out. The influence of flow generated by the penetration at the face and soil layering are taken into account.

2 NUMERICAL ANALYSIS AND DISCUSSION

2.1 *Model built-up*

A sketch of TBM tunnelling with active face support through a semi-confined aquifer is shown as Figure 1. An aquifer (sand layer) is overlain by a semi-permeable layer (peat layer). An impermeable layer is beneath the aquifer. The tunnel is built in the centre line of the aquifer. For an unconfined aquifer, it is assumed that the soil is homogeneous.

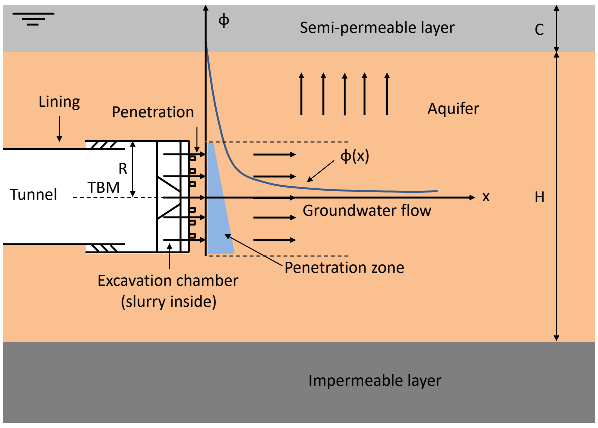


Figure 1. Sketch of TBM tunnelling in a semi-confined aquifer

Table 1. Inputs for the model

Parameter	Semi-permeable layer	Aquifer	Impermeable layer
γ' (kN/m^3)	10.5	16	16
k (m/s)	10^{-6}	4.0×10^{-4}	-
K_0 (-)	0.66	0.47	0.43
E (kPa)	1.5×10^3	5.0×10^4	7.5×10^5
v (-)	0.15	0.3	0.3
φ (°)	20°	36.3°	32°
ψ (°)	0°	1°	2
c (kPa)	3	1	1

A two-dimensional (2-D) model was set up in the finite element code PLAXIS 2D, see Figure 2. The model dimensions 480 m × 60 m (length × depth) were selected to balance the boundary effect and the computational efficiency. A 14.5 m diameter tunnel centre is situated at NAP -27.2 m. The inputs for the model are listed in Table 1. Mohr-Coulomb model was selected for the different soil layers.

To bring the complex boundary problem to a simple solution, some simplifications and assumptions were made. The lining is assumed to be finished and thus the gap is fully filled with grout and the volume loss behind the shield is neglected. Hence, the tunnel and shield are not modelled. No vertical displacements of tunnel crown and invert are assumed, see Figure 2.

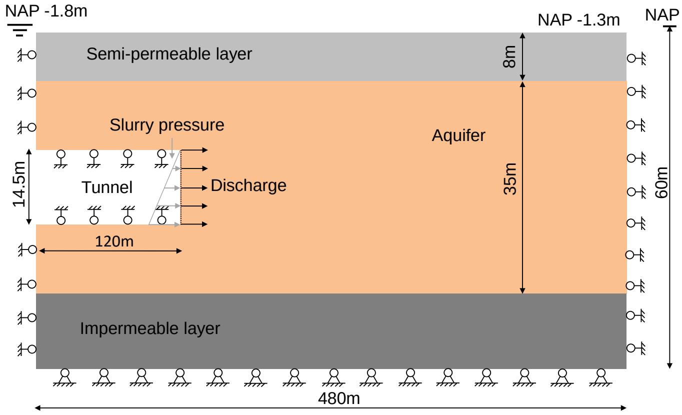


Figure 2. Model built-up

The support pressure can be assumed consisting of slurry pressure and excess pressure. The slurry pressure due to slurry self-weight is set as a linear pressure along the depth and the density of slurry is 1300 kg /m^3, which was a medium value that monitored in Second Heinenoord Tunnel (COB, 2000). In case of penetration velocity larger than the excavation rate, a discharge is expected at the face. The discharge is assumed as constant. Therefore, a hydraulic boundary of constant discharge at the face is defined. In this condition, according to Broere (2001), the volume of the water discharged by the penetrating slurry is roughly equal to the total pore volume of the excavated material. The discharge per unit area of tunnel was set as, according to:

