Analysis of face stability and ground settlement in EPB shield tunnelling for the Nanjing metro

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Abstract: Nanjing Metro No.1 line is 16.92 km in total length, of which 6.11 km is above ground level and 10.81 km is underground. It passes through very complicated geological terrain such as: shallow hills, rock layers and ancient floodplains. Shield tunnelling was carried out using an earth pressure balanced (EPB) shield along the line. The EPB shield tunnelling machine bored through variable ground conditions, such as: a shallow embedded clay under water; a watery clay under a buildings; and a shallow rock under a buildings.

The EPB shield behaviour during shield tunnelling may cause deformation of ground surface on the line. To predict and control ground surface deformation observed data was obtained during the EPB shield tunnelling, and a numerical analysis of face stability and ground settlement was conducted. The results for the simulation of shield behaviour show that local collapse of the ground sometimes occurred at the crown part of the shield face. Furthermore, over-excavation caused loosening of the ground around the shield crown and large ground surface settlement. These also caused the release of the ground pressure acting on the shield. This analysis demonstrated an overall agreement between the simulated results and the observed data.

Résumé: La ligne de la métro No.1 de Nanjing est de 16.92 kilomètres dans la longueur totale, de laquelle 6.11 kilomètres sont au-dessus du niveau du sol et 10.81 kilomètres sont souterrains. Il traverse le terrain géologique très compliqué comme : collines peu profondes, couches de roche et floodplains antiques. Le perçage d'un tunnel de bouclier a été effectué à l'aide d'un bouclier équilibré par pression de la terre suivant la ligne. La machine de perçage d'un tunnel de bouclier a alésé par des conditions au sol variables, comme : un argile incorporé peu profond sous l'eau ; un argile aqueux sous bâtiments ; et une roche peu profonde sous bâtiments.

Le comportement de bouclier pendant le perçage d'un tunnel peut causer la déformation de la surface au sol sur la ligne. Prévoir et commander la déformation extérieure au sol ont observé des données ont été obtenus pendant le bouclier de perçant un tunnel, et une analyse numérique de stabilité de visage et de règlement de la terre a été conduite. Les résultats pour la simulation du comportement de bouclier prouvent que local effondrez-vous de la terre s'est parfois produit à la pièce de couronne du visage de bouclier. En outre, l'au-dessus-excavation a causé se desserrer de la terre autour de la couronne de bouclier et du grand règlement extérieur au sol. Ceux-ci ont également causé le dégagement de la pression au sol agissant sur le bouclier. This analysis demonstrated an overall agreement between the simulated results and the observed data.

Keywords: 3D models, elastoplastic materials, finite element, monitoring, plastic deformation, settlement.

INTRODUCTION

Nanjing Metro No.1 line is 16.92 km in total length, of which 6.11 km is above ground level and 10.81 km is underground. During the Spring of 2003, the City of Nanjing supervised a segment of tunnel staring from Zhangfuyue to Sanshanjie, then from Sanshanjie to Zhangfuyue, and then from Zhangfuyue to Xinjieke. The excavated diameter (D) of the tunnel was 6.4 m over a total length of 10.81 km, with an average overburden cover (H) to the tunnel axis of 6.4 m. The tunnel was bored by an earth pressure balance (EPB) shield and the advance rate varied from 6 m to 8 m per day depending on the ground conditions encountered. Details of the construction sequence and the soil conditions are reported by Sun et. al (2005) and Pei (2005).

An extensive field instrumentation programme was devised to monitor the relationship between the maximum vertical settlements and the actual support pressures induced by tunnelling. A lot of surface settlement points were installed to detect the vertical settlement profiles along / across the tunnel axis.

The paper describes the ground conditions encountered and the soil characteristics. Details of numerical analysis are given and data indicating the ground response as the tunnel passed the measured points. The results are analysed for comparison with previously reported ground responses to tunnelling.

GEOLOGICAL CONDITIONS

Almost all of the subsoil in the Nanjing Area was deposited during the Holocene Epoch. The deposits consist largely of material derived from rocks in the vicinity and they are composed primarily of clay and silt. The source deposits consist of large masses of clay from the Lower Cretaceous System, Gecun Stratum (Klg).

