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Tunnelling Induced Ground Movement in Soil and its Control Using Face Pressure

V. B. Maji, Abite Aguda*

Dept. of Civil Engineering, Indian Institute of Technology Madras, Chennai, India
**Email:vbmaji@gmail.com*

ABSTRACT

Tunnelling through cities underlain by soft soil is commonly associated with soil movement around the tunnels and subsequent surface settlement. The predication of ground movement during the tunnelling and optimum support pressure is always a great concern. The commonly used Earth pressure balance (EPB) tunnelling machines, uses the excavated soil in a pressurised head chamber to apply a support pressure to the tunnel face during excavation. This face pressure is a critical responsibility in EPB tunnelling because as the varying pressure can lead to collapse of the face. The objective of the present study is to evalute the critical supporting face pressure by observing the vertical deformation and horizontal displacement of soil body during tunnelling. The face pressure and grout pressures were varied to see how they might influence the magnitude of surface settlements. A numerical model using PLAXIS-3D tunnel has been developed to analyse the soil movement around the tunnel that includes various geotechnical conditions. The ground surrounding the tunnel is found to be very sensitive to the face pressure and grout pressure in terms of surface settlement and collapse of the soil body.

Keywords: Earth pressure balance (EPB); Face pressure; Ground movement; Critical support pressure; Surface settlement

1. INTRODUCTION

The construction of new infrastructures in urban setup frequently involves construction of tunnels. Increasing population drives the need for more infrastructures and underground construction and will continue to flourish as a preferred solution. The prediction of tunnel induced ground deformation therefore becomes a key issue in the planning process. As tunnel face stability is directly related to the safe and successful construction of a tunnel. The earth pressure balance (EPB) shields have been developed in part to minimize surface settlement, where a rotating cutter head is propelled forward by a series of jacks pushing from the concrete lining that has been previously installed. It is the excavated material in the spoils chamber itself that acts as a support to the face. The measurement of the pressure is crucial in EPB tunnelling because, if the pressure is not constant, the varying pressure can lead to collapse of the face. Many researchers have proposed analytical approaches to determine the required pressure to stabilize the tunnel face mostly based on limit equilibrium approach or limit analysis (Atkinson and Potts, 1977; Davis et al., 1980; Leca and Dormieux, 1990;

Jancsecz and Steiner, 1994; Anagnostou and Kovari, 1994; Anagnostou and Kovari, 1996; Broere, 2001). An examination of field data of subsidence in soft ground tunnelling by Attewell (1977) indicates that a major proportion of total soil deformation occurs immediately after construction. New and O'Reilly (1991) reviewed the ground movements associated with tunnelling and found that the main hazards associated with the tunnel construction in urban areas include poor ground conditions, presence of water table above the tunnel, shallow overburden and ground settlements induced by tunnelling with potential damage to the existing structures and utilities above the tunnel. Mair and Taylor (1997) studied the components of ground deformation associated with closed shield tunnelling. The EPB machines with full tunnel face support reduce the total volume loss significantly as the tunnel advances. Clough and Schmidt (1977) observed that the tunnel face ground loss contributed from one-quarter to one-third of the total volume loss.

The predication of ground movement during the tunnelling and optimum support pressure could be based on analytical, empirical or the numerical methods. The empirical methods for settlement analysis usually neglect in situ stress in contrast to the finite element methods. Nevertheless, they do not include the interaction between the soil and lining, therefore cannot account for the support stiffness. Normally, the empirical methods are used as a preliminary verification to get an idea about the displacement that occurs at the ground surface. Finite element analysis became more acceptable tool for the tunnel induced settlement study. Present work aimed to evaluate the critical face supporting pressure by observing the deformation of soil mass during tunnelling. Settlement analysis is carried out using PLAXIS-3D tunnel program to study the face pressure balance and grouting to overcome any collapse.

