Three-dimensional finite-element analysis on ground responses during twin-tunnel construction using the URUP method

Z.X. Zhang ^{a, 1}, C. Liu ^a, X. Huang ^a, C.Y. Kwok ^b and L. Teng^c

^a *Department of Geotechnical Engineering, School of Civil Engineering, Tongji University, Shanghai 200092, China*

^b *Department of Civil Engineering, Haking Wong Building, The University of Hong Kong, Hong Kong, China*

^c *Shanghai Urban Construction(Group) Corporation, Shanghai 200023, China*

Abstract

The paper presents a finite-element analysis of a metro tunnel project using the URUP method in which the shield machine is launched and received at the ground surface level. During the tunnelling process, the cover depth varied from 0.7D (D is the excavation diameter) to -0.3D in which case the shield machine was partially above the ground surface. A three-dimensional finite element model is proposed via the commercial software ABAQUS considering the actual geological condition and tunnelling procedures. Elasto-plasticity constitutive models are utilised for the top three strata in the finite element analysis (FEA). Constant gradients corresponding to material density are assumed for the face supporting pressure and the grouting pressure in the model. The ground contraction method is employed to simulate the shield-induced volume loss. The numerical model is firstly validated against the field measurement data considering the surface settlement. Parametric studies are performed subsequently to investigate the influence of some key tunnelling variables including cover-to-diameter ratio and face supporting pressure on the ground responses. According to the FEA, a critical cover depth of 0.55D is proposed for URUP method

¹ zxzhang@tongji.edu.cn (Zixin Zhang), corresponding author.

below which value instability and collapse of surrounding soils will be highly likely.

Author keywords: URUP method, Full-scale test, Numerical simulation, EPB, Case history.

1 Introduction

As documented in various tunnelling cases, TBM (Tunnel Boring Machine) has become the prevalent choice for tunnelling activities in congested urban areas due to its safe working environment and small disturbance to surrounding ground (Fargnoli et al., 2013; Fargnoli et al., 2015; Liu et al., 2014; Standing and Selemetas, 2013; Wongsaroj et al., 2007). As a predominant style of the mechanised tunnelling method, TBM tunnelling method also owns the merit of largely reducing the excavation volume in comparison to traditional tunnelling approaches, e.g. the open-cut tunnelling method. Nevertheless, launching and receiving shafts still need to be excavated prior to the initiation of conventional TBM tunnelling, which restricts the application of TBM tunnelling in cases with extremely narrow construction space. In order to shorten the construction period and minimise the tunnelling impact, the URUP (Ultra Rapid Under Pass) method was developed in Japan in 2006 (Hayashi et al., 2006; Izawa et al., 2006) and adopted thereafter in multiple tunnelling projects of Oi Area (Fujiki et al., 2009; Nakamura and Hazama, 2010; Zick et al., 2013). As shown in Fig.1, when this method is used, TBM is directly driven into and out of the ground without the need of working shafts (Zhang et al., 2013). This new approach is normally associated with Earth Pressure Balanced (EPB) TBM as the excavation face may be exposed to the atmosphere. Since in the URUP method TBM is normally super-shallowly buried or sometimes even partially above the ground surface, the control of face supporting pressure and grouting pressure during TBM driving is of extreme importance (Nakamura and Hazama, 2010). Two projects in the western part of the Netherlands (i.e. an underpass at Goes and the Rijnlandroute tunnel, respectively) were used to demonstrate the feasibility of the URUP method (Beijer, 2010; Oldenhave, 2014). According to these two case studies, the URUP method is a reasonable option for the construction of both underpass and large diameter tunnel in the western part of the Netherlands, despite that it is economically not competitive compared to the conventional TBM

tunnelling approach.

The underground space has been massively exploited in Eastern China to meet the increasing demand for underground space and transportation in urban areas (Lin et al., 2015; Liu et al., 2014; Min et al., 2015; Ng et al., 2013; Shen et al., 2014; Xu et al., 2015; Zhang and Huang, 2014). The densely-distributed underground and overground infrastructure systems greatly squeeze the construction space and require rigorous ground movement control. Thus, the URUP method becomes a reasonable solution for tunnelling projects under such conditions. Since there is no prior experience of URUP tunnelling in China, a full-scale test using the URUP method was conducted to excavate the twin tunnels between the Moling Station and Jiangjun Ave. Station of the Nanjing Metro Line S1 (NMS1) in China. Comprehensive monitoring programme was designed to investigate ground responses due to UPUP tunnelling work.

The paper presents the results of a three-dimensional finite element analysis (FEA) on the aforementioned full-scale test performed in Nanjing, aiming at evaluating the influence of multiple tunnelling variables on the ground responses by comparing the FEA results with the field measurement data.

2 Project overview and geotechnical conditions

2.1 URUP tunnelling section of Nanjing Metro Line S1

The URUP section investigated in this study is a part of Metro Line S1 of Nanjing, which is the capital of Jiangsu Province located in the eastern part of China. The URUP tunnelling section has a total length of 123.7 m along its eastbound direction and 124.6 m in the westbound direction respectively (Fig. 2). In this section, two twin tunnels spaced by 8.5 m between the central axes were excavated using an EPB shield 7.4 m in length and 2×10^5 kg in weight. The outer diameters of the cutter head and shield body are 6.38 m and 6.34 m respectively. The designed maximum thrust force of the shield machine is 33396 kN. Four grout injection openings are circumferentially

aligned at the shield tail with an interval angle of 90° . Single-layered lining is employed with the outer and inner diameters of 6.2 m and 5.5 m, respectively. Each lining ring is fabricated from precast concrete segments which have a width of 1.2 m and a thickness of 0.35 m.

The shield machine was launched from the entrance pit and driven along the eastbound route on Jun 26, 2012; it reached the shield audit on Oct 10, 2012. The westbound tunnelling was commenced on Nov 11, 2012 and completed on Dec 30, 2012. The TBM was advanced towards the shield audit along the eastbound direction with an up-gradient of 2.8%, while it was driven along the westbound direction with a down-gradient of -2.8%. Marginal/negative cover depths were encountered during the tunnelling along the both routes (Fig.3). Negative cover depth herein refers to the case when part of the TBM is over the ground surface. A negative overburden (OB) tunnelling phase ((i.e. EB-IV in Fig. 3) was encountered in the eastbound tunnelling (i.e. the shield receiving portion). At this stage, supporting pressures at both excavation face and shield tail were controlled carefully to avoid excessive soil deformation which could probably give rise to ground failure. According to the data recorded by the shield machine, during advancement the average tunnelling speed was 5 mm/min for negative OB phase, and 10 ~ 15 mm/min for marginal ones.

