

**A NUMERICAL STUDY OF THE EFFECTS OF FACE SUPPORT PRESSURE AND SOIL  
MATERIAL PROPERTIES ON FACE DEFORMATION AND LINING STRESSES IN EPB  
SHIELD TUNNELING - A CASE STUDY OF MASHHAD METRO LINE 2**

*\*Saba Gharedash<sup>1</sup> and Milad Barzegar<sup>1</sup>*

*<sup>1</sup>Department of Mining and Metallurgical Engineering*

*Amirkabir University of Technology*

*Tehran, Iran*

*\*milisaba.46@gmail.com*

# **A NUMERICAL STUDY OF THE EFFECTS OF FACE SUPPORT PRESURE AND SOIL MATERIAL PROPERTIES ON FACE DEFORMATION AND LINING STRESSES IN EPB SHIELD TUNNELING - A CASE SYUDY OF MASHHAD METRO LINE 2**

## **ABSTRACT**

In this study the results of a series of numerical simulations which are conducted for one of the tunnels of the Mashhad metro line 2 project are presented to evaluate the role of tunnel face support pressure and soil strength parameters on deformation of tunnel lining and face deformation. To achieve this goal, comparative numerical simulations of a mechanized tunnel advance in soft non-homogeneous soil on top of the ground water table and beneath the building are conducted and sensitivity analysis is performed. Development of a step by step tunnel construction process is modeled using a three-dimensional finite difference model (FLAC<sup>3D</sup>) taking into account all the relevant components of shield tunneling. In order to understand the mechanism of the effects of a variety of support pressures on face deformation and lining stresses, numerical simulations are conducted. It is found that the use of different face supports causes a considerable influence on the forces of tunnel lining. In addition, it is revealed that building weight help to change the stress paths and the stress distributions in the tunnel lining.

## **KEYWORDS**

Earth pressure balance shield tunneling, Finite difference method, Lining stress, Face extrusion

## **INTRODUCTION**

Construction of tunnels for a variety of purposes, such as underground transportation and sewage disposal in urban areas, usually necessitates excavation in soil or semi-rigid media. Unlike hard and massive rocks, these types of ground usually need to be supported immediately after excavation to avoid collapse and constrain surface settlements as a critical issue in urban areas. New methods of tunneling such as pipe jacking and mechanized tunneling by tunnel boring machines (TBMs) have made it possible to excavate tunnels in soft ground and concurrently install the support system. Application of these methods combined with proper lining design results in better rates of advancement, higher stability and lower surface settlements. Walter et al. (2010) investigated tunnel face stability using a deformable tunnel lining with a centrifuge model. Muir-Wood (1975) presented a simple approach to the problem of proportioning the tunnel linings which can be useful in many situations. The main objectives of this paper include not only an analysis of stresses and deformations of the face, but also the TBM movement and deformation and loading of the tunnel lining. In this paper, effects of face support pressure and soil strength parameters is evaluated based on a variety of important tunneling design criteria, such as face deformation and loading of the tunnel lining.

## **MASHHAD METRO PROJECT**

Excavation of such a long tunnel is a major component of the Mashhad metro project. Excavation of the tunnel is to be performed by an earth pressure balance shield machine with an outside diameter of 9.15 m. The shallowest and deepest ground section is partially under building with cover-to-depth ratio (C/D) of 1.3 where C and D are cover depth and diameter of tunnel. Considering unfavorable geological conditions and dimensions of the excavating face, face stability of the tunnel is one of the key technical

issues in this project. The one most hazardous section along the tunnel is located under buildings with C/D ratio of 1.3, whose detailed analysis is shown in Table 3. At this one section excavation diameter is 9.15 m with a cover thickness of 7.94 m. The tunnel is partially excavated underneath the building (building which is located in the left side of the tunnel, see Figure 1). On the basis of the site survey results, Mashhad soil can be differentiated into a number of well-defined strata based on physical properties and soil types as illustrated in Table 1.

