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# Numerical modelling of tunnelling induced ground deformation and its control

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ARTICLE HISTORY

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ABSTRACT

Tunneling through cities underlain by soft soil, is commonly associated with soil movement around the tunnels and subsequently surface settlement. The predication of ground movement during a tunnelling project and optimum support pressure could be based on analytical, empirical or the numerical methods. The commonly used Earth Pressure Balance (EPB) tunneling machines, make use of the excavated soil in a pressurized head chamber to exert a support pressure to the tunnel face during excavation. This face pressure is a critical paramater in EPB tunnelling because the varying pressure can lead to the total failure and collapse of the face. The objective of the present study was to evalute the critical supporting face pressure and grout pressure by observation of the vertical deformation and horizontal displacement of soil body during tunneling. The face pressure and grout pressures were varied to see how they might influence the magnitude of surface settlements/heave. A numerical model using PLAXIS-3D Tunnel software package was developed to analyze the soil movement around the tunnel which can involves different geotechnical conditions. The ground surrounding the tunnel was found to be very sensitive to the face pressure and grout pressure in terms of surface settlementand the possibility of collapse in the body of soil.

**Keywords:** *Earth Pressure Balance (EPB); Face pressure; Ground movement; Critical support pressure; Surface settlement; Subsidence*

## 1. Introduction

Planning and construction of new infrastructures in urban areas frequently involve tunneling projects. Incessant population growth is demanding for more infrastructure and underground construction and this will continue to flourish as the favorite solution. Therefore, the prediction of tunnel-induced ground deformation has become a key issue in the planning process. Safe and successful construction of a tunnel is directly related to the tunnel face stability. The Earth Pressure Balance (EPB) Shields have been developed in part to minimize surface settlement. The rotating cutterhead is propelled forward by a series of jacks pushing from the concrete lining that has been installed in previous stage. It is the excavated material in the spoils chamber itself that acts as the support of the face. Measurement of the pressure is crucial in EPB tunneling; because if the pressure is not kept almost constant, the varying pressure can lead to the collapse of the face. Many researchers have proposed different analytical methods to determine the required pressure to stabilize the tunnel face mostly based on limit equilibrium approach or limit analysis [1-7]. An examination of field data of settlement in soft ground tunneling by Attewell [8] indicates that a major proportion of total soil deformation occurs immediately after construction. New and O'Reilly [9] reviewed the ground movements associated with tunneling 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 tunneling with potential damage to the existing structures and utilities on top of tunneling area. Mair and Taylor [10] studied the components of ground deformation associated with closed shield tunneling. The use of EPB machines with full tunnel face support significantly reduce the total volume loss as the tunnel advances. Clough and Schmidt [11] observed that the ground loss of the tunnel face contributed 1/4 to 1/3 to the total volume loss.

The prediction of ground movement during the tunneling and optimum support pressure could be based on the analytical, empirical or numerical methods. The empirical methods for settlement analysis usually neglect in-situ stress in contrast to the numerical methods such as finite element method. Nevertheless, they don't take the interaction between the soils and lining into consideration, therefore they are able to account for the support stiffness. Normally, the empirical methods are used as a preliminary verification step to grasp an idea about the displacement that occurs at ground surface. Finite element analysis has become the more acceptable tool for tunnel-induced settlement study. Present work was aimed to evaluate the critical supporting pressure of

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the face and the grout pressure by observing the deformation of soil mass while tunnelling is under way. Subsidence analysis was carried out using PLAXIS-3D Tunnel code to investigate the face pressure balance and grouting pressure required for overcoming the possibility of collapse.

