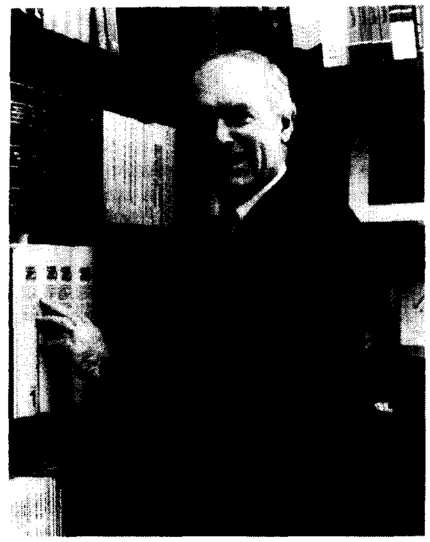
The Future of Mechanized Tunnelling: Introduction to the ITA Open Session at the Toronto Congress 1989

Z. Eisenstein



Dr. Z. Eisenstein

The four papers in this special section of Tunnelling and Underground Space Technology *were presented at the ITA Open Session held September 5, 1989, in Toronto, Canada, at the annual congress of the ITA. The subject of the session was "The Future of Mechanized Tunnelling." Dr. Z. Eisenstein, a professor of civil engineering at the University of Alberta (Canada) and First Vice-President of the ITA, organized and moderated the session. His opening remarks are summarized below.*

The tunnelling scene is presently dominated by two different technological concepts: the New Austrian Tunnelling Method (NATM) and the Tunnel Boring Machine (TBM). The NATM, after almost three decades of continuous development and refinement, has reached a level of reliable maturity. As such, its place among world tunnellers is guaranteed. Younger in age but parallel in development has been a rapid, intense development in mechanized tunnelling. The two approaches are not in conflict; they simply serve two different sets of requirements.

The intensity and pace at which mechanized tunnelling is developing today led to the decision to devote the ITA Open Session at the Toronto Congress entirely to this topic. It was expected that the Session would not only reflect the exciting recent developments in this area, but also should include speculation about the future trends in this field. All aspects of TBMs should be included—their ground control and ground support capabilities, as well as the associated lining technologies.

In preparation for the Session, in my capacity as moderator, I formulated several specific questions to be dealt with by the speakers and during the subsequent discussion:

1. Under what conditions is mechanized tunnelling preferable to the NATM or to conventional (drill-and-blast) techniques?
2. What are the geological, geotechnical and structural support features that control mechanized tunnelling and, as such, should be considered in design and procurement of a mechanized tunnelling system?
3. What would constitute an "ideal" Tunnel Boring Machine? Can a machine be developed that is capable of handling an entire geological spectrum, from hard rock to soft clay or water-bearing sand, including mixed faces? Is there any value to developing such a machine?
4. Considering the eternal dilemma of soft ground tunnelling—i.e., whether to change the ground conditions to suit the available technology, OR whether to select a technology to suit a particular ground—what role can the earth-pressure-balance shield and the slurry shield play in solving it? Or have these two exciting techniques already solved the dilemma once and for all?
5. Will the extruded lining system be capable of replacing the more conventional types of support, particularly in soft ground, where ground deformations are of concern?

In responding to the above questions and adding to them their own concerns, the speakers came with several exciting presentations, most of them published in this issue. There is no question that mechanized tunnelling has quickly become the "high tech" chapter in underground construction. That chapter is far from being finished—and I very much doubt that it ever will be. □

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Tunnelling Machines in Soft Ground: a Comparison of Slurry and EPB Shield Systems

Siegmund Babendererde

Abstract—*Thanks to advances in tunnelling machine technology, tunnels can be driven by machines even in very difficult ground conditions. The two most important objectives in machine tunnelling are: (1) to support the ground reliably during excavation; and (2) to hold the surrounding ground behind the shield tail in balance by grouting while the lining is being erected. The author compares two established machine tunnelling methods—the slurry shield and the earth-pressure balanced shield tunnelling system—with regard to how they achieve these two objectives.*

Résumé—*Grâce aux progrès faits en matière de technologie de tunnelier, les tunnels peuvent être creusés par des machines même dans des sols très difficiles. Les deux objectifs les plus importants pour le creusement au tunnelier sont: (1) soutenir le sol de façon fiable pendant l'excavation; et (2) maintenir en équilibre le sol environnant derrière la queue du bouclier par des injections effectuées lors la mise en oeuvre du revêtement. L'auteur compare deux méthodes de creusement au tunnelier en usage—le bouclier à boue de bentonite et le bouclier à pression de terre—en analysant leurs possibilités d'atteindre ces deux objectifs.*

Introduction

In Berlin, Germany, a section of metro recently was driven with a hydro-shield, i.e., a slurry shield. Three years previously, the same shield with the same crew had driven another section of metro very successfully only a few kilometres away.

This time, however, the drive was not in the quaternary sand strata, but in ice-age glacial deposits such as boulder clay with enclosed sand lenses and many boulders. The same tunnelling machine which had been a thoroughbred in sand was now, in boulder clay, suddenly an old stubborn mule.

Satisfactory advance rates could only be achieved with a new design especially constructed for the particular ground conditions.

This example confirms yet again that there is no universal tunnelling machine for soft ground. The machine and components must be matched to the specific geological conditions. On the other hand, technical machine developments are now available which allow us to mechanically drive tunnels in even very difficult ground.

By "difficult ground", we mean geological conditions in which, during the tunnel drive, the face is only partly or not at all stable, the tunnel lies in the ground water, the ground conditions often change, or the strength of the excavated material is extremely variable (see Fig. 1).

As is generally the case in soft ground, the main objective is to preserve the initial stresses and, particularly, to avoid unintentional over-excavation. Thus, the body of ground surrounding the tunnel drive should not be damaged by overbreak or by loosening and relaxation. Settlements that appear at the surface are an indication of an inadequate tunnelling method. They show the extent to which the tunnel lining is embedded in the ground. A lining that is unsatisfactorily bedded deforms, is locally overstressed, and becomes damaged. Repairs to such a lining result in high maintenance costs.

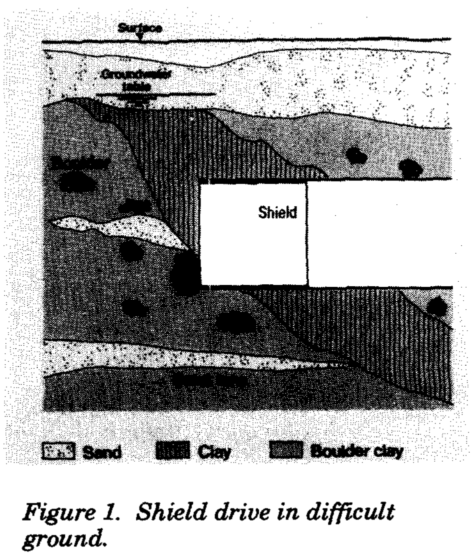

Figure 1. Shield drive in difficult ground.

When tunnelling with a machine in soft ground, the face must be reliably supported during excavation and the surrounding ground behind the shield tail must be held in balance by grouting during the erection of the lining.

