Rozgonyi, T. G., Kieffer, D. S., Maidl, U., Balci, C., TBM considerations for soft-ground tunnels.
International Symposium on the Fusion Technology of Geosystem Engineering, Rock Engineering and Geophysical Exploration
Seoul, Korea, Nov. 18-19, 2003

TBM considerations for soft-ground tunnels

T. G. Rozgonyi[1], D. S. Kieffer[1], U. Maidl[2], Cemal Balci[3]
[1]Mining Engineering Department, Colorado School of Mines, Colorado, USA
[2]Maidl + Maidl Consulting Engineers, Bochum, Germany
[3]Visiting Scholar from Istanbul Technical University, at Colorado School of Mines, Colorado, USA

Abstract: The global demand for underground facilities has increased substantially in the past decades, and a substantial number of underground projects have had to deal with challenging ground conditions in urban environments. Particularly challenging are weak and unstable water bearing soils. Advancements in shielded TBM tech-nology have led to significant improvements regarding the ability to control ground deformations in soft ground. Nonetheless, ground collapse may occur even when the most advanced TBM designs are employed if unexpected adverse ground conditions are encountered or if insufficient stabilizing pressure is transferred to the tunnel face. This paper reviews common approaches for face stability and face pressure transmission calculations, and provides an overview of some of the latest technological developments and considerations for soft ground TBM applica-tions.

1. Introduction

Soft ground tunneling in urban environments must often confront the significant challenges of unstable water-bearing soils and strict ground deformation allowances. Due to advancements in shielded TBM technology over the past 20 to 30 years, significant improvements related to the ability to control ground deformations in soft ground have been realized. However, ground collapse may occur even when the most sophisticated TBM technology is embraced if unexpected ground conditions are encountered or if insufficient stabilizing pressure is transferred to the tunnel face.

2. Tunnel face stability

As discussed by Peck (1969), one of the main issues that must be considered in the design of soft ground tunnels includes excavation stability during construction, most notably stability of the tunnel face. Over the past 35 years several methods for evaluating tunnel face stability of tunnels constructed in soft ground have been proposed. The following sections summarize some of these methods as related to face stability evaluations in cohesive and cohesionless soils.

1.1 Face stability in cohesive soils

The classical contributions regarding tunnel face stability in soft ground considered the undrained behavior of clay soils (Broms and Bennermark, 1967; Peck, 1969). Stability conditions were evaluated on the basis of an overload factor, N, given by:

$$N = \frac{\sigma_s + \gamma H - \sigma_t}{Su}$$

where σ_s is the overburden surcharge; γ is the unit weight of soil; H is the depth to tunnel spring line; σ_t is the support pressure applied to the tunnel face; and s_u is the undrained shear strength of the soil. Face instability conditions were shown to be associated for overload factors exceeding 5 to 7. Subsequent works (Mair, 1979; Schofield, 1980; Davis et al., 1980) showed that the value of critical overload factor depends on the ratio if overburden depth to tunnel diameter.

1.2 Face stability in sands and other noncohesive soils

For tunnels excavated in granular soils, the required face support pressure to prevent face instability is generally based on limiting equilibrium methods that consider an assumed mechanism of tunnel face instability (for example, Horn, 1961; Krause; 1987; Leca and Panet, 1988; Mokham and Wong, 1989; Leca and Dormieux, 1990; Jancsecz and Steiner, 1994; Anagnostu and Kovari, 1996).

Leca and Dormieux (1990) proposed a limit equilibrium method based on an assumed three-dimensional failure mechanism involving the motion of two intersecting conical blocks. For granular soils above the water table, the limiting support pressure is expressed as:

$$\sigma_t = \alpha_s \sigma_s + \alpha_\gamma \gamma D$$

where σ_t is the required support pressure; α_s and α_γ are weight factors that depend on the internal friction angle of the soil and the ratio of overburden depth to tunnel diameter; γ is the unit weight of soil; and D is the tunnel diameter. Leca and Dormieux (1990) also proposed a more general expression for soils exhibiting cohesive and frictional shear strength characteristics. Recent contributions extended this approach to tunnels driven through water bearing soils (Leca et al., 1997), wherein the effects of water are treated as additional loads applied to the bounding surfaces of the conical blocks, as determined from numerical three-dimensional seepage analyses.