2. SURFACE SETTLEMENT

The most common empirical method to predict ground movements is based on a Gaussian distribution. Peck (1969) and Schmidt (1969) were the first to show that the transverse settlement trough, after construction of a tunnel, in many cases can be well described by the Gaussian function (Fig. 1). Two parameters, namely the ground loss (GL) and the standard deviation 'i' of the curve, are needed to fit the surface settlement. Cording and Hansmire (1975) defined the ground loss as the volume of soil that displaces across the perimeter of a tunnel. It is often defined in terms of volume lost per unit length of tunnel constructed. The percentage ground loss (GL) is defined as follows,

$$GL = \frac{V_i}{V_0}.100\% \tag{1}$$

where V_i = trough volume and V_0 = tunnel volume. Based on the shape of the normal distribution curve, Peck (1969) showed that the maximum settlement $S_{v,max}$ is given by,

$$S_{v,\max} = \frac{0.314.GL.D^2}{i} \tag{2}$$

Where, D = the diameter of the tunnel, $S_{v,max}$ = maximum settlement above the tunnel axis. The settlement at various point of the trough is then given by,

$$S_v(y) = S_{v,\max} . \exp\left(\frac{-y^2}{2i^2}\right) \tag{3}$$

where y is the vertical distance from the tunnel axis and 'i' is the horizontal distance from the tunnel axis to the point of inflection of the settlement trough. Peck (1969) suggested that percentage of ground loss (GL) is usually in the range of 1-2% in stiff clay, 2-5% in soft clay and less than 1% in sandy soil. Mair (1993) also suggested that subsurface settlement profiles could be reasonably approximated in the form of a Gaussian distribution. Volume of ground loss due to tunnel construction can also be used to assess the quality of construction process. O'Reilly and New (1982) have proposed relationship between ground loss and tunnelling quality. For a good practice in firm ground, ground loss is assumed to be 0.5%, where as for poor practice in soft ground, it may be 4.0% or more. The volume of the settlement trough per unit length of tunnel, V_s is obtained by integrating Eq. 3 yields,

$$V_s = \int S_v(y)\, dx = \sqrt{2\pi}\, .i\, .S_{v,\max} \tag{4}$$

In addition to the settlement volume $V_{s,}$ one has to consider the ground loss V_t which is the volume of the ground that has deformed into the tunnel after the tunnel has been constructed. For tunnelling in undrained ground, the settlement volume is more or less equal to the ground loss, but the settlement volume tends to be somewhat smaller for drained excavations. The dilation and swelling due to unloading may result in soil expansion, such that $Vs<V_t$ (Cording and Hansmire, 1975). However, differences tend to remain small and it can be assumed that $V_s \approx V_t$.

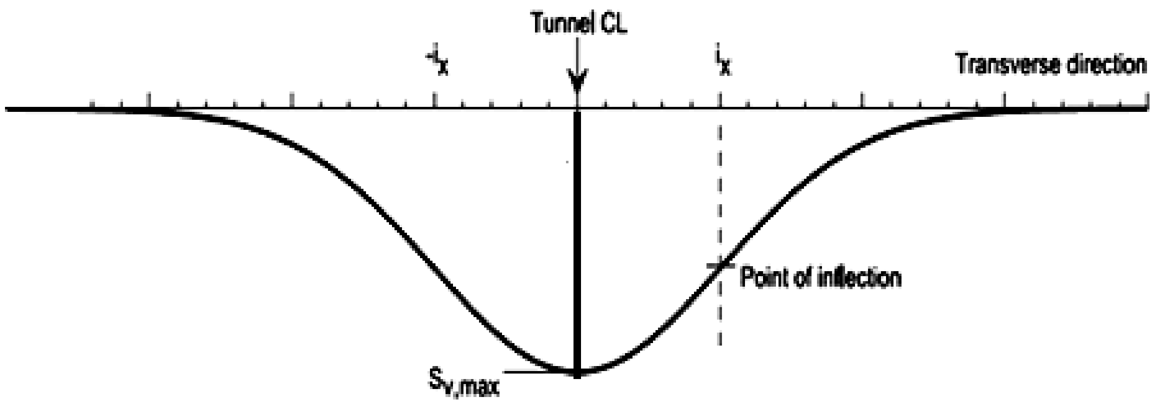


Figure 1: Gaussian curve for transverse surface settlement trough

3. WIDTH OF THE SETTLEMENT TROUGH

The distance from the tunnel axis to the inflection point 'i', and determining the width of the settlement trough is subject of many investigations. Peck (1969) suggested a relationship between tunnel depth Z_o and tunnel diameter D, depending on ground conditions. After Peck (1969), many other researchers have come-up with similar relationships, namely Cording & Hansmaire (1975) and Clough & Schmidt (1981). O'reilly and New (1982) presented results from multiple linear regression analysis performed on field data, confirming the strong correlation of 'i' with tunnel depth. He did not find significant correlation with tunnel diameter or method of construction (except for very shallow tunnels with a cover to diameter ratio less than one). They stated that for most practical purposes, the regression lines may be simplified to the form (Eq. 5),

