This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

https://www.issmge.org/publications/online-library

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.
Face stability in slurry and EPB shield tunnelling

G. Anagnostou
RSE Ltd, Consulting Engineers, Zürich, Switzerland

K. Kovári
Swiss Federal Institute of Technology, Zürich, Switzerland

ABSTRACT: Slurry and EPB-shield tunnelling have been successfully applied worldwide in recent years. Under extremely unfavourable geological conditions, however, face instabilities may occur. The present paper aims at a better understanding of the mechanics of face failure. For the case of slurry shield, attention will be paid to the time-dependent effects associated with the infiltration of slurry into the ground ahead of the face. Related topics, such as the stand-up time and the effect of the excavation advance rate, will be discussed. In the case of an EPB-shield, a distinction will be drawn between fluid-pressure and effective pressure in the chamber. The effective pressure can be visualized as a “grain to grain” contact pressure between the muck and the ground at the face. The water pressure in the chamber reduces the hydraulic head gradient in the ground and, consequently, the seepage forces acting in front of the face. The face is thus stabilized both by the direct support of the pressurized muck and by the reduction of the seepage forces in the ground. Normalized diagrams are provided which allow the assessment of tunnel face stability.

1 INTRODUCTION
Tunnel construction in saturated soils is being carried out ever more frequently using closed shields, which allow the control of surface settlement and limit the risk of tunnel face failure through the continuous support of the face during excavation (Stack 1982). Slurry shields use a bentonite suspension as supporting fluid (Fig. 1a). By means of an air cushion, the fluid pressure may be accurately applied and maintained without fluctuations (Babendererde 1991). Earth pressure shields provide continuous support of the tunnel face using freshly-excavated soil (Fig. 1b), which under pressure completely fills up the work chamber (Fujita 1981). The supporting pressure is achieved through control of the incoming and outgoing materials in the chamber, i.e. through regulation of the screw-conveyor rotation and of the excavation advance rate.

2 SLURRY-MODE OF OPERATION

2.1 The computational model
Face stability in homogeneous ground will be assessed by considering the limit equilibrium of a wedge loaded by a prismatic body (Fig. 2). The critical inclination of the slip surface ABFE is determined iteratively (minimization of safety factor). This sliding mechanism was proposed by Horn (1961) and it takes into account the formation of slip surfaces, which may frequently be observed during the failure of tunnel faces in shallow tunnels. Mohr-Coulomb failure condition and drained conditions will be assumed, i.e., all computations are carried out in terms of effective stresses. The load of the prism is computed based upon silo-theory (Janssen 1895). Besides this load, the submerged weight of the wedge, the normal and shear forces along the slip surfaces ADE, BCF and ABFE, as well as the supporting force of the slurry are introduced into the equilibrium equations.

The calculation of the support force of the slurry deserves especial attention. To prevent a seepage flow towards the excavation face, the slurry pressure must be higher than the ground water pressure in the soil. Due to this pressure difference Δp (Fig. 3), the slurry infiltrates into the soil. It is well known (cf. Müller - Kirchenbauer 1972, Xanthakos 1979) that
the stabilizing force of a slurry depends essentially on the extent of infiltration of the slurry into the ground: the less the slurry penetrates, the greater the support force will be.

Under optimum operational conditions, a filter-cake forms on the tunnel face, acting like a membrane and inhibiting the infiltration of the suspension into the ground. The support force results from the difference in hydrostatic pressure between the slurry and the ground water (so-called "membrane-model"). A filter-cake is formed when the suspension contains aggregated solid matter which is filtered out at the beginning of the slurry infiltration. Often, suspended material is present in the slurry due to mixing with the excavated soil; aggregates may be added to the suspension when required - for example, in a uniform, coarse and poorly-graded ground.

