NUMERICAL MODELLING OF MECHANISED TUNNELLING IN CLAY

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ABSTRACT: Mechanised tunnelling method using Tunnel Boring Machine (TBM) is widely used in urban settings due to the lesser construction duration and minimal surface disruption. However, field monitoring studies report ground movements and damages to the buildings under which the TBM ploughed through. Hence, it is important to attain a detailed understanding of the TBM- ground interaction before the initiation of any new tunnel construction activity so as to predict the variation in the *in- situ* ground conditions, avoid any damage to the structures adjacent to the tunnel alignment and optimize the TBM performance. This paper comprises a brief review of various studies on the simulation of tunnelling in soft soils followed by the numerical back- analysis of a typical field monitoring study involving underground tunnelling in clay using the general purpose finite element suite, ABAQUS. A circular tunnel with 8.16 m diameter, running straight and embedded in clay with 19.0 m overburden is considered. The deformation and pore pressure response is monitored along the ground surface and near the tunnel crown, respectively, for each excavation stage of 1.5 m. The numerical model clearly demonstrated the progressive changes in these parameters as the excavation advances.

Keywords: Mechanised Tunnelling, TBM, Finite Element, ABAQUS, Ground Movement

1 INTRODUCTION AND BACKGROUND

The increasing demand for transportation facilities in the urban set- ups has urged the need for extensive underground networks. Mechanised tunnelling techniques involving Tunnel Boring Machine (TBM) is widely used for building up these networks due to the lesser construction duration and minimal surface disruption. Conversely, several field monitoring studies (Chen et al. 2011, Standing and Selemetas 2013 and Houhou et al. 2016) report ground disturbance and structural damage induced by the mechanised tunnelling methods. Hence, it is important to understand the ground- TBM interaction in order to predict the variation in the *in- situ* ground conditions, avoid structural damage and control the TBM operational parameters to minimise the ground disturbances.

Majority of the researches on underground tunnelling in soft grounds are more focused on the conventional open- face tunnelling concept (Mair et al. 1993, Addenbrooke and Potts 2001, Ng. and Lee 2005, Marshall et al. 2012 and Pinto and Whittle 2014). Kasper and Meschke (2004), Lambrughi et al. (2012), Comodromos et al. (2014), Do et al. (2014) and Zheng et al. (2015), are a few studies discussing the ground deformation and stress transfer mechanism during closed- face tunnelling, which is the most commonly used tunnelling concept nowadays. It was noted that these studies involve essentially numerical techniques like finite difference and finite element methods due to their flexibility in accommodating the TBM as well as ground variables compared to the experimental and analytical approaches. However, attempts of rigorous modelling by incorporating the different TBM components like cutter- head torque, face pressure and grout pressure are still evolving. This research paper describes the numerical modelling procedure for mechanised tunnelling through stiff clay by appropriately accounting for all the TBM components. The results are then compared with an actual field monitoring data in order to demonstrate the efficiency of the numerical model to predict the ground responses as the tunnel advances.

2 DEVELOPMENT OF THE COMPUTATIONAL DOMAIN

In order to investigate the tunnelling induced ground responses, a full three- dimensional model with circular twin-tunnel configuration was developed using the general purpose finite element suite, ABAQUS. Details of tunnelling and site conditions were referred from

Standing and Selemetas (2013). The tunnels are of 8.16 m diameter, running straight with 16.0 m centre- to centre distance and embedded in clay at a depth of 19.0 m. Figure 1 shows the cross- section of the twin- tunnel alignment and the site stratigraphy up to a depth of 35.0 m below the ground surface.

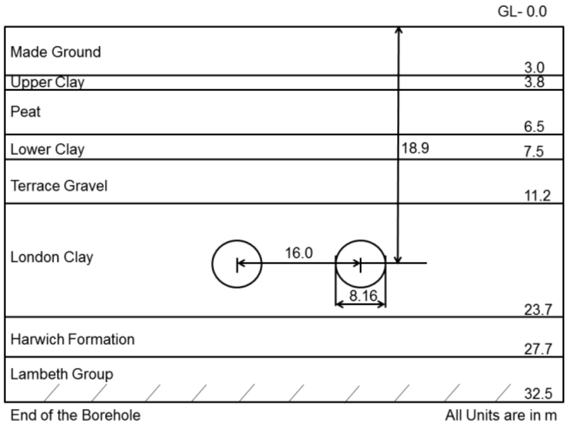

Fig.1 Cross- section of the twin- tunnel alignment

The dimensions of the computational domain was chosen to be H + 12 D along the X- axis (across the direction of tunnelling), H+ 15 D along the Y- axis (along the direction of tunnelling) and H + 6 D along the Z- axis, where, H is the height of the overburden with respect to the tunnel- axis and D is the tunnel diameter.