Table 1 – Physical and mechanical properties of the host tunnel media

Layer	Soil type	$\gamma_{\text{bulk}}$ (KN/m <sup>3</sup> )	C (KPa)	$\Phi$ degree	$\nu$ -	E MPa	$K_0$ -
I	CL-ML	18.5	10	25	0.35	12	0.53

### FINITE DIFFERENCE SIMULATION MODEL

It is generally accepted that numerical modeling of construction sequence of a soft ground shield tunnel is a complicated matter because the actual field conditions involving non-linear soil behavior are three-dimensional in nature and is highly indeterminate (Rowe et al., 1983; Lo et al., 1984). In order to demonstrate the fundamental three-dimensional aspect of the problem, a comprehensive numerical study was conducted employing the FLAC<sup>3D</sup> code (Itasca Consulting Group, 2006). Dimensions of the model were selected large enough to avoid boundary effects. The soil layer is assumed to be of elasto-plastic material in conformity with the Mohr-Coulomb failure criteria. It is assumed that the tunnel is driven by means of an earth pressure balance TBM in soft soil. Figure 1 shows various components of the obtained numerical model. Neglecting deformations of the shield skin, the TBM was modeled as a rigid contact body. According to the TBM specifications, the length of the shield was assumed to be 10 m. The step-by-step excavation process was modeled by repeated rezoning of the element mesh at the cutting face and repeated insertion of elements representing the tunnel lining and the tail void grout, respectively. Furthermore, the boundary conditions for the face support and grouting pressure were adjusted based on progress of the simulated tunnel advance. Elements of the grout are directly connected to elements of the tunnel lining on the inner side and the soil on the outer side. The concrete lining was therefore modeled in a simplified manner as a continuous isotropic elastic tube. Grouting pressure was modeled by pore pressure boundary conditions on the grout element nodes at the shield tail based on the assumed grouting pressure with a variation of 15 KN/m<sup>2</sup>/m over the height. In Table 2, vertical soil pressures for the largest and shallowest depths are presented. Table 3 shows the mechanical properties of encountered materials used in the model.

Table 2 – Boundary pressures for grout injection

Chainage (km)	5.950
Tunnel	<i>Top</i>
	<i>Bottom</i>
$\sigma_v$ (kN/m <sup>2</sup> )	$\pm 141$
	$\pm 279$
Grout pressure (bar)	$\pm 3.4$
	$\pm 4.8$

Table 3 – Material models and applied parameters

Material type	Constitutive model	Thickness			E Bulk modulus)	$\nu$ (Poisson's ratio)	$\phi$ (friction angle)	C (cohesion)	$\rho_{bulk}$ (density )
		(cm)	(m)	(m)					
Shield	elastic		10		210000	0.17	-	-	7850
Grout	Mohr-coulomb		12		40	0.25	35	600	1500
Lining	elastic		35		30000	0.2	-	-	2600
Building	elastic	Building height (m)	Building width(m)	Building length (m)	25000	0.2	-	-	364
Foundation	elastic	9	12	20	25000	0.2	-	-	2400
			3						

