A Different View on TBM Face Equilibrium in Permeable Ground

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ABSTRACT
The construction of mechanized tunnels in soft ground has evolved significantly over the last 20 years, especially in the control of the face pressure and the closure of the soil-lining void to reduce the induced settlements. On the other hand, several mechanisms of the TBM excavation cycle are still not taken into account for routine design calculations, such as the increment of water pressures in front of the tunnel face, the flow of excavation fluids around the shield, the dynamic equilibrium between the grout pressures and the excavation convergence, among others. This paper discusses specifically the issue of face pressures and how several mechanisms, which are routinely not considered, can be easily verified and incorporated into the state-of-practice of design.

INTRODUCTION
The stability of underground excavations is traditionally assessed through analytical solutions based on the lower bound theorem of plasticity (Atkinson and Potts 1977; Mühlhaus 1985) or the limit equilibrium method (Anagnostou and Kovári 1994; Messerli et al. 2010). These methodologies depend on the tunnel geometry, certain ground parameters and the support pressure. When conventional construction methods are used, the internal support pressure is an abstract representation of the combined effects of partial excavation, ground reinforcement, and the different support elements.

The use of closed-face tunnel boring machines (TBM) changed this perspective, as the magnitude of pressures acting on the excavation boundary could be controlled in a more direct way. In front of the TBM the ground is excavated as the cutterhead rotates and the cutting tools scrape the ground from the tunnel face while additives are injected to condition the material. Water, polymers, bentonite, and foam can be used as additives under different conditions (Thewes et al. 2012). The loosened ground with additives, herein referred to as the mixture, flows through the openings at the cutterhead and into the excavation chamber. This mixture is kept pressurized to support the face.

These and all other TBM processes can be actively controlled through the operation of mechanical or hydraulic systems that make up the machine. It is self-evident that for each of these TBM actions there will be a reaction from the ground to achieve equilibrium. However, this last point is frequently overlooked, resulting in idealized concepts of how mechanized tunneling works. Most importantly, these frameworks fail to explain important features of ground response that have been observed in the field (Bezuijen and Talmon 2008; Dias and Bezuijen 2015). Nonetheless, it is fair to say that this is something generally known and discussed in the specialized academic community. However, to the
authors’ knowledge, most projects still use these idealized concepts, with a few exceptions of challenging projects (Aime et al. 2004; Kaalberg et al. 2014) and post-mortem investigations of projects that did not go as expected. Therefore, a realistic improvement in the design practice of mechanized tunnels in soft ground would be to understand why that is the case and how these models can be incorporated into the state-of-practice.

In this paper, the framework describing the mechanisms around the face of a TBM will be discussed. The first point to be recognized is that the supporting mixture is the medium through which the machine forces are transferred to the face of the tunnel. In earth pressure balance (EPB) machines, the mixture is composed of the excavated soil and additives, and is removed from the chamber mechanically, through a screw conveyor. In slurry pressure balance (SPB) and mixshield machines, the mixture is mostly composed of a slurry suspension, and is removed through a hydraulic circuit. The chambers of mixshield machines are divided by a submerged wall, in a working chamber, completely filled with slurry, and a pressure chamber, partially filled with a pressurized air bubble that controls the pressure at the chamber and prevents significant fluctuations.

**STRESS TRANSFER**

The structure of the mixture is very important to the understanding of how the face pressure is transferred to the tunnel face. The slurry suspensions in SPB and mixshield machines, which can be extracted through a hydraulic circuit, are normally more fluid than the paste consistency necessary to control the pressure gradient along the screw conveyor on EPB machines. However, in both cases, the mixture presents an open matrix, where the solid particles are in a suspension with negligible effective stresses. The rheology can then be considered equivalent to a fluid and, as the mixture flows slowly, viscous forces can be disregarded.

These fluid mixtures can only support isotropic stress states, represented by an equivalent scalar pressure. Adversely, the undisturbed ground at the tunnel face will, in most cases, be standing under an anisotropic stress state, set by the coefficient of earth pressure at rest ($k_0$). Therefore, it is fundamentally infeasible to transfer a face pressure that will match the in situ stress of the ground in every direction. Take, as an example, a pressure that matches the vertical stress at the tunnel roof, as shown in Figure 1. The same pressure will be acting at the horizontal direction, where the horizontal stress in ground is probably of a different magnitude. The same is true at the tunnel invert, where a perfect balance cannot even be attained in the vertical direction, because of the differences between the volumetric weight of the mixture and the ground.