Anagnostou and Kovári (1996) also proposed a computational method based on the limiting equilibrium of a three dimensional sliding surface, which draws on work introduced by Horn (1961). As shown in Figure 1, the assumed failure mechanism involves wedge loaded by a rectangular prism. The critical wedge geometry, having the lowest factor of safety, is determined in an iterative process. The load of the rectangular prism, bounded by CDEF and KLMN, is computed based on the full overburden weight or silo theory (Janssen, 1895), as appropriate for the tunneling depth and soil characteristics. The support force (face pressure) necessary to stabilize the wedge is determined by considering the rectangular prism load, together with the weight of the wedge, and the normal and shear forces acting on the sides and back of the wedge. The stability analysis is conducted in terms of effective soil stresses, with the addition of groundwater seepage forces as appropriate.

For TBMs operating in the EPB mode, the face pressure calculation must consider an effective soil stress state, since water pressures within the front chamber cannot be supported. For tunnels driven through granular soils beneath the groundwater table, seepage forces must also be considered. If the water pressure within the front chamber is less than the in-situ groundwater head, seepage toward the TBM face will occur, with potentially destabilizing effects.

As elaborated in the following section, determination of the required support force for a slurry shield must consider the degree of slurry infiltration into the soil ahead of the cutterhead, as the required support force is inversely proportional to the depth of infiltration.

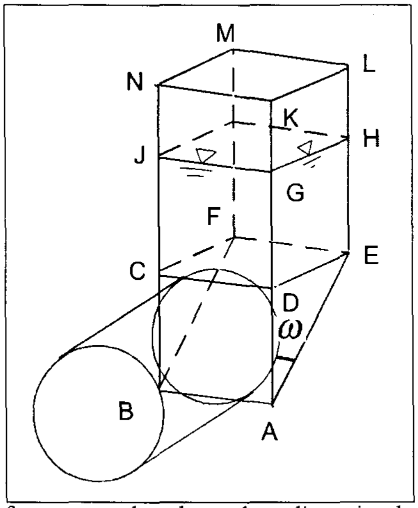

Figure 1. Model for determination of the face pressure based on a three-dimensional sliding mechanism, from Horn (1961) and Anagnostou and Kovari (1996).

3. Load transfer mechanisms for EPB and slurry shield TBMs

Both EPB and slurry shield TBMs provide continuous positive tunnel face control, but differ with regards to the mechanism of stress transference to the excavated tunnel face. In slurry shield TBMs the earth and water pressures are transferred to the tunnel face hydraulically by means of a bentonite suspension that is kept under pressure in the front chamber of the TBM. Total stresses or water pressures can be controlled by this mechanism of stress transference; however, effective stresses in front of the chamber cannot be supported directly. As shown in Figure 1, the hydraulic pressure transferred to the face in a slurry shield is controlled by means of a compressed air cushion located behind a submerged wall.

In EPB shields, face control is provided by loosened ground within the front chamber that has been excavated by the cutterhead. Supporting pressures are transferred to the tunnel face by an earth pressure mechanism, and the magnitude of pressure is thus dependent on the degree of soil deformation. As shown in Figure 2, the magnitude of face pressure in an EPB shield is controlled by means of a pressurized screw conveyor that extracts spoil from the front chamber, and the excavation advance rate. Total stresses or effective stresses can be controlled in an EPB shield, but water pressures inside of the front chamber cannot be supported. Ground conditioning to achieve more favorable support and spoil conveyance characteristics in EPB shields can be achieved by the introduction of bentonite, polymer, or foams in front of the cutterhead.

