Computational modelling of a TBM tunnelling through alluvial ground at design stage

Rohola Hasanpour[1], Jamal Rostami[2], Jürgen Schmitt[1], and Babak Sohrabian[3]
[1]Hochschule Darmstadt – University of Applied Sciences, Fachbereich Bauingenieurwesen, Haardtring 100, 64295 Darmstadt, GERMANY
[2]Colorado School of Mines, Department of Mining Engineering, 1600 Illinois St., Golden, 80401 Co, USA.
[3]Hacettepe University, Department of Mining Engineering, Beytepe 06800, Ankara, TURKEY

1 ABSTARCT

This paper reviews the issues involved with estimation of face pressure for first section of Glass Tunnel project, where extended reach of the tunnel length will be excavated in the alluvial soil. A single shield TBM with EPB mode is planned to mine the tunnel with maximum overburden can reach 103 m in this section. The alluvial grounds include several geological layers with various martial properties should be accounted for estimation of contact frictional forces on the shield. Similarly, the high groundwater table requires calculation of hydrostatic and lithology to estimate earth pressures. Given the high overburden and presence of groundwater, possibility of shield jamming and tunnel face instabilities should be investigated. This study focuses on estimation of the TBM thrust force for the Glass tunnelling project using FLAC3D to estimate face pressure and contact load on the shield based on a 3D model of the machine and tunnel lining. Key parameters including mechanical and hydraulic properties of the ground, tunnel geometry, TBM components and their dimensions were incorporated in the 3D model. Ground parameters were calculated by the model are compared to the values in the geological reports and similar case studies.

2 INTRODUCTION

Tunnel excavation using a shielded TBM through alluvial ground with high overburden is a challenging process. Proper investigation at the design stage for selecting the suitable TBM is critical to achieve the project success. An important aspect of proper machine selection is taking face pressure into account to stabilize the tunnel face during boring and for accurate estimation of required thrust force to propel the machine forward. Several analytical studies have been devoted to tunnel face stability and estimation of suitable face pressure. Most analytical results are based on the limit equilibrium method ([1] to [3]) and the limit analysis method ([4] to [6]). Various computational studies have also been introduced for investigation of face pressure and surface settlements within TBM tunnelling at shallow depth. To examine stress-strain behaviour of the ground, three-dimensional models have been developed in recent years by ([7] to [11]).

[1] E-Mail-Adresse: ra.hasanpour@gmail.com & juergen.schmitt@h-da.de
[2] E-Mail-Adresse: rostami@mines.edu
[3] E-Mail-Adresse: babak_sohrabian@hacettepe.edu.tr

Numerical analyses for estimation of the required thrust force for shielded TBM tunnelling in squeezing ground were also given in the studies by [12] and [13]. However, studies on face stability and the required thrust force of EPB (mixed) shielded driven tunnels through the multi-layer alluvial ground with high overburden are scarce. In addition, most of the existing theoretical methods are not fully applicable in such situations for evaluation of face pressure and the required thrust force, and 3D numerical simulations should be used for realistic evaluation of pressures.

This paper discusses the application of 3D numerical analysis of ground pressure for estimation of the face pressure in the first stage of Glass Tunnelling project, which is planned to mine through soft ground at depth of just over 100 m. This tunnel will be used to convey water to the catchment area of Urmia lake. An estimated 17% of the excavations located in the alluvial ground. A single shield EPB machine is selected for completion of the tunnel, and after completion of the tunnel in the soft ground, the machine is expected to cut through rock for the remainder of the tunnel while dealing with high pressure groundwater table. Considering the amount of overburden and presence of underground water, shield jamming and tunnel face instabilities could be anticipated and to prevent such incidents, more detailed site investigation and proper selection of machine specifications is critical.

The focus of this study is in the evaluation of face pressure and thrust forces required to avoid face instabilities in tunnelling through alluvial ground. Numerical modelling by FLAC3D is used to estimate tunnel face pressure and contact loads on the shield in the laminated alluvial ground. All parameters including mechanical and hydraulic properties of the ground, tunnel geometry, TBM main components and their performances were simulated accurately in a 3D model. Ground parameters were obtained from the geological reports of the study area or the similar case histories. The results of numerical modelling and related studies including face pressure, chamber pressure, and required thrust force to prevent shield jamming as will be explained in this paper.