2.2 Geotechnical conditions

The Nanjing city, is built on the lower Yangtze River valley with an average altitude of about 12 m (Xia and Han, 2009). Ground profile of the test is illustrated in Fig. 3. Note that “ER” and “WR” in Fig. 3 denote the tunnel rings in eastbound and westbound directions respectively with the number followed indicating the advanced distance from the entrance pit (ER) and shield audit (WR), e.g., ER72 represents the eastbound tunnel ring that is 72×1.2 metres ahead of the entrance pit. The tunnelling work was mainly carried out in made ground (i.e. ①₂) and silty clay (④_{-1b1}), with a small portion performed in completely weathered andesite (i.e. J31-1). Different from the silty clay usually encountered during geotechnical activities in Nanjing, the type involved in this

project can be classified as Nanjing Xiashu loess (Li, 2011). According to previous research, the major concern for loess (i.e., hydro-collapse) is rarely observed in Xiashu loess (Xia and Han, 2009). According to the geotechnical investigation (Li, 2011), soil layers ①₂ and ④_{-1b1} had relatively high compressibility and low shear strength which could significantly influence the deformation and stability of surrounding ground during tunnelling. In addition, the two strata also had high permeability which may cause a quick build-up of excess pore water pressure during the arrival of TBM and a slow consolidation speed post the passage of TBM. Geotechnical properties of presented strata are summarised in Table 1.

3 Field measurements

3.1 Field monitoring programme

A comprehensive field monitoring programme was conducted in the vicinity of the shield audit. Two representative sections characterised by different cover depths were selected (i.e. C-C and E-E in Fig. 4). For each section, three kinds of measurements were performed including ground displacements, earth pressure, and pore water pressure using inclinometers/extensometers, earth pressure cells (EPCs), and pore water pressure piezometers (PWPPs) respectively. As Fig. 5 shows, the inclinometers and extensometers were positioned inside PPC (Poly propylene carbonate) pipes, with the tops fixed at the ground surface for shallow ones (i.e. XE-2) and the bottoms fixed in the stratum J31-2 for others, ensuring that cumulative integration starts from a stationary point. Fig. 6 presents the arrangement of EPCs around tunnel lining rings ER84 and WR22. These EPCs were welded onto reinforcing cages of the segments prior to concrete casting. Instrument cables of the EPCs were encapsulated inside PPC pipes (Poly propylene carbonate). Ring magnets measuring ground settlement were anchored at a spacing of 1.0 m along corresponding PPCs. Initial elevations of the PPC pipe tops were determined based on the monitoring data recorded 30 days after the instrumentation. In addition, five surface settlement points (SSPs) were placed at ER34 at a spacing

of 6 m (see Fig. 4). It should be clarified that herein only the ground deformation during the URUP tunnelling process is analysed and compared to the FEA results.

Conditioning of soil ahead of the tunnel face is always performed during shield tunnelling process. This kind of treatment is able to provide a “pulpy” supporting medium made of soil, additives, foam, and polymers, etc. (Vinai et al., 2008). As for the URUP tunnelling process, continuous transmission of the excavated soil should be maintained under a low (even zero) pressure inside the excavation chamber. Therefore, a series of experiments were carried out on soil conditioning prior to tunnel construction, with the purpose of obtaining the most proper ingredients of the conditioned soil inside the excavation chamber. According to the experiment results, a mixture, which consisted of additive (4.1%) and water (95.9%), was adopted as the most appropriate soil conditioning agent for the URUP tunnelling. The slump of the conditioned soil was about 10 cm. Isolation piles were pre-installed between eastbound and westbound routes (as shown in Fig. 4), aiming at reducing the influence of westbound tunnelling on the eastbound tunnel.

3.2 Effectiveness of tail grouting during tunnelling with OBs of $-0.3D$ and $0D$

According to the field data, multiple EPCs “survived” after segment installation, making it possible to investigate the grouting pressure distribution around the outer surface of the tunnel lining under different OBs. Several studies have been focused on the effectiveness of tail grouting (Bezuijen et al., 2004; Dai, 2010; Nagel and Meschke, 2011) during the conventional shield tunnelling. By analysing the field measurement data of the Sophia rail tunnel, Bezuijen et al. (2004) concluded that the grouting-induced vertical grouting pressure gradient at the tail void was not constant with the magnitudes varying between the two values respectively determined by the water pressure and grout hydrostatic pressure. The maximum gradient, which corresponded to the unit weight of the grout material, was never reached owing to the presence of the buoyancy forces. According to the measurement results in the present study, the grouting pressure gradient was much

lower than the value determined by the water pressure during tunnelling with $OB = 0.3D \sim 0.5D$, probably attributable to the excessive pumping pressure of the two upper injection openings.

Fig. 7 illustrates the grouting pressure variation and distribution at ER84 and WR22 during the URUP tunnelling. Only the two lower grouting openings are activated with a grouting ratio (i.e., the ratio of the grout volume to the tail-void volume (Chen et al., 2011)) between 100% and 120%. It should be noted that ER84-T4 was out of order when the shield tail was >1.2 m ahead. The backfill process can be revealed by the grouting pressure variations at the installed EPCs. For the three normally-working EPCs, i.e., ER84-T5, ER84-T6 and ER84-T7, despite notable fluctuations the grouting pressures generally increase before reaching almost constant values when the shield tail has advanced 5 rings (6.0 m) forward. The hydrostatic pressure envelope of the grout mortar with a gradient of 19.5 kN/m is superimposed in Fig. 7 (a) (black dashed line). It can be observed that the measured final grouting pressure values collapse to the hydrostatic pressure envelope, indicating a good quality of grouting control. The EPCs in the vicinity of the working grouting openings exhibited more pronounced grouting pressure changes than other ones. It is worth noting that grouting pressure at ER84-T3 dropped to zero while the shield tail was 6.0 m away. This might be attributed to the grout overflow induced by the large deformation of the slope between two tunnel routines adjacent to ER84 (see Fig. 3). As shown in Fig. 7 (b), the backfilling process can be reflected by the grouting pressure changes at WR22-T8 (westbound tunnelling). As highlighted by the green ellipse in Fig. 7 (b), the grouting pressure at WR22-T8 reached its stable value of about 28 kPa 28 hours after the injection, which is close to the initial hardening time (about 24 hours) of the adopted grout material. The stable value agrees with the local geostatic stress; therefore, it is reasonable to employ a grouting pressure distribution during zero-cover tunnelling phase in the FEA which will be discussed in the following sections.