As a consequence, the tunnel face will always undergo stress increments, and the associated strains. Along the radial direction, the excavated perimeter can either contract or expand. If it contracts while in contact with the cutterhead, an additional volume of ground will be excavated. For an expanded section, there will be a gap between the ground and the cutterhead. This allows the supporting mixture to flow around the shield depending on the pressure at the face and the grout pressure at the back of the shield. Another point to consider is that if volumetric strains are induced in a saturated ground, they will generate increments of pore water pressure that will lead to consolidation. Of course, the time frame for dissipation will depend on the ground permeability and drainage conditions.
The fact that the supporting mixture has such a loose matrix that it acts as a fluid raises the question of whether the face pressure should be considered by its hydraulic head or just as a total stress boundary. Here, a parallel is normally traced with diaphragm walls (slurry walls), where the supporting fluid creates an impermeable layer on which the fluid pressure is applied and the hydraulic head is dissipated. In this way, the pressure can be transferred to the ground without changing the hydraulic boundary conditions. The same thing should occur at the TBM face, through the so-called filter cake. However, one should consider that the ground at the tunnel face is constantly being removed while the filter cake is being formed, which can affect the process.

This problem was identified when excess pore pressures were measured in front of SPB (Bezuijen et al. 2001; 1999) and EPB machines (Bezuijen 2002), revealing that the ideal process of cake formation is not always achieved and depends on the properties of the ground, the additives and the excavation speed (see Figure 2 for example measurements). To quantify these effects, one must first understand how the supporting fluid creates an impermeable layer on the ground. The pressure in the supporting fluid must be higher than the water pressure in the ground, inducing the fluid to flow into the ground. The fluid carries suspended material that clogs the ground pores, reducing its permeability. As far as this process is concerned, the foam bubbles used to condition permeable soils on EPB TBMs have the same purpose as the slurry particles on SPB and mixshield TBMs.

The second step is to quantify the gradient inducing the flow from the face. An analytical formulation can be derived (Bezuijen 2002; Bezuijen et al. 2001), based on the approximation that there is a constant infinitesimal hydraulic source at every point in the tunnel face. This distributed head is defined with reference to the in situ water pressure. By equating the volumetric flow rate from the source \( A=dr.r.d\theta \) with the flow rate at a certain radial distance \( s \) along a semi-spherical domain in front of the tunnel \( A=2.\pi.s^2 \), one obtains:

\[
q.r.d\theta.dr = k \cdot \frac{d\phi}{ds} \cdot \frac{4.\pi.s^2}{2}
\]  

(1)
\[ q \text{ is the discharge from the point source, assumed constant all over the tunnel face.} \]

By integrating Equation 1 along the following limits: \( \phi = [\phi(S), \infty]; s = [S, \infty]; r = [0, R]; \theta = [0, 2\pi] \), and defining \( s = \sqrt{x^2 + r^2} \), one obtains:

\[
\phi(x) = \frac{\phi_0}{R} \left( \frac{\sqrt{x^2 + R^2}}{x - R} \right)
\]

where \( \phi_0 \) is the incremental piezometric head at the tunnel face (\( x=0 \)).

From Equation 2 it is possible to calculate the hydraulic gradient at the tunnel face as:

\[
\frac{d\phi}{dx}\bigg|_{x=0} = -\frac{\phi_0}{R} \left( \frac{\sqrt{x^2 + R^2}}{x - R} \right)\bigg|_{x=0} = -\frac{\phi_0}{R}
\]

The penetration velocity can then be defined as:

\[
v_p = \frac{k \phi_0}{nR}
\]

where \( n \) is the ground porosity and \( k \) is the ground hydraulic conductivity to the penetration fluid.