$$i = K.Z_o \tag{5}$$

where K is a trough width parameter, with $K \approx 0.5$ for clayey ground and $K \approx 0.25$ for sandy ground. The approach of Eq. 5 has also been confirmed by Rankine (1988), who presented a variety of tunnel case histories in different type of soils. Mair and Taylor (1997) presented a large number of tunnelling data with different linear regressions for tunnels in clays, sands and gravels. The regressions analysis confirmed the findings of O'reilly and New (1982) for clayey soils, with a trough width parameter ranging between 0.4-0.6, with a mean value of K=0.5. However, for sandy soils 'K' obtained ranging between 0.25-0.45 with a mean value of 0.35, indicating somewhat wider settlement troughs.

4. DEPTH OF THE SETTLEMENT TROUGH

Craig and Wood (1978) found that the volume of settlement trough at the surface is approximately equivalent to the volume of ground loss in the tunnel. The ground loss ratio in Eq. 2 is used for an initial estimate of $S_{v,max}$. The method of construction of the tunnel has a considerable effect on the ground loss ratio. Depending on equipment, control procedures and experience of the crew, ground loss ratio can vary between 0.5% and 2% in homogenous ground. In sands, a loss upto 1% may be seen, where as in soft clays it ranges from 1% to 2%, as reported by Mair (1996). Considering data for mixed ground profile with sands or fills overlaying clays, Mair and Taylor (1997) reported values between 2% and 4%. For tunnels in undrained clays, Clough and Schmidt (1981) proposed a relationship between mobilized stability number N and ground loss ratio. For N less than 2, the response is elastic with small ground movements where the tunnel face is stable. For N between 2 and 4, load increases and a limited plastic yielding occurs, while for N between 4 and 6, the yielding zone spreads, leading to large movements. For N greater than 6, yielding zone is significant, leading to tunnel face instability with large ground movements.