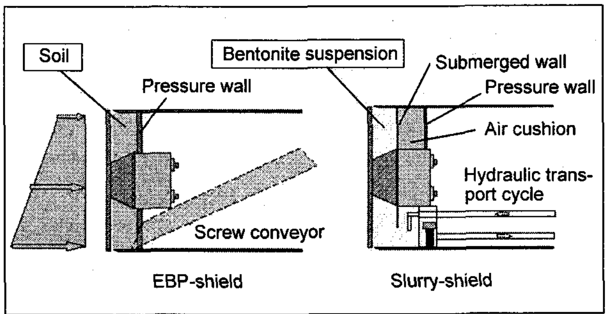

Figure 2. Operation principles of EPB and slurry shields.

The transmission of stresses to the excavated tunnel face in shielded TBMs is customarily described by the *membrane model* or the *penetration model*, shown schematically in Figure 3. As enumerated below, the model that best describes the stress transfer depends on characteristics of both the supporting medium and ground.

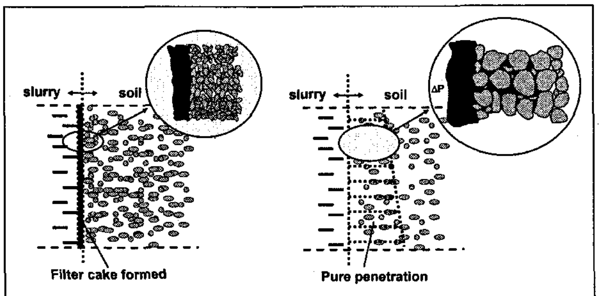

Figure 3. Membrane and Penetration models for slurry shields, (from Müller and Kirchenbauer, 1977).

A. Membrane model

In order to prevent groundwater seepage flow toward the excavation face and the associated deleterious effects of groundwater drawdown, the pressure transferred to the face of a slurry shield must exceed the in-situ groundwater pressure. This pressure differential results in slurry infiltrating the ground to varying degrees. The membrane model is applicable to conditions where the ground in front of the tunnel cutterhead is sufficiently impermeable to effectively prevent infiltration of the bentonite slurry suspension. As depicted in Figure 4, a filter cake is formed at the interface between the slurry and the ground being excavated, which can be idealized as an impermeable membrane. In this case, the pressure change across the membrane is equal to the difference between the in-situ groundwater pressure and slurry pressure.

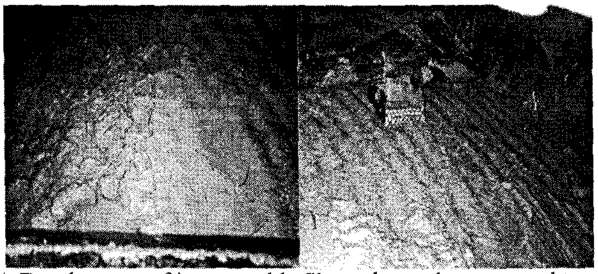

Figure 4. Development of impermeable filter cake on the excavated tunnel face.

The membrane model is most applicable to slurry shields operating in fine sands; however, this model is not suited to slurry shields operating in coarse granular materials, as significant infiltration of the slurry into the ground is likely to occur. The membrane model is also not applicable to EPB shields with foam conditioning, as the service time of foam agents is generally limited, and the lack of the solid fraction in the foam hinders the formation of a filter cake membrane (Maidl 1995).

The transfer of the pressure to the excavated face takes place in form of total stresses (σ_{tot}) over the filter cake. The small amount of filtration water in the filter cake results in excess pore water pressures Δu immediately in front of the tunnel face. The generation of excess pore water pressures is confirmed by measurements made by Broere (2001). The dissipation of excess pore pressure over time depends on the surrounding ground conditions. However, considering the typical grain size ranges for slurry shield applications, it can be reasonably assumed that excess pore pressures dissipate quickly and the membrane remains in a total stress state. If the total stresses σ_{tot} remain constant, the dissipation of excess pore water pressure Δu is accompanied by an increase in effective stresses $\Delta\sigma_{eff}$. This mechanism results in an instantaneous transmission of pressure to the tunnel face, which is highly effective in limiting ground movements in front of the shield.