3 PROJECT DESCRIPTION

The Glass tunnel located in the West-Azerbaijan province in Iran, about 7 km northeast of city of Piranshahr and 8 km to the southeast of Naghadeh (Figure 1). The Glass dam and tunnel water conveyance in the project area are planned for conveying the flood water of Lavin river to Urmia lake and Naghadeh plain. This project is specifically funded to provide water for Urmia Lake and prevent its drying, which has been an alarming environmental issue in Northwest of Iran. The water will be transferred through a tunnel with a length of 35,660 m. Roughly 6000 m of tunnel length (17% of the excavations) will be through the alluvial ground. The entrance portal of the tunnel is located at upstream of Glass dam and the outlet portal is situated at North of Beigom Ghaleh mountain, which is a part of Urmia Lake basin. The excavation diameter and final diameter of the tunnel is 6.40 m and 5.77 m, respectively. The tunnel is designed to convey flood or excess water with anticipated volume of about 623 million m^3 per year to Urmia Lake. For application in both alluvial ground and in the hard rock, a single shield TBM with capabilities to operate in mixed modes (pressurized face and open) is considered.

Figure 1: Project location

4 GROUND PROPERTIES

The geomechanical parameters for the soil have been obtained from the field measurements and laboratory tests on the drilling samples. The groundwater table was determined to be at 8 m depth. As shown in Table 1, there are different layers of high and low plasticity clays, clayey sands, silty clays, gravel clay, gravel silt, etc. in the overburden. The upper layer is composed of high plasticity clays of about 5 m thickness and show loose and moderate density with SPT measurement of N_{SPT} = 9. This layer is underlain by the 3 m thick alluvium and then 6 m low to high plasticity clays (N_{SPT} = 11). The alluvium in this part is composed of clayey sand with gravel-sand-clay mixtures (N_{SPT} > 50). Under the high plasticity clays lies a 10 m thick alluvium, made of silty gravels and clayey gravels with N_{SPT} > 50. The alluvium is followed by two layers of medium to high plasticity of clays with 63 m and 9 m thickness. An alluvium with 3 m thick and N_{SPT} > 50 is located between these two layers. This is followed by a 12 m thick alluvium layer composed of silty gravels, clayey gravels and silty sands. The tunnel will pass through various layers, however, for the most part through the alluvium layer (Figure 2).

5 3D NUMERICAL MODELLING

To analyse and estimate face pressures and contact loads around the shield in the excavation of Glass tunnel with a single shielded TBM, a full 3D model have been developed in FLAC3D so that all geometrical details of the EPBM shielded TBM can be taken into the accounts. Numerical simulation was implemented in two steps as follows;

1) block modelling, entering ground properties, the definition of ground behaviour model, assigning boundary conditions, application of in-situ stress, and solving the unexcavated model for stabilising the unbalanced forces.
2) Modelling TBM advance using step by step excavation, application of interface elements for simulation of interactions, applying face pressure, segment extrusion and loading, and grouting behind segments.

Table 1: Geomechanical and hydraulic properties of the geological layers of the study area

Field data						Laboratory data							
Depth (m) from	Depth (m) to	Length (m)	Ground	N_{SPT}	PI	γ_d (kN/m^3)	γ_{sat} (kN/m^3)	c (kN/m^2)	φ (◦)	e	s (%)	K (m/s)	E (MPa)
0	5	5	CH	9	27	14.5	19.00	43	28	0.95	0.59	1.0E-07	12.0
5	8	3	SC-GP-GC	50	9	20.1	22.50	7	33	0.48	0.83	2.7E-06	75.0
below groundwater table													
8	14	6	CH-CL	11	23	14.5	19.0	61	27	0.95	0.95	7.5E-08	15.0
14	24	10	GC-GM	50	7	20.4	22.7	9	33	0.36	0.74	5.0E-06	82.0
24	87	63	CL-CH	14	19	14.5	19.0	79	27	0.72	0.74	2.0E-07	15.0
87	90	3	GC	50	11	19.3	22.0	20	31	0.39	0.61	5.0E-06	84.0
90	99	9	CL-ML	13	11	14.5	19.0	81	29	0.71	0.67	6.7E-08	16.0
99	111	12	GC-GM-SM	50	5	20.2	22.6	18	33	0.43	0.62	5.0E-06	95.0
111	114	3	CL-ML	18	6	14.5	19.0	58	29	0.84	0.47	1.0E-07	17.0
114	124	10	SC-SM	50	10	18.8	21.7	31	31	0.52	0.61	5.0E-06	65.0
124	140	16	ROCK	50	NP	22.5	24.0	500	35	0.40	0.33	1.0E-08	800.0