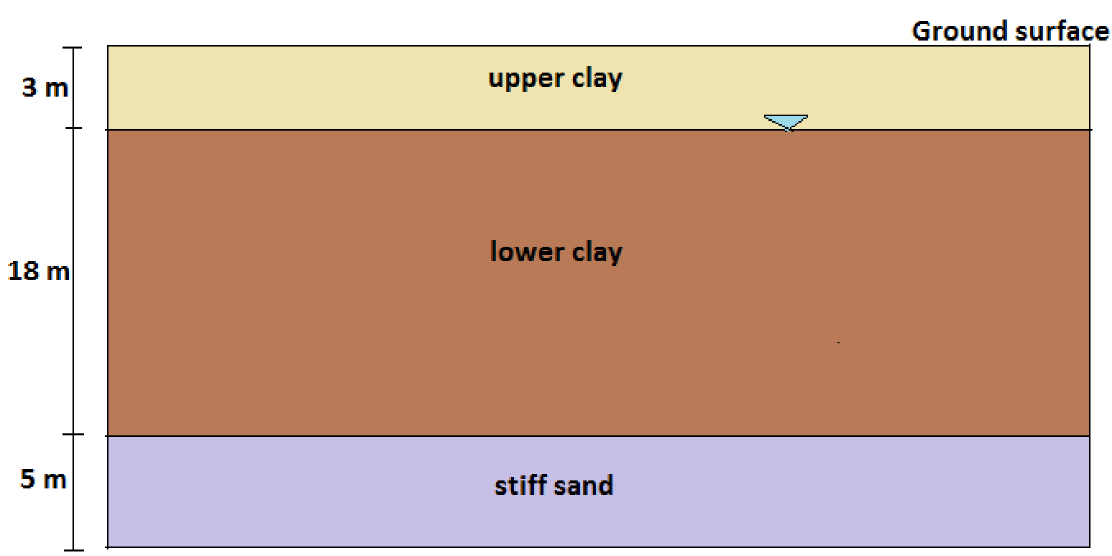


Figure 2: Soil profile showing different layers

5. FINITE ELEMENT MODEL USING PLAXIS

A tunnel has been modelled to demonstrate the effect of tunnelling and face pressure on the surface settlement/ground deformation. The water table in the model has been 3m deep from the ground surface. The depth to the centre of the tunnel is 12m and the inside and outside diameters of the tunnel are 8.5m and 9m, respectively (Fig. 2). The 3D model used in the analysis is shown in Fig. 3 is 80m long, 26m high and 20m wide. To model the excavation section, twenty slices each 1.5m wide is considered at the centre portion, at the front and end portion 25m sections were included to reduce the influence of boundary conditions. Table 1, shows various soil material properties used for the FE analysis using Mohr-Coulomb material model. PLAXIS can handle cohesionless sands (c=0), but it is advised to enter a small value of cohesion (c > 0.2 kPa) (PLAXIS 3D tunnel manual 2004). Tables 2, shows the structural element properties that were used in the analysis. The normal stiffness *EA*, flexural rigidity *EI*, and weight *w*, were selected based on the material properties of the shield used. It should be noted that the *EA* and *EI* related to the stiffness per unit width and *w* is the specific weight in units of force per unit area. Poisson's ratio *v* is set to zero in PLAXIS for long, slender structural elements such as sheet-pile walls and, in this model, cylindrical steel plate. The interface strength is set to be 0.9 for real soil-structure interactions, the interface is weaker and more flexible than the associated soil layer, means the value is less than 1. Table 3, shows the concrete lining properties that were used in the analysis. The elastic parameters *E* and *v* were also based on the material properties of concrete lining. The interface strength reduction factor was set as one, in the shield case. The input for dry unit weight of the lining is 24kN/m^3 as of standard concrete.

Table 1: Material properties of different types of soil

Parameter	Upper clay	Lower clay	Stiff sand
Material model	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb
Type of material behaviour	Undrained	Undrained	Drained
Dry unit weight (kN/m^3)	16	16	17
Saturated unit weight (kN/m^3)	18	18	20
Young's modulus (kN/m^2)	8.4 x 10^3	1 x10^4	2.5 x 10^4
Poisson's ratio (υ)	0.35	0.35	0.3
Undrained shear strength (kN/m^2)	48	70	3
Friction angle (°)	10	20	30
Permeability (m/day)	0.001	0.05	1
Dilatancy angle (ψ)	0	0	0

Table 2: Material properties of structural plates representing shield

Parameter	Value
Type of behaviour	Elastic
Normal stiffness (kN/m)	8.20 X 10^6
Flexural rigidity (kN/m^2/m)	8.38 X10^4
Equivalent thickness (m)	0.35
Unit weight (kN/m^3)	38.5
Poisson's ratio (υ)	0.3
Interface strength reduction	0.9