## 2. Surface settlement

The most common empirical method to predict ground movements is based on Gaussian distribution. Peck [12] and Schmidt [13] were the pioneers to show that the transverse settlement trough, after construction of a tunnel, in many cases can be well described by the Gaussian function (Figure 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 [14] defined the ground loss as the volume of soil that is displaced 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,

$$GL = \frac{V_i}{V_0}.100\% \quad (1)$$

Where, $V_i$ = Trough volume, $V_0$ = Tunnel opening volume ($\pi.r^2$) and r = Radius of the tunnel. Based on the shape of the Normal distribution curve, Peck [12] showed that the maximum settlement $S_{v,max}$ can be given by,

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

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

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

Where, y is the horizontal distance from the tunnel axis and 'i' the horizontal distance from the tunnel axis to the point of inflection of the settlement trough. Peck [12] suggested that the percentage of ground loss (GL) is usually in the range of 1-2% for stiff clay, 2-5% for soft clay and less than 1% for sandy soil. Mair [15] 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 the construction process. O'Reilly and New [16] has proposed a relationship between ground loss and tunneling quality. For good practice in firm ground, ground loss is assumed to be 0.5% maximum, whereas for poor practice in soft ground it may be as high as 4.0% or more. The volume of the settlement trough per unit length of tunnel, '$V_s$' is obtained by integrating equation 3 yields,

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

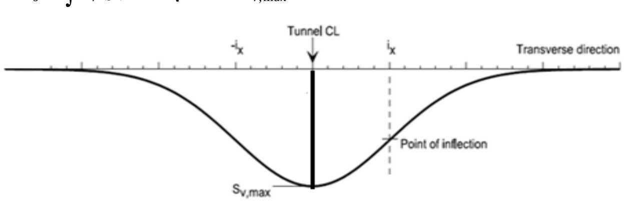


**Figure 1. Gaussian curve for transverse surface settlement trough.**

In addition to the settlement volume Vs, one has to consider the ground loss Vt which is the volume of the ground that has deformed into the tunnel after the tunnel has been constructed. For tunneling in undrained soil, the settlement volume is more or less equal to the ground loss, but the settlement volume tends to be somewhat smaller for water-drained excavations. The dilation and swelling due to the unloading may result in soil expansion, such that Vs<Vt [14]. However, differences tend to remain small and it can be assumed that Vs=Vt.

## 3. Width of the settlement through

The distance from the tunnel axis to the inflection point 'i', and determining the width of the settlement trough has been the topic of many investigations. Peck [12] suggested a relationship between tunnel depth Zo and tunnel diameter D, depending on ground conditions. After Peck (1969), many other researchers [11,14,16] have come up with similar relationships. O'reilly and New [16] presented results from multiple linear regression analysis performed on field data, confirming the strong correlation between 'i' and tunnel depth. They didn't report significant correlation between 'i' and tunnel diameter or method of construction (except for very shallow tunnels with a ratio of overburden to diameter less than one). They stated that for most practical purposes, the regression lines may be simplified to the following form (equation 5),

$$i = K.Z_o \quad (5)$$

Where, $K$ is a trough width parameter, with $K$ = 0.5 for clayey ground and $K$ = 0.25 for sandy soil. The approach of equation 5 has also been confirmed by Rankine [17], who presented a variety of tunnel case histories in different types of soils. Mair and Taylor [10] presented a large number of tunneling data with different linear regressions for tunnels in clays, sands and gravels. The regressions analysis confirmed the findings of O'reilly and New [16] for clayey soils, with a trough width parameter ranging between 0.4 to 0.6 with a mean value of 0.5. However, for sandy soils '$K$' was ranging between 0.25 to 0.45 with a mean value of 0.35, indicating partially wider settlement troughs.

## 4. Depth of the settlement through

Craig and Muir Wood [18] 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 equation 2 is used for an initial estimation of $S_{v,max}$. The construction method of the tunnel has a considerable effect on the ground loss ratio. Depending on the equipment, control procedures and level of experience of the crew, ground loss ratio can vary between 0.5% and 2% in homogenous ground. In sands, a loss up to 1% may be seen, whereas in soft clays it ranges from 1% to 2%, as reported by Mair [19]. Considering data for mixed ground profile with sands or fills overlaying clays, Mair and Taylor [10] reported values between 2% to 4%. For tunnels in undrained clays, Clough and Schmidt [20] 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 and the tunnel face would be 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 considerably big, leading to instability in tunnel face with large ground movements.