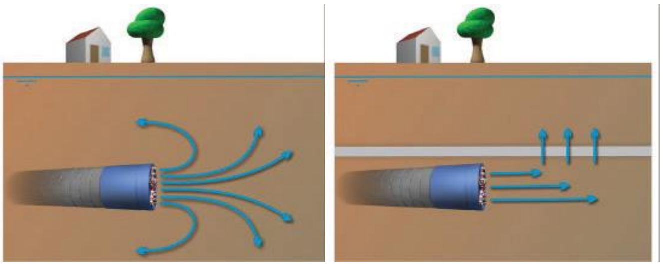
$$q = -n \cdot v \tag{1}$$

with n the porosity of the soil and v the excavation rate of the TBM. In the GHT, the excavation rate is about 40 mm/min and the porosity of the soil is about 0.37. The discharge at the face is estimated as 0.000247 m/s according to Equation (1). According to Broere (2001), the starting value of the head at $x = 0$ is 4.4 m. Hence, Equation (1) is only valid for excavation face pressures higher than 4.4 m height of water and the infiltration rate is higher than the TBM velocity. If not, there should be a head boundary condition.

A transient fluid-solid coupled analysis was performed. Undrained condition was assumed to semi-permeable layer and aquifer, the impermeable layer was set as non-porous. The analysis step lasted 60 mins, corresponding to the normal excavation period of 40 to 60 mins for one ring excavation.

2.2 *Numerical result of excess pore water pressures*

The soil layering may affect the pressures at the face and thus the face stability. Two common types of soil layering are shown in Figure 3. For an unconfined aquifer, contour of the excess pore pressures is a radial distribution, see Figure 4.



(a) Homogeneous soil (b) Semi-confined aquifer

Figure 3. Sketch groundwater flow caused by shield tunnelling. (After COB, 1998, drawings by Bezuijen jr, used with permission)

Figure 4. Contour of the excess pore water pressures around the face in an unconfined aquifer (negative value indicates pressure in PLAXIS 2D)

Figure 5. Contour of the excess pore water pressures around the face in a semi-confined aquifer (negative value indicates pressure in PLAXIS 2D)

It is clear from Figure 5 that the magnitudes of excess pore water pressure caused by TBM excavation in a semi-confined aquifer are the same at any depths, when the flow reaches the place where is at least three times of tunnel diameter from the face. This may be helpful for improving the analytical solution for the excess pore pressure caused by TBM tunnelling in a semi-confined aquifer proposed by Xu et al. (2019).

Bezuijen (2001) proposed a steady state model to describe the piezometric head in the case of homogeneous soil:

$$\phi(x) = \phi_0 \left[\sqrt{1+(x/R)^2} - x/R \right] \quad (2)$$

Where $\phi(x)$ the piezometric head (m) at a distance x (m) in front of the tunnel face, ϕ_0 is the piezometric head at the tunnel face (m), and R (m) the radius of the tunnel, assuming a piezometric head of zero far from the tunnel in the pore water.

For a semi-confined aquifer, the steady state piezometric head at the face can be derived from the one-dimensional solution of pore pressure generated by TBM tunnelling Broere (2001):

$$\phi(x) = \phi_0 \exp(-x/\lambda) \quad (3)$$

with λ the leakage length of the aquifer (m).

Figure 6 shows that, for an unconfined aquifer the model of Bezuijen (2001) excellent match the numerical results. For the tunnel in a confined aquifer the agreement is not so good (see Figure 7). The reason is that Eq. (3) is a purely 1D solution and the flow close to the tunnel face has 3D aspects, see Bezuijen & Xu (2018).