4 Three-dimensional FEA analysis of the URUP tunnelling process

4.1 Numerical model

A three-dimensional nonlinear FEA, consisting of 100,680 nodes and 84,869 elements in the mesh is conducted to investigate the entire URUP tunnelling process using the general purpose finite element program ABAQUS (Hibbitt, 2012). According to previous research on FEA for tunnelling (Liu et al., 2014), finite strain FEA does not present obviously better performance than small strain FEA in predicting tunnelling-induced soil deformation. Therefore, considering the computational cost and divergence possibility, small strain FEA is employed using ABAQUS standard with the “Soils” step for coupling pore water flow and soil solid skeleton deformation.

One of the most important components in FEA on tunnelling process is the simulation of shield machine, which accounts for the possible shield-driven-induced disturbance to the ground. As a matter of fact, apart from the kinematic behaviour (e.g. advancing and rotating), the geometry, especially the conicity of shield machine, will have practically considerable impact on the ground. In recent FEAs for tunnel construction, the shield machine is usually considered in two ways: (i) using deformable elements (Fagnoli et al., 2015; Liu et al., 2014; Pang et al., 2005) or rigid body with certain conicity (Kasper and Meschke, 2004) in conjunction with artificially-increased self-weight in some cases (Liu et al., 2014); (ii) the displacement method, namely the “fictitious” shield method (Do et al., 2014; Mollon et al., 2013), in which displacement conditions are applied upon the nodes of the shield skin (Shen, 2009) or surrounding ground (Mollon et al., 2013). Previous studies revealed that the deformability of the shield machine might not be negligible (Festa, 2015). However, for shield simulation using representative elements, the computational cost might become unaffordably significant especially when every detail is taken into consideration, e.g. the shield machine components, the material anisotropy and the shield-ground interaction. Hence, certain reasonable simplifications were normally adopted in many tunnelling FEAs. In the current

study, the “fictitious” shield method is employed without considering shield deformation.

The difference between the diameter of cutter head and that of shield body is the major source of ‘overcutting’ during shield tunnelling. Other factors such as the curved shield-driven process will also contribute to overall overcutting. As shown in Fig. 8, the maximum annular void space between the shield and surrounding soil is 2 cm in the current case. This void space can be reduced by multiple factors such as forward penetration of the grout mortar at the shield tail. According to the field measurement data, the excavation-induced volume loss is much smaller than the theoretical prediction made from the shield geometry. The maximum volume loss in the URUP tunnelling was 0.2%, which is in accordance with the average level of volume loss with respect to previous tunnelling practices in this area. In order to investigate the influence induced by different tunnelling variables, e.g. the face supporting pressure and the grouting pressure, the ground contraction around the shield machine is evaluated in the first place using a benchmark case (Case 0, corresponding to the section of ER34). Both uniform and non-uniform (refer to Fig. 9 (b)) ground convergence patterns have been considered in the FEA. For the adopted non-uniformly distributed ground contraction, the maximum displacement (i.e. δ in Fig. 9 (b)) occurred at the tunnel crown and the “contracting centre” coincides with the shield centre.

The face supporting pressure was configured according to the lithostatic condition at the tunnel face. Additives were injected into the soils inside the excavation chamber to get better fluidity and homogeneity of the excavated soil. Different from slurry shield tunnelling, distribution of face pressure of EPB tunnelling over the excavation face does not follow a precisely linear relationship with a certain gradient. Nevertheless, a linear distribution of face pressure related to the lateral earth pressure at rest is a fair assumption for simplicity as shown in Fig. 9 (a). The face supporting pressure at the tunnel centre-level can be calculated roughly by:

$$\sigma_t = K_0 \gamma H \quad (1)$$

where σ_r is the face supporting pressure at the tunnel centre axis, H is the depth of the tunnel centre, γ is the average unit weight of the upper soils, and K_0 is the lateral earth pressure coefficient that is determined by the stress state in the ground ahead of the tunnel face. Liu et al. (2014) adopted a K_0 of 0.7 in the case study of West Changjiang Rd. Tunnel in Shanghai where muddy clay is frequently encountered. According to the inherent properties of the clay involved in this project, a K_0 value of 0.4, corresponding to the coefficient of the earth pressure at rest, is employed in the benchmark FEA. This value also agrees with the field data recorded by the EPCs in the excavation chamber. A group of basic face supporting pressures (under different covers) was obtained from the sensors instrumented inside the excavation chamber (refer to Table 3). A gradient of 7 kN/m^3 is applied for face pressure in the benchmark case (Case 0) according to the field observation.

Grouting pressure is an important factor affecting the ground deformation during the shield tail passage. Grouting pressure distribution in the tail void cannot be reflected directly by field measurement, unless EPCs are pre-installed at the outer surface of the tunnel lining. However, due to the low survival rate of EPCs after segment installation, this kind of data with high quality and enough integrity are rarely obtained. As discussed in section 3.2, it is reasonable to assume a linear distribution of grouting pressure along the depth. Considering that grouting pressure gradient is not a constant prior to complete grout hardening, multiple gradient values are employed in the FEA to examine the induced discrepancy.

The FEA model has a dimension of 108.0 m (length) $\times$ 59.6 m (width) $\times$ 40 m (depth) (see Fig. 10) with the excavation region being far away from the set boundaries. The model is comprised of 75,373 C3D8P elements (8-node trilinear displacement and pore pressure) for involved strata, 4,864 C3D8I elements (8-node linear brick, incompatible modes) for tunnel linings, and 4,592 C3D8I elements for grouting layers. The horizontal displacement of the four vertical

boundaries was constrained while they were allowed to move freely vertically. Furthermore, the model's base boundary was fully fixed in both horizontal and vertical directions. The pore water pressure boundary was fixed to be zero at the ground surface.