B. Penetration model

The penetration model is applicable to the condition where the ground ahead of the cutterhead is sufficiently permeable to permit infiltration of bentonite slurry suspension or other soil conditioning agents, such as foam. The infiltration effectively hinders the formation of an impermeable filter cake membrane. For slurry shield tunneling, the penetration model is applicable to coarse sands and gravel-sand mixtures, while for EPB shields with foam conditioning, the model is applicable to sandy soils.

In the penetration model, the slurry or other ground conditioning agents introduced in front of the cutterhead infiltrates the pore space of the ground ahead of the face, thereby reducing the permeability of the ground (Figure 5). Excess pore water pressures result from the penetration, and the extent of penetration is largely governed by the soil grain size distribution and yield strength of the infiltrating medium. As the infiltration occurs, the supporting medium exerts a force which is equivalent to the pressure gradient across the zone of infiltration. In slurry shields the mobilized excess pore pressures are generally small due to the relatively high permeability of the ground, provided the surrounding ground conditions provide for adequate drainage pathways.

In the penetration model, the zone of infiltration can be described by a quasi-membrane model (Maidl, 1995) and the stabilizing face pressure takes place by transference of the shear stresses to the soil particles.

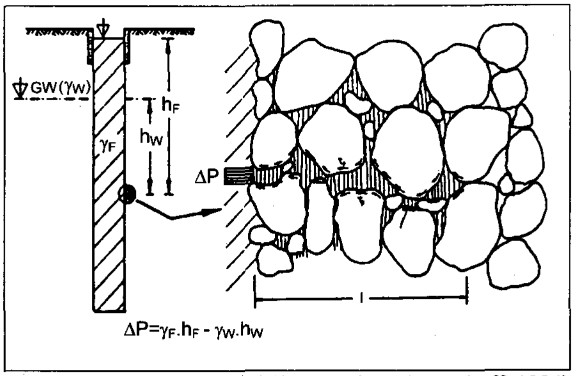

Figure 5. Penetration model (from Walz and Steinhoff, 1994)

C. Limits of the membrane and penetration models

Neither the membrane nor penetration models are applicable to conditions where excess pore pressure dissipation does not occur. Such conditions may be encountered in low permeability clay materials or in stratified systems containing lenses of clay or granular materials (Figure 6).

Due to the essentially impermeable characteristics of the surrounding ground, which may include the prevention of drainage through granular lenses, neither penetration of slurry or foam into ground, nor formation of a membrane takes place. In these cases, the pore water pressure in front of the tunnel face rises to the magnitude of the applied slurry or earth pressure.

For undrained sand lenses having relatively low initial effective stresses, the rise in pore pressures can theoretically lead to liquefaction and instability of the tunnel face. However, this is largely a theoretical consideration, since only a slight amount of penetration is necessary in order to satisfy the conditions of the membrane or penetration models. The potential for liquefaction is increased when using slurry shield tunneling methods because the excavation chamber is filled with a suspension that has a lower specific weight than the in-situ soil and thus cannot impede the development of a liquefied soil state. In EPB shield tunneling this situation is more controllable since the excavation chamber is filled with the spoil.

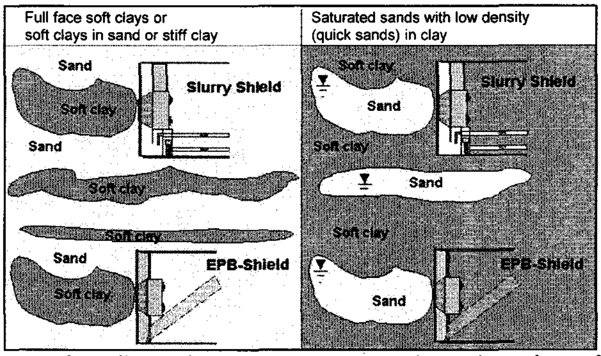

Figure 6. Ground conditions that may prevent penetration and membrane formation.

4. Soil types suited to EPB and slurry shield applications

The typical ranges of soil grain sizes that are considered appropriate for EPB and slurry shield applications are shown in Figure 7. Sands and fine gravel represent the typical application range for slurry shields, while EBP shields are better suited for soils having a significant fraction of fines. The use of slurry shields in soils having significant fines has proven uneconomical due to high costs related to technologies for the slurry/spoil separation plant.