If the penetration velocity \( (v_p) \) is smaller than the TBM drilling velocity then the excavation tools will be scraping deeper than the slurry/muck penetration and \( \phi_0 \) will be equal to the face pressure. On the other hand, when \( v_p \) is larger than the TBM drilling velocity an impermeable layer will be formed. However, excess pore pressures will still occur in front of that layer, as the layer moves through the ground at a rate equal to the TBM drilling velocity. For this condition Equation 4 can also be used, but now the penetration velocity \( (v_p) \) is known (equal the TBM drilling velocity) and the incremental piezometric head \( (\phi_0) \) can be calculated, resulting in a value smaller than the pressure in the mixing chamber.
This situation occurred during drilling of the N/S line in Amsterdam (Kaalberg et al. 2014), where the measured excess pore pressure close to the TBM was only 40% of the applied excess pressure in the mixing chamber. This process is explained more in detail in Bezuijen (2016). There are some recent attempts to simulate this process numerically with a model for the slurry penetration within a numerical groundwater flow calculation (Zizka et al. 2015).

**PHASE BALANCE IN THE MIXTURE**

The increments of pore water pressure in front of the tunnel and the consequent water outflow can have a significant impact on foam conditioning, which depends heavily on the amount of water in the supporting mixture. The foam is formed by mixing a surfactant solution, which presents a certain liquid volume (QL), with compressed air. This forms a structure where gas is trapped in the foam bubbles. The volume of foam (QF) is used to calculate the foam expansion ratio (FER=QF/QL), dividing it by the original liquid volume of the solution, and the foam injection ratio (FIR=QF/QS), dividing it by the volume of excavated ground (QS).

Once the foam blends into the supporting mixture, its additional volume will increase the initial porosity of the mixture (n₁), described in Figure 3a, to a porosity that is suitable for the TBM operation (n₂). For an initially dry mixture, the foam will occupy the air spaces (Figure 3b), so the volume of foam needed to increase the porosity from n₁ to n₂ can be calculated as:

\[ V_f^{(dry)} = V_i \cdot \frac{n_2(1-n_1)}{1-n_2} \]

(5)

If the ground is originally saturated, one must consider the possibility that the face pressure will induce groundwater flow from the face, in which case the initial amount of water will be reduced or even increased, depending on the flow conditions. Considering the hypothesis that there is no water flow (Figure 3c), the necessary volume of foam can be calculated as:
For the case where water flows out of the mixture (Figure 3d), the necessary volume of foam can be calculated as:

\[ V_{F}^{(\text{flow})} = V_{i} \cdot \frac{n_{2} - n_{1}}{1 - n_{2}} \]  \hspace{1cm} (6)

where the Foam Water Replacement Ratio (FWR) is defined as the volume of water that flows out of mixture over the initial volume of water.

Using Equations 5, 6 and 7 one can calculate the necessary foam injection ratio (FIR) for these three conditions. A detailed calculation example is presented in the next section.

**FACE STABILITY**

Nowadays, the most commonly used method to assess the stability of a mechanized tunnel is the limit equilibrium wedge stability analysis (Anagnostou and Kovári 1994; Messerli et al. 2010). The minimum required face pressure is composed of two parts: one to guarantee the wedge stability (S), considering effective stresses; and another to support the water pressures (W). The first part (S, see Figure 4) depends on the vertical forces due to the overlying prism (V) and self-weight of the wedge (G), and on the shear resistance along the vertical triangular walls of the wedge (T). The magnitude of the resultant along the inclined plane (R) is unknown. However, its direction at limit equilibrium is \( \phi \), the friction angle of the ground, with respect to the normal vector. Therefore, it is possible to calculate equilibrium along the direction perpendicular to R, based on the wedge angle (\( \theta \)), so that R can be ignored. Referring to the trigonometric scheme in Figure 4, one can derive the following, where the wedge angle (\( \theta \)) should be set to maximize the value of S:

\[ S = \frac{(V + G)}{\tan(\theta + \phi)} - \frac{2.T.\cos(\phi)}{\sin(\theta + \phi)} \]  \hspace{1cm} (8)

For a hydrostatic distribution of water pressure, the components V, G and T can be calculated explicitly. It should be noted that Terzaghi’s arching theory is often used to alleviate the overburden of the prism to calculate the V component. There are contrasting views on how the horizontal stresses should be calculated for that, so the authors decided not to consider this effect herein. Therefore the three components can be calculated as follows:

\[ V = \sigma''_{v}(Zt).D^{3} \cdot \tan \theta \]  \hspace{1cm} (9)

\[ G = \frac{D^{3} \cdot \tan \theta}{2} \cdot \gamma' \]  \hspace{1cm} (10)

\[ T = k_{o} \cdot D^{2} \cdot \tan \phi \cdot \tan \theta \left( \sigma''_{v}(Zt) + \frac{D \gamma'}{3} \right) \]  \hspace{1cm} (11)

where \( \sigma''_{v}(Zt) \) is the vertical effective stress at the depth of the tunnel crown and \( \gamma' \) is the volumetric weight of the soil immersed in water.