This paper explains how these two objectives are achieved with current technical systems.

Classification of Tunnelling Machines According to Type of Face Support

An unstable face can be supported during the excavation by:

- Mechanical means.
- Compressed air.
- A fluid;
- The excavated material itself.

Mechanical support in soft ground below the groundwater level is incomplete and very risky.

Compressed air support is also problematic in some soils. In addition, the shield tail seals usually are not airtight. Therefore, it is not possible to support the face by merely holding the working chamber at the front of the shield under compressed air. Because the whole tunnel must be kept under pressure, this method has high labour costs.

In recent years two tunnelling machine systems have established themselves:

1. The Slurry shield system, in which the face is supported by a fluid.
2. The Earth pressure balanced (EPB) shield system, in which the soil material itself supports the face.

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Below, these two types of tunnelling systems (pictured in Figure 2) are compared with respect to face support and excavation, and erection of the tunnel lining.

Comparison of Slurry and EPB Shields: Face Support

Support Medium

Slurry shield

The support medium of the face on slurry shields is virtually a frictionless fluid. It consists of water and an additive that can filter out and settle on the surface of the face to form an impervious layer. This layer then transfers the pressure of the support fluid to the ground. In Europe, bentonite (a technically processed clay) is generally used as the filterable additive; in Japan, natural clay is usually added.

The support medium technology for bentonite has been developed by Prof. Muller-Kirchenbauer (Germany); and for natural clay, by Dr. Hironobu Yamazaki (Japan).

Earth pressure balanced (EPB) shield

Because the support medium on EPB shields is the excavated material itself, it is generally not open to precise definition. Even when the groundwater is included, the properties of the support medium are at best those of a non-frictionless, high-viscosity fluid. If the water inflow from the groundwater is not adequate or if the soil does not contain enough very fine particles, water or a clay water suspension is introduced into the tool gap at the cutting head in order to achieve fluid properties suitable for pressure transfer.

Mechanical systems, such as mixing bars mounted on the revolving cutting wheel and the stationary pressure bulkhead, mix the excavated material and the added suspension to a homogeneous mass. The more homogeneous and consistent the soil, the more successful is this process.

Transfer of Constant Pressure to the Face

How well an unstable face is supported depends particularly on effectively transferring a constant pressure from the support medium to the surface of the face. If the support pressure is not constant, but instead oscillates, the varying pressure inevitably leads to collapse of the face. Figure 3 compares the slurry shield and EPB shield with regard to face support.

Slurry shield

On a slurry shield with a frictionless support fluid, the suspension is contained in the working chamber at a predetermined support pressure. The suspension reacts sensitively with pressure changes to volume imbalance between the supply of suspension pumped into the working chamber and that, enriched with the excavated material, removed from the working chamber.

In practice, however, in spite of all control and regulation systems, these volume variations cannot be avoided. Therefore, in order to avoid pressure variation in the support medium, a compressed air reservoir is incorporated in the German Hydro-shield (Fig. 4) as an elastic spring element. This reservoir provides, by way of a normal compressed air regulation unit, an almost constant support pressure (maximum deviation of +/-0.05 bar).

Japanese designs maintain the constant support pressure by compensating for volume differences between inlet and outlet with the pumps and valves. Computer-controlled monitoring and steering systems are employed. However, a major element in the maintenance of a constant support pressure is the inertia of the support suspension, which flows with an almost constant speed through the pipeline. A Japanese slurry shield system is shown in Figure 5.

EPB shield

For the EPB shield, the support pressure in the working chamber is generated by thrusting the tunnelling machine against the face with the shield jacks. The rotating cutting wheel scrapes off the ground that is pressed into the working chamber. At the same time, the equivalent amount of spoil is removed from the working chamber by the screw conveyor. The rotation speed of the screw conveyor regulates the volume extracted and thus the support pressure in the working chamber.

In this case, the support medium is not a frictionless fluid, but often rather a compressible soil mass. The pressure state in the working chamber is monitored by pressure cells. Not infrequently, there are pressure differences in the support medium between the face and the pressure bulkhead of more than 1 bar. However, the effect on face

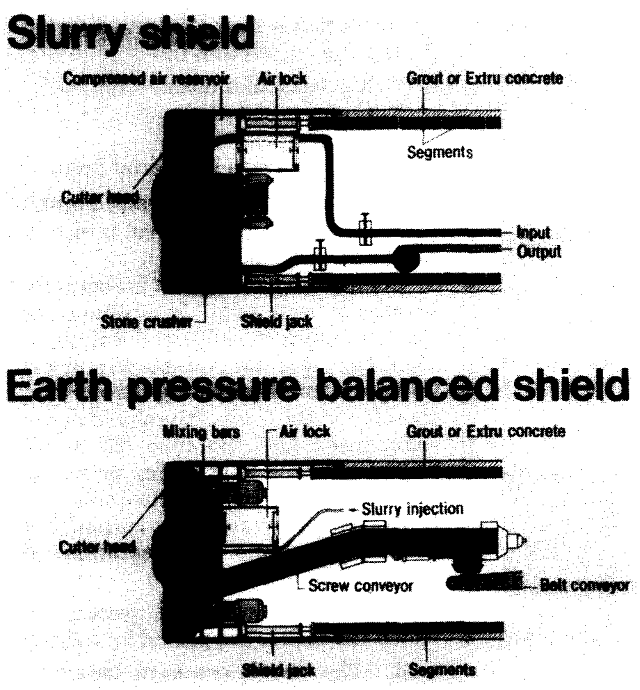

Figure 2. Comparative design of the slurry shield and earth pressure balanced shield tunnelling machines.

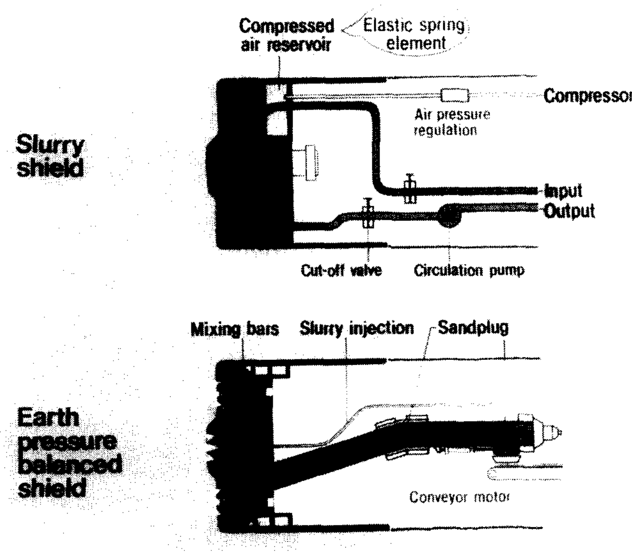

Figure 3. Comparison of face support for the two types of tunnelling machine systems.

instability is not as serious as it is for a slurry shield. Even large stress differences between ground and water loading on the face support pressure from the spoil material in the working chamber can result in only limited face deformation, because the stress in the spoil mass would immediately rise.

With the slurry shield, in contrast, the face would collapse due to the lack of friction in the support medium.