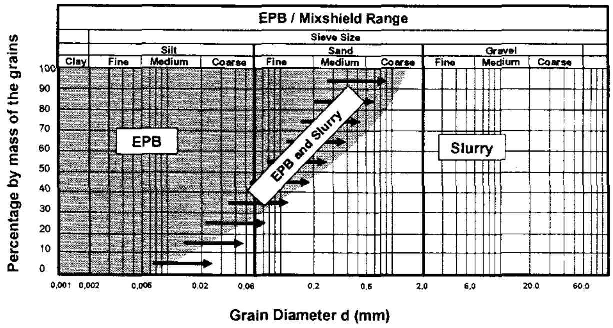

Figure 7. Typical grain size distribution ranges for the application of EPB and slurry shields.

EPB shields have been successful in sandy and gravely materials with the introduction of soil conditioning such as bentonite, polymers, and foams.

5. Problematic ground conditions for EPB and slurry shields

A. EPB shields in coarse high-permeability soils

In gravels and coarse sands, a sedimentation process in the mining chamber commences, especially when high foam injection ratios (FIR) are used. As depicted in Figure 8, the sedimentation process causes the specific weight of the supporting medium to vary between the top and bottom levels inside the excavation chamber (represented by Line 1). In the crown area, the specific weight may be below that of loosely packed sands, meaning that the face pressure in the crown is only transmitted through the foam (air pore pressure). The higher specific weight in the lower part of the excavation chamber promotes the transference of face pressure chiefly through effective grain to grain contact stresses.

If the EPB shield remains at a standstill for a significant period of time, the face support pressure in the excavation chamber is reduced (represented by Line 2 of Figure 8), and may approach external hydrostatic pressures. The pressure reduction results from the limited service life of the foam. By means of foam conditioning in front of the rotating cutterhead and in the excavation chamber, it is possible to raise the support pressure during the standstills back toward Line 1 of Figure 8; however, it is difficult to influence the unfavorable specific weight distribution caused by sedimentation process.

When re-starting the shield advance, relatively high face pressures are transmitted by effective stresses in the invert area, as shown by Line 3 of Figure 8. This results in poor material flow and possibly the development of obstructions in the lower half of the chamber. This can cause operational difficulties due to higher cutterhead torque and propulsion thrust requirements, and can make governing the shield more difficult.

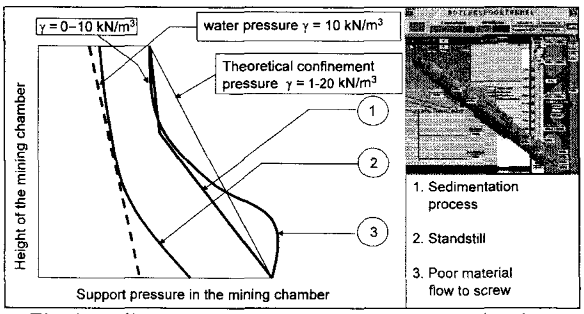

Fig. 8. Sedimentation process in the excavation chamber.

B. Slurry shields in plastic clays

Advancing a slurry shield through highly plastic clays can result in significant tunneling difficulties. Figure 9 depicts two critical scenarios that may develop. In the first scenario, advancing the TBM above a critical rate results in a build-up of soil in the pressure chamber (Phase 2). The slurry in the suction area then circulates only within the chamber, instead of transporting the material away from the face (Phase 3). If the TBM advance rate increases further, the soil in the pressure chamber thickens, and in the worst case the entire suction area will be blocked (Phase 4). It then becomes necessary to clean the machine under high pressure before it can be redeployed, which may require divers to enter the pressure chamber.