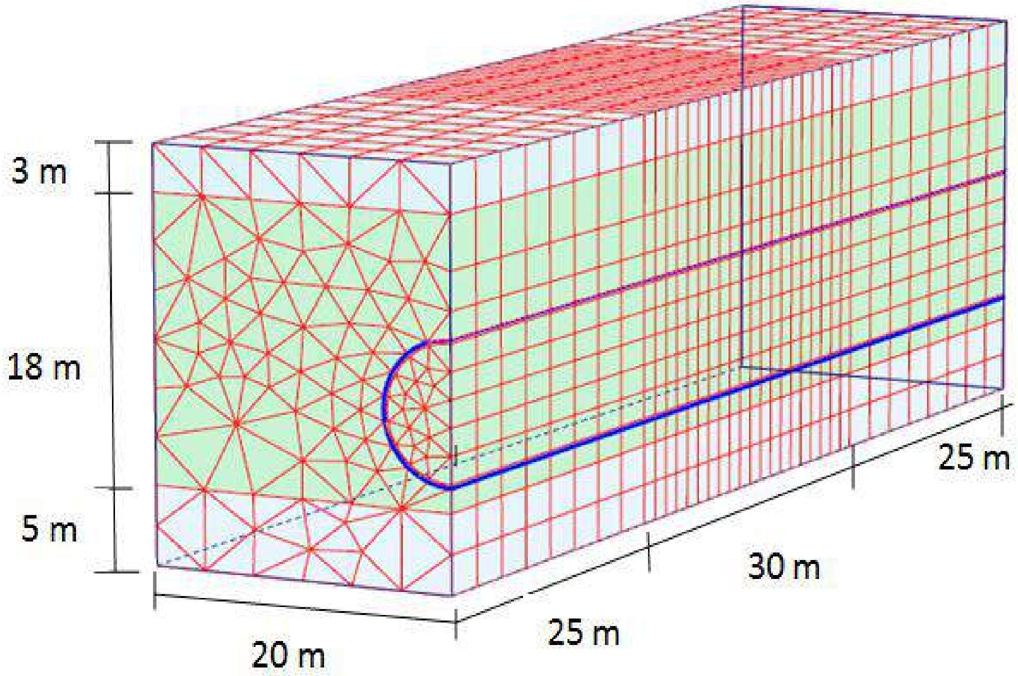


Figure 3: 3D mesh for the tunnel to be excavated using PLAXIS-3D

Table 3: Material properties of the concrete lining

Parameter	Concrete Lining
Identification	concrete
Type of material behaviour	Linear-elastic
Material type	Non-porous
Volumetric weight (kN/m^3)	24.0
Young's modulus (kN/m^2)	3.1×10^7
Poisson's ratio (υ)	0.2
Interface strength reduction	1.0 (Rigid)

6. FACE PRESSURE AND CORRESPONDING SETTLEMENT

During tunnel construction, soil is removed from the tunnel face and the soil layer in front and above the face exerts active earth pressure. The presence of infrastructures or surcharge contributes as additional earth pressure. For the tunnel alignment below the groundwater table, water pressure is another significant component of pressure acting at the tunnel face. For stability, the layers of soil at the tunnel face should have sufficient strength to balance these forces. In many projects, tunnels will encounter several layers of loose soils or weathered rock. The face may not be strong enough to bear such pressures and may be unstable leading to the collapse of soil mass resulting in excessive settlement at the surface. Support pressure (face pressure) needs to be built up at the face of tunnel, to counter balance the pressure generated by the soil, water and overlying infrastructures. Sometimes, even with stable geology, face support pressure needs to be built up in order to prevent the inflow of water into the tunnel. A decrease in the groundwater level may result in consolidation and thereby surface settlement. In cases of mechanised tunnelling, support mediums will be used to build the required face support pressure. Common support mediums used are bentonite slurry, earth paste, and compressed air. Choosing a support medium depends on various factors like properties of soil and the type of TBM used. There are some adverse effects of applying excessive face pressure, as it may lead to surface heave and ground distortion. On the other hand inadequate support pressure may cause surface settlement. Therefore, an adequate range of face support pressure is needed to stabilize the face, which in turn will minimize settlement, avoid soil body collapse.

The pressure exerted on the face is controlled by the relative amounts of material that enter and exit the spoils chamber of the EPB machine. A higher rate of ingress than egress will gradually increase the face pressure and vice versa. Researchers have formulated a relationship between the stresses acting on the face and the undrained shear strength of the soil for a circular tunnel constructed in homogeneous, plastic clay (Peck, 1969 and Leca, 2000). Peck (1969) reported that the overload factor (N) (Eq. 6) should not exceed about 6. Tunnelling could be carried out without unusual difficulty in plastic clays if N remains below 5. It was also noted in shield tunnelling, that the values of N much greater than 5 may cause the clay to infiltrate the tail void too rapidly, so that the annulus space cannot be filled with grout satisfactorily. In addition, for values of N approaching 7, tunnel advance may become slow and difficult as the shield has a tendency to tilt.

The overload factor is given as,

$$N = \frac{\sigma_s + \gamma . H - \sigma_T}{S_u} \tag{6}$$

Where, σ_T = support pressure applied at the centre, σ_s = surcharge load, γ = Soil unit weight, H = depth to centre of tunnel and S_u= undrained shear strength.

7. ANALYSIS OF FACE PRESSURE

Different face pressures, corresponding to different values of over load factor (N), were applied at the face in order to observe the settlement profile for each case. The aim was to find the case where very small longitudinal deformations observed at the face, may be the true earth pressure balance.
Table 4 shows the overload factors and corresponding average face pressures that were modelled.