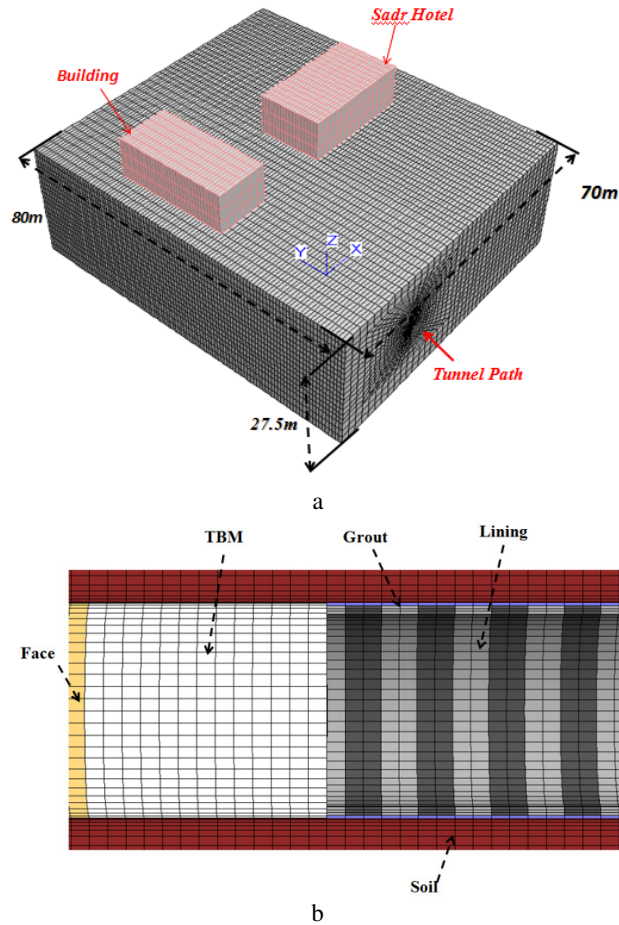


Figure 1 – a) 3-D finite difference mesh, b) Representation of different shield tunneling components in the model

## NUMERICAL STUDIES

Pressure distribution of the fresh excavated soil acting on the tunnel face was considered linear, which increases with depth based on the density of the conditioned muck (13 KPa/m) in the excavation chamber. The initial muck pressure at the center level of the tunnel face is supposed to equal to earth pressure at rest. The conditioned muck pressure is increased (or decreased) in each construction phase (ranging from 10–200% of lateral earth pressure,  $\sigma_T/\sigma_y$ ), where  $\sigma_y$  is lateral earth pressure and  $\sigma_T$  is face support pressure.

### Tunnel Face Extrusion

Lunardi (2008) has shown that deformations of the tunnel face can be a useful indicator to evaluate the ground response. Figure 2 shows the relationship between the face extrusion (Ex), normalized by the tunnel diameter (d), and tunnel face support pressure ( $P_1$ ). The curves are drawn for a 20% increase and decrease in strength parameters. Generally the normalized face extrusion is decreased as soil strength parameters are increased. At around the face support pressure of 100%, the normalized face extrusion is independent of cover depth and building effect and soil strength parameters. From Figure 2b we see again

that uncontrollable movement of the face is more likely to happen when the tunnel lies beneath the building.

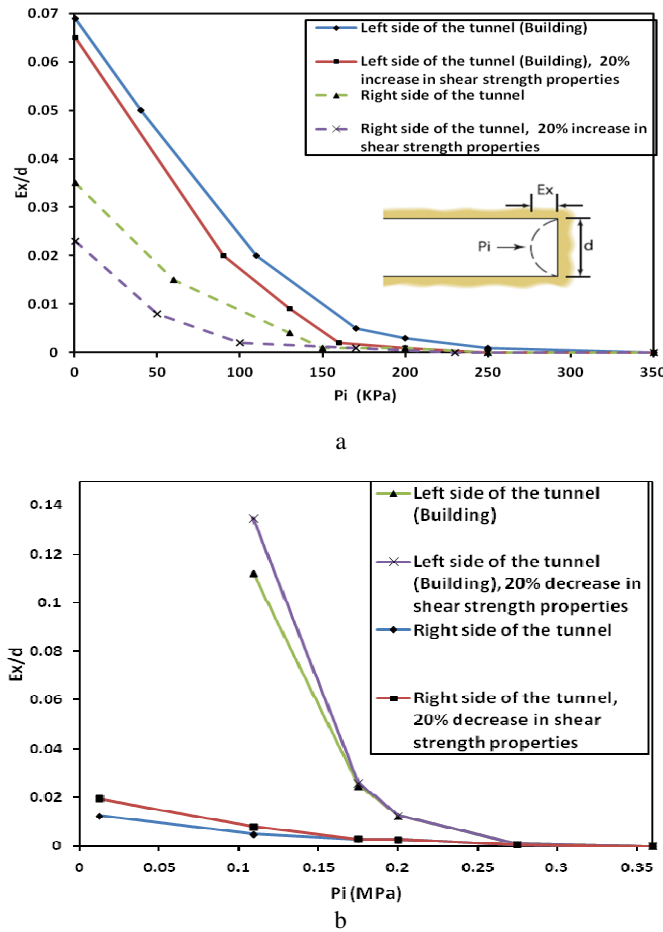
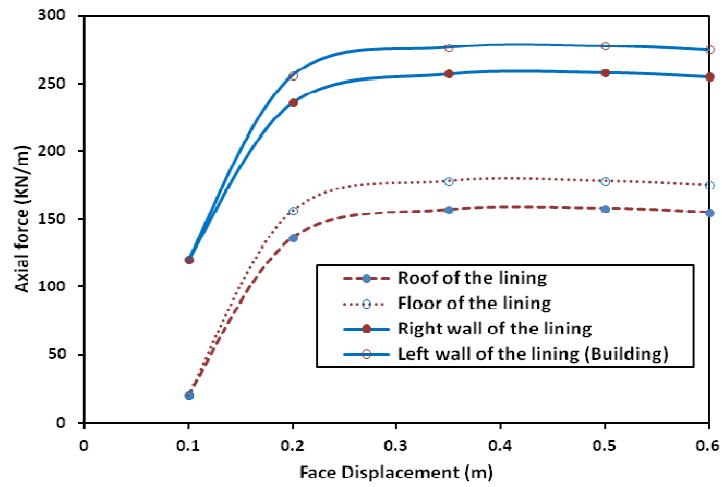