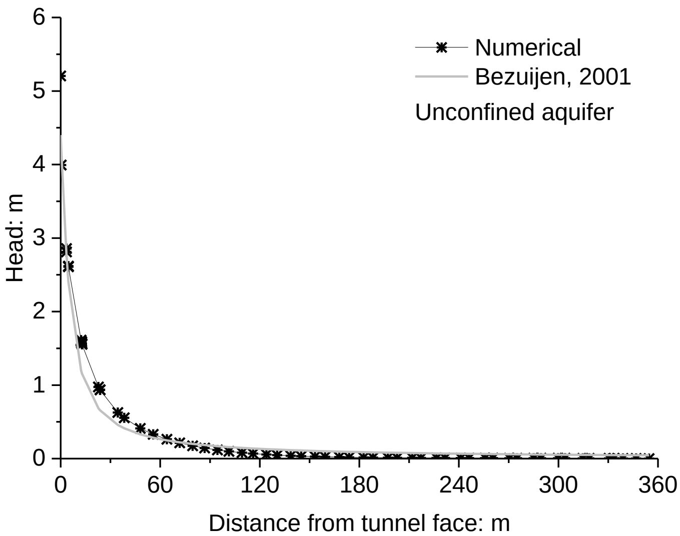


Figure 6. Steady state head change with distance from tunnel face in an unconfined aquifer compared at the axis of the tunnel

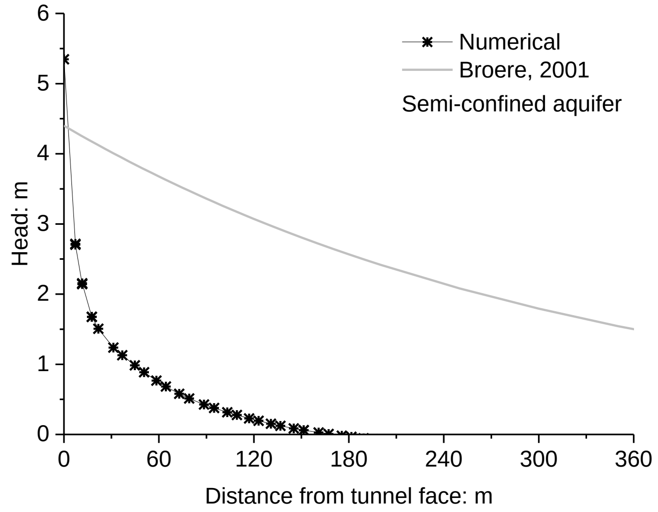


Figure 7. Steady state head change with distance from tunnel face in a semi-confined aquifer compared at the axis of the tunnel

2.3 *Numerical result of displacements*

Vector contour of the ground displacements induced by TBM tunnelling in a semi-confined aquifer can be seen in Figure 8. Neither face collapse nor blow-out potential was observed. The ground surface heave was clear. The maximum displacement was 226.5 mm. Figure 9 shows vector contour of the ground displacements induced by TBM tunnelling in an unconfined aquifer. There was a small blow-out potential at the face, but the maximum displacement around the face was only about 2.5 mm.

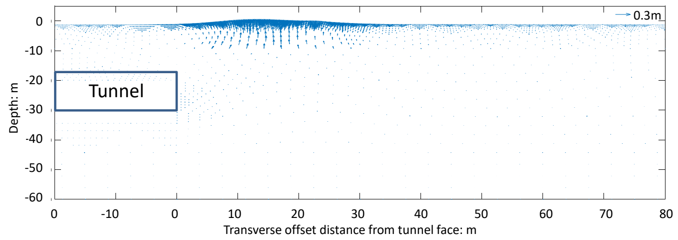


Figure 8. Vector contour of the ground displacement for a semi-confined aquifer

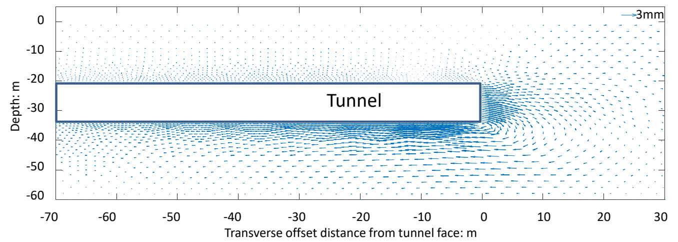


Figure 9. Vector contour of the ground displacement for an unconfined aquifer

3 CONCLUSIONS

A simple FEM modelling was carried out. Though some simplifications and assumptions were made, some new findings are found.