## 5. Finite elements model using Plaxis-3D

A tunnel has been modeled to demonstrate the effect of tunneling and face pressure on the surface settlement and ground deformation. The water table was modeled 3m deep from the ground surface. The depth of the center of the tunnel is 12m and the inside and outside diameters of the tunnel are 8.5m and 9m, respectively (Figure 2). The 3D model used in the analysis is shown in Figure 3, is 80m long, 26m high and 20m wide. In order to model the excavation section, twenty slices each of which 1.5m wide were considered at the center portion, at the front and end portion. 25m sections were included to reduce the influence of boundary conditions. In Table 1, one can observe the various soil material properties used for the FE analysis using Mohr-Coulomb material model. PLAXIS can handle cohesion-less sands (c=0), but it is advised to enter a small value of cohesion (c > 0.2 kPa) [21]. Table 2, shows the structural element properties that were used in the analysis. The normal stiffness $EA$, flexural rigidity $EI$, and weight $w$, were chosen based on the properties of the materials of the shield used. It should be noted that *EA* and *EI* are 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 as well. 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, which means the value is less than 1. In Table 3, one can see 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 to one, in the shield case. The input for dry unit weight of the lining is 24kN/m³ as of standard concrete.

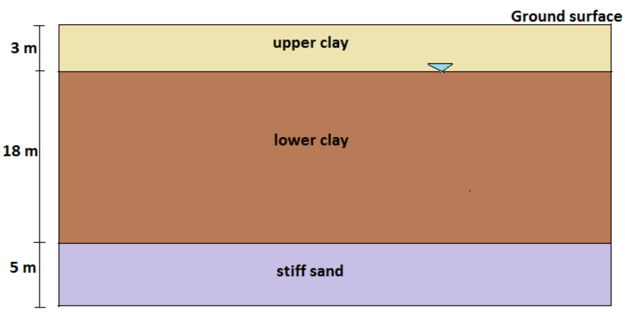


Figure 2. Soil profile showing different layers.

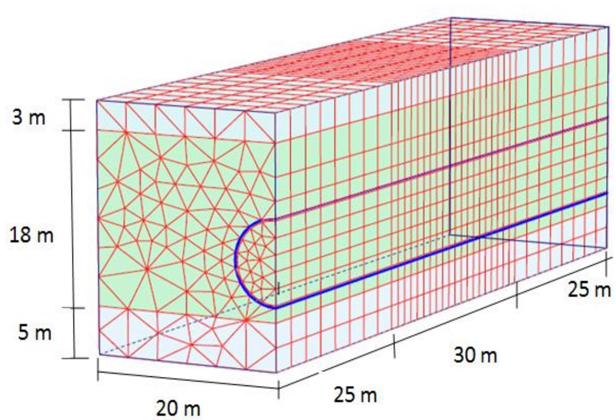


Figure 3. 3D mesh for the tunnel to be modeled using PLAXIS-3D.

Table 1. Material properties - Soil.

| Parameter | Name | Upper clay | Lower clay | Stiff- Sand |
|---|---|---|---|---|
| Material model | model | Mohr-Coulomb | Mohr-Coulomb | Mohr-Coulomb |
| Type of material behavior | Type | Undrained | Undrained | Drained |
| Dry unit weight (kN/m³) | $\gamma_{unsat}$ | 16 | 16 | 17 |
| Saturated unit weight (kN/m³) | $\gamma_{sat}$ | 18 | 18 | 20 |
| Young's modulus (kN/m²) | $E$ | 8.4 x 10³ | 1 x10⁴ | 2.5 x 10⁴ |
| Poisson's ratio | $v$ | 0.35 | 0.35 | 0.3 |
| Undrained shear strength (kN/m²) | $c$ | 48 | 70 | 3 |
| Friction angle (°) | $\varphi$ | 10 | 20 | 30 |
| Permeability (m/day) | $K_y, K_x$ | 0.001 | 0.05 | 1 |
| Dilatancy angle | $\psi$ | 0 | 0 | 0 |