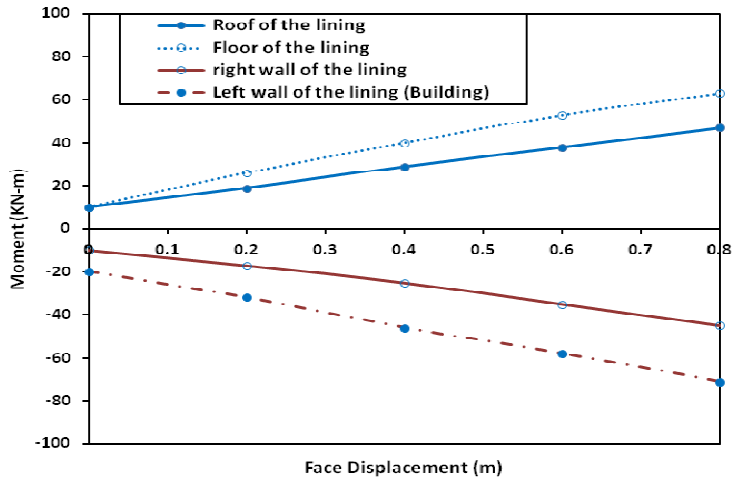
Figure 2 – Normalized face extrusion as a function of face support pressure in two side of the tunnel, a) 20% increase in soil strength parameters, b) 20% decrease in soil strength parameters.

### Face Pressure and Tunnel Lining Deformation

The face displacement as a function of face pressure for EPB shield tunneling in the one section of Mashhad metro is shown in Figures 3a and 3b. It is evident that following a sudden drop or rise in the face support pressure, stresses are distributed to the overburden soil. The strain measurements at the roof, wall, and floor of the tunnel were selected to calculate moments and axial forces in the tunnel lining. These values were calculated on a three dimensional basis. The surrounding tunnel experiences plasticity regions, depending on different inner pressures provided by the tunnel face and face support pressure. The objective is to evaluate the stress and displacement redistributions after excavation for different face support pressures. Figures 3a and 3b summarize the study results of effects of thickness and degree of degradation within the degraded zone around the tunnel lining. The figures exhibit initial internal forces in the lining as a function of face displacement. It can be seen from Figures 3a and 3b that axial forces increase with the amount of face displacement. Similar to the case observed for moments of tunnel lining, moments increase with size of the disturbed zone. Furthermore, moments follow the same pattern as the axial forces according to Figures 3a and b.