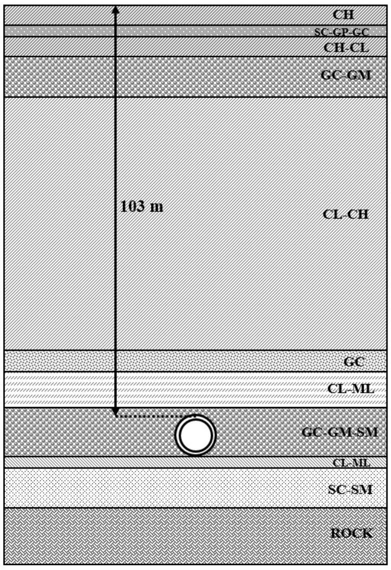

Figure 2: Geological cross section of the study area with tunnel location

3D modelling was set up to include 103 m overburden per geotechnical and hydrological properties in Table 1. The ground was assumed to follow a linear elastic-perfectly plastic behaviour according to Mohr-Coulomb failure criterion. A screen shot of the 3D model of the tunnel with the geological sections of the study area is shown in Figure 3. The in-situ state of stress is assumed to vary linearly with the depth and its ratio ($K_0=\sigma_h/\sigma_v$) is assumed to be 1, due to the high depth, presence of water in the ground, and anticipated behaviour of soft plastic clay and alluvial layers. To simulate ground layers and in situ pressures in the virgin ground, damping (solving) of the unbalanced forces was needed, where unbalanced forces reached zero. The contour lines of estimated ground displacement are presented in Figure 4.

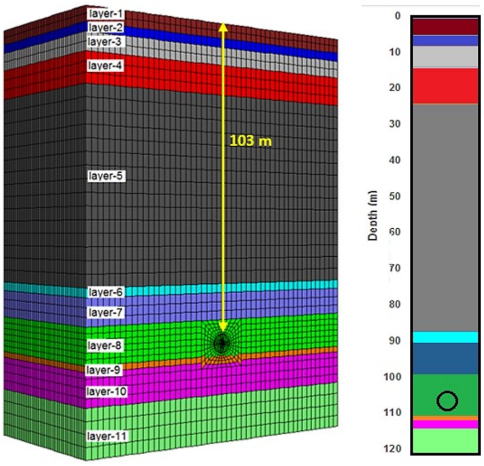

Figure 3: 3D model with the geological section of the study area

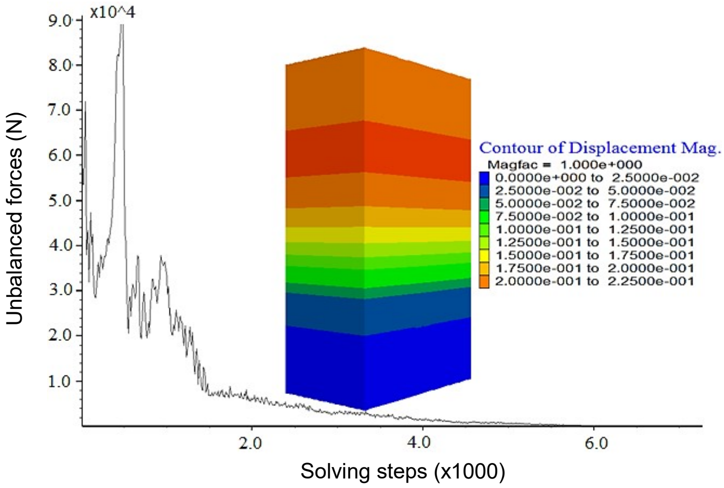

Figure 4: Contour lines of ground displacement after solving the block model for applying in situ pressures

Tunnel excavation and simulation of the shield advances followed the damping step. For simulating of tunnelling, the 3D model of TBM and relevant dimensions were selected and implemented based on machine specifications. In this study, a mixed mode boring machine with 6.4 m excavation diameter with the capability of 14 cm overcut and shield length of 10.5 m was used for computational analysis. The main geometric features of the EPBM used in the modelling are given in Table 2. The shield and annular gap backfill were considered to behave as linear elastic material, with pertinent properties listed in Table 3.