Table 2. Material properties - Structural plates representing shield.

| Parameter | Name | Value |
|---|---|---|
| Type of behavior | Material type | Elastic |
| Normal stiffness (kN/m) | $EA$ | $8.20 \times 10^6$ |
| Flexural rigidity (kN/m²/m) | $EI$ | $8.38 \times 10^4$ |
| Equivalent thickness (m) | $d$ | 0.35 |
| Unit weight (kN/m³) | $w$ | 38.5 |
| Poisson's ratio | $v$ | 0.3 |
| Interface strength reduction | $R_{inter}$ | 0.9 |

Table 3. Material properties - The concrete lining .

| Parameter | Name | Concrete Lining |
|---|---|---|
| Identification | - | concrete |
| Type of material behavior | Model | Linear-elastic |
| Material type | type | Non-porous |
| Volumetric weight (kN/m³) | $\Upsilon_{unsat}$ | 24.0 |
| Young's modulus (kN/m²) | $E_{ref.}$ | $3.1 \times 10^7$ |
| Poisson's ratio | $\upsilon$ | 0.2 |
| Interface strength reduction | $R_{inter}$ | 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 overhead or surcharge contributes to additional amounts of earth pressure. For the tunnel alignment below water table, water pressure is another significant component of pressure acting at the tunnel face. For stability, the layers of soil at the tunnel face must have adequate strength to counterbalance these forces. In many projects, tunnels will encounter several layers of loose soils or weathered rock. The face might not be strong enough to bear such pressures and may be unstable leading to the collapse of soil mass resulting in excessive settlement on the ground surface. Support pressure (face pressure) needs to be built up at the face of tunnel, to balance the pressure generated by the weight of soil, water and overlying infrastructure. 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 soil consolidation and thereby surface settlement. In case of mechanized tunneling, support media will be employed to build the required face support pressure. Common support media used are bentonite slurry, earth paste, and compressed air. Choosing suitable support medium depends on various factors such as 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 required to stabilize the face, which in turn will minimize settlement, and prevent from soil body collapse.

The pressure exerted on the face is controlled by the relative amounts of material that go in and go out 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 in homogeneous, plastic clay [12, 22]. Peck [12] reported that the overload factor (N) (equation 6) shall not exceed about 6. Tunneling could be carried out without unusual difficulty in plastic clays if N remains below 5. It was also noted that in shield tunneling, the values of N much greater than 5 may cause the clay to infiltrate the tail void too rapidly, so that the annulus space can't 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, $\sigma_T$ = Support pressure applied at the center
$\sigma_S$ = Surcharge load
$\Upsilon$ = Soil unit weight
$H$ = Depth to center of tunnel
$S_u$ = Undrained shear strength

## 7. Analysis of face pressure

Different face pressures, corresponding to different values of overload 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 demonstrates the overload factors and corresponding average face pressures that were modeled. 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 supporting pressure is less than the earth pressure at rest, the face deformation occurs inside the tunnel, and in extreme cases, the face collapse occurs (N=3). When the face supporting 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).