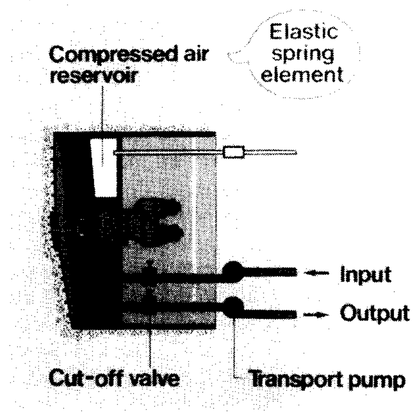

Figure 4. German Hydro-shield system.

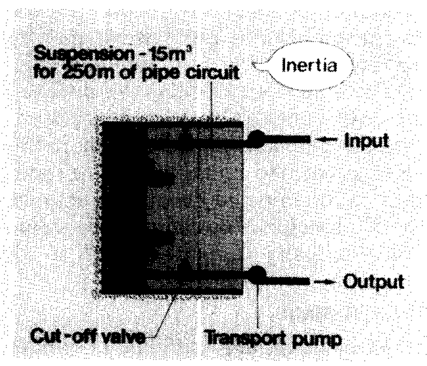

Figure 5. Japanese slurry-shield system.

Sealing the Working Chamber

The pressure on the face can only be kept constant if the extraction points for material from the working chamber are equipped with reliable and controllable sealing units.

Slurry shield

On the slurry shield, the excavated material is extracted from the working chamber with the support medium by centrifugal pumps, generally to a separation plant above ground.

The hydrostatic pressure in the transport line resulting from the position of the separation plant is usually higher than the support pressure in the working chamber. If the separation plant has to be installed in the tunnel, non-return valves have to be fitted to ensure that when the circulation pumps do not operate the support medium does not flow out of the working chamber leaving the face unsupported.

EPB shield

In the EPB shield, the material reconstituted in the working chamber is removed from it by screw conveyors. The amount of material in the screw conveyor depends upon the consistency and uniformity of the spoil. It is not to be expected that the conveyor will always be completely filled with soil material; if it were, a sealed closure would form on the outlet for the material.

In Japan, two screw conveyors are often mounted one behind the other, using different diameters and different speeds to interrupt the screw process. In the intermediate, screw-less section (the "sand plug"), the material should compact to form a bung, thus sealing the working chamber. Thereafter, the extracted material from the second screw conveyor can pass at atmospheric pressure through a chute to the conveyor belt. A hydraulically operated gate creates a second seal, if required, by forming an additional sand plug at the exit of the second conveyor.

In the design of the T1 for the Channel Tunnel, Robbins has adopted another solution (see Fig. 6). Instead of the second screw conveyor, a double reciprocation pump has been installed behind the first screw conveyor, thus achieving a reliable sealing system for the working chamber. Even if this method does not operate entirely satisfactorily, it is a logical step in the solution of a technical problem.

Comparison of Slurry and EPB Shields: Excavation

The energy required to turn the cutting wheel in an EPB shield is considerably greater than that required for a slurry shield because the internal friction of the supporting soil mass in the working chamber of the EPB shield has to be overcome for each rotation. The support medium in a slurry shield, on the other hand, is a frictionless fluid. The two systems are compared in Figure 7.

Therefore, a 525 KW cutter wheel drive has been installed for the 6.60-m-dia. slurry shield drive in Berlin, while in Essen the drive for an EPB shield (7.10 m dia.) was 1.34 KW.

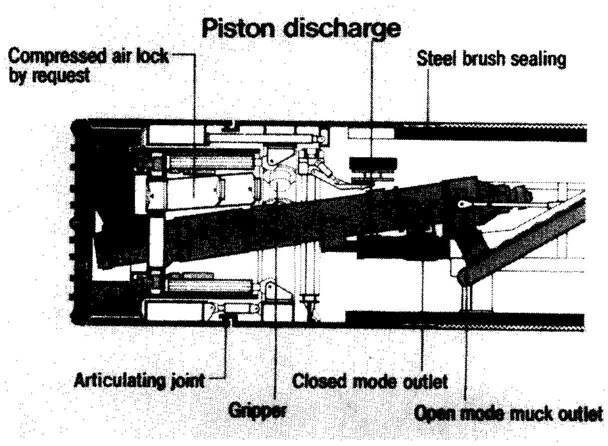

Figure 6. The design for the Robbins T1 tunnel boring machine incorporates a double reciprocation pump behind the first screw conveyor to achieve a reliable sealing system for the working chamber.

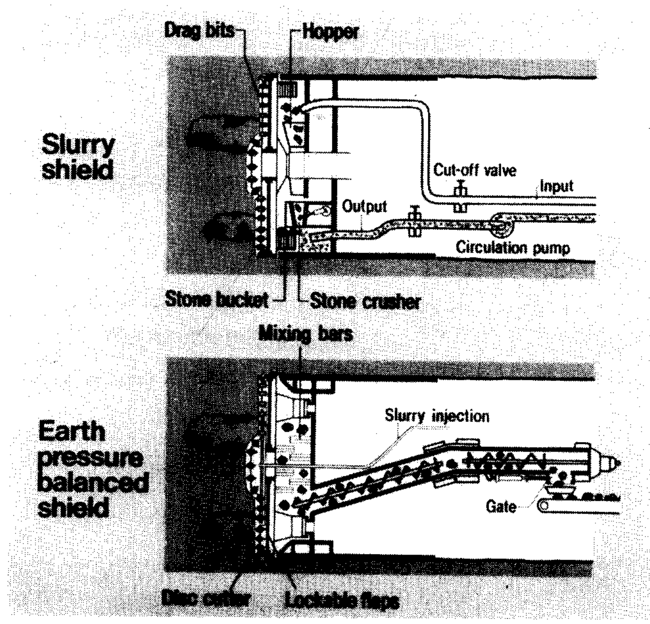

Figure 7. Differences in the design of the slurry shield and earth pressure balanced shield with respect to excavation.

Excavation Tools

The type of excavation tools is determined solely by the soil properties and, thus, are selected independently of the decision to use a slurry shield or an EPB shield.

Tools for sand and gravel soils can be very simple and are subject to the usual aspects of wear.

In cohesive soils, the tools can reduce the danger of sticking by only removing small pieces of ground. Here, small excavator teeth ("drag bits") that can be quickly changed have proven themselves.

In variable ground, particularly when boulders are embedded, it is necessary to mount the cutting wheel with a combination of various tools. Disc cutters should break up the boulders while trailing drag bits scrape off the soft ground. The disc cutters, mounted about 8–10 cm apart, also reduce hard stones in softer ground deposits and limit the width of strips of cohesive soil shaved off by the trailing drag bits.

Removal of Stones

The potential to break up large stones and remove broken material is different for slurry and EPB shields.

Slurry shield

Even when the stones are basically broken up in the face by disc cutters, it is impossible to avoid smaller stones or fragments from larger boulders falling into the working chamber. They are caught there by stone scoops attached to the rear of the cutting wheel and fed, via a hopper, to a stone crusher that has a jaw width of 40 cm. The stone crusher reduces the stones to a material with a maximum edge length of 12 cm. This material can then be transported without difficulty by the centrifugal pumps along a 25–30 cm pipeline to the separation plant.