4.2 Material properties

The soil mass was assumed to behave as an elasto-plastic material with the combination of linear elasticity and linear Drucker-Prager (DP) yield criterion (Drucker and Prager, 1952). DP model is popular in FEA of excavation when limited data of associated soil strata are available. The model considers the influence of the intermediate principle stress and thus is widely used in simulating soils and rocks (Karakus and Fowell, 2003). In addition, compared with Mohr-Coulomb model, DP model is computationally more efficient. The yield surface of DP models can be expressed in terms of three stress invariants:

$$p = -\frac{1}{3}\boldsymbol{\sigma}:\mathbf{I} \quad (2)$$

$$q = \sqrt{\frac{3}{2}\mathbf{S}:\mathbf{S}} \quad (3)$$

$$r^3 = \frac{9}{2}\mathbf{S}:\mathbf{S}:\mathbf{S} \quad (4)$$

where p is the mean (hydrostatic) stress, q is the equivalent (von Mises) stress, and r is the third invariant of deviatoric stress tensor $\mathbf{S}$ which can be written as:

$$\mathbf{S} = \boldsymbol{\sigma} + p\mathbf{I} \quad (5)$$

where $\mathbf{I}$ is a diagonal unit matrix. The linear DP criterion is therefore expressed as:

$$F = t - p \tan \beta - d = 0 \quad (6)$$

$$t = \frac{1}{2}q \left[1 + \frac{1}{K} - \left(1 - \frac{1}{K} \right) \left(\frac{r}{q} \right)^3 \right] \quad (7)$$

where t is a deviatoric stress measure, d and β are material constants which can be obtained

respectively from Eq. 8 and Eq. 9 based on Mohr-Coulomb's material constants c (cohesion) and φ (internal friction angle) for three-dimensional problem, while the flow stress ratio K can be determined using Eq. 10.

$$\tan \beta = \frac{6 \sin \varphi}{3 - \sin \varphi} \quad (8)$$

$$d = 2c \frac{\cos \varphi}{1 - \sin \varphi} \quad (9)$$

$$K = \frac{3 - \sin \varphi}{3 + \sin \varphi} \quad (10)$$

Since strata J31-2 and J31-3 are not located in the vicinity of the tunnel, their plastic behaviour is ignored and the linear elastic model is employed. The tunnel lining as well as the grout layer is also assumed to behave as a linear elastic material. It is noteworthy that the grout hardening process is not taken into consideration to reduce uncertain factors (e.g. property variation during hardening); thus the progressive development of soil movement may not be accurately captured. Detailed material properties used in the FEA of the URUP tunnelling are listed in Table 2.

4.3 Modelling procedure

A typical “step-by-step” approach is adopted to simulate the URUP tunnelling process, incorporating with the “element death” approach which is widely employed in FEA of excavation problems (Liu et al., 2014). As shown in Fig. 11, the following modelling sequence is followed:

- (i) Initially, the geostatic stress in the soil mass is configured, making the initial displacement in the ground smaller than 1E-6 m.
- (ii) Thereafter, the elements of the first 7 slices (each slice is of one ring width of 1.2 m) in EB route which approximately equal to the length of the TBM shield are deleted simultaneously to simulate the first boring of TBM into the ground. Non-uniformly distributed displacement boundaries are applied on soil nodes in the perimeter of the

tunnelling space to account for the volume loss induced by the shield machine. It should be clarified that the nodes at the shield tail (highlighted by the blue line in Fig. 11 (a)) are freed to enable the occurrence of grout forward-penetration. The face supporting pressure and the grouting pressure are activated at the front face and shield tail respectively, as marked by the yellow arrows in Fig. 11 (a).

- (iii) In the sequential excavation, at each excavation step one slice of soil elements is deactivated in the model and the corresponding displacement boundaries are activated/deactivated as illustrated in Fig. 11 (b); grouting pressure is applied on surrounding ground within a length of two slices behind the shield tail. One slice of lining and grouting layer is activated in the model as shown in Fig. 11 (c).

5 FEA Results and discussions

5.1 Benchmark cases for tunnelling variables

Benchmark simulations for tunnelling variables using the proposed numerical model need to be performed prior to FEA parametric studies of the URUP tunnelling process. This is achieved by the comparison between the field measurements and FEA results. Surface settlement, which directly reflects the real-time tunnelling condition, is selected as the reference to examine the reliability of the FEA results. Peck R (1969) proposed that the Gaussian curve family can be used to empirically evaluate the surface settlement:

$$S(x) = S_{\max} \exp\left(-\frac{x^2}{2i^2}\right) \quad (11)$$

$$S_{\max} = \frac{V_s}{i\sqrt{2\pi}} \quad (12)$$

$$V_s = V_L \pi R^2 \quad (13)$$

where $S(x)$ denotes the ground surface settlement, $S_{\max}$ is the maximum greenfield vertical

surface displacement above the tunnel centre-line, x is the horizontal offset from the tunnel centre-line, i represents the transverse distance between the tunnel centre-line and the inflexion point of the settlement trough, while V_s , V_L , and R stand for the volume of the settlement trough (per meter length of tunnel), the volume loss, and the outer radius of the shield machine, respectively. According to Mair et al. (1993), the trough width i can be estimated by (assuming no volumetric changes with depth) Eq. 14,

$$i = k(z_0 - z) \quad (14)$$

$$k = \frac{0.175 + 0.325(1 - z/z_0)}{1 - z/z_0} \quad (15)$$

where z is the depth from the ground surface at which the Gaussian curve is required, z_0 is the depth of the tunnel centre-line. Based on the Gaussian fit of the measured data in section ER34 (Fig. 12), the trough width is equal to 3.7 m with $k = 0.47$ which agrees well with the prediction by Eq. (15).

Figure 12 compares the surface settlement troughs at face arrival and tail passage obtained by the FEA with those from field measurement for Case 0 (see Table 4). The surface settlement troughs predicted by Eq. 11 are also overlaid in Figure 12 for comparison. It is clear that in both cases the FEA results agree with both the field measurements and empirical prediction. Comparing with the data of FEA and field measurement, the empirical prediction tends to provide a narrower settlement trough during shield approaching. This might be due to the assumption of constant soil volume, which is not the case in reality, especially during the excavation at the tunnel face. The FEA overestimates the maximum settlement (-1.2 mm) comparing to the monitoring data (-1.05 mm). The volume loss at face arrival obtained from the FEA is 0.06%, which is higher than the volume loss predicted by Gaussian curve 0.03%. As depicted in Fig. 12 (b), the FEA and Gaussian

curve provide similar settlement troughs at the tunnelling stage of tail-passing which are almost identical to the field data. Furthermore, the predicted volume loss of FEA is 0.12%, which is close to that of Gaussian curve (i.e. 0.11%).