**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 | |
| 2 | 130 | 42.28 | 48.62 | 109.75 |
| 1 | 200 | 6.42 | 57.74 | 63.70 |
| 0 | 270 | - | 58.21 | -67.55 |
| -1 | 340 | - | 126.62 | -166.59 |
| -2 | 410 | - | 434.42 | -390.02 |

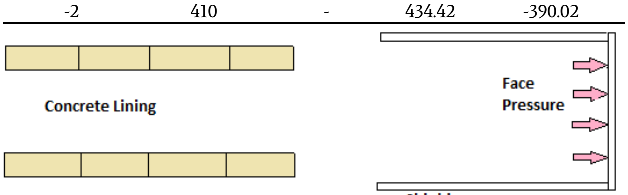


**Figure 4. Schematic diagram showing the application of the face pressure.**

When the face supporting pressure is approximately in balance with the lateral earth pressure, the settlement/heave start balancing (N=1). In this modeling, overload factors above 3 were also tested but the soil body collapsed inward through the face due to the extremely low face pressure. In the present case, the critical face pressure is found to be approx. 200kPa. In the horizontal displacement column, the positive result indicates the soil displacement inward and the negative sign indicates outward displacement. Figure 5(a) shows total vertical displacement from the numerical analysis, when a face pressure of 410kPa was applied at the face of the tunnel. Since a high face pressure was applied, no ground surface settlement occurred but a 434.42mm heave was observed. Figure 5(b) shows the maximum horizontal displacement of 1100mm in the outward direction. Similarly Figure 6(a) 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 6(b) shows a maximum of 67.55mm horizontal displacement of soil outward. Figure 7(a) shows total vertical displacement, when face pressure of 60kPa was 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 7(b) shows 109.75mm of maximum horizontal displacement of soil inward which denotes total collapse.