EPB shield

A stone crusher cannot be installed in the working chamber of an EPB shield because the excavated material has to be mixed there to a more or less homogeneous support medium.

Smaller stones or fragments of boulders broken up at the face are mixed with the earth mass in the working chamber and reach the conveyor belt over the screw conveyor. However, in order to handle the stones before they can be reduced by a crusher, the screw conveyor has to have a very large diameter (up to 1.40 m).

Stones that have not been reduced sufficiently by the disc cutters on the cutting wheel are more difficult to remove from an EPB shield than from a slurry shield.

In summary:

- It is easier to enter the working chamber in a slurry shield. The replacement of the suspension with compressed air is quicker for the slurry shield system.
- A stone crusher can be installed in the working chamber of the slurry shield.
- In the EPB shield, stones endanger the mixing bars and the screw of the screw conveyor.

Erection of the Tunnel Lining

The tunnel lining, which is generally a segmental lining, can be erected in the same way behind a slurry shield as behind an EPB shield (see Fig. 8). Whatever technique is applied should fulfill the requirements discussed above. In order to minimize subsequent maintenance costs, the tunnel lining must not be overstressed. Therefore, it must be free of cavities and, if possible, placed while maintaining the primary stress state in the ground.

Shield Tail Seal

The shield tail seal is the most important design element (see Fig. 9). It must effectively seal the joint between the shield tail and the back of the segments so that the grout injected behind the segments can balance the surrounding soft ground and ground water. If the seal fails, grout can flow into the interior of the shield, causing the pressure in the grout to fall immediately , allowing groundwater and soil to penetrate the shield tail joint. The ground around the tunnel lining could relax and loosen, thereby destroying the bedding.

Sealing of this shield tail joint is technically difficult, not only because the individual segments cannot be positioned accurately enough to avoid steps or ledges of up to 1.5 cm, but also because the advance of the shield leaves a void behind the shield tail that has simultaneously filled with an equal volume of grout. At the same time, the grout must be at a pressure somewhat higher than the pressure from the surrounding soil and groundwater. These requirements are not always met by current designs.

The easiest way to solve this problem is to introduce an elastic spring element that compensates for differences in the volume vacated by the shield tail and that of the injected grout. To achieve this the shield tail seal must be movable and elastically supported.

Rubber seals

At present, the stiffness of rubber seals has prevented them from closing steps between inaccurately placed seg-

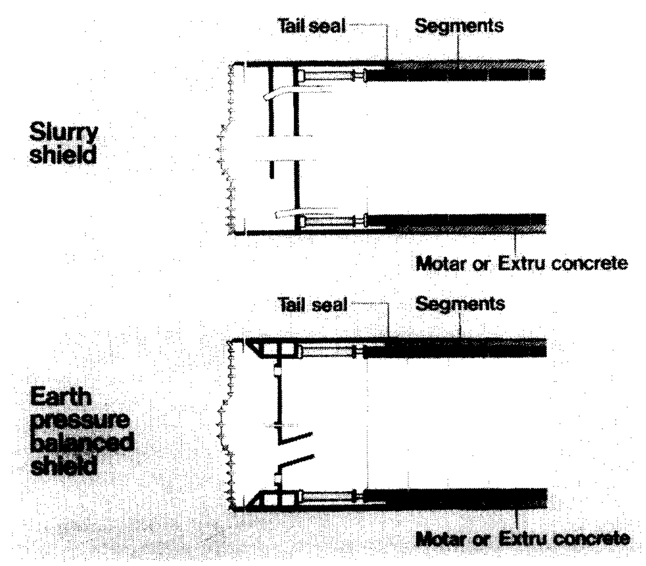

Figure 8. Erection of lining is similar for the slurry shield and EPB shield.

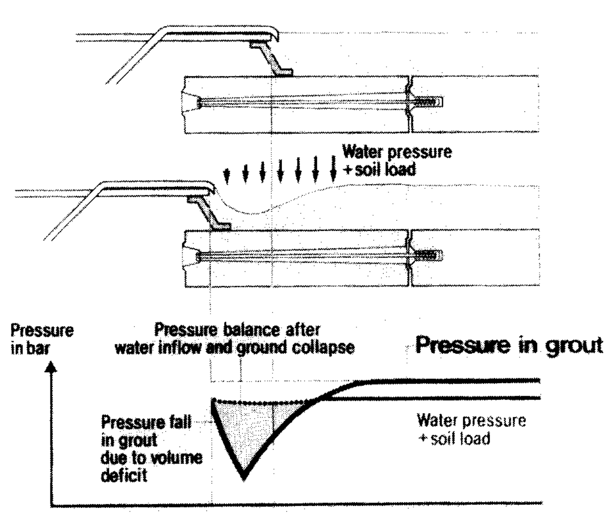

Figure 9. Variable volume grouting with a fixed tail shield.

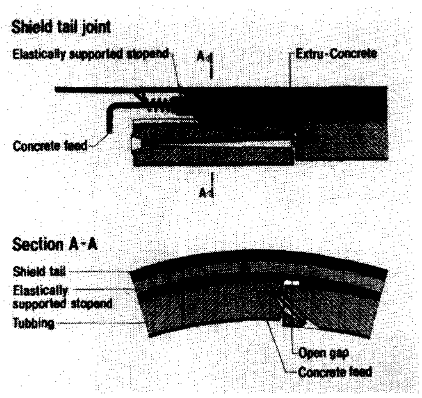

Figure 10. Extru-concrete seal.

ments. The more inexperienced the crew, the greater the problems these seals cause.

A shield tail seal system with rubber seals on a movable and elastically supported basis has yet to be developed. Therefore, we must be skeptical about undisturbed tunnel lining bedding in water-bearing cohesionless ground.

Steel brush seals

The steel brush seal, developed in Japan, can seal the gaps between inaccurately placed segments. The seals are arranged in as many as five rows, one behind the other. Grease is pumped into the chambers between the individual rows and held at a particular pressure, whereby the pressure in each chamber is increased towards the shield tail so that the pressure in the last chamber is about 2 bar higher than that of the grout. The grout is thus prevented from penetrating through the brushes to the shield interior.

The higher grease pressure means that grease constantly flows into the grout. However, the grease consumption is in fact significant.

It is still questionable whether soft ground can be supported at a constant pressure in groundwater.

Extru-concrete seal

A reliable shield tail seal that meets all of the requirements is the seal made with Extru-concrete (see Fig. 10).

A movable, ring-shaped steel device that closes the shield tail joint is elastically supported by hydraulic jacks from the body of the shield. The hydraulic jack circuit is linked to a regulated gas reservoir, which provides the elastic spring.

The steel device is sealed against the shield tail with a rubber seal, and with spring strips against the back of the tubing segments.

Extru-concrete is pumped through the movable steel device into the shield tail joint. The extru-concrete is a normal concrete with chemical additives to improve its flow properties.

The gaps between the inaccurately placed segments are sealed by the concrete itself in the Extru-concrete seal. Although initially some of the cement mortar flows out, a support matrix of coarser material quickly forms which, even for gap widths of 1.5 cm, provides enough support for the finer material. Even at pressures of 8 bar on gaps of 1.5 cm, this seal is effective for several hours.