The component to support the water pressures (W) can also be calculated explicitly, multiplying the hydrostatic pressure at the depth of the tunnel centerline by the area of the wedge (D²). However, as
discussed in the previous section, the distribution of pore water pressure in front of a TBM is often not hydrostatic during drilling. The difference between the face pressure and the hydrostatic pressure creates a groundwater flow pattern that can be roughly described with Equation 2.

This changes the face pressure calculation, as the water pressure reduces the effective stresses to compute $V$, $G$ and $T$, which in turn reduce the required effective support ($S$), while it also increases the support necessary for the water pressure ($W$).

This iterative balance can be calculated by discretizing the wedge area along the vertical direction ($dy$). Each slice of the wedge will be incrementally shorter ($dy\cdot \tan \theta$) in the horizontal direction. By integrating Equation 2 from $x=0$ to $x=x_i$, and dividing the result by $x_i$, one can obtain the average increment of water pressure at each level $i$ from 0 to $n$ (see Figure 4). The resultant expression is:

$$
\phi_i^{avg} = \frac{1}{2} \left( R^2 \ln \left( \frac{R^2 + x_i^2}{R^2 + x_{i-1}^2} \right) + \frac{R^2}{2x_i} \ln \left( \frac{R^2 + x_i^2}{R^2 + x_{i+1}^2} \right) - \frac{1}{2x_i} R^2 \ln(R) \right)
$$

(12)

This pressure is then used to recalculate the vertical effective stress acting along the wedge, remembering that $\phi$ denotes a pressure increment over the hydrostatic pressure. The prism component ($V$) can still be calculated through Equation 9, considering the increment of pore water $\phi_i^{avg}$ calculated with Equation 12. On the other hand, the components of self-weight ($G$) and side friction ($T$) have to be calculated discretely, as in:

$$
G = D \sum_{i=0}^{n} \left( \frac{x_i + x_{i-1}}{2} \cdot \frac{\sigma_{v,i} + \sigma_{v,i-1}}{2} \right)
$$

(13)

$$
T = k_o \cdot \tan \phi \sum_{i=1}^{n} \left( dy \cdot \frac{x_i + x_{i-1}}{2} \cdot \frac{\sigma'_{v,i} + \sigma'_{v,i-1}}{2} \right)
$$

(14)

Finally, the component for the support of the water pressure ($W$) can be calculated considering the pressure increment along the inclined edge of the wedge, using Equation 2 for $x=x_i$. The resultant will be

\[\text{Figure 4. Geometry and scheme of forces for the limit equilibrium wedge stability analysis}\]
the integral of these values, summed with the hydrostatic pressure, along the y direction and the thickness of the wedge (D).

For both cases, the resultant $F=S+W$ is scaled to the circular area of the tunnel. There are several rules of thumb to determine the operational face pressure of the machine. For now, a safety margin of 10 kPa above the minimum pressure will be considered. This operational pressure is then verified against the possibility of blowout, considered to occur when the pressure exceeds the vertical total stress at the tunnel crown plus the weight of the supporting mixture from the roof to the tunnel centerline.

**EXAMPLE**

Consider the following: A tunnel of 10 m in diameter with the crown at a depth of 20 m; the ground volumetric weight is 18 kN/m³ and $k_0=0.5$; the groundwater level is at the surface and the volumetric weight of the mixture is 12 kN/m³.

First consider that an impermeable layer can be formed, during stand still for example, and that the groundwater remains hydrostatic. In this case, the effective support ($S$) is maximized at a wedge angle ($\theta$) of $22.2^\circ$ with the following components: $V=6530$ kN; $G=1632$ kN; and $T=1100$ kN, resulting in $S=3921$ kN. The water support ($W$) is 25 MN, resulting in an operational support pressure of 299 kPa, which is less than the blowup limit of 420 kPa. Here is where the traditional design would stop.