Fig. 13 illustrates the lateral displacement at XE-1 and XC-1 when the shield tail passed by. As can be observed in Fig.13, despite some minor discrepancies the FEA results of Case 0 generally agree with the field data. For the tunnelling phase with $OB = 0.5D$ (Fig. 13(a)), the maximum lateral displacement occurred at the location 2 m above the tunnel centre axis in both the FEA and field measurement. The FEA tends to underestimate the lateral displacement above the tunnel bottom level, probably owing to the simplifications in soil properties. The FEA results deviate from the monitoring data below the level of tunnel bottom: the ground moves towards the tunnelling space in the FEA, while the opposite tendency can be observed in the field data. This might be caused by the overestimation of soil upheaval at the tunnel bottom after the tail passage in the FEA due to the idealization of the subgrade layer J31-2 as a linear elastic material. Nevertheless, this disagreement is acceptable since that the lateral displacement above the tunnel bottom is of more interest during the URUP tunnelling. As presented in Fig. 13(b), the distribution of lateral displacement along the vertical direction for the tunnelling process with a zero OB (e.g. shield-driven across section C-C) is different from that for the tunnelling process with a marginal OB in which the maximum lateral displacement is observed at the ground surface due to the lack of constrain from upper soils.

The aforementioned analysis shows that the current finite element model is able to capture the ground response of the real project. Therefore, Case 0 (details in Table 4) is selected as the reference case to investigate the influence of different tunnelling variables.

5.2 Evolution of surface settlement under different C/D

Although in modern shield tunnelling activities the face pressure and grouting pressure can be

well controlled, shield passage may still lead to unexpected impact to the surrounding ground, owing to the interaction between the shield machine and soil mass. For mechanised tunnelling projects with ultra-shallow depth, the surface settlement is even more sensitive to disturbance caused by the shield machine. The settlement induced by the shield-passing process $\Delta S_{\max,s}$ can be quantified by the difference between surface settlements before and after tail passage using Eq. 16:

$$\Delta S_{\max,s} = S_{\max,f} - S_{\max,t} \quad (16)$$

where $S_{\max,f}$ and $S_{\max,t}$ are the settlements at face-approaching and tail-passing instants respectively.

As shown in Fig. 14 (a), for the receiving portion (i.e. eastbound) in which the shield is driven out of the ground, the surface settlement decreases with the reducing C/D, owing to the sufficient face supporting pressure comparing to the local lateral earth pressure. Regarding the face approaching process during the launching portion (i.e. westbound), $S_{\max}$ exhibits an evident development (from -0.61 mm to -1.139 mm for $S_{\max,f}$ and from -2.9 mm to -3.8 mm for $S_{\max,t}$) with decreasing OB before OB reaches 0.55D. This may be caused by shield-induced stress relief when the shield passes the region with OB smaller than 0.55D. In this case, the soil above the tunnel crown is not able to form an arching-like structure and thus experiences more significant deformation. As shown in Fig. 14 (a), $S_{\max}$ varies linearly with OB when OB is greater than 0.55D.

The relationship between $\Delta S_{\max,s}$ and C/D (refer to Fig. 9) is plotted in Fig. 14 (b) for the benchmark case (case 0). According to the FEA results (Case 0), a general trend that $\Delta S_{\max,s}$ increases (in terms of the absolute value, similarly hereinafter) with the reducing cover depth (C/D) is evident for both eastbound and westbound routes. In both cases, an approximately linear variation of $\Delta S_{\max,s}$ with C/D is observed when C/D is greater than 0.55D. For tunnelling under a

cover depth smaller than this value, abrupt changes in $\Delta S_{\max,s}$ are present. Thus, an OB about 0.55D can be inferred as the critical cover depth for the URUP tunnelling under the prescribed geological conditions. When the cover depth is below this value, it is recommended that the tunnelling variables (particularly the driving speed, the face pressure and the grouting pressure) should be very carefully controlled.

5.3 Influence of different patterns of ground contraction around the shield machine

As mentioned above, two typical ground contraction modes (uniform and non-uniform) are usually adopted to simulate the effects of volume loss in the analysis of tunnelling process. The non-uniform contraction mode is determined using Eq. 17 according to the geometric relationship shown in Fig. 9:

$$\delta = \frac{\delta_{\max}}{2(R - R_0)} \left[\sqrt{R^2 - 2R(R - R_0)\cos(\theta + \pi/2) + (R - R_0)^2} - R_0 \right] \quad (17)$$

where δ is the assumed contraction around the shield, $\delta_{\max}$ is the contraction at the tunnel crown. R and R_0 are the radii of the maximum excavation and shield body respectively and θ is the angle of the polar coordinate system with the origin located at the centre of the excavation region.

Case 3 is employed with a uniformly distributed contraction ($\delta = 6$ mm) pre-defined at the surrounding ground during the shield passage. The transversal displacements at XE-1 computed by Case 0 and Case 3 are plotted in Fig. 15, together with the corresponding measured data. As can be observed, both of the two cases yield a distribution of the lateral movement that is similar to the measured data. Case 3 which assumes a uniformly-distributed contraction displacement underestimates the maximum lateral displacements at both of the tunnel axis level and ground surface. The maximum inward displacement computed by Case 3 locates at the tunnel centre level, slightly below that of Case 0 and field measurement. Both of Case 0 and measured data indicate a reversal of the lateral displacement at the depth of about 2.5 m, probably owing to the excessive

soil movement towards the tunnel crown. However, this phenomenon is not present in Case 3 due to the assumption of uniform ground contraction. Fig. 16 shows the comparison of the settlement troughs (ER34) computed from Case 0 and Case 3. It is obvious that the ground contraction mode has limited influence on the settlement trough induced by the tail-passing. The FEA of Case 3 shows a $S_{\max}$ slightly greater than that of Case 0 and the measured data.

5.4 Influence of face supporting pressure on surrounding ground

Although face supporting pressure is usually determined according to the geotechnical properties of local soil mass, the actual value fluctuates continuously during the shield advancement. The disturbance caused by the insufficient/excessive face supporting pressure can be reflected by the soil movement ahead of the tunnel face. Three cases (i.e. Case 0, Case 1 and Case 2 in Table 4) with different supporting pressure at the tunnel centre level are employed to investigate the influence of face supporting pressure on the soil responses. Figure 17 illustrates the longitudinal displacement measured and computed at XE-1 (tunnel shoulder) and XE-2 (tunnel crown). It should be noted that positive displacement in the figure stands for movement departing away from the tunnel face. According to the field measurement, soil above the shield (XE-2) tended to move towards the opposite direction of the shield advancement (Figure 17(a)). This was probably caused by the active stress state behind the cutting face. It is noteworthy that only the data of Case 1 shows the similar distribution along the depth. For the other two cases, including Case 0 which is selected as the benchmark case, the soil ahead of the tunnel face moved forward as a result of excessive face supporting pressure, particularly in the upper part of the tunnel face. As shown in Fig. 17, the measured data indicates insufficient supporting pressure below the level of the tunnel axis, whereas the computed results of Case 0 and Case 2 do not exhibit a similar trend. Therefore, it can be inferred that the supporting pressures in Case 0 and Case 2 are excessive above the tunnel axis, presumably owing to the high gradient adopted in the FEA of the tunnelling across this section (E-