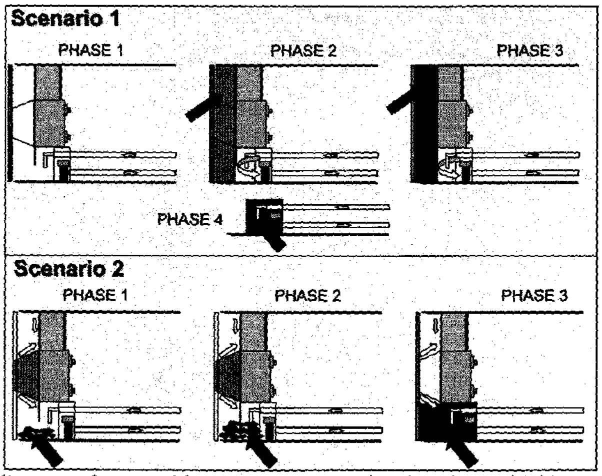

Figure 9. Schematic diagram of two problematic scenarios related to slurry shield tunneling in plastic clays.

In the second scenario, suspension flow near the bulkhead is restricted and large blocks of clay may collect in the invert of the pressure chamber (Phase 1). It then becomes necessary to utilize mechanical devices to move the clay blocks in front of the cutter hole (Phase 2). A shortened holding time in the pressure chamber decreases the ability to soften the clay blocks, which can lead to a complete blockage of the suction area (Phase 3).

When considering actual operation, it is not only cohesion and adhesion of the clay that are important, but the mechanical characteristics of the suspension in the pressure chamber. The speed of the bore fluid leaving the slurry pipe is typically about 1.5m/s to 3.7m/s, while the speed of the bentonite-suspension near the dividing wall openings depends solely on the conditions in the pressure chamber. But even with 100% supply from the diving wall, this results in a maximum velocity of only about 0.14m/s near the suction area. The drag forces that can be exerted on large clay blocks may therefore prove insufficient to guarantee transport of material without delay. For this reason, agitators are introduced to assure adequate suction flow.

6. Ground improvement echnologies

In order to achieve adequate face support in Slurry and EPB shield tunnels, the soil at the excavated face must be relatively impermeable. In slurry shields, the bentonite suspension is effective in forming an impermeable filter cake on the tunnel face, or it may form a quasi-membrane by penetrating a certain distance into the tunnel face. In EPB shields, however, soil conditioning agents often have to be introduced in front of the cutterhead in order to create a soft and compressible paste in the pressurized chamber. Conditioning agents commonly include bentonite, polymers, and foams. Figure 10 shows a Herrenknecht TBM with foam being injected in front of the cutterhead.

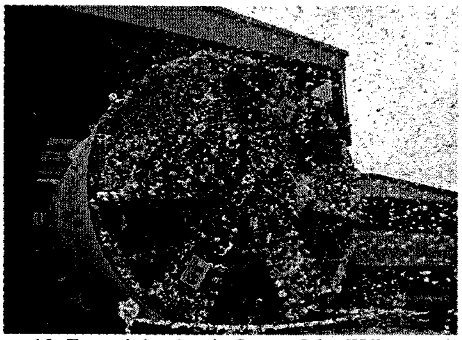

Figure 10. Foam injection in front of the EPB cutterhead.

The main goals of conditioning are to create a plastic soil having a low angle of internal friction and low permeability. By achieving such a soil state, controlled flow of the spoil through the screw conveyor is facilitated and TBM performance is enhanced. The main benefits of soil conditioning include:

- reduced power and torque requirements;
- reduced abrasion and machine wear;
- improved face pressure control;
- reduced soil permeability; and
- reduced soil adhesion and clogging of the cutterhead.

Additionally, as shown in Figure 11, by creating soft and compressible homogenous paste, spoil conveyance can be facilitated greatly.

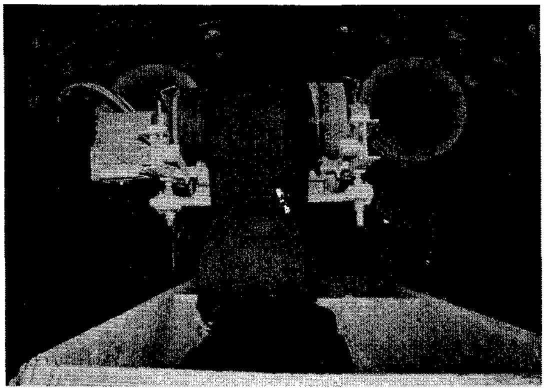

Figure 11. Use of foam conditioning in an EPB shield for creating an ideal soft and compressible paste.