Table 1. Parameters of soils for research area

No.	soil type	thickness (m)	Poisson's ratio (υ)	Young's modulus E (MPa)	undrained shear strength C (kPa)	unit weight γ (kN/m)	shear strength	
							C (kPa)	φ (°)
①	artifical soil	4.7	0.34	2.52	4.8	16.4	10.0	20.0
②	clayey soil~silty soil	13.5	0.32	4.34	10.2	19.2	10.0	22.0
③	silty clay~silt	7.4	0.31	4.51	5.7	19.6	34.1	20.0
④	silty clay mixed with gravel cobble	12.4	0.32	4.58	17.2	20.0	55.8	25.0

FINITE ELEMENT ANALYSIS

FEM model

The ANSYS program was used for three-dimensional and axi-symmetric FEM analysis. The finite element mesh consists of 8-node linear brick, reduced integration and hourglass control (H/D = 1) (Figure 1). In addition to the above case, the results obtained for a three-dimensional mesh with hybrid elements are also compared. These elements are primarily intended for use with incompressible and almost incompressible material behaviour.

Soil Parameters

In this paper a type of elastoplastic model was considered: Drucker-Prager model. The Drucker-Prager failure surface is perfectly plastic (no hardening), but plastic flow on this surface produces inelastic volume increase, which causes the cap to soften. The parameters for the Drucker-Prager model are presented in Table 1.

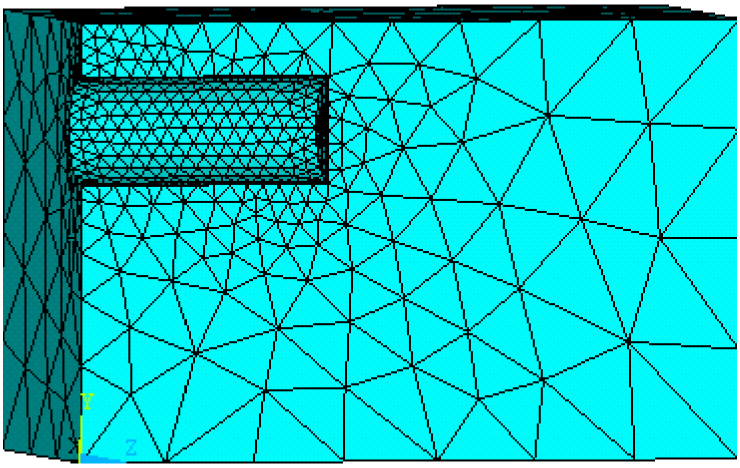

Figure 1. A model of finite element method

Pressure at the face of the shield

A uniform pressure is applied to the exposed face elements. The average pressure level for shield analysis is 114 kPa (2400 psf), Kasali (1982). Finno (1983) applied uniform induced pressure for EPB shield of 71.9 kPa (1200 psf) for the longitudinal analysis, and for the transverse analysis the pressure was 57.5 kPa (1200 psf). An average heaving pressure of 60.7 kPa (1260 psf) was used by Shirasuna (1985) in his research to induce displacements in his model and compare to those observed in the field.

In this paper, the authors used pressures of less than 55.26 kPa, 70.0 kPa, 75.66 kPa and more than 75.66kPa at the face of the shield for FEM analysis.

A face stability index (N)

A face stability index (N) is the ratio between the difference between the natural pressure and the pressure applied to the tunnel face, and the undrained shear strength to analyze tunnel face stability (Broms and Bennermark 1967). The stability ratio is devised based on cohesive ground (clay) and it was found that the tunnel face would be stable when the index is less than six.

$$N = (\sigma_s + \gamma H - \sigma_t)/S_u \quad (1)$$

where γ = unit weight of soil,

H = depth of tunnel axis,
σ_s = surface surcharge,
σ_t = tunnel support pressure and
S_u = undrained shear strength at tunnel axis

Schofield (1980) and Davis et al. (1980) substantiated the claims of N<6 for stability of a tunnel face, and concluded that the stability ratio depends on the depth of cover to diameter (*H*/*D*) ratio. They also concluded that stability ratio (*N*) is equal to between five and seven for a depth of cover to diameter ratio of 1.5. Davis et al. (1980) found that their results estimated from the limit theorems of plasticity yield critical values of *N* that showed a significant variation of the depth of cover to diameter ratio from that stated in Broms and Bennermark (1967). A larger range of *N* for tunnel face stability was given by Kimura and Mair (1981) who carried out tests on reconsolidated clay and verified that the stability of the tunnel face can be confirmed for values of *N* between five and ten, depending on the depth.