E). This could be possible since the supporting material in the excavation chamber is a mixture of excavated soil and prepared slurry. The slurry is supposed to be injected into the excavation chamber to maintain the fluidity and homogeneity of the supporting material. It should be noted that for XE-2, in spite of the difference in the distribution shape of longitudinal displacement, Case 0 gets the closest agreement with the measured data in magnitude especially at the location 1m above the tunnel axis. Figure 17 (b) shows that a K_0 value of 0.4 (i.e., Case 0) also gives the best fitting of the FEA results to the measure data at the location XE-1.

As shown in Fig. 18, $S_{\max}$ is plotted against K_0 under different values of C/D. The westbound tunnelling is not considered in this analysis due to the existence of eastbound tunnel which makes the westbound tunnelling different from greenfield condition. As illustrated in Figure 18, $S_{\max}$ in Case 1 experienced an abrupt decrease from C/D = 0.58 to C/D = 0.51, which is favourable in the tunnelling practices. For Case 0, $S_{\max}$ varies as shown in Fig. 14. It is noteworthy that for Case 2, the $S_{\max}$ does not change with the value of C/D, indicating that the soil ahead of the tunnel face is well supported under this supporting pressure distribution.

6 Conclusions

The paper presents a case history study of a metro tunnel project using the URUP tunnelling method, in which the shield machine was launched and received at the ground surface. Field monitoring programme was carried out to investigate the ground responses to the URUP tunnelling process. A complete three-dimensional finite element model was employed to investigate the ground responses during the construction of twin tunnels. Ground contraction method was utilised to simulate the tunnelling-induced ground loss. Both of the uniform and non-uniform ground contraction models were examined. Face supporting pressure, as well as grouting pressure, was considered in the model. The numerical model was initially validated by comparing the numerical

results with the field measurement data considering the ground surface settlement for a benchmark case. The numerical model was then exploited to investigate the influence of different tunnelling variables on the ground responses. Based on the presented analysis, the following conclusions can be drawn:

(i) The earth pressure recorded by EPCs installed on the tunnel lining could generally reflect the backfill process of the grout mortar. Based on existing research and the measured data in the paper, the grouting pressure gradient in the tail void cannot reach the value corresponding to the unit weight of the grout material. The grouting process could be simplified as a non-uniformly distributed pressure within a length of two rings (2.4 m) behind the shield tail.

(ii) Six cases with different tunnelling variables were selected to investigate the soil displacement during the URUP tunnelling process under different cover depths. As can be drawn from the FEA using the benchmark case which was in close agreement with the measured data, the surface settlement trough at shield approaching could not be well-described by the empirical method using Gaussian curve ($k = 0.47$). However, the Gaussian curve got satisfactory agreement with the settlement trough at tail passage.

(iii) The maximum settlement induced by the shield-passing process generally increased linearly with the reducing C/D when C/D was greater than 0.55. For tunnelling with a C/D smaller than 0.55, an abrupt increase of maximum settlement was observed. Thus, a critical cover depth can be inferred to be 0.55D.

(iv) As expected, the FEA results on transversal displacement at the two selected sections agreed well with the measured data. However, the longitudinal displacement did not get satisfactory fitting by FEA, owing to the unpredictable gradient of the supporting material at the tunnel face. The adopted distribution of the face supporting pressure in the benchmark case tended to be slightly excessive above the level of the tunnel axis. Nonetheless, the magnitude of the longitudinal

displacement computed by the FEA was still acceptable in comparison with the measured data.

(v) As for the URUP tunnelling method, the ground responses to the tunnelling activities were relatively small, comparing to other metro tunnel projects in the same area. This is probably owing to the carefully control of the tunnelling variables, including the advancement speed, the face supporting pressure, the grouting pressure, and some temporary treatments for unexpected affairs during the tunnelling process.

It can be concluded that despite certain discrepancies exist between the FEA prediction and the measured data, this study shows that the FEA is an efficient tool to investigate the impact of URUP tunnelling on the surrounding ground. Since the geotechnical properties of the strata encountered in this project are better than those of adjacent areas, e.g. the silty clay in Nanjing downtown and the muddy clay in Shanghai, feasibility of the URUP method in these areas still needs to be carefully examined prior to construction. This can be achieved following the approach developed in the current study.

Acknowledgement

The research was conducted with funding provided by the National Science Foundation of China (Grant No.41372276). The first and second authors would also like to acknowledge the support provided by Shanghai Tunnelling Engineering Co., Ltd...

References

- Beijer, A.J., 2010. Ultra Rapid UnderPass: the adaption of the URUP method for the Netherlands. Delft University of Technology, Delft.
- Bezuijen, A., Talmon, A., Kaalberg, F., Plugge, R., 2004. Field measurements of grout pressures during tunnelling of the Sophia rail tunnel. *Soils and Foundations* 44, 39-48.
- Chen, R.P., Zhu, J., Liu, W., Tang, X.W., 2011. Ground movement induced by parallel EPB tunnels in silty soils. *Tunn Undergr Sp Tech* 26, 163-171.
- Dai, Z., 2010. The Mechanism and Engineering Application Study of Simultaneous Backfill Grouting During Shield Tunneling in Soft soils. Tongji University, Shanghai.
- Do, N.-A., Dias, D., Oreste, P., Djeran-Maigre, I., 2014. Three-dimensional numerical simulation of a mechanized twin tunnels in soft ground. *Tunn Undergr Sp Tech* 42, 40-51.
- Drucker, D.C., Prager, W., 1952. Soil mechanics and plastic analysis or limit design. *Quarterly of applied mathematics* 10.
- Fargnoli, V., Boldini, D., Amorosi, A., 2013. TBM tunnelling-induced settlements in coarse-grained soils: The case of the new Milan underground line 5. *Tunn Undergr Sp Tech* 38, 336-347.
- Fargnoli, V., Gragnano, C.G., Amorosi, A., Boldini, D., 2015. 3D numerical modelling of soil–structure interaction during EPB tunnelling. *Geotechnique* 65, 23-37.
- Festa, D., 2015. On the Interaction between a Tunnel Boring Machine and the Surrounding Soil. Delft University of Technology, Delft.
- Fujiki, H., Nakamura, T., Izawa, M., 2009. URUP (Ultra rapid Under Pass) method - the first Shield Tunneling method for launching and arrival at the ground level, World Tunneling Congress. ITA, Budapest, Hungary, pp. 112–116.
- Hayashi, M., Miki, K., Yokomizo, F., Yoshida, M., Izawa, M., 2006. The demonstration work of URUP method, Proceedings of the 61st Annual Conference of Japan Society of Civil Engineers, Shiga Prefecture, pp. 485-486.
- Hibbitt, K., and Sorensen, Inc., 2012. Abaqus users manual, Version 6.12, Pawtucket, Providence, R.I.
- Izawa, M., Miki, K., Yokomizo, F., Yoshida, M., Hayashi, M., 2006. The outline of the development of