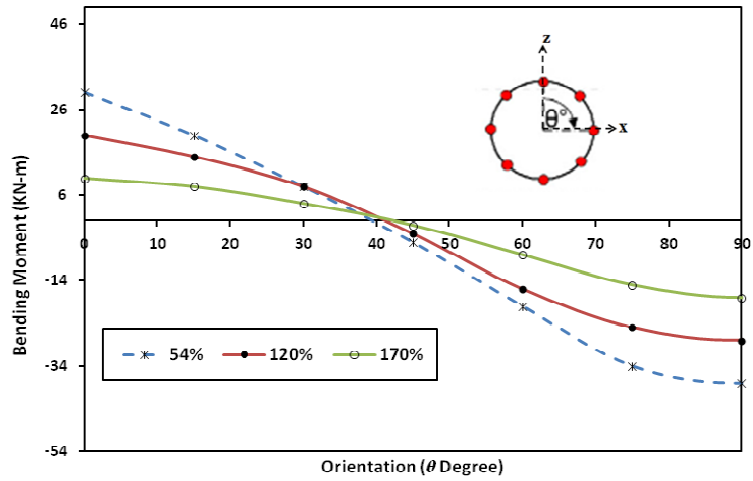
a



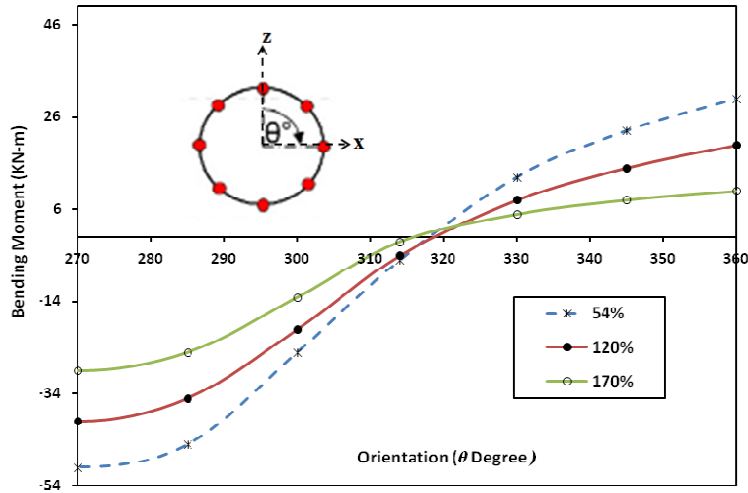
b

Figure 3 – Effects of face displacements on the internal forces of tunnel lining, a)axial forces, b)moments

Moments and axial forces due to relief of the in situ stresses (e.g., the post-construction stresses) for various positions around tunnel lining and different values of face support pressure are shown in Figures 4 and 5. The angle of incidence plays an important role in the selection of the critical section for tunnel lining. Varying face support pressure was followed by only a slight change in the axial forces and moments. The biggest compressive forces were observed for face support pressure ratios of lower than 100% near the invert, as expected. For example, for  $\theta = 90^\circ$  the turning point will be at  $45^\circ$  location.