2.1 Finite Element Mesh

Figure 2 shows the 3 D finite element mesh for the underground tunnel model considered in this study.

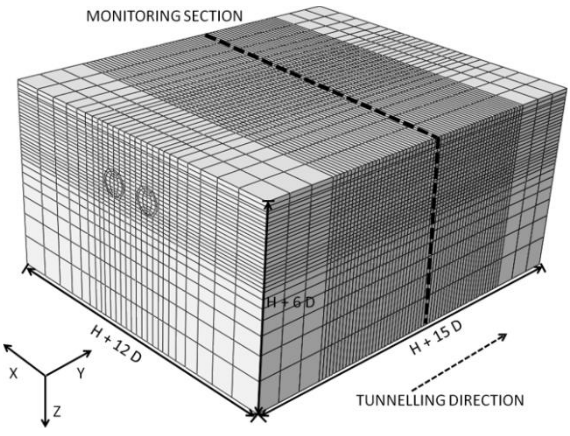

Fig.2 The 3 D finite element mesh for the underground tunnel model considered in this study

It consists of a heterogeneous soil section with eight distinct layers, TBM shield, lining and grout components. C3D8RP (Eight- noded linear brick element with trilinear displacement, pore pressure and reduced integration scheme) was used to discretise soil and grout sections, whereas, S4R (four- noded conventional stress/ displacement shell element with reduced integration scheme) was used for the TBM shield and tunnel lining.

Translations in X and Y directions were constrained along the vertical boundaries of the soil domain in YZ- plane and XZ- plane, respectively. Translations in all directions were constrained along the bottom boundary of the model. All the three rotational degrees of freedom were constrained for the TBM shield and lining.

2.2 Materials and Constitutive Relations

Soil properties used in the analysis are shown in Table 1. Ground water table is assumed at the ground level. TBM shield, grout and tunnel lining material parameters used in the analysis are presented in Table 2.

Table 1 Soil Parameters

Material	**Depth (m)**	**γ (kN/ m^3)**	**E_u (MPa)**	**ν**	**e_0**	**c_u (kPa)**
Made Ground	0- 3.0	18	3	0.49	1.08	20
Upper Clay	3.0- 3.8	15	0.9	0.49	8.1	20
Peat	3.8- 6.5	15	0.45	0.49	9	20
Lower Clay	6.5- 7.5	15	0.9	0.49	1.62	20
Terrace Gravel	7.5- 11.2	20	24	0.2	0.79	-
London Clay	11.2- 23.7	19	54	0.49	0.68	60 + 3z*
Harwich Formation	23.7- 27.7	20	60	0.49	0.68	97.5 + 3z*
Lambeth Group	27.7- 32.5	21	70	0.49	0.68	109.5 + 3z*

*z = depth below top of London Clay

Table 2 TBM Shield, Grout and Tunnel Lining Material Parameters

Material	**Outer Diameter (m)**	**Thickness (m)**	**γ (kN/ m^3)**	**E (GPa)**	**ν**
Shield	8.12	0.1	80	1500	0.13
Grout	8.12	0.155	20	0.5	0.4
Lining	7.81	0.35	25	26.3	0.2

Drucker Prager constitutive relation with a cap is used to model the undrained behaviour of clay deposits. The addition of a cap yield surface helps to control volume dilatancy when the material yields in shear. Drucker Prager constitutive relation without a cap is used to model the terrace gravel. The friction angle is 39 degrees for the Terrace gravel layer. Shield, lining and grout are modelled as simple linear elastic materials with an assumption that elastic strains developed in the material are less than 5 %.

3 MODELLING PROCEDURE FOR MECHANISED TUNNELLING

The *in- situ* stress conditions were initially established with the introduction of gravity load in the soil domain. To eliminate boundary effects, initially tunnel is excavated 25.0 m away from the front boundary in the X- Z plane. Loads/ pressures are applied as summarised in Table 3. Tunnel is excavated for a length of 1.5 m, equivalent to one lining ring width. The step time period is 3.9 hours which is equivalent to the rate of advance of the TBM (0.375 m/ hr). A coupled pore fluid diffusion and stress analysis offered by ABAQUS/ Standard (ABAQUS 2012) was carried out. Accordingly, the porous medium is modelled as a multiphase material considering the presence of two fluids i.e. wetting liquid and gas. The finite element mesh is attached to the solid phase allowing the fluid flow through this mesh. An effective stress principle is used to describe the porous medium behaviour.