The steel seal device is advanced by the pressure in the Extru-concrete itself (see Fig. 11). If the pressure in the concrete exceeds that calculated to be necessary to support the surrounding soil and ground water, the elastic hydraulic support gives and the steel shield tail seal assembly glides forward, but only so long as the pressure on the stop-end exceeds that required to counteract the pressure from the soil and groundwater (see Fig. 12). The gas reservoir in the hydraulic circuit ensures that a constant support pressure is maintained, even for small volume imbalances.

This system has already proven itself many times, the most impressively in Lyon, France, on the drive under the Rhone and Saone rivers, in water-bearing gravel where the distance between shield and river bed was less than 5 m and the shield diameter was 6.50 m.

On the Lyon drive, it was proven that it was possible to drive a shield in cohesionless ground without relaxing or loosening the surrounding soil. Settlements could be completely avoided.

Conclusion

Quality of the Tunnel

The development of machine technology, particularly because the advantages of electronic steerage and control systems can be increasingly applied, have made it possible to successfully drive tunnels in difficult soft ground with tunnelling machines.

When properly applied, the slurry and EPB shield systems now available permit considerable improvement in the quality of the completed tunnel. Up to now, the concept of quality in tunnel construction was of no great relevance—probably because the first priority, naturally, was to complete the designed tunnel in the specified time with the finances available.

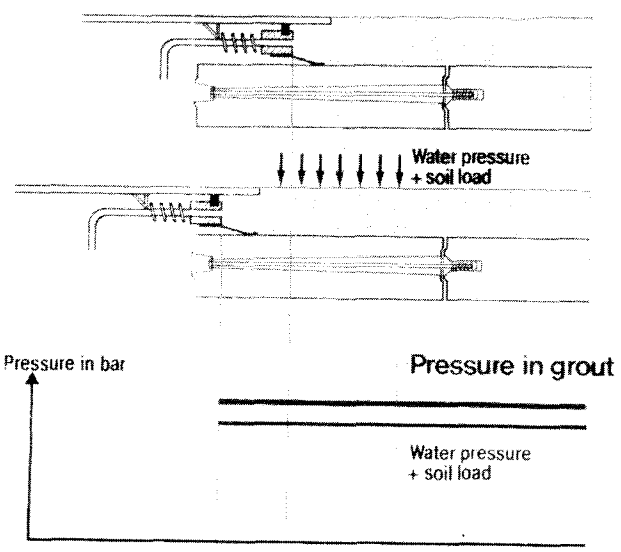

Figure 11. Constant volume grouting with an elastic shield tail seal.

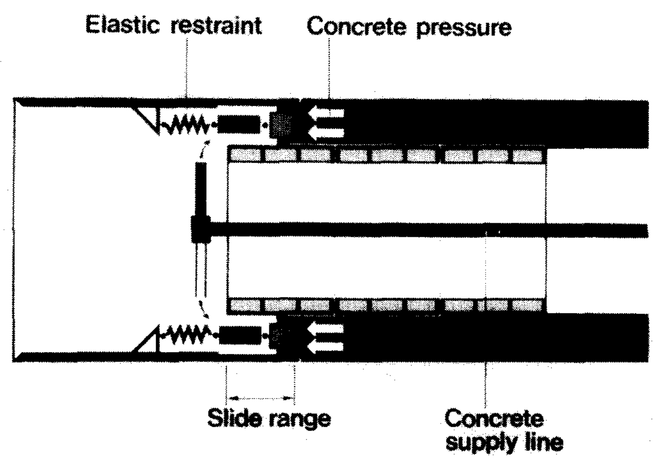

Figure 12. Elastically restrained stop-end slip form allows the steel shield tail seal assembly to glide forward only as long as the pressure on the stop end exceeds that required to counteract the pressure from the soil and groundwater.

However, with increasing safety during construction, the expectations with regard to quality have also risen.

In this context, the importance of impeccable bedding of the lining for the life of a tunnel and, thus, its quality, cannot be overemphasized.

Role of the Design Engineer in Mechanical Drives

If a tunnel is driven with a slurry or EPB shield, additional action to improve or change the property of the ground is not necessary. The protection of buildings or installations beneath which the tunnel is to pass can be achieved more effectively and cheaply if the mechanical and technical potential inherent in the TBM design are exploited. Here (and not only here) there is great scope for the planning engineer in an expanding field.

The introductory example showed that for particular geological conditions, a particular tunnelling machine must also be constructed. It is always necessary to define the geological conditions precisely.

In addition, tunnelling methods and machine systems designed to meet the challenge of geological difficulties must be evaluated and their effectiveness during construction monitored. For example, it would be a great advantage if a calculation method were developed for tunnel lining design that would take into account the influence of the tunnelling method on the bedding (see Fig. 13). □

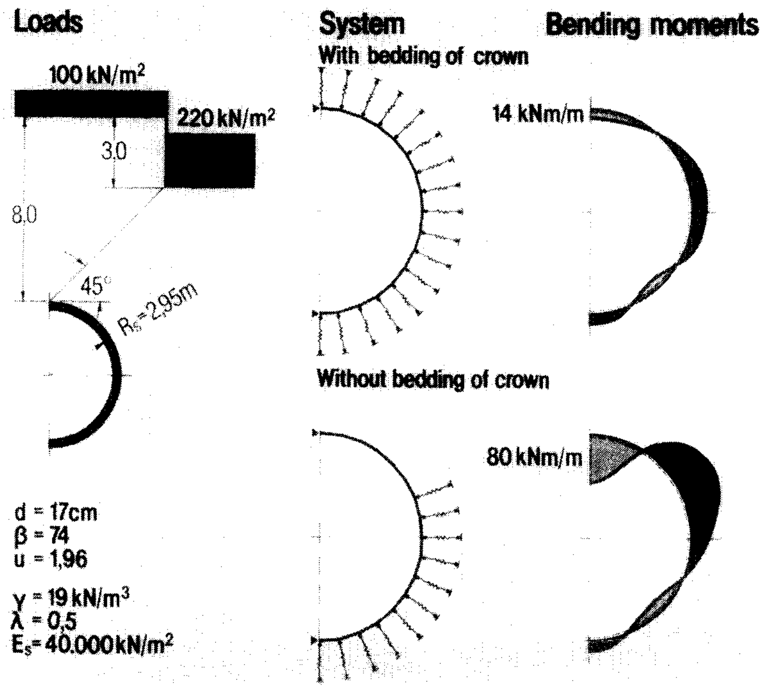

Figure 13. Influence of bedding of lining on the bending moments.