a

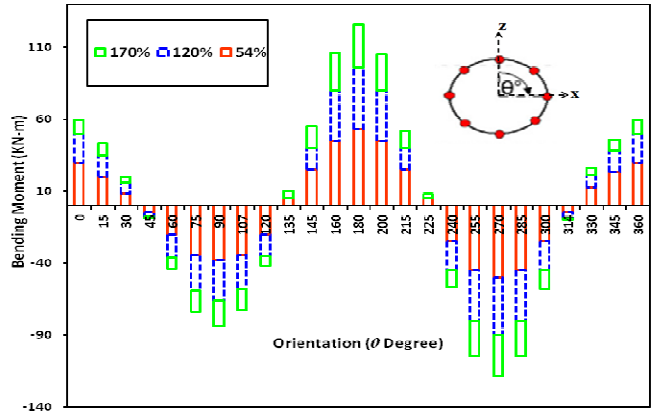


b

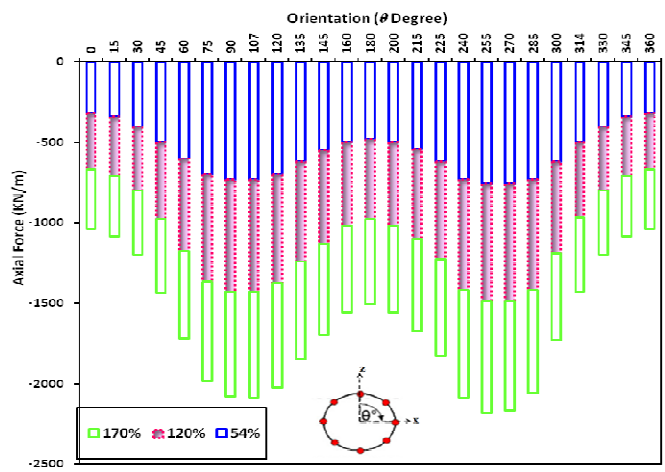
Figure 4 – Effects of angle of incidence on the internal moments of the tunnel lining, a) right side of the tunnel ( $\theta = 0-180^\circ$ ), b) left side of the tunnel (building,  $\theta = 180-360^\circ$ )

Numerical results of bending moments, axial forces and stresses along the lining for different values of face support pressure are shown in Figure 5 for the one section. In these figures,  $\theta = 0^\circ$ ,  $90^\circ$  and  $180^\circ$  indicate the crown, spring line and invert of the tunnel, respectively. The maximum values of bending moments, axial forces and stresses appear at the crown, springline and invert. However, values of bending moments, axial forces, and stresses in the lining for lower than 100% face support ratios are larger than those for higher face support pressures. As is indicated, maximum bending moments, axial forces, and stresses are reduced by about 20% using different face support pressure conditions.

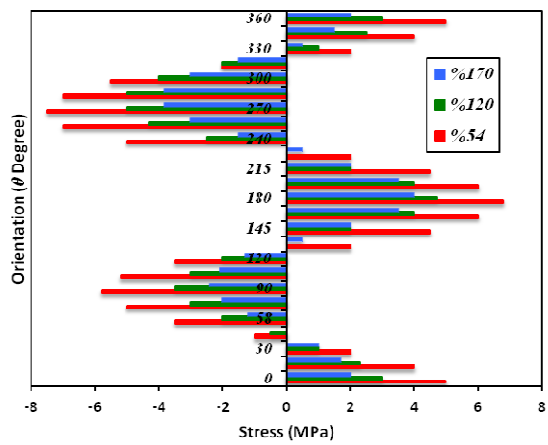




a



b



c

Figure 5 – Lining stresses a) bending moments, b) axial forces, c) stresses

Figure 6 shows the effects of buildings weight on lining stresses. Figure 6 shows that the lining stresses are increased as face displacement in the transverse section of tunnel lining is increased. Lining stresses increase when tunnel passes below buildings. Distribution of lining stresses is asymmetrical due to the effects of buildings weight.