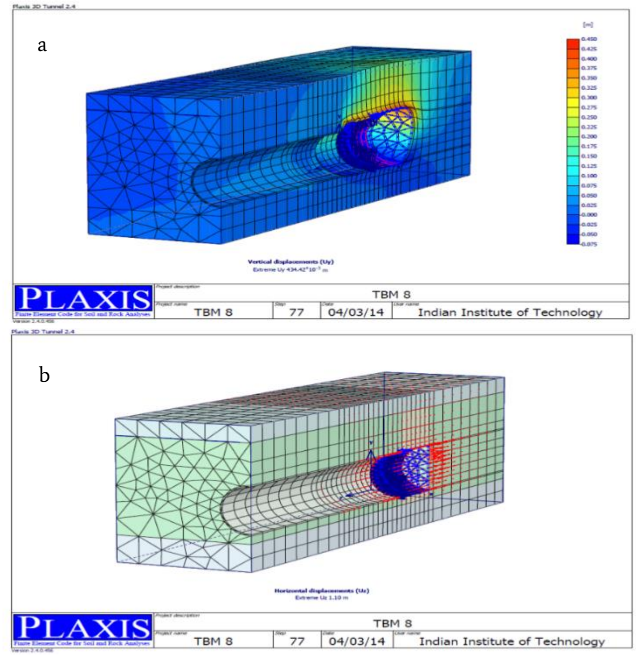


**Figure 5. (a) Vertical displacement contours (b) Horizontal displacement vectors (with face pressure = 410kPa).**

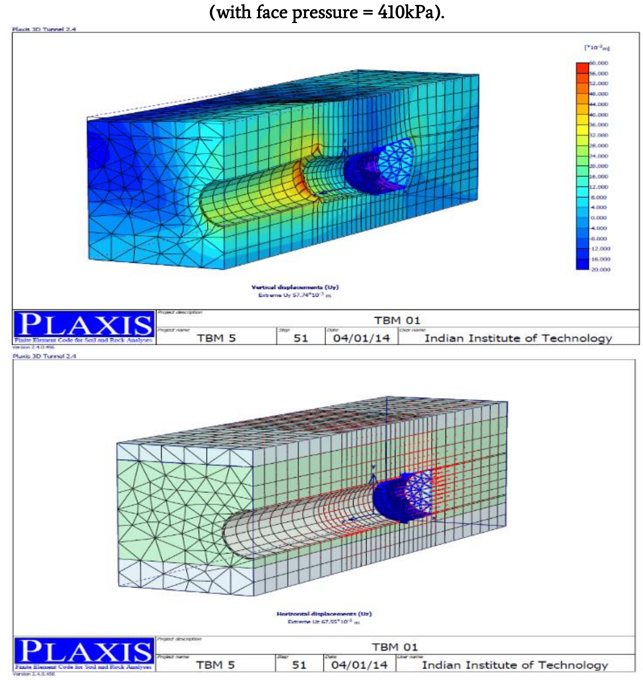


**Figure 6. (a) Vertical displacement contours (b) Horizontal displacement vectors (With face pressure = 200kPa).**

The Figure 8 combined 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 were caused by the change of the supporting pressure applied in the EPB shield. This can be divided into three phases; in the first phase, when the face supporting pressures is greater than the earth pressure at rest, the compressional deformation of soil in front of tunnel face occurs. In the second phase, when the face supporting pressure is in the range between the earth pressure at rest and critical supporting or transition stage pressure, the face deformation caused by decreasing supporting pressure is insignificant. In the third phase, when the face supporting pressure decreases further, a significant deformation or total collapse of soil body occurs. For this simulation, the magnitude of surface settlement at N=1 becomes very small with increasing face pressure to 200kPa, while the heave increases slightly. So in the present case, the face pressure at which the transition could occur was found to be 200kPa (N=1), which is considered to be the true face balance pressure.

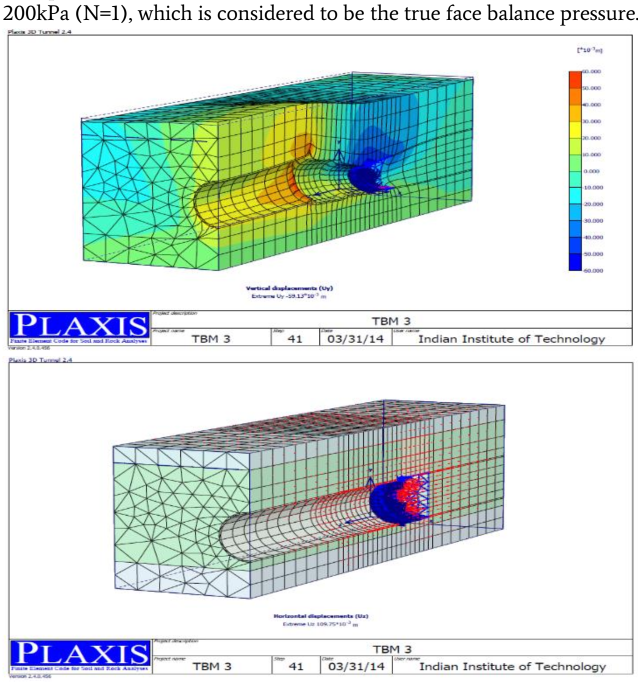


Figure. 7 (a) Vertical displacement contours (b) Horizontal displacement vectors (with face pressure = 60kPa).

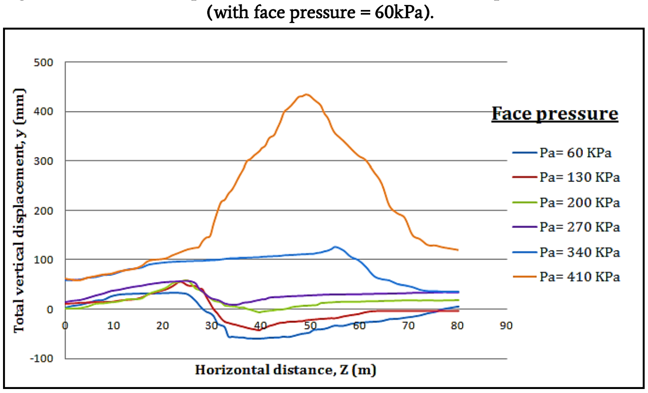


Figure 8. Vertical displacement versus horizontal distance for different face pressures.