- URUP method, Proceedings of the 61st Annual Conference of Japan Society of Civil Engineers, Shiga Prefecture, pp. 483-484.
- Karakus, M., Fowell, R.J., 2003. Effects of different tunnel face advance excavation on the settlement by FEM. *Tunn Undergr Sp Tech* 18, 513-523.
- Kasper, T., Meschke, G., 2004. A 3D finite element simulation model for TBM tunnelling in soft ground. *Int J Numer Anal Met* 28, 1441-1460.
- Li, Y., 2011. Geotechnical investigation report for Nanjing-Gaochun Intercity Metro project: Moling Rd. Station to Jiangjun Ave. Station in Contract DJ-XX01. Jiangsu Province Transportation Planning and Design Institute Co., Ltd, Nanjing.
- Lin, C., Wu, S., Xia, T., 2015. Design of shield tunnel lining taking fluctuations of river stage into account. *Tunn Undergr Sp Tech* 45, 107-127.
- Liu, C., Zhang, Z.X., Regueiro, R.A., 2014. Pile and pile group response to tunnelling using a large diameter slurry shield – Case study in Shanghai. *Comput Geotech* 59, 21-43.
- Mair, R.J., Taylor, R.N., Bracegirdle, A., 1993. Subsurface settlement profiles above tunnels in clays. *Geotechnique* 43, 315-320.
- Min, F., Zhu, W., Lin, C., Guo, X., 2015. Opening the excavation chamber of the large-diameter size slurry shield: A case study in Nanjing Yangtze River Tunnel in China. *Tunn Undergr Sp Tech* 46, 18-27.
- Mollon, G., Dias, D., Soubra, A.-H., 2013. Probabilistic analyses of tunneling-induced ground movements. *Acta Geotech* 8, 181-199.
- Nagel, F., Meschke, G., 2011. Grout and bentonite flow around a TBM: Computational modeling and simulation-based assessment of influence on surface settlements. *Tunn Undergr Sp Tech* 26, 445-452.
- Nakamura, T., Hazama, H., 2010. URUP (Ultra Rapid under Pass) Method the First Implementation in a Public Project, 5th Civil Engineering Conference in the Asian Region and Australasian Structural Engineering Conference 2010. Engineers Australia, Sydney, pp. 1175-1180.
- Ng, C.W.W., Liu, G.B., Li, Q., 2013. Investigation of the long-term tunnel settlement mechanisms of the first metro line in Shanghai. *Can Geotech J* 50, 674-684.
- Oldenhave, A., 2014. Starting a large diameter TBM from surface: Feasibility study for the Netherlands.

Delft University of Technology, Delft, Netherlands.

- Pang, C.H., Chow, Y.K., Yong, K.Y., 2005. Three-dimensional numerical simulation of tunnel advancement on adjacent pile foundation, *Underground Space Use. Analysis of the Past and Lessons for the Future*, Two Volume Set. Taylor & Francis.
- Peck R, B., 1969. Deep Excavations and Tunneling in Soft Ground, 7th International Conference on Soil Mechanics and Foundation Engineering. Sociedad Mexicana de Mecánica, pp. 225-290.
- Shen, J., 2009. Research and Application on Numerical Simulation Method for Shield Tunneling. Shanghai Jiao Tong University, Shanghai.
- Shen, S., Wu, H., Cui, Y., Yin, Z., 2014. Long-term settlement behaviour of metro tunnels in the soft deposits of Shanghai. *Tunn Undergr Sp Tech* 40, 309-323.
- Standing, J.R., Selemetas, D., 2013. Greenfield ground response to EPBM tunnelling in London Clay. *Geotechnique* 63, 989-1007.
- Vinai, R., Oggeri, C., Peila, D., 2008. Soil conditioning of sand for EPB applications: A laboratory research. *Tunn Undergr Sp Tech* 23, 308-317.
- Wongsaroj, J., Soga, K., Mair, R.J., 2007. Modelling of long-term ground response to tunnelling under St James's Park, London. *Geotechnique* 57, 75-90.
- Xia, J., Han, A.-M., 2009. Cyclic variability in microstructure and physio-mechanical properties of the Xiashu Loess–palaeosol sequence in Nanjing, China. *Engineering Geology* 104, 263-268.
- Xu, Q., Zhu, H., Ma, X., Ma, Z., Li, X., Tang, Z., Zhuo, K., 2015. A case history of shield tunnel crossing through group pile foundation of a road bridge with pile underpinning technologies in Shanghai. *Tunn Undergr Sp Tech* 45, 20-33.
- Zhang, Z., Huang, M., 2014. Geotechnical influence on existing subway tunnels induced by multiline tunneling in Shanghai soft soil. *Comput Geotech* 56, 121-132.
- Zhang, Z.X., Hu, W., Liu, C., Teng, L., 2013. Investigation of ground pass shield tunnelling method based on large-scale model test and its engineering application. *Chinese Journal of Rock Mechanics and Engineering* 32, 2161-2169.
- Zick, P., Sugihara, H., Takatoku, Y., Sada, S., 2013. URUP (Ultra Rapid Underpass) – TBM Excavation in

Soft Ground from the Surface Elevation (Zero Overburden) Without Shaft and/or Large Scaled Open Cut Pit, Transportation Research Board 92nd Annual Meeting. Transportation Research Board, Washington DC, pp. #13-1080.

Figure Captions

Fig. 1. Sketch illustrating the URUP tunnelling process (Zhang et al., 2013).

Fig. 2. Overview of the project (MAPWORLD: <http://en.tianditu.com/map/index.html>; annotations made by the authors).