During FEM analysis, at first, a geostatic stress was put into Gauss point as an initial stress field (Figure 2), and then FEM numerical simulations were conducted with different pressure at the face of the shield while a face stability index (N) was calculated, respectively.

RESULTS

Plastic deformation

Maximum plastic deformation occurred at the bottom of the excavation face and the face stability index (N) was equal to 6 when the pressure (σ_t) was 70 kPa (Figure 3). When the pressure was greater than or equal to 75.66 kPa, the face stability index (N) was 5.3, and the plastic strain at the front of the shield was greater than 1 with a pressure of 70 kPa (Figures 4 and 5). When the pressure was less than or equal to 55.26 kPa, the index (N) was 7.34 and the plastic deformation was obviously larger than that at 70.0 kPa (Figure 6).

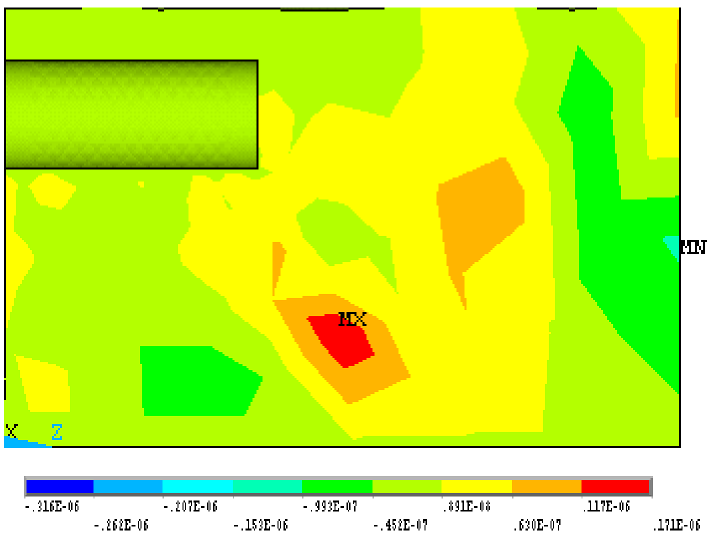

Figure 2. A initial stress field for FEM numerical simulation

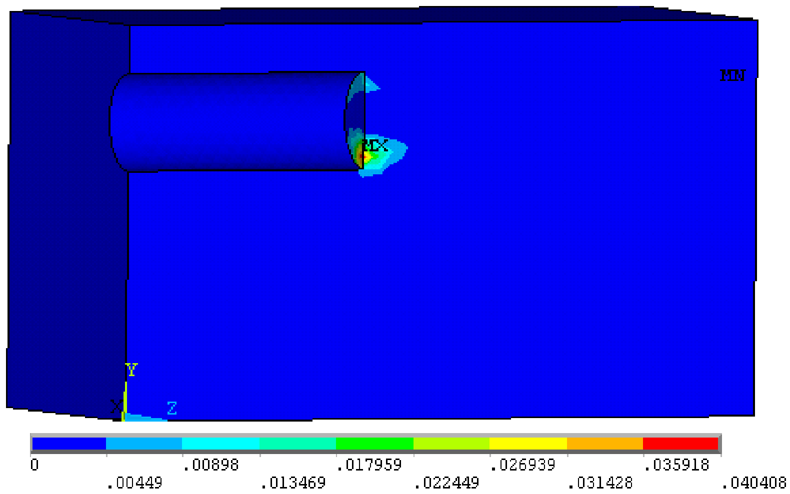

Figure 3. Plastic deformation at $\sigma_t = 70$ kPa

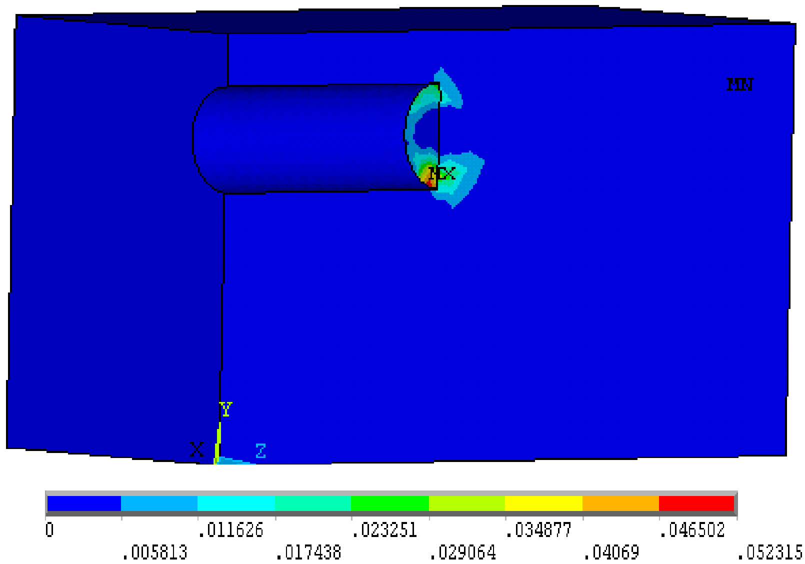

Figure 4. Plastic deformation at $\sigma_t = 75.66$ kPa