By simple geostatic calculations one can assess the total horizontal and vertical stresses along the tunnel boundary. The normal stress can then be calculated through a coordinate transformation operation and compared with the face pressure distribution (Figure 5). One can see that the face pressure falls short of matching the normal boundary stresses, which will induce the excavation to converge.

To analyze the hydraulics of the face pressure transfer, one can start by using Equation 3 to calculate the hydraulic gradient at the tunnel face. The face pressure is 49 kPa above the hydrostatic pressure at the tunnel center, so the hydraulic gradient is around 1 ($i = \phi_0/R = 5 \text{ m}/5 \text{ m} = 1$). Assuming a granular material with a permeability of $10^{-5}$ m/s and a porosity of 0.4, one can then use Equation 4 to calculate the water penetration velocity from the mixture to the ground ($v = k.i/n = 0.025 \text{ mm/s}$). This velocity can be assumed an upper bound to the penetration of slurry or foam bubbles, which will always have a higher viscosity than water. A typical TBM drilling velocity is 1 mm/s. Therefore, for this set of parameters, one can easily see that, during drilling, the supporting fluid will not be able to penetrate further than the depth that is scraped away during each cutterhead rotation.

Another way to look at this is to consider the rotation speed of the cutterhead. Take, for example, a rotation speed of 3 rpm, and consider that each point along the face is scraped two times per revolution. This means that the supporting fluid can penetrate 0.25 mm in the 10 seconds it remains in contact with the ground before it is removed. Other studies have scaled this dissipation by the ratio between the penetration depth and the maximum thickness of the filter cake, through which the whole pressure difference is dissipated (Broere and van Tol, 2000). However, considering that a regular filter cake is stable at the scale of centimeters, it is fair to say that the dissipation through a layer of 0.25 mm will be negligible. Therefore, the whole pressure difference is used in Equation 2 to assess the field of pore water pressure increments ahead of the face.

Through the iterative methodology described in the previous section, the new wedge equilibrium is set at a wedge angle ($\theta$) of $27.9^\circ$ with the following components: $V=5411$ kN; $G=1351$ kN; and $T=917$ kN, resulting in $S=2356$ kN. The effective support is 40% less than for the hydrostatic case. The water,
support (W), on the other hand, is 31 MN, 23% higher than before. Their combined effect requires an operational face pressure of 342 kPa, which is less than the blowup limit of 420 kPa.

Another consideration is the volume of water that flows out of the supporting mixture. Considering a Darcy velocity of $5 \times 10^{-5} \text{ m/s}$ through the area of the tunnel face, the flow rate is about $3.6 \times 10^{-3} \text{ m}^3/\text{s}$. A certain FIR needs to be specified to take the porosity of the saturated ground from the original 0.4 to about 0.5. If no flow is considered, an FIR=20% should suffice (Equation (6)). However, the water flow rate represents about 5% of the initial water in the amount of muck excavated in a certain time (FWR=0.05). Therefore, an FIR=22% is necessary to compensate for the water loss (Equation (7)).

CONCLUSION
The way the processes around TBM operations are understood is constantly evolving, helping designers and contractors to achieve more reliable tunneling systems. However, the quantitative models that represent these processes are frequently disregarded in general design and face stability analyses. This paper presented one step of a general approach to incorporate these models in the state-of-practice.

A different view regarding the equilibrium conditions around the face of a TBM was presented, through analysis of the following: the stresses around the face, along the cross section, and the longitudinal direction; the field of pore water pressure increments in front of a TBM where no filter cake could be formed; the phase balance in the supporting mixture, of special concern to design foam injections; and finally the assessment of face stability, where the increment of pore water pressure will normally require a higher support pressure at the face. The example that was presented illustrates the relative magnitude of these factors for the chosen parameters. Other conditions can possibly reach different results. However, the point of this study is to show that these analyses can provide an objective and accessible framework for the state of practice, where any condition or set of parameters can be processed and the results analyzed for a more realistic design of the TBM excavation cycle.

Figure 5. Example calculation for the face pressures (FP) and in situ stresses (SX-horizontal, SY-vertical, P0-normal to the boundary) around the tunnel perimeter.
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