The Present and Future of Mechanized Tunnel Works in Soft Ground

G. Fukuchi

***Abstract**—In considering the recent progress of shield tunnelling in very soft ground, a new segmental lining design model is presented. The model takes into account the lower rigidity of joints between segments and rings. In showing some* in-situ *measurements during and after construction, the paper recommends referring to actual behaviour of the lining for determining design parameters. The current state of and future outlook for backfill grouting for segmental lining is also discussed. The paper summarizes various developments in extruded concrete lining with steel-bar reinforcement, which is regarded as a promising alternative to segmental lining in very soft ground.*

***Résumé**—En considérant les progrès récents dans l'excavation par bouclier dans un terrain très meuble, l'auteur présente un nouveau modèle prend en compte la faiblesse de rigidité aux joints voussoir-à-voussoir et anneau-à-anneau. A ce propos, il indique quelques résultats de mesures faites,* in situ, *pendant et après la construction sur le comportement de l'anneau, et il conseille de se référer à ces données empiriques pour déterminer les paramètres de calcul. Il discute ensuite du problème du coulis de remplissage en fonction des perspectives présentes et futures. Enfin, il expose sommairement différents systèmes d'extrusion du béton, avec armature, considérés comme une alternative prometteuse pour le revêtement voussoirs en terrain très meuble.*

Introduction

The cut-and-cover method has becomes increasingly difficult to adopt for constructing tunnels beneath streets in densely built-up areas because of conflicts between construction works and traffic. In contrast, the shield tunnelling technique has been developed in such a way that its application now permits safe construction even in very soft water-bearing ground such as quaternary alluvial deposits. Considerable surface settlement is likely to occur in cases involving relatively small depth of cover. However, such settlement can be kept small, and serious damages to neighboring structures avoided, through the use of well-controlled backfill grouting.

In shield tunnelling, the lining is erected by assembling prefabricated segments. A certain number of segments, fabricated in a factory, are transported and deposited temporarily at a yard near the work site. It is not easy to ensure such yard space, even for a temporary period, in a densely built-up area.

For tunnel linings erected by segment assembly, the various methods of design that have been proposed for various soil and underground water conditions ensure the mechanical strength of the lining in most cases. The water-stopping faculty of the lining is provided by sealing and caulking materials inserted in the joint between segments. However, long-term water sealing is more difficult to achieve; if there is a high need for it, a secondary lining, sometimes further reinforced by steel bars, must be used.

In parallel with the recent development of shield tunnelling, the so-called NATM—using rock bolts and shotcrete—is being applied to urban tunnelling. This method has become fashionable because it is superior in economic terms to shield tunnelling, provided that the ground displacement that results from water-stopping measures is not a cause for environmental concern.

Some evaluations of the segmental lining method have concluded that the high price of segments, combined with the occasional requirement of a secondary lining, and expenses for transporting segments and renting yard space, result in costly tunnelling work. During the last decade, the extruded concrete lining (ECL) system has been developed in Europe, especially in Germany, as an alternative to segmental lining. This technique extrudes plain concrete or steel-fibre concrete into the space between the excavated ground surface and the inner forms. However, in consideration of the very loose ground and possible seismic forces, stronger reinforcement of concrete by steel bars is considered necessary. Such a modification of the ECL method is now underway in Japan.

In any case, the use of the ECL system makes it theoretically possible to lessen ground settlement owing to the extrusion of concrete, thereby eliminating the need for extra works such as backfill grouting. The ECL system is thought to be a promising and, ultimately, economical method, although it requires installations peculiar to the extrusion of concrete, as well as inner forms, which are difficult to reuse.

Shield Tunnelling in Japan

Structural Design Model

In shield tunnelling, the lining is erected by assembling prefabricated segments by means of joints such as bolt and nut, tenon and mortise, or knuckle and pin. Unlike *in-situ* concrete lining in rock tunnelling, segmental lining is bestowed with supporting ability immediately after its erection. Segments are generally placed in an alternating pattern with those of adjacent rings.

Various two-dimensional models, approximated by a continuum ring with uniform rigidity, have been proposed, although the assumptions for loading and bedding are diverse.

In sound ground, e.g., well-compacted diluvial soil layers, the segment ring hardly suffers any excess of bending moment, thanks to the horizontal soil reactions, which act effectively. Hence, there is no actual hindrance, even when a complete continuum ring model is used for design without considering the behaviour of joints.

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Meanwhile, closed-shield tunnelling (slurry and earth-pressure balanced types), which has developed rapidly for the past 10 years, now makes it possible to drive with ease in very soft and water-bearing ground (e.g., N-value < 5). In such ground, because the horizontal soil reaction no longer remains, it is necessary to avoid large deformation of the ring by strengthening the joints. However, it is impossible to strengthen joints to the same rigidity as that of the segment. Consequently, a calculation based on a continuum ring model produces an underestimated bending moment of the ring.

In consideration of this situation, the Japanese National Railways took the initiative to improve the above-mentioned structural model of common use by introducing an effective factor ($\eta < 1$) for the ring rigidity lowered due to segment joints, and a rigidity increment ξ, supported by adjacent rings through ring joints. This modification is also in common use in Japan for soft ground tunnelling.

Further, Kubo & Yuki (1968), Yamamoto (1976), and Murakami and Koizumi (1978) successively proposed more sophisticated structure models, which similarly represent segment joints by flexural springs. However, none of these models took into account the relative displacement between rings. In 1978, Yamaguchi et al., attaching importance to the relative displacement, proposed a new model, which provides shear springs in radial as well as in peripheral directions, in addition to the flexural springs.

Figure 1 shows the said two-ring structure model. A computer programme is also available for a three-ring model. The model may be converted universally to all kinds of models, including a continuum ring model, by choosing suitable spring constants. Going further, Hanya (1985) extended the model to the case involving a secondary inner lining, adding shear springs between two linings. This extended model is capable of estimating forces acting not only at each joint, but also shear forces between the linings.

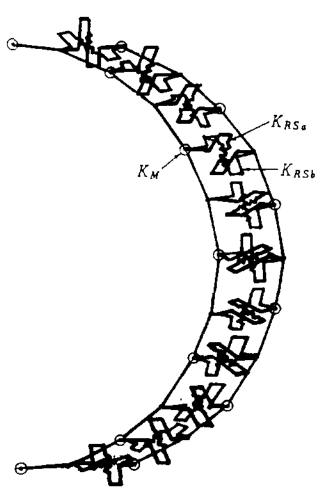

Figure 1. Two-ring model with flexural spring and shear springs between rings.

Table 1. Coefficients of horizontal earth pressure and soil reaction (Japan Society of Civil Engineers).

Type of Soil	Coefficient of Horizontal Earth Pressure	Soil reaction (kgf/cu. m)	N-value
Well-solidified sandy soil	0.35–0.45	3.0–5.0	N ≥ 30
Solidified cohesive soil			N ≥ 25
Compacted sandy soil	0.45–0.55	1.0–3.0	15 ≤ N < 30
Hard cohesive soil			8 ≤ N < 25
Medium cohesive soil		0.5–1.0	4 ≤ N < 8
Loose sandy soil	0.50–0.60	0–1.0	N < 15
Soft cohesive soil	0.55–0.65	0–0.5	2 ≤ N < 4
Very soft cohesive soil	0.65–0.75	0	N < 2

Loading Conditions

The loading conditions adopted thus far are diverse, reflecting the nature of the soil formation in each region. Assumptions related to horizontal earth pressure, soil reaction, modulus of rigidity, and vertical earth pressure are the essential design factors. The coefficients of horizontal earth pressure and of soil reaction given in Table 1 are derived from the Standard Specification of JSCE for shield tunnelling.