Table 3 Loads Considered in the Mechanised Tunnelling Simulation Procedure

Load type	Magnitude*	Region of application
Cutter- head torque	1.12 MN m	Excavation face at every 1.5 m interval
Face pressure	2.0 bar	Excavation face at every 1.5 m interval
Traction from shield- skin	5.54 MN	Excavated soil surface in contact with the shield
Tail grout pressure	1.6 bar	Excavated soil surface after shield removal

*Magnitudes are taken from Standing & Selemetas (2013)

4 RESULTS AND DISCUSSION

Ground response discussed in this section include vertical displacement and pore pressure variation at selected locations, monitored for each excavation stage of 1.5 m during the excavation of the left- tunnel. Results from the finite element study are compared with the field monitoring data reported in Standing and Selemetas (2013). Figure 3 shows the vertical displacement profile on the ground surface along the monitoring section against the distance of the TBM face from the monitoring section as the tunnel advances. The co- ordinates (x, y, z) = (0, 0, 0) corresponds to the location on the ground surface above the centre- line of the left- tunnel lying in the monitoring section. Both the field data and finite element analysis results shows an initial heave (< 1 mm) as the TBM approaches the monitoring section with the distance of face at about y= -20.0 m. This is followed by a progressive settlement of about 3 mm in the field study and about 6 mm in the numerical study.

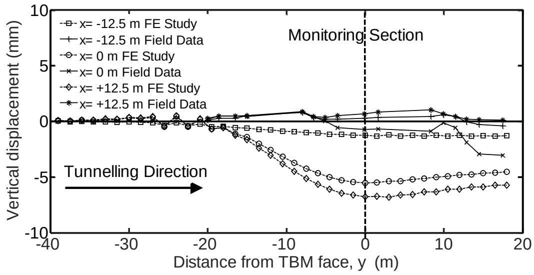

Fig. 3 Vertical displacement at selected locations during the tunnel advancement plotted against the distance of the TBM face from the monitoring section.

Figure 4 shows the peak pore pressure responses obtained from the finite element study, during the withdrawal of tail grout pressure and installation of the tunnel lining. Excess pore water pressures close to 100 kPa was recorded in both the field as well as finite element study as the TBM face approaches the monitoring section. This can be attributed to the sudden settlement of soil above the tunnel- crown due to the withdrawal of shield and installation of lining together- with the fresh grout. As the tunnel advances beyond the monitoring section, it was observed that the pore pressure reduce gradually which can be attributed to the stress release above the tunnel due to grout hardening and transfer of overburden effectively to the tunnel lining.

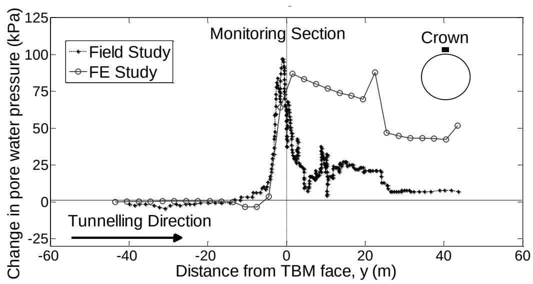

Fig. 4 Pore water pressure variation near the tunnel crown during the tunnel advancement plotted against the distance of the TBM face from the monitoring section.

The finite element study overestimates the pore water pressure variation beyond the monitoring section which might be due to the undrained analysis type. The adoption of tail grout pressure values gauged directly from the pump would have also influenced the pore pressure response. The values gauged from the pump will not be actually exerted in the field due to the different losses which include the grouting pipe and outlet friction resistance. However, the dissipation pattern of pore water pressure beyond the monitoring section in the finite element study is similar to the field study.

5 CONCLUSION

A three dimensional finite element model is developed for simulating the mechanised shield tunnelling procedure in soft ground incorporating all the relevant components like torque, face pressure, shield- skin friction, tail grout pressure, lining installation, back- up trailer weight, grout hardening and excavation. Profiles were plotted demonstrating the ground responses in terms of deformation and pore pressure variation as the tunnelling progresses. The results were found to be in good agreement with the actual field data. The pore pressure variation from the developed model highlights the significance of properly accounting for the relevant TBM components which are usually ignored in the numerical simulation of mechanised tunnel construction problems. Modelling methodology proposed in this study is relevant in the estimation of ground behaviour for situations with similar ground conditions and excavation procedure.

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