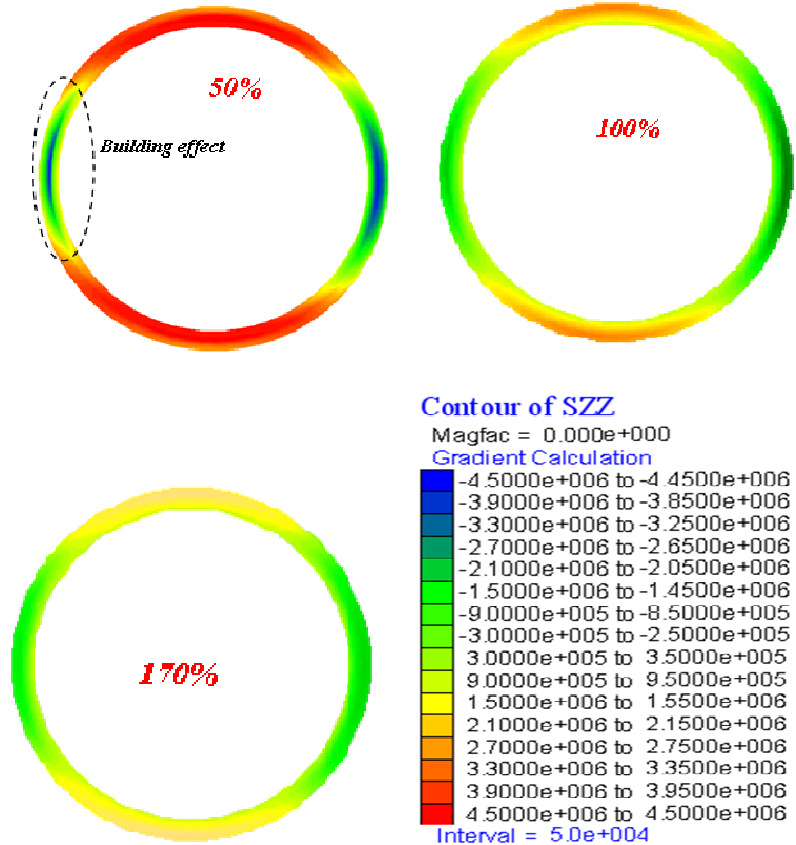


Figure 6 – Distribution of lining stresses as a function of face displacement and buildings weight

**CONCLUSIONS**

For a shield-driven tunnel advance in soft soil, the influence of face support pressure and soil strength parameters was analyzed numerically for shield tunneling in the Mashhad metro line 2 project. The results obtained from analyses demonstrate the complexity of various interactions involved in shield tunneling. Conclusions drawn from this work can be summarized as follows:

- Soil strength parameters and face support pressure are the most influential factors on face deformations in the vicinity of the face zone and have a considerable influence on the internal forces of tunnel lining.
- When face support pressure is close to the in situ horizontal earth pressure, the face extrusion normalized by the tunnel diameter is independent of soil strength parameters.
- Values of bending moments, axial forces, and stresses in the lining can be reduced using high face support pressure.
- Using buildings weight, the maximum compressive stress and tensile stress are increased by about 30% and 55% respectively.

- It has been found that different points of lining of tunnels which are executed in a step by step manner are affected by face displacement. Axial forces and bending moment increase with face displacement.
- The more the degree of disturbance in the degraded zone around a tunnel, the higher is bending moment and axial forces in the lining.
- Effects of incidence angle on moments and axial forces in the tunnel lining were examined. It was found that the highest tensile stresses occur at the crown.

#### REFERENCES

- Itasca Consulting Group, Inc. (2006). *FLAC3D Manual*, third Ed. (FLAC3D Version 3.1).
- Lo, K.Y., RMC, Ng., Rowe, R.K. (1984). Predicting settlement due to tunneling in clays. *Tunneling in Soil and Rock*, Lo, K.Y. (Ed), *ASCE Geotech III Conference*, Atlanta, Georgia, 48–76.
- Lunardi, P. (2008). Design and construction of tunnels using the approach based on the analysis of controlled deformation in rocks and soils. In: Davis, J. (trans), Springer Publ., 8–22.
- Muir-Wood, A.M. (1975). The Circular Tunnel in Elastic Ground. *Geotechnique*, 25(1), 115–127.
- Rowe, R.K., Lo, K.Y., Kack, G.J. (1983). A method of estimating surface settlement above tunnels constructed in soft ground. *Canadian Geotechnical Journal*, 18(1), 11–22.
- Walter, H., Coccia, C.J., Ko, H.Y., & McCartney, J. (2010). Investigation of tunnel face stability using a deformable tunnel lining with a centrifuge model. *Geo Florida 2010*, West Palm Beach FL, USA. Feb 20–24, 2010.