Table 4: Face pressure variation corresponding to displacement variation

Overload Factor (N)	Face Pressure (kPa)	Vertical Displacement		Horizontal Displacement Z (mm)
		Settlement (mm)	Heave (mm)	
3	60	59.30	32.41	109.75
2	130	42.28	48.62	63.70
1	200	6.42	57.74	-67.55
0	270	-	58.21	-166.59
-1	340	-	126.62	-390.02
-2	410	-	434.42	-1100.0

Frictional force or drag along the tail skin is obtained from the thrust force of the shield onto the lining (jack force) and the force exerted on the face, arrived at 10469kN. Figure 4 shows the diagram depicting application of face pressure in the model. It was found that, when the face support pressure is less than the earth pressure at rest, the face deformation occurs inside the tunnel, and in extreme cases, the face collapse occures (N=3). When the face support pressure is larger than the earth pressure at rest, the compressional deformation of soil in front of the tunnel face occurs and the ground surface appears to heave (N= -2). When the face support pressure approximatly balances the lateral earth pressure, the settlement/heave start balancing (N=1). In this modelling, overload factors above 3 are also attempted but the soil body collapses inward through the face due to the extremely low face pressure. In the present case, the critical face pressure is found to be 200kPa. In the horizontal displacement column, the positive result indicates the soil displacement inward and the negative sign indicate outward direction. Figure 5a shows total vertical displacement from the numerical analysis, when face pressure of 410kPa applied at the face of the tunnel. Since, a high face pressure was applied, no ground surface settlement occurs but a 434.42mm heave is observed. Figure 5b shows the maximum horizontal displacement of 1100mm in the outward direction. Similarly Fig. 6a shows total vertical displacement from the model when face pressure was 200kPa at the tunnel face. Ground surface shows 6.42 mm settlement and 57.74 mm heave. Figure 6b shows a maximum of 67.55mm horizontal displacement of soil outward. Figure 7a shows total vertical displacement, when face pressure of 60kPa applied at the face of the tunnel. Since, a very low face pressure was applied; ground surface shows 59.30mm settlement and 32.41mm heave. Figure 7b shows 109.75mm of maximum horizontal displacement of soil inward depicting total collapse. The Fig. 8 combines all the analysis results and depicts the total vertical displacement corresponding to various face pressure applied. It can be seen that the deformation and failure of the tunnel face caused by the change of the support pressure applied in the EPB shield. This can be divided into three phases, in the first phase, when the face support pressure is greater than the earth pressure at rest, the compressional deformation of soil in front of tunnel faces occurs. Second phase, when the face support pressure is located between the earth pressure at rest and critical support or transition stage pressure, the face deformation caused by decreasing the support pressure is very small. The third phase, when the face support pressure decreases further, a significant deformation or the total collapse of soil body occurs. For this simulation, the magnitude of surface settlement at N=1 become very less with increasing face pressure to 200kPa, while the heave increases slightly. So in the present case, the face pressure at which the transition occurred is found to be 200kPa (N=1), which is considered to be the true face balance pressure.

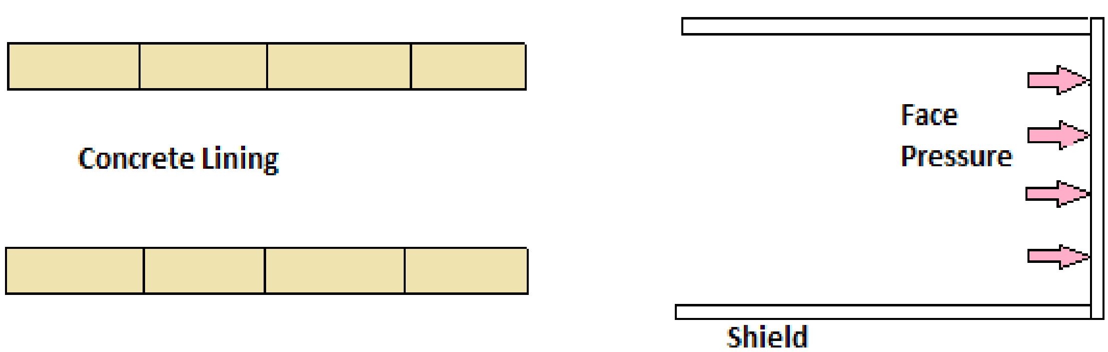
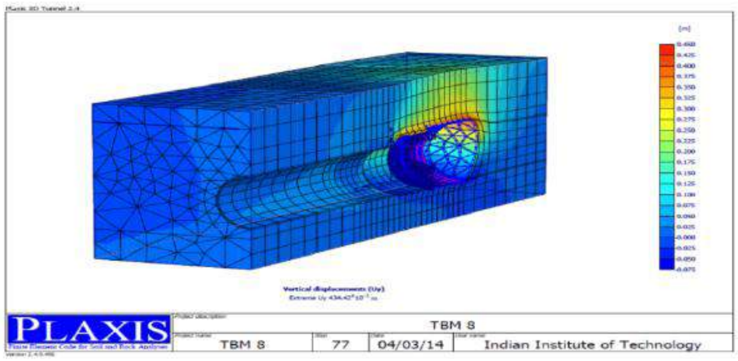
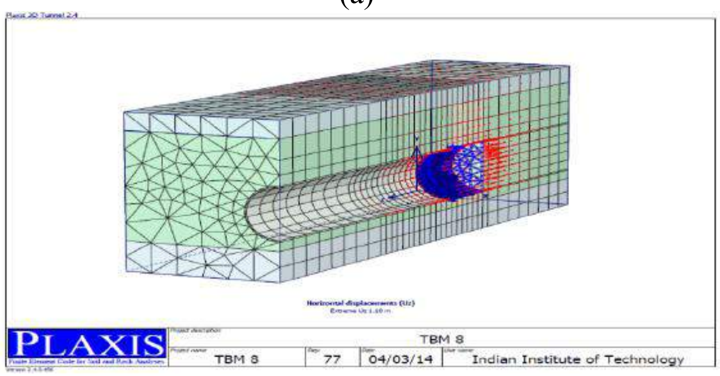