The steady state model proposed by Bezuijen (2001) has been used to describe the measurements from sites in semi-confined aquifers, but no data from unconfined aquifer was used. In this study, the model was validated by the numerical results.

The magnitudes of excess pore water pressure caused by TBM excavation in a semi-confined aquifer are the same at any distance, when the flow reaches the place where is at least three times of tunnel diameter from the face. This is helpful to improve the analytical solution for the excess pore water pressure caused by TBM tunnelling in a semi-confined aquifer proposed by Xu et al. (2019). In that model, the groundwater flow is assumed to be a cylindrically symmetric flow in the horizontal plane, when the flow reaches the place where is tunnel diameter from the face.

The soil displacement around the face may suggest that penetration at the face is positive for the face stability. With proper face support, both collapse and blow-out can be avoided. This also depends on the soil layering condition.

Further work on the effects of such as diameter, grout pressure, excavation process in the ground response to TBM tunnelling with active face support can be made based on but is not the focus of this study.

ACKNOWLEDGEMENT

The research is funded supported by Science and Technology Development Fund, Macao Special Administrative Region of China (File numbers: FDCT/0035/2019/A1 and FDCT/193/2017/A3).

REFERENCES

Anagnostou, G. & Kovári, K. 1994. The face stability of slurry-shield-driven tunnels. *Tunnelling and Underground Space Technology*, 9 (2): 165-174.

Bezuijen, A. & Xu, T. 2018. Excess pore water pressures in front of a tunnel face when drilling in a semiconfined aquifer. ITA-AITES World Tunnel Congress, Dubai, 21-25 April 2018.

Broere, W. 2001. *Tunnel Face Stability & New CPT Applications*. Ph.D. thesis, Delft University of Technology, Delft, The Netherlands.

Broere, W. 2015. On the face support of microtunnelling TBMs. *Tunnelling and Underground Space Technology* 46, 12-17.

COB (Centre for Underground Construction). 2000. *Second Heinenoord tunnel evaluation report*, COB report K100-06. Gouda, The Netherlands: COB.

Kasper, T. & Meschke, G. 2004. A 3D finite element simulation model for TBM tunnelling in soft ground. *International*

Journal of Numerical and Analytical Methods in Geomechanics 28, 1441–1460.
Kim, S. H. & Tonon, F. 2010. Face stability and required support pressure for TBM driven tunnels with ideal face membrane – Drained case. *Tunnelling and Underground Space Technology* 25, 526–542.
Lambrughi, A., Rodríguez, L. M. & Castellanza, R. 2012. Development and validation of a 3D numerical model for TBM–EPB mechanised excavations. *Computers and Geotechnics* 40, 97–113.
Perazzelli, P., Leone, T., Anagnostou, G. 2014. Tunnel face stability under seepage flow conditions. *Tunnelling and Underground Space Technology* 43: 459-469.
Stack, B. 1982. Handbook of Mining and Tunnelling Machinery. Chichester, U.K.: John Wiley & Sons.
Ukritchon, B., Yingchaloenkitkhajorn, K. & Keawsawasvong, S. 2017. Three-dimensional undrained tunnel face stability in clay with a linearly increasing shear strength with depth. *Computers and Geotechnic*s 88, 146-151.
Xu, T. & Bezuijen, A. 2019. Bentonite slurry infiltration in sand, filter cake formation under various conditions. *Géotechnique* 69 (12), 1905-1106.
Xu, T., Bezuijen, A. & Broere, W. 2021. Analytical solutions for groundwater flow caused by TBM tunnelling in a semi-confined aquifer. In preparation.
Zhao, C. Y., Lavasan, A. A., Barciaga, T., Zarev, V., Datcheva, M & Schanz T. 2015. Model validation and calibration via back analysis for mechanized tunnel simulations – The Western Scheldt tunnel case. *Computers and Geotechnic*s 69, 601-614.