In addition to these loads, temporary forces acting on the lining ring during shield driving works should be considered. These are:

- The reaction of shield thrust.
- Backfill grouting pressure.
- Soil pressure just after the shield tail is pulled off.
- Lateral forces generated by a second tunnel driving, in the case of a twin tunnel.
- In earthquake regions, it is also necessary to assess the possible effects of seismicity.

Examples of measurement are shown in Figuers 2, 3 and 4. Figure 2 shows the result of stress measurement in the reinforcement six months after the erection of a railway tunnel lining, as compared with the calculation assuming the full overburden and Terzaghi's loosened soil pressure. The tunnel lies at a depth of 25 to 30 m in a diluvial sand layer. In this case, the assumption of the loosened soil pressure seems more reasonable than the full overburden pressure.

Figure 3 shows stresses in the reinforcement measured before and after the second shield driving had passed by. The ground consisted mainly of alluvial cohesive soil layers. The figure implicates the redistribution of ground stresses as a result of soil disturbance associated with the second shield drive.

After the lining is erected, it will undergo a change in loading condition due to a rheological transition of soil reaction, construction error, backfill

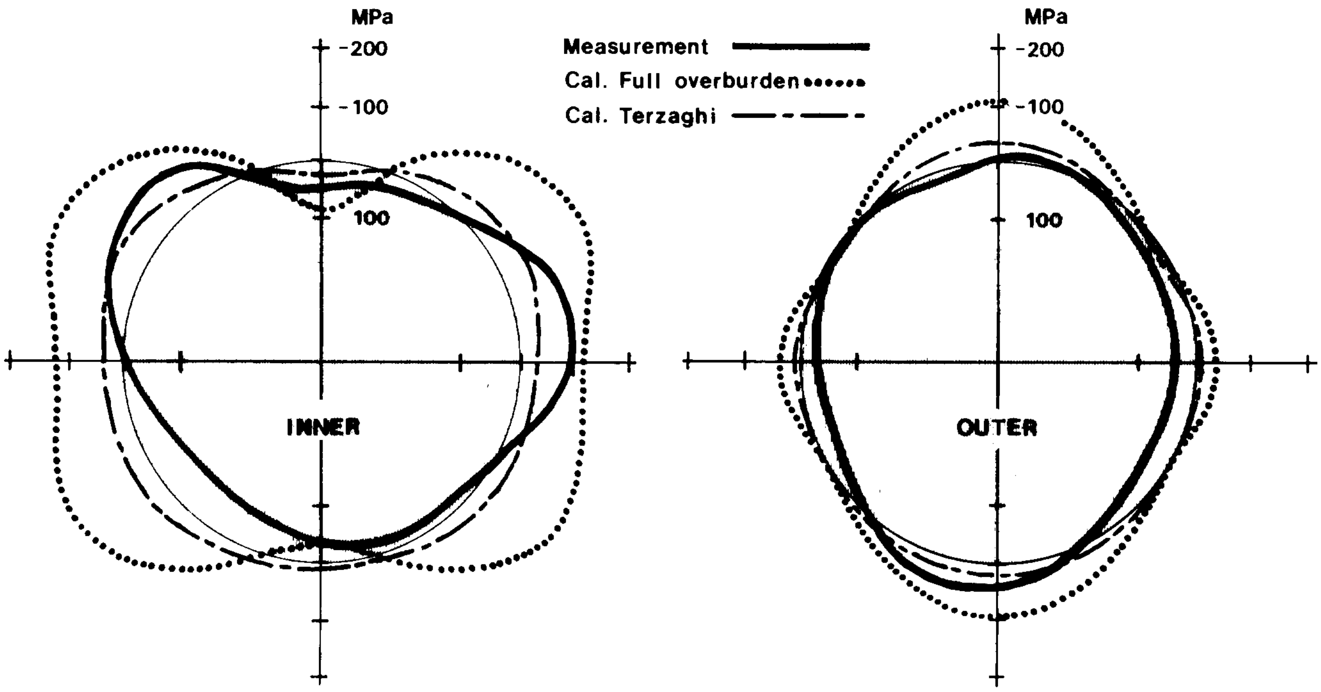

Figure 2. Calculated and measured stresses in reinforcement for the Yurakucho Tunnel (after Yamaguchi and Kawata).

grouting, or a change in air pressure. Figure 4 shows the result of long-term observation of stresses in steel bars for the same tunnel as that shown in Figure 2. In the figure, no large difference is seen between measurements taken one month later and those taken six months later, as compared with the difference between measurements taken a week later and those taken a month later. This finding means that the ground is likely stabilized approximately six months after shield driving is complete. This tunnel was constructed in 1972 with compressed air; at that time, the closed-type shield had not yet been developed.

Designers should be aware that tunnels are subject to various disturbances to the loading condition, during construction and after completion for several months. Even a well-devised structural model cannot predict precisely what will happen in actuality; however, it is of great value to assess the influence of change in loading conditions by means of a parametric study. In this regard, a data-bank backed up with various kind of *in-situ* measurements is strongly recommended.

Backfill Grouting

Shield driving in soft ground is usually subject to disturbances of the ground, which consequently undergoes displacement due to soil consolidation. Almost all shield tunnelling works in soft ground are assisted by backfill grouting to a volume greater than that of the tail void. The surrounding ground then heaves once more, and is disturbed again. The consolidation takes place simultaneously with this disturbance, and the ground settlement thus attains a final value. Ground heaving is sometimes harmful, too, although it

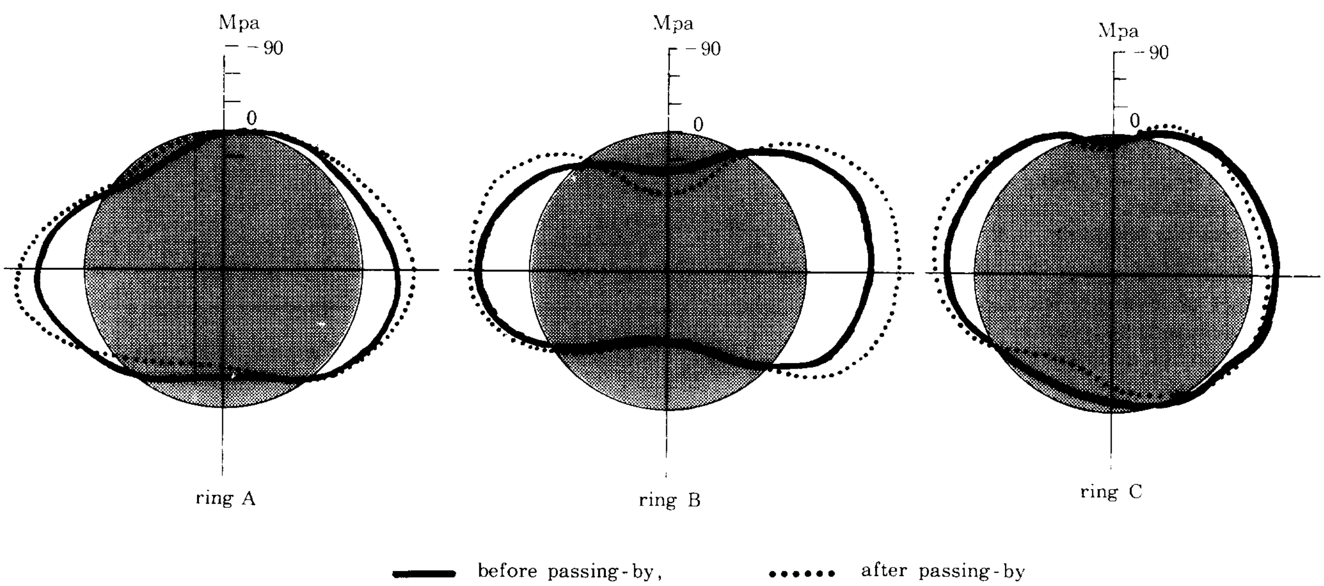

Figure 3. Change of stresses in inner reinforcement, before and after the passing by of the second shield, for the Hamamatsucho Tunnel (after Yamaguchi and Kawata).