Figure 5 illustrates the schematic view of the machine arrangement as simulated in FLAC3D. It should be noted that the machine was modelled based on speculated machine specifications. The excavation stages and the total number of steps for the numerical modelling were selected based on the anticipated design of the cutterhead and shield for a suitable single shield TBM. The contact between the cutterhead with the alluvial ground as well as between the shield and the ground has been modelled by using the interface elements on both tunnel and shield boundaries by considering the gap between them according to non-uniform overcut around the shielded TBM. This refers to the fact that the shield lies against the floor and the maximum gap will be on the crown side of the excavated space.

Table 2: Geometric parameters of TBM components used in numerical modelling

TBM components	unit	value
Shield length	[m]	10.5
Shield thickness	[cm]	5
Segment width	[m]	1.5
Segment thickness	[cm]	30
Segment number	-	5+1

Table 3: Mechanical properties of main TBM components used in modelling

Material properties	Unit	Shield	Soft backfill	Hard backfill
Elastic modulus	[GPa]	200	0.5	1.0
Poisson's ratio	-	0.3	0.4	0.3
Unit weight	[kg/m^3]	7680	2100	2400

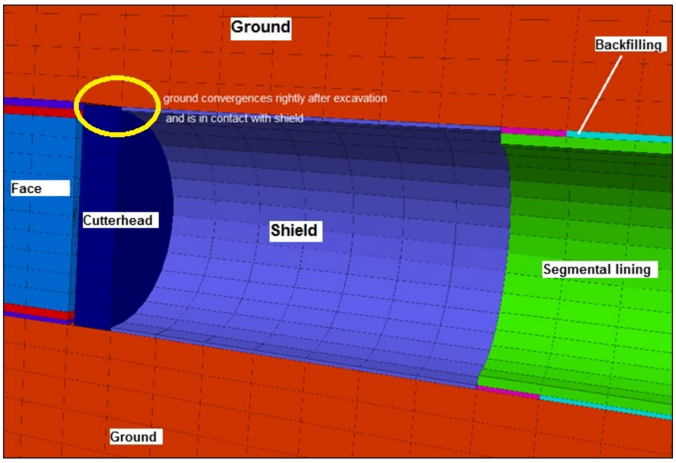

Figure 5: Screen shot of geometric configuration of the model of a EPBM in FLAC3D

6 RESULTS OF NUMERICAL ANALYSIS

6.1 Control of displacements

Calibration and adjustment of the numerical simulations was done based on controlled displacements at contact points between the shield and ground with respect to applied overcut. Figure 6 shows the contour line drawing of calculated displacements for 36 m of tunnel length, including the cutterhead, shield and selected length of lined tunnel. As it can be seen, the maximum displacement around the shield at the crown of the tunnel was calculated at 12 cm. This is 2 cm smaller than the expected amount (the overcut value in this point is 14 cm). Furthermore, the maximum displacement at the invert was obtained 3 cm, 2 cm more than the anticipated value (the overcut value in the invert is assumed to be 1 cm). However, the total displacement is 15 cm, this means that the overcut and interactions between shield with ground were simulated accurately, but due to uplifting forces, the overcut sizes were changed at the invert and crown around the shield.