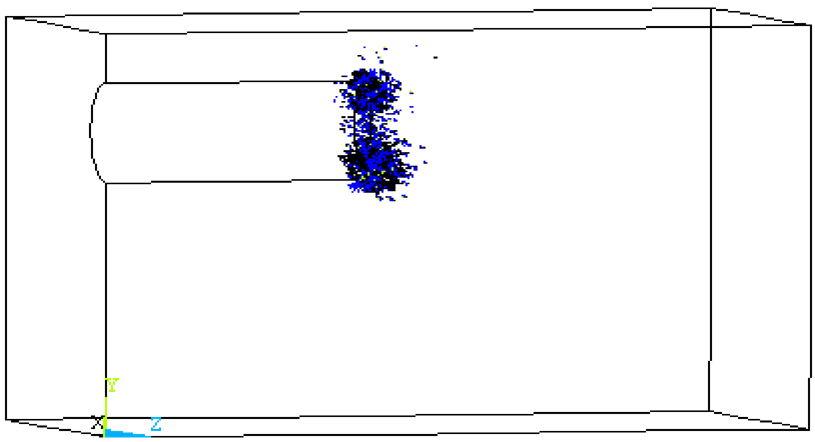

Figure 5. Plastic deformation vector

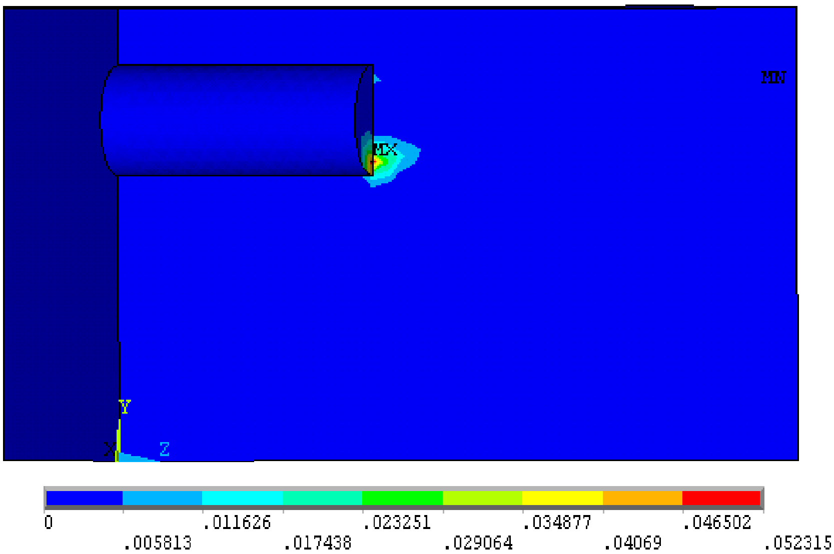

Figure 6. Plastic deformation at σ_t = 55.26 kPa

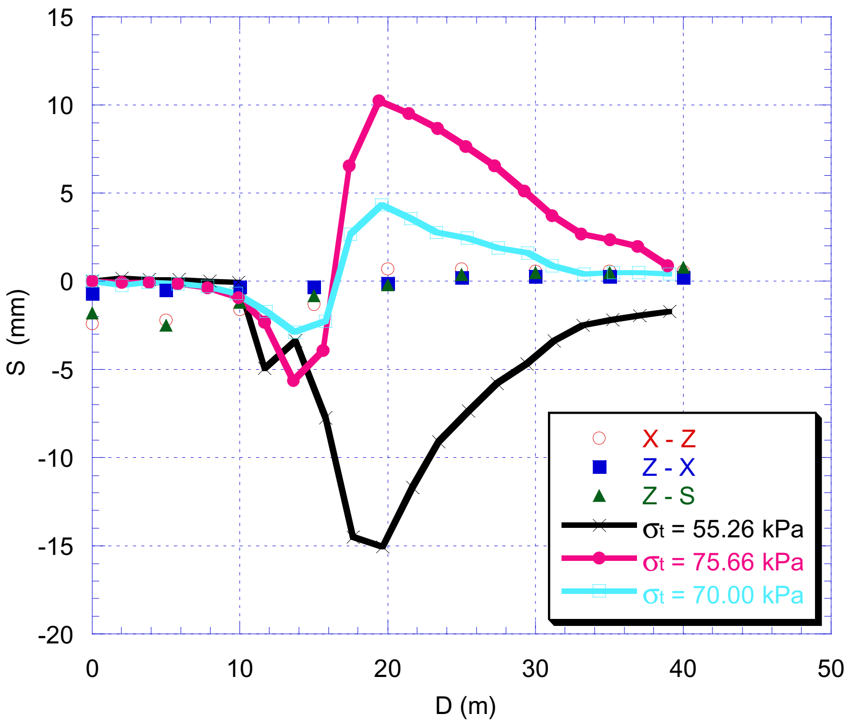

Figure 7. A comparison of the surface settlements profiles calculated and measured from Zhangfuyue to Xinjieke (Z – X), from Zhangfuyue to Sanshanjie (Z – S), and from Xinjieke to Zhangfuyue (X – Z), S--surface settlements (mm), D--distance (m)

Ground settlement

In tunnel design one is also concerned with the damage causing differential settlement, i.e. the gradient at the inflection point and the trough width. Figure 7 presents a comparison of the surface settlement profiles calculated under different σ_t and measured during shield tunnelling from Zhangfuyue to Xinjieke (Z – X), from Zhangfuyue to Sanshanjie (Z – S), and from Xinjieke to Zhangfuyue (X – Z).

CONCLUSION

Earth pressure balanced shields provide continuous support to the tunnel face using the freshly-excavated wet soil, which under pressure completely fills up the work chamber. EPB-shield tunnelling has been successfully applied worldwide in recent years. Under extremely unfavourable geological and hydrogeological conditions, however, face instabilities may occur. The deformation due to shield tunnelling at present is mainly caused by two reasons, namely the deformation occurring when the shield passes through and the subsequent deformation.

Based on the following aspects, the deformation due to shield tunnelling can be predicted in four stages:

1. Three-dimensional FEM (3D-FEM) analysis is conducted using a range of tunnel support pressures σ_t.
2. A face stability index (N) is calculated using equation (1) for each σ_t considered.
3. The tunnel support pressure σ_t is estimated with an elastoplastic axisymmetric FEM analysis taking account of the process of tunnel excavation such as the earth pressure at the cutting face.
4. Based on the predicted plastic deformation, that stability ratio (*N*) is dependent on the depth of cover to diameter (*H*/*D*) ratio, and the smaller σ_t is, N will be larger.

The surface settlement profiles calculated under different σ_t were compared with one measured during shield tunnelling for the Nanjing metro.

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