For soils with exceptionally high permeability or when the shear resistance of the slurry is low, the bentonite will penetrate into the ground to a certain degree. The final distance of infiltration $e_{max}$ can be estimated by means of the following empirical formulae (DIN 4126):

$$e_{max} = \frac{\Delta p d_{10}}{2 \tau_f}$$

where $\Delta p$, $d_{10}$ and $\tau_f$ denote the excess slurry pressure, the characteristic grain diameter of the soil and the yield strength of the slurry, respectively. Accordingly, the extent of slurry infiltration is governed by the finer soil particle fraction. The yield strength $\tau_f$ of the suspension depends essentially on the bentonite concentration. When the slurry infiltrates into the soil, it exerts a mass force, which is equal to the pressure gradient. The support force of the slurry is calculated by integrating the mass forces over the suspension-saturated zone of the wedge.

### 2.2 Numerical Example

The effect of slurry infiltration on the face stability will be demonstrated by a numerical example concerning a tunnel in a cohesionless ground. For given geometric data, shear parameters and unit weights (see Fig. 3), the safety factor depends on the excess pressure $\Delta p$ and on the infiltration depth, i.e. (see eq. 1), on the characteristic grain size $d_{10}$ of the soil and on the yield strength $\tau_f$ of the slurry.

It is, therefore, possible to represent the safety factor as a function of the characteristic grain size $d_{10}$ (Fig. 4). On the abscissa of this diagram, the grain size range for gravels and sands is given as well; a $d_{10}$-value of 0.60 mm, for example, characterizes a poorly-graded medium sand with a small silt or clay fraction. The curves A, B and C in Fig. 4 correspond to different values of slurry yield strength $\tau_f$ (i.e., bentonite concentration) and of excess pressure $\Delta p$. These two parameters can be directly controlled by the engineer in practice.

Consider curve A. In a soil with a $d_{10}$-value which is smaller than the grain size of medium sand, the deviation from the membrane model (safety factor $=1.50$) is negligible, i.e., the slurry acts as though the face were sealed. Within the range of coarse sand (0.60-2.00 mm), the safety factor experiences a steep decrease. It should be noted, however, that soils with a $d_{10}$-value higher than 0.60 mm occur rather infrequently. At a characteristic grain size in the magnitude of fine gravel, the safety factor becomes equal to 1; in a coarser subsoil, heading failure will occur.

The question then arises as to what extent the stability of the tunnel face can be increased by raising the excess pressure or the bentonite concentration. Comparison of curves A and B (Fig. 4) shows that raising the excess pressure causes an increase in safety, but only in finer soils. When the characteristic grain size is bigger than approximately 2 mm (i.e.,
poorly graded gravel without a fine-grained fraction), increasing the fluid pressure will only cause further infiltration and fluid loss. The deeper infiltration of slurry into the ground represents a safety risk that cannot be compensated by raising the fluid pressure. In extremely coarse and poorly-graded soils, an important technique for stabilizing the face - the control of the slurry pressure - becomes ineffective.

However, by selecting a higher bentonite content, the grain-size range of soils that can be supported by a bentonite slurry becomes, in this example, larger by an order of magnitude (compare curves A and C in Fig. 4). Note that the bentonite content has little effect when the soil is fine-grained. Consequently, a slurry with a low concentration of bentonite might also be used. Such a suspension is favourable due to the easier separation and better handling of the excavated material. Conflicts may occur, however, when the ground consists of different layers with extreme variations in the finer fraction.

2.3 Time-dependent effects

During an excavation standstill, the infiltration distance increases gradually over the course of time. Since the support force decreases with increasing infiltration distance, the safety factor will decrease gradually. Quantification of this effect is based upon the calculation of the time-development of the infiltration depth. This can be done analytically (Anagnostou and Kovári 1994).

Fig. 5 shows the safety factor as a function of time for the example of Fig. 3. At t=0 the infiltration depth is zero, and the safety factor obtains its maximum value of the membrane model (approximately 2 for \( \Delta p = 40 \) kPa). The initial safety margin vanishes after a critical time-period \( t_{cr} \), i.e. at \( t_{cr} \) the limit equilibrium is achieved. Accordingly, \( t_{cr} \) represents the stand-up time of the slurry-supported tunnel face. Obviously, permeability has a decisive influence on the stand-up time. The lower the permeability is, the slower the infiltration and the consequent loss of stability will take place. By a dimensional analysis, one can verify that the stand-up time is proportional to the reciprocal value of permeability. This relationship is represented by a straight line in a double logarithmic plot (Fig. 6). In this example, the tunnel face will remain stable during an excavation standstill of up to several hours, provided that the permeability is lower than \( 10^{-4} \) m/sec. In highly-permeable ground (e.g. \( k=10^{-2} \) m/sec), face instability will occur within few minutes.