## 8. Face pressure with grouting

Grouting around a tunnel lining is the determinant of the loading on it and is an important parameter in order to predict the settlement above the tunnel. The injection of clay grout into the tailpiece void after lining erection may be expected to decrease settlements. However, much of the soil movement will occur prior to the ground injection. As an initial approximation, it is assumed here that no initial soil deformations due to the excavation of the tunnel and lining installation occurs prior to clay grout injection. In this modeling, at the tail of the TBM, grout is injected to fill up the gap between TBM and the final tunnel lining segments located just behind the shield i.e. from 25m to 28m. The grouting process is simulated by the application of pressure on the surrounding soil. The grout is applied as a water pressure in the model in an outward radial direction, as shown in the Figure 9. Table 5 shows the average grout pressures corresponding to total vertical settlement using the base face pressure of 200kPa. When the grout pressure is less than 100kPa the soil body would have a tendency for settlement and heave but at grout pressure of 110kPa, no ground settlement would occur but a small degree of heave could be observed. When the grout pressure becomes greater than 110kPa, the magnitude of heave increases further. This implies that the grout pressure of 110kPa is the optimum amount to counterbalance for the present case. Figure 10 shows total vertical displacement contours, when grout pressure of 120kPa is injected to fill up the gap between TBM and the final tunnel lining segments. As high grout pressure was applied, no ground surface settlement but a heave of 67.50mm was observed. Figure 11 shows total vertical displacement when the grout pressure was around 110kPa. Due to the balanced grout pressure applied, no ground surface settlement was observed and the heave reduced to 60.06mm. Figure 12 illustrates total vertical displacement when the grout pressure of 70kPa was injected. Due to very low grout pressure, the ground surface shows a 29.39mm settlement and 55.47mm heave. Figure 13 shows a wavy vertical displacement profile as observed due to applying grout pressure between concrete lining and shield. It can be concluded that 110kPa grout pressure is the approximate optimum counterbalancing grout pressure for the present case.

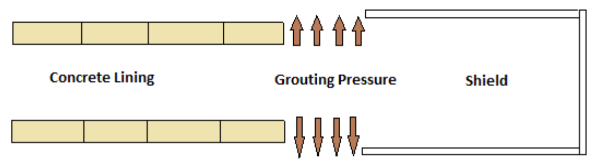


Figure 9. Schematic diagram showing application of grout pressure.

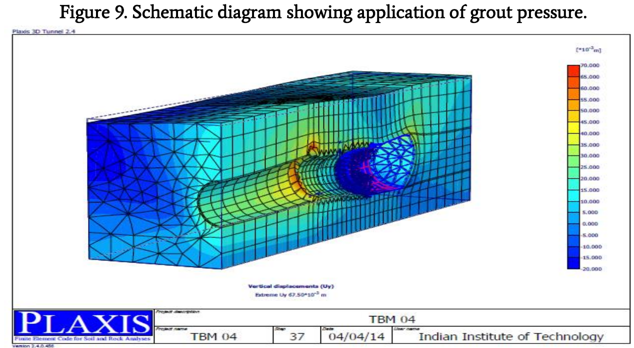


Figure 10. Vertical displacement contours for grout pressure = 120kPa.

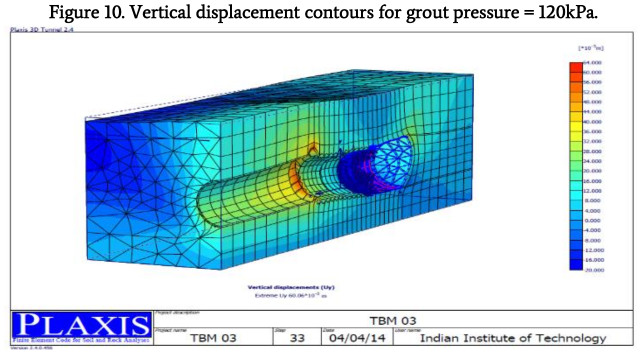


Figure 11. Vertical displacement contours for grout pressure = 110kPa.