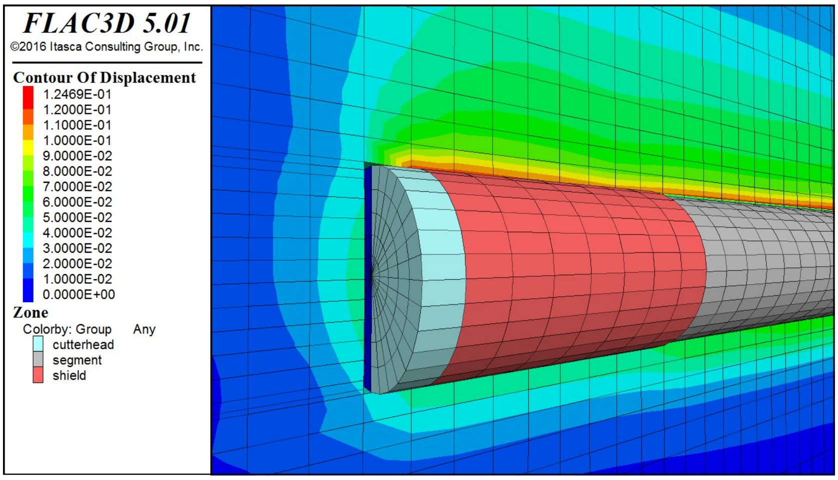

Figure 6: Contour of displacements around shield and segmental lining

6.2 Face pressure

In order to estimate the total face pressure (chamber pressure + cutterhead (disc) force) which is required to excavate the face during tunnelling operation, different amounts of pressures including 6, 8 and 10 bar were applied to the face and then the related displacement values for each pressure level were calculated. As shown in Figure 7, application of face pressures of 6 and 8 bar cause to face to be deformed by 46 mm and 23 mm, respectively. However, application of 10 bar pressure leads to face displacement about 2 mm in the direction of tunnel axis. The estimated face displacement amounts were drawn against the total face pressure values in Figure 8. It proves the presence of a linear relationship between the face displacement and the applied face pressure. It can be concluded that the total face pressure of 9.5 bar would be enough for stabilizing the tunnel face.

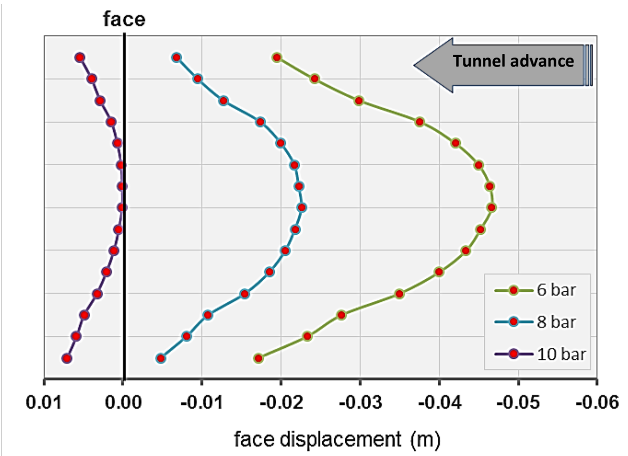

Figure 7: Displacement diagram at tunnel face resulted by applying different pressure values

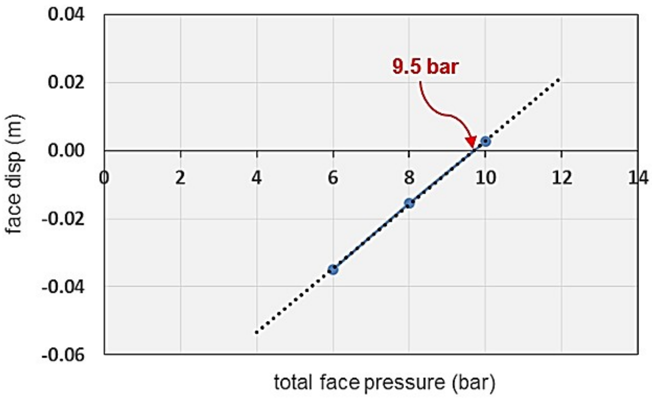

Figure 8: Displacement at tunnel face versus applied total face pressure

6.3 Chamber pressure

Chamber pressure or operating face pressure from the machine side, which is needed to prevent water leakage into the tunnel, was investigated using computational analysis, assuming that the total face pressure of 9.5 bar would be applied to the face. For this purpose, fluid mechanical principals were applied to estimate the hydrostatic pressure at tunnel face. This means that instead of determining water level at a depth of 8 m from the surface and applying it to the whole model, because of existing permeable alluvial layers and impermeable clay layers, the porosity, permeability, saturation, humidity and other parameters were defined distinctly for each layer. The result presents that the maximum hydrostatic pressure at tunnel face in drained condition could be estimated at 3.5 bar. Meanwhile, during the operation, the chamber pressure to prevent face collapse and allow limited water ingress should be more than 3.5 bar (Figure 9).