Figure 4: Schematic diagram showing application of face pressure

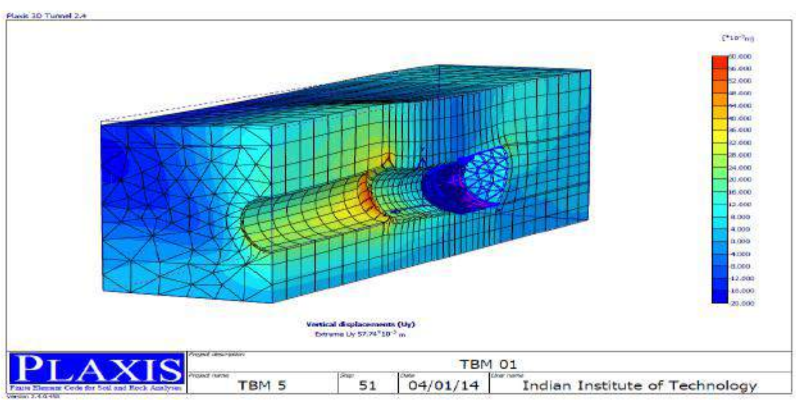


(a)



(b)

Figure 5: (a) Vertical displacement contours and (b) Horizontal displacement vectors (with face pressure 410kPa)



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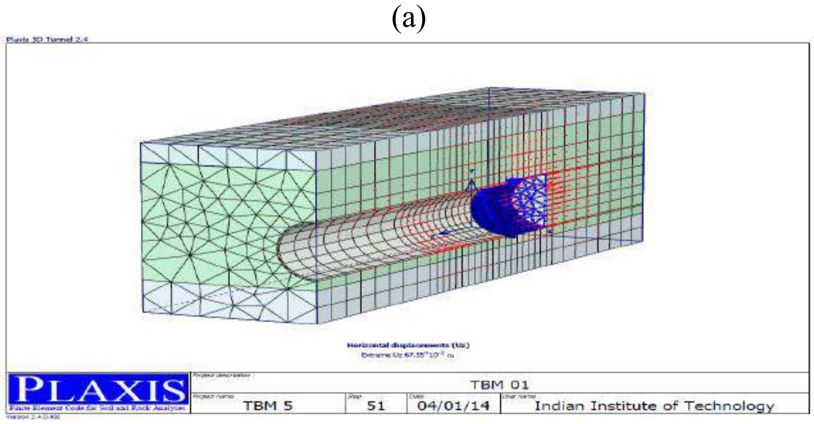
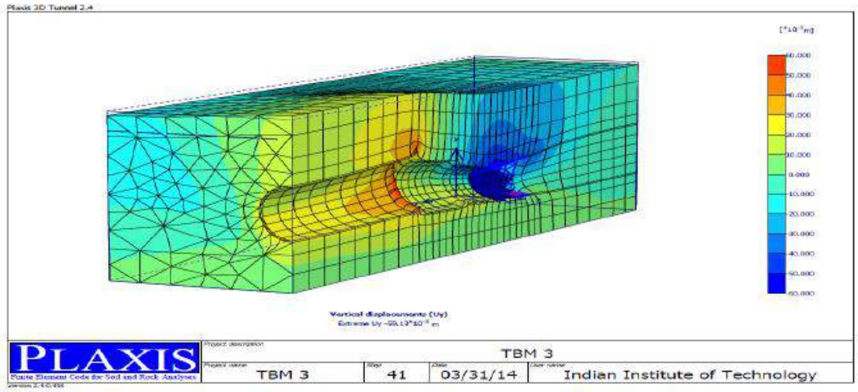
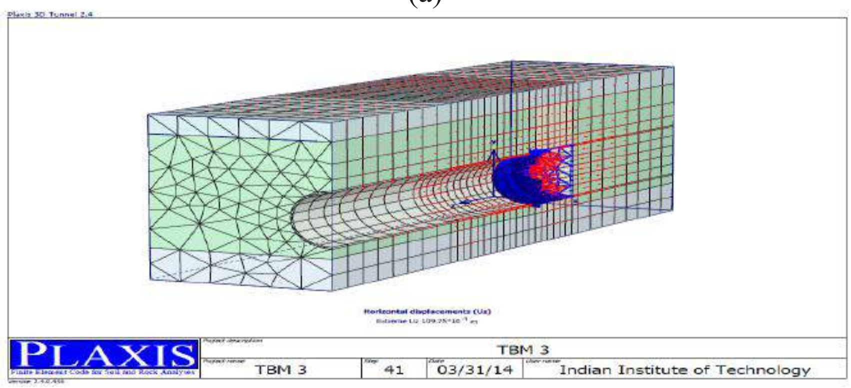