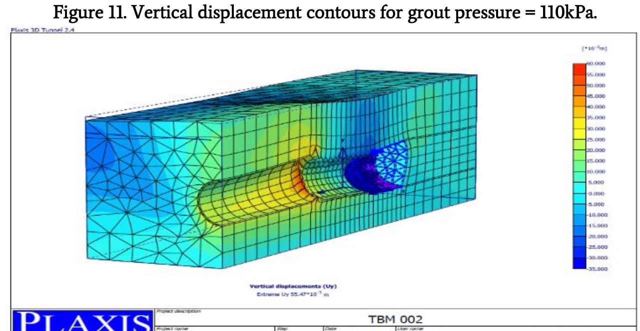


Figure 12. Vertical displacement contours for grout pressure = 70kPa.

Table 5. Grout pressure variation and corresponding deformation.

| Grout pressure (kN/m²) | Face pressure (kN/m²) | Settlement (mm) | Heave (mm) | Displacement (mm) |
|---|---|---|---|---|
| 70 | 200 | 29.39 | 55.47 | 42.16 |
| 80 | 200 | 16.24 | 56.62 | -67.58 |
| 90 | 200 | 10.25 | 58.75 | -67.43 |
| 100 | 200 | 4.68 | 58.92 | -80.29 |
| 110 | 200 | - | 60.06 | -94.29 |
| 120 | 200 | - | 67.5 | -122.18 |

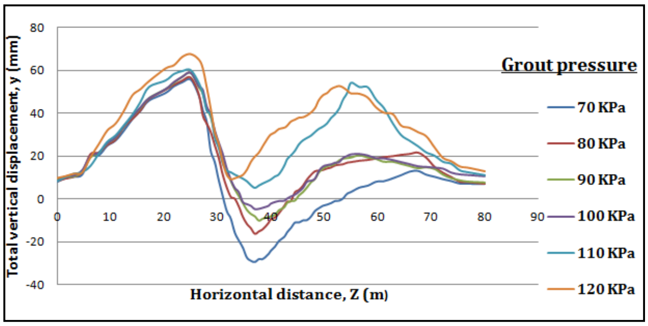


Figure 13. Vertical displacement versus horizontal distance for different grout pressures.

## 9. Summary and conclusion

Face pressure is a critical parameter in EPB tunneling as the varying pressure can lead to the total collapse of the face. A numerical model using PLAXIS-3D was developed in order to analyze the soil movement around the EPB-excavated tunnel that takes various geotechnical conditions into consideration. The face pressure and grout pressures were varied to see how they might influence the magnitude of total vertical displacement as well as horizontal movement directly. The deformation and failure of the tunnel face caused by the change of the supporting pressure applied in the EPB shield tunnel face can be divided into three different stages. In the first stage, when the face supporting pressures was greater than the earth pressure at rest ($N \leq -2$), the compressional deformation of soil in front of tunnel face would occur. In the second stage, when the face supporting pressure is somewhere between the earth pressure at rest and critical (transition-phase) supporting pressure ($N=1$), the face deformation caused by decreasing supporting pressure would be very small. In the third stage, when the face supporting pressure is less than the face pressure ($N \geq 3$), a significant deformation or the total collapse of soil body would occur at was at least probable. The same was true for the case when grout pressure was applied, i.e. ground settlement decreased and increased according to the grout pressure at the void space filled behind shield tail. Results show that the ground surrounding the tunnel is very sensitive to alterations in the grout pressure in terms of surface settlement and possibility of collapse of the soil body. It was found that the present numerical model is capable of predicting the tunnel-induced ground deformation and applicable face pressures to control it.

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