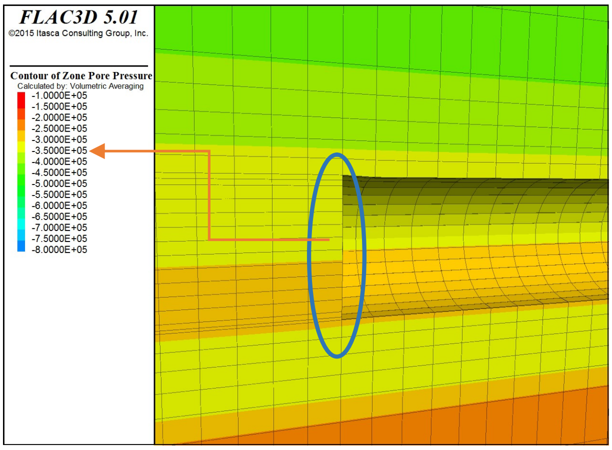

Figure 9: Contours of pore pressure (hydrostatic pressure) around tunnel face

6.4 Contact friction load on the shield

For calculation of the required thrust force to overcome frictional load on the shield, the contact loads were examined using the 3D model. Figure 10 depicts the contour plot of the contact pressure around the shield. As can be seen in this Figure, the closure of overcut between ground and the shield occurs right after excavation at early stages of the tunnelling that causes redistribution of ground stresses and reduction in contact pressures on the shield. The induced contact pressures are redistributed with different magnitude around the shield because of non-uniform overcut (the overcut is14 cm at the crown that reduces gradually to 2 cm at the invert).

For calculation of the required thrust force to overcome frictional forces on the shield to propel the TBM forward, the total contact pressures over the shield is multiplied by the skin friction coefficient μ and the reduction coefficient β which is the ratio of the real shield radius r over the tunnel radius R. This allows for calculation of the required maximum thrust force. If μ is assumed to be 0.45, the required thrust force to overcome skin frictional loads is resulted equal to 5.4 MN according to Figure 11.

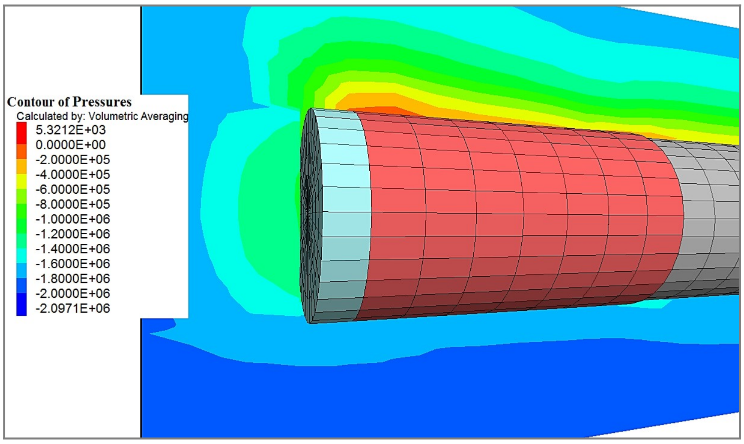

Figure 10: Contour of contact pressures around the shield

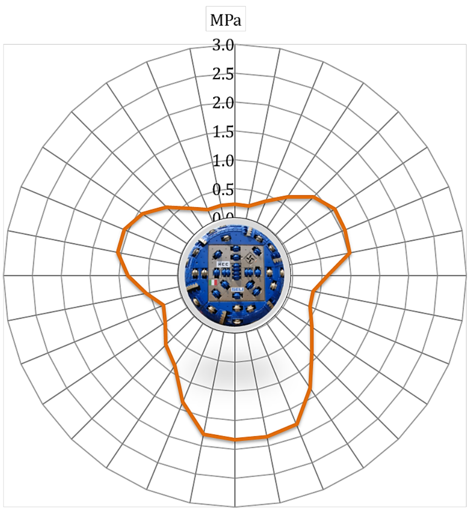

Figure 11: The total contact pressure profile over the shield

7 CONCLUSIONS

In this study, a comprehensive numerical analysis of tunnelling process through the alluvial ground using a mixed mode TBM was performed. Ground, tunnel, TBM and groundwater parameters were selected based on data from field and laboratory tests and applied in the simulated model. The results of 3D simulation of ground-shield interaction are presented as contour plots and diagrams.