Although soil conditioning agents have been widely used in conjunction with EBP shield tunneling, the specification of soil conditioning requirements remains largely empirical, and is generally based on experience and trial-and-error.

7. Mixed-shield TBMs

The ultimate goal in TBM design is to have a single machine that is effective in all ground conditions. While no such TBM yet exists, substantial recent efforts have been devoted to the design of convertible mixed-shield TBMs (Herrencknecht, 2000) that are capable of tunneling through a broad array of heterogeneous materials. Depending on specific geologic conditions, a mixed-shield TBM may have design features for cutting hard rock and excavating soft soils, operating in an open or closed face configuration, and operating in either the EPB or slurry shield mode. As an example, Figure 12 shows a Herrenknecht mixed-shield TBM having the following design features:

- disc cutters for cutting hard rocks;
- soil picks for excavating soft soils;
- center cutterhead for reducing torque requirements and reducing adhesion of soft clays;
- jaw crusher for breaking down hard boulders and blocks; and
- slurry shield configuration for providing ground control in weak water bearing soils.

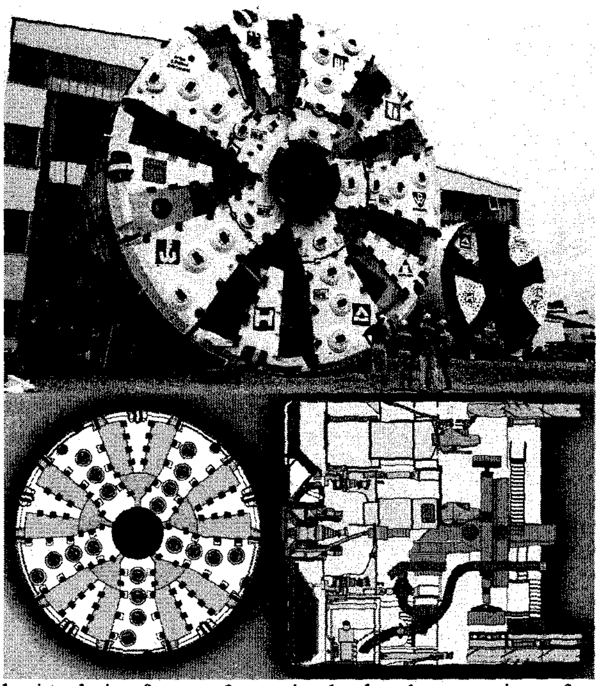

Figure 12. Mixed-shield TBM having design features for cutting hard rock, excavating soft soil, operating in closed or open face mode, and operating in the slurry shield mode.

Figure 13 shows schematically how a mixed-shield design can be converted from the slurry shield mode to the EPB mode. This may require a full day of work to complete while the TBM is at a standstill, and a substantial number of mode changes along an alignment would prove impractical. However, with sufficient information regarding ground conditions along the alignment, the optimum locations for operational mode changes can be determined.

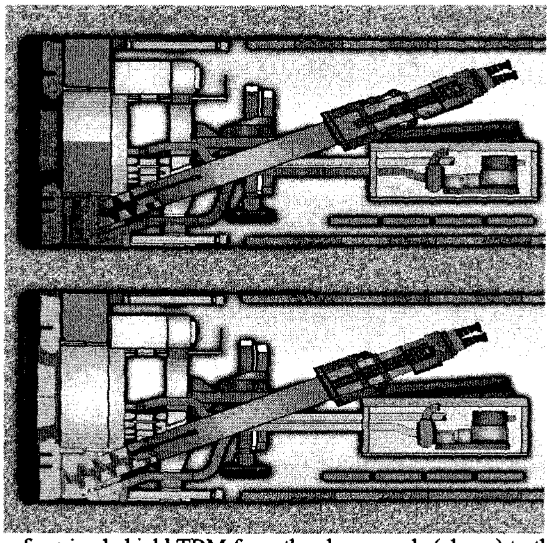

Figure 13. Conversion of a mixed-shield TBM from the slurry mode (above) to the EPB mode (below).