(b)

Figure 6: (a) Vertical displacement controur and (b) Horizontal displacement vectors (with face pressure 200kPa)



(a)



(b)

Figure 7: (a) Vertical displacement contour and (b) Horizontal displacement vectors (with face pressure 60kPa)

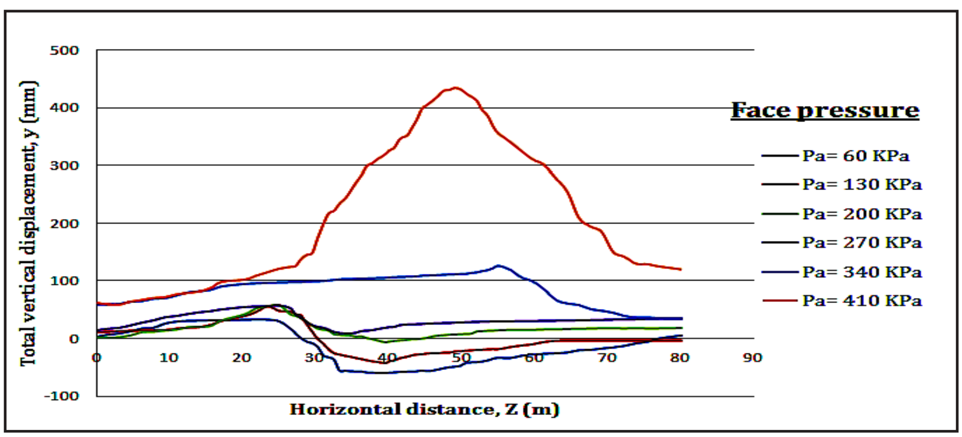


Figure 8: Vertical displacement versus horizontal distance for different face pressure

8. SUMMARY AND CONCLUSION

A numerical model using PLAXIS-3D has been developed to analyse the soil movement around the EPB tunnel that includes various geotechnical conditions. The face pressures are varied to see how they might directly influence the magnitude of total vertical displacement as well as horizontal movement. The deformation and failure of the tunnel face caused by the change of the support pressure (applied in the EPB shield tunnel face) is divided into three stages. First stage, when the face support pressures was greater than the earth pressure at rest (over load factor, $N \leq -2$), the compressional deformation of soil in front of tunnel faces occurs. Second stage, when the face support pressure is located between the earth pressure at rest and critical support or transition stage pressure (N=1), the face deformation caused by decreasing the support pressure is very small. The third stage, when the face support pressure is less than the face pressure ($N \geq 3$), a significant deformation or the total collapse of soil body occurs. In the present case, the face pressure at which the transition occurs is found to be 200kPa (N=1), which is considered to be the true face balance pressure and could capture nicely using the numerical model. The model is found to be capable of modelling the tunnel induced ground deformation and its control using face pressure.

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