The results show that the total pressure is needed to be applied in order to stabilise the tunnel face within excavation process and the chamber pressure for preventing water leakage into the tunnel were determined at 9.0 bar and 3.5 bar, respectively. This means that a cutterhead pressure about 6.0 bar (9.0-3.5=6) would be required to be thrusted to the face for boring. Furthermore, the required thrust force in order to avoid shield jamming were calculated to be equal to 5.4 MN. In machine stand-still, the required thrust force to move machine forward increases up 6.5 MN.

In tunnelling through the alluvial ground, it is essential to predict and apply the accurate face pressure and to prevent face failures. Applying face pressure smaller than the required may cause to a larger volume of over-excavations and create huge sinkholes at ground surface that must be avoided. This is due fact that applying lower than required face pressures leads to large deformations at tunnel face and consequently makes an extended zone of overstressed ground above the tunnel. Thus, the ground above tunnel is loosened and flows towards the face, while the continuation of over excavation extends the ground movement to the surface. The immediate consequence of over excavation is slower machine advance rate, additional material handling in the tunnel, and ultimately forming of sinkholes.

The results of modelling show the potential for simulation of shielded TBM application in the alluvial ground at higher depth and implementation of various ground condition parameters (i.e. in-situ stresses and properties for multi-layers' soil and alluvium) for evaluation of the face pressure and required thrust forces. This allows for assessment of the possibility for machine jamming and face instabilities in given ground conditions and may lead to the development of possible solutions. The results are realistic and plausible and show the potential for use of this approach to assess the risk of driving a mixed mode type of shielded TBM in the alluvial ground under high overburden.

8 REFERENCES

[1] G. Anagnostou, K. Kovári, "The face stability of slurry shield-driven tunnels" Tunnelling and Underground Space Technology, (1994).

[2] C. Carranza Torres, "Computation of factor of safety for shallow tunnels using Caquot's lower bound solution", Technical report for Geodata, Turin. (2004)

[3] W. Broere, "Tunnel face stability and new CPT applications", Ph.D. Thesis, Delft University of Technology, Delft University Press, The Netherlands, (2001).

[4] E.H. Davis, M.J. Gunn, R.J. Mair, H.N. Seneviratne, "The stability of shallow tunnels and underground openings in cohesive material", Géotechnique, (1980).

[5] P. Chambon, J.F. Corté, "Shallow tunnels in cohesionless soil: stability of tunnel face" Journal of Geotechnology Engineering ASCE, (1994).

[6] A.H. Soubra, "Kinematical approach to the face stability analysis of shallow circular tunnels", Proceedings of the Eight International Symposium on Plasticity, Canada, British Columbia, (2002).

[7] D. Dias, R. Kastner, M. Maghazi, "Three dimensional simulation of slurry shield tunneling", Proceeding of Geotechnical Aspects of Underground Construction in Soft Ground, Balkema, Rotterdam, (2000).

[8] M. Melis, L. Medina, J.M. Rodriguez, “Prediction and analysis of subsidence induced by shield tunnelling in the Madrid metro extension”, Canadian Geotechnical Journal, (2002).
[9] J.A. Schmitt, “Spannungsverformungsverhalten des Gebirges beim Vortrieb mit Tunnelbohrmaschinen mit Schild”, Heft 89 Dissertation, Institut für Grundbau und Bodenmechanik, Heft 89-Dissertation, Institut für Grundbau und Bodenmechanik, Technische Universität Braunschweig, (2009).
[10] H. Chakeri, R. Hasanpour, M.A. Hindistan, B. Ünver, “Analysis of Interaction between Tunnels in Soft Ground by 3D Numerical Modeling” Bulletin of Engineering Geology and Environment, (2011).
[11] R. Hasanpour, H. Chakeri, Y. Özcelik, H. Denek, “Evaluation of surface settlements in the Istanbul metro in terms of analytical, numerical and direct measurements”, Bulletin of Engineering Geology and Environment, (2012).
[12] R. Hasanpour, “Advance numerical simulation of tunneling by using a double shield TBM”, Computer and Geotechnics, (2014).
[13] R. Hasanpour, J. Rostami, G. Barla, “Impact of Advance Rate on Entrapment Risk of a Double Shield TBM in Squeezing Grounds”, Rock Mechanics and Rock Engineering, (2015).