Convertible mixed-shields designs can aid substantially in ground deformation control and excavation efficiency particularly when tunneling at shallow depths through a broad array of heterogeneous materials.

References

Anagnostou, G. and Kovari, K. 1996. Face Stability in Slurry and EPB Shields Tunnelling. Tunnels and Tunnelling, Dec. 1996, pp. 27-29.

Broere, W. 2001. Tunnel Face Stability & New CPT Applications, Dissertation, TU Delft 2001.

Broms, B.B., and Bennermark, H., 1967. Stability of Clay at Vertical Openings. ASCE Journal of the Soil Mechanics and Foundation Division, SM1, pp. 71-94.

Davis, E.H., Gunn, M.J., Mair, R.J. and Seneviratne, H.N., (1980). "The stability of shallow tunnels and underground openings in cohesive material". Géotechnique, V. 30, No. 4, pp. 397-419.

Herrencknecht, M., 2000. Development of Mixshields from Hera to the 4th Elbe Tunnel Tube". Tunnel, No. 3/2000, pp. 11-25.

Horn, N. 1961. Horizontaler Erddruck auf senkrechte Abschlussflächen von Tunnelröhren. Landeskonferenz der Ungarischen Tiefbauindusrtie.

Jancsecz, S., Steiner, W. 1994. Face support for a large mix-shield in heterogeneous ground conditions. Tunneling '94, London.

Janssen, H.A., 1895. Versucheueber Getreidedruck in Silozellen. Zeitschrift des Vereins Deutscher Ingenieure, XXXIX, No. 35, 1045-1049.

Krause, T. 1987. Schildvortrieb mit Fluessigkeits und Erdgestuetzer Orstbrust. Dissertation, TU Braunschweig.

Leca, E. and Dormieux, L. 1990. Upper and Lower Bound Solutions for the Face Stability of Shallow Circular Tunnels in Frictional Material. Geotechnique, V. 40, pp. 581-606.

Leca, E., Garnier, J., Atwa, M., Chambon, P., Skiker, A., Dormieux, L., Garnier, D., Maghous, S., 1997. Analyse théorique et expérimentale de la stabilité du front de taille des tunnels à faible profondeur. Proc. 14th Int. Cong. on Soil Mech. and Found. Eng.

Leca, E. and Panet, M. 1988. Application du Calcu a la Rupture a la Stabilite du Front de Taille d'un Tunnel. Revue Francaise de Geotechnique, V. 45, pp. 5-19.

Maidl, U. 1995. Erweiterung der Einsatzbereiche des Erddruckschildes durch Bodenkonditionierung mit Schaum, Dissertation, Ruhr-Universität Bochum.

Mair, R.J., 1979. "Centrifugal modelling of tunnelling construction in soft clay". PhD Thesis, University of Cambridge.

Mohkam, M., Wong, Y.W. 1989. Three dimensional stability of the tunnel face under fluid pressure, Numerical Methods in Geomechanics, Innsbruck.

Müller-Kirchenbauer, H. 1977. Stability of slurry trenches in inhomogeneous subsoil. In N.N., editor, 9th International Conference on soil Mechanics and Foundation Enginering.

Peck, R.B., 1969. Deep excavations and tunneling in soft ground. Proc. 7th Int. Conf. Soil Mechanics and Foundation Engineering, Mexico, State-of-the-Art Volume, pp. 225-290.

Schofield, A.N., 1980. "Cambridge Geotechnical Centrifuge Operations". Géotechnique, V. 30, No. 3, pp. 227-268.

Walz, B., Steinhoff, J. 1994. Stützung von Erdwänden mit viskosen Flüssigkeiten. Festschrift des Arbeitskreises Baugruben zum 65. Geburtstag von Univ.-Prof. Dr.-Ing. Habil. Anton Weienbach