Fig. 3. Ground profile and monitoring sections (EB: eastbound; WB: westbound; ER: ring number along the eastbound route; WR: ring number along the westbound route;): (a) strata profile along the eastbound route and positions of sections C-C and E-E; and (b) strata profile along the westbound route and positions of sections C-C and E-E.

Fig. 4. Plan view of the field monitoring instrumentation (ER: ring number along the eastbound route; EB: eastbound; WB: westbound; XC: inclinometer at section C-C; XE: inclinometer at section E-E; S34: surface settlement points at ER34).

Fig. 5. Arrangement of the inclinometers in selected sections (ER: ring number along the eastbound route; WR: ring number along the westbound route; XC: inclinometer at section C-C; XE: inclinometer at section E-E): (a) section C-C (OB = 0.1D); and (b) section E-E (OB = 0.5D).

Fig. 6. Arrangement of the EPCs on the outer surface of the tunnel lining (ER: ring number along the eastbound route; WR: ring number along the westbound route): (a) ER84 with OB = -0.3D; and (b) WR22 with OB = 0.

Fig. 7 Grouting pressure variation and distribution: (a) grouting pressure distribution of ER84 during the eastbound tunnelling; and (b) grouting pressure variation of WR22-T8 during the westbound tunnelling.

Fig. 8. Simulation procedure in FEA for the URUP tunnelling (void space between shield and soil is exaggerated to make a clear illustration).

Fig. 9. Face supporting pressure distribution and shield-induced ground contraction: (a) face supporting pressure distribution; and (b) non-uniform ground contraction subjected to shield passage.

Fig. 10. Sketch of the mesh in FEA of the URUP tunnelling process.

Fig. 11. Modelling procedure adopted in FEA.

Fig. 12. Comparing surface settlements obtained from the FEA results (Case 0) with the field measurements and the Gaussian settlement distributions (field data were based on the starting values 30 days measured prior to the face arrival): (a) face arrival; (b) tail passage.

Fig. 13. Lateral displacements during tail-passing obtained from FEA (Case 0) and field data (field data were based on the starting values measured 30 days prior to the face arrival): (a) comparison at XE-1 with OB between $0.5D$ and $0.6D$; and (b) comparison at XC-1 with $OB = 0$.

Fig. 14. Relationship between the maximum settlement and C/D (Case 0 in FEA): (a) the maximum settlement during tunnel face-approaching and tail-passing; and (b) the relative maximum settlement due to shield-passing.

Fig. 15. Comparison between the uniform and non-uniform contraction, via lateral displacement measured at XE-1 during tail-passing (measured data were based on the starting values measured 30 days prior to the face arrival; XE indicates the inclinometer at section E-E).

Fig. 16. Comparison between the uniform and non-uniform contraction, via surface settlement trough during tail-passing.

Fig. 17. Longitudinal displacement measured and computed in section E-E (XE indicates the inclinometer at section E-E): (a) results of XE-2; and (b) results of XE-1.

Fig. 18. Influence of face supporting pressure on surrounding soil mass during the westbound tunnelling.

ER84

WR22

List of tables

Table 1. Geotechnical properties of the strata involved in the full-scale test (Li, 2011)

Stratum	$\gamma : kN/m^3$	$W : \%$	$W_L : \%$	$W_p : \%$	e	$c_q : kPa$	$\varphi_q : \text{deg}$	$k_h : cm/s$	$k_v : cm/s$	$E_s : MPa$	STP (N -Value)	
① ₂	19.4	32.6	42.2	22.6	0.802	16.0	16.4	2E-4 – 4E-6	2E-4 – 5E-6	5.71	6.3	
④ _{-1b1}	19.9	24.0	37.3	19.8	0.701	55.2	11.9	3.3E-6	3.5E-6	8.39	15.0	
J31-1	20.9	15.3	—	—	—	41.0	17.0	2E-4 – 8E-5	2E-4 – 8E-5	9.50	29.6	
J31-2	22.3	1.29	—	—	—	50.0	37.0	5E-4 – 2E-3	2E-4 – 5E-3	19.0	—	
Stratum	$\gamma : kN/m^3$	$f_r : MPa$				$c_q : MPa$	$\varphi_q : \text{deg}$	$k_h : cm/s$	$k_v : cm/s$	$E : MPa$	$v_p : m/s$	
				v_{pr}	v_{pm}							
		In situ	Saturated									
J31-3	25.4	34.59		21.37		1.0	48.8	8.0E-5	8.0E-5	2100	3432	3961

γ : Gravity density; w : Water content; W_L : Liquid limit; W_p : Plastic limit; e : Void ratio; c_q : Cohesion obtained from direct shear tests; ϕ_q : Friction angle obtained from direct shear tests; k_h : Permeability coefficient in the horizontal direction; k_v : Permeability coefficient in the vertical direction; ν : Poisson's ratio; f_r : Uniaxial compressive strength; E_s : Compressive modulus; E : Elastic modulus; v_p : Compressive wave velocity; v_{pr} : Compressive wave velocity of rock; v_{pm} : Compressive wave velocity of rock mass.

Table 2. Material constitutive parameters used in the FEA

Stratum	$E : MPa$	ν	$\beta : \text{deg}$	$d : kPa$	K
① ₂	15	0.30	31.9	42.8	0.828
④ _{-1b1}	25	0.27	23.9	55.2	0.871
J31-1	25	0.25	32.9	110.8	0.822
J31-2	60	0.20			
J31-3	2.1E3	0.21			
Lining	3.55E4	0.18			
Grout	5	0.4			

Table 3. Average face supporting pressure obtained from the field measurement and adopted in the FEA

	EB tunnelling			WB tunnelling		
	OB: 0.5D	OB: 0.1D	OB: -0.3D	OB: 0D	OB: 0.3D	OB: 0.5D
Field	60~70	10~30	0~10	0~20	30~40	60~70
FEA	60	30	0	20	40	60

Note: All of the face supporting pressures are based on the values at the tunnel centre level. The FEA face supporting pressures presented in the table are used in benchmark tests of the model (Case 0; $K_0 = 0.4$). All pressures are in kPa.

Table 4. Parameters used in presented FEA cases

Case	K_0	Grouting pressure at tunnel crown	Grouting pressure gradient (kN/m)	Ground contraction	δ (mm)
0	0.4	Earth pressure	7	non-uniform	6
1	0.2	Earth pressure	7	non-uniform	6
2	0.6	Earth pressure	7	non-uniform	6
3	0.4	Earth pressure	7	Uniform	6