In order that the face remains stable (safety factor \( \geq 1 \)), the support force should not underpass a minimum value, and, consequently, the infiltration distance should not exceed a critical maximum value \( e_{cr} \). During excavation standstill in a coarse-grained and

---

**Fig. 4. Safety factor as a function of characteristic grain size**

**Fig. 5. Safety factor as a function of time**

**Fig. 6. Stand-Up Time as a function of permeability**

**Fig. 7. Critical advance rate as a function of permeability**
poorly-graded soil, the infiltration distance may reach this critical value after certain time (Fig. 5). During continuous excavation, however, the infiltration of slurry takes place simultaneously with the removal of ground at the face, i.e. the infiltration is partially compensated by the excavation. Continuous excavation is, therefore, more favourable than excavation standstills.

Details on the analytical determination of the effect of advance rate on the slurry infiltration can be found in Anagnostou and Kovári (1994). The infiltration distance is governed by the ratio $v/k$ of advance rate to soil permeability. The higher the advance rate, the less the slurry penetrates into the ground, and, consequently, the higher the support force and the safety factor will be. In particular, if the advance rate is higher than a critical value $v_{cr}$, the critical infiltration depth will not be reached, i.e. the face will remain stable. Fig. 7 shows the critical advance rate as a function of permeability for the example of Fig. 3. Accordingly, in a soil with a permeability of, e.g., $10^{-3}$ m/sec, the advance rate should amount to at least 17 mm/min. At higher advance rates a safety margin will be present; at lower advance rates, the tunnel face becomes unstable.

3 EPB-MODE OF OPERATION

3.1 The computational model

The assumed sliding mechanism is the same as previously. However, as the work chamber is filled with excavated soil under pressure, a distinction must be drawn between the total and effective stress acting upon the face. Only the effective normal stress can be denoted as actual support pressure on the excavation face. This will be termed "effective support pressure" and denoted in the following text by $s'$ (Fig. 8).

What effects does the pore water pressure, i.e. the piezometric head $h_F$ in the work chamber have from a stability point of view? If it is lower than the piezometric head $h_0$ in the undisturbed state, then the groundwater will seep through the tunnel face. So seepage forces $f$ will act towards the tunnel face and could endanger its stability.

Determination of the seepage forces calls for a three-dimensional numerical seepage-flow analysis. The numerical model is described elsewhere (Anagnostou and Kovári 1996). Fig. 9 shows typical results (contour-lines of the piezometric head in the plane of symmetry). The increasing density of the potential lines close to the tunnel face indicates an increasing value of the seepage forces. As the seepage forces are oriented perpendicularly to the potential lines, the resultant seepage force acting on the wedge slopes slightly downward, while that in the prism above is practically vertical. The destabilizing effect of the seepage forces acting on the wedge is thus clearly apparent. An approximately horizontal load is exercised on the wedge, while the vertical load from the prism is simultaneously increased. The first effect is taken into account by introducing the seepage forces acting within the wedge into its equilibrium equations. The second effect requires a modification of the silo-theory. Computational details are given in the above-cited publication.

3.2 The necessary effective support pressure

At limit equilibrium, the effective support pressure $s'$ depends on the tunnel diameter $D$, on the overburden $H$, on the piezometric head in the chamber $h_F$, on the elevation of the water table $h_0$, on the shear strength parameters $c$ and $\phi$, on the submerged unit weight $\gamma'$ (for the soil beneath the water table) and on the dry unit weight $\gamma_d$ (for the soil above the water table).
With a dimensional analysis and by taking into account the linearity of the equilibrium and failure equations, one obtains the following general form of the limit equilibrium condition (Anagnostou and Kovári 1996):

\[ s' = F_0 \gamma' D - F_1 c + F_2 \gamma' \Delta h - F_3 c \frac{\Delta h}{D} \]  

(2)

where \( F_0 \) to \( F_3 \) are dimensionless coefficients that depend on the friction angle \( \phi \), on the geometric parameters \( H/D \) and \( (h_o-D)/D \), and on the ratio of the dry to the submerged unit weight \( \gamma_d/\gamma' \).

Fig. 10 shows these coefficients as functions of the friction angle \( \phi \). The diagrams have been computed numerically with the model described above. The curves cover the relevant range of the geometric parameters \( H/D \) and \( (h_o-D)/D \). The ratio \( \gamma_d/\gamma' \) was taken to 1.60, a good enough approximation for practical purposes. Eq. (2) together with the diagrams in Fig. 10 provide a simple but powerful tool to characterize quantitatively the face stability at EPB-operation in a given particular case.

3.3 Example

The Great Belt project involves two single-lane undersea tunnels (ϕ 8.7 m) bored by four EPB tunnel boring machines. The tunnels cross glacial tills and fissured marls. The upper till is a preconsolidated undisturbed soil with up to 20% clay. The lower till is an extremely heterogeneous material containing irregular sand lenses, gravels and glacial boulders. Due to the high porewater pressures (up to 4 bar in the tills), extensive drainage works have been carried out. This so-called Moses project reduced the seepage forces, simplified cross passage constructions and made entry into the working chamber possible at low air pressure (Biggart et al. 1993).

In order to provide a better understanding of face stability conditions, statical calculations have been performed based on the model described above (Kovári and Anagnostou 1994). In the following, the interaction between the main factors will be demonstrated by means of a numerical example concerning a tunnel section in the tills with an overburden and a sea depth of 30 m and 10 m, respectively.

By using eq. (2) and the nomograms of Fig. 10 (with \( \phi =32.5^\circ \) and \( \gamma' =13 \) kN/m\(^3\)), one obtains the limit equilibrium relation between the effective support pressure \( s' \), the head difference \( (h_o - hF) \) and the cohesion \( c \) (Fig. 11).

Fig. 10. Nomograms for the dimensionless coefficients
According to Fig. 11, the compensation of the water pressure (i.e., \( h_0 = h_F \)) suffices for face stability even when the ground shows a cohesion \( c \) as low as 10 kPa (point A in Fig. 11). If the difference between the water level \( h_0 \) and the head \( h_F \) in the chamber for some reasons is high, a high effective support pressure \( s' \) is required to ensure face stability. For example, in the extreme case of atmospheric pressure in the chamber (and for a cohesion \( c \) of 30 kPa - a reasonable value for the tills), the necessary effective support pressure amounts to 160 kPa (point B in Fig. 11).

A high effective support pressure, however, has considerable operational disadvantages such as excessive wear of the cutter head, high torque, or arching of the muck at the entrance to the screw conveyor. The head difference between chamber and ground should, therefore, be kept as low as possible. In this example, when reducing the head difference by 20 m, the required effective support pressure \( s' \) decreases from approximately 160 kPa to 60 kPa (point C). A low head difference can be maintained either by decreasing the pore water pressure in the ground (by pump operation) or by maintaining a sufficient head \( h_F \) in the chamber. The latter depends upon the ground properties, the properties of the muck and machine-specific details. An overview of existing techniques can be found elsewhere (Anagnostou and Kovári 1996).

Fig. 11 shows, furthermore, that, in a cohesionless ground, the same reduction in the required effective support pressure of approx. 56 kPa can be achieved either by reducing the head difference by 10 m or by increasing the ground cohesion by approximately 28 kPa. In other words: The effects of \( \Delta s' \approx 56 \text{ kPa} \), \( \Delta(h_0 - h_F) = 10 \text{ m} \) and \( \Delta c \approx 28 \text{ kPa} \) in stabilizing the tunnel face are statically equivalent.

### REFERENCES