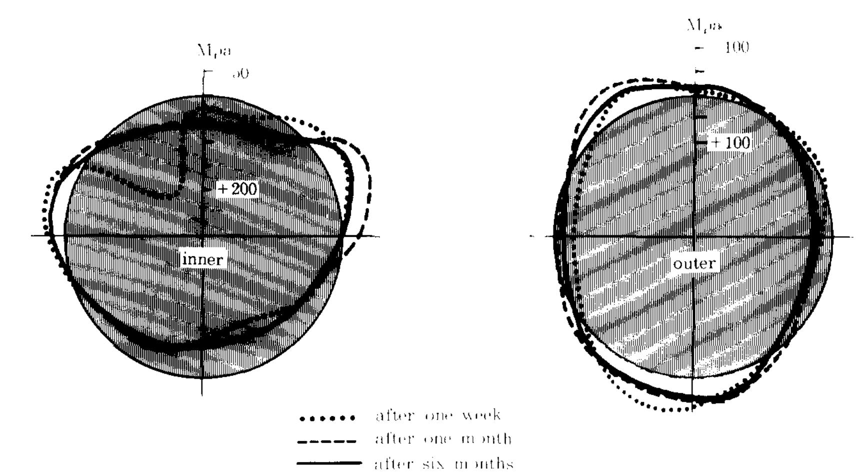

Figure 4. Change of stresses in reinforcement of the Yurakucho Tunnel after some time elapsed (after Yamaguchi and Kawata).

is momentary; but the harm is in the settlement, rather than in the heaving, which is a more serious issue. Backfill grouting is aimed primarily at suppressing the final settlement, and secondarily at improving the impermeability of the ground.

Some *in-situ* observation in very soft cohesive soil layers has shown that the tail clearance is completely closed within 30 to 60 minutes by ground pressure. Mori et al. (1984) warn that if backfill grouting is not performed in a timely manner, it accelerates the stress redistribution in the ground. In such a case, the more the grout is increased, the more the ground is disturbed, resulting in an adverse effect. Accordingly, the following conclusion can be derived; the grouting should be well controlled and performed in a very short time—or without the slightest delay, if possible—after the shield tail is advanced.

Backfill grouting systems are classified by the timing of injection, as shown in Table 2.

According to an inquiry carried out by Japan Tunnelling Association in 1984, the share of each backfill grouting system adopted in Japan is as follows:

Delayed Grouting, Other:	6.4%
Quick Grouting:	67.5%
Simultaneous Grouting:	25.0%
No reply:	1.1%

The fact that the quick grouting system is adopted most often is thought to be due to its simple application. However, in response to a question about future prospects for choice of system, there were 47 replies for simultaneous grouting versus 57 replies for quick grouting. This response pattern may imply a superior effectiveness of the former mthod for suppressing ground settlement.

With regard to the amount of grout, theoretically the volume of grout used to fill up the tail void would be equal to the volume of the void. In practice, however, the actual amount of grout used may be as much as twice the volume of the void, due to stray run-out of grout or separated water into the ground or roundabout to the face.

To specify the grouting pressure, an empirical rule is applied, such as a pressure higher than the soil pressure acting at the tunnel crown (approximated to that of full overburden at the level of the crown), or a pressure higher than the ground water pressure by 50 to 150 KPa. Excessive grouting pressure not only accentuates the ground settlement, but also exerts a locally concentrated force capable of causing a large deformation or failure of segment ring.

There are two ways of controlling grouting:

1. By regulating the rate of grout so as to maintain the pressure within a certain range of variation.
2. By grouting an amount equal to a certain proportion of the theoretical void.

The control system is in urgent need of automatization because of a shortage of skilled laborers. However, ultimately it is human intelligence that selects suitable values to be given and modifies them according to changes in ground conditions or tunnelling processes. Therefore, it seems unlikely that a fully automatized backfill grouting system will be realized in the near future.

There are two types of grout mix: the one-solution type and the two-solution type. The former excels the latter in pumpability and resistivity to dissolution of the alkaline component. However, the one-mix grout takes 5 to 12 hours to harden, during which time settlement of the ground progresses around the tail void. In contrast, the two-solution grout, in which the water-glass is mixed with the former type of grout immediately before the grouting process, can harden in from several seconds to several tens of seconds, depending on the mixing ratio. However, this latter mix is liable to stick to the inside of grouting pipes, and is more expensive than the one-solution grout.

From the aforementioned inquiry in 1984, the share of use for each of these types has been derived as follows:

Sand-gravel, sand:
35% for one-solution
65% for two-solution

Silt, clay:
47% for one-solution
53% for two-solution

These results suggest a trend toward the two-solution type of grout. Because more than five years have passed since the time of that inquiry, the share of use of the two-solution grout undoubtedly is increased today, in spite of its higher price.

The Extruded Concrete Lining (ECL) system described below does not require such intricate handling of grouting. However, the segmental lining system will not be wholly replaced by ECL system in the near future. Ample scope therefore remains for improving the backfill grouting system, as well as the grout mix.

Table 2. Classification of backfill grouting methods.

Type of Grouting Method	Procedure
Delayed grouting	Grouted through a grout hole in the centre of a segment placed several rings in the back
Quick grouting	Grouted for just one ring after advance of machine by the width of one ring
Simultaneous grouting	Grouted in following the tail, with no gap

Development of the Extruded Concrete Lining (ECL) System in Japan

Review of Developments

For shield or TBM tunnelling, the use of the ECL system offers a monolithic concrete lining by extruding concrete between the excavated surface and the inner form.

This method shows promise as a powerful alternative to the segmental lining system. According to Novak (1984), the ECL concept was born about 80 years ago in Europe, although it was not until the latter half of the 1970s that the idea was finally realized. Tunnel projects that have used the ECL system and that have been discussed in the literature are shown in Table 3.

It should be noted that all of the European ECL works have involved plain concrete or steel-fibre concrete; some were assisted by a partial secondary lining. On the whole, the technique of the European ECL system may be considered as state-of-the-art. In Japan, in contrast, the first ECL tunnelling work, which took place in 1967, was assisted by steel-bar reinforcements. From that time to the present, the impetus for ECL development has been halted.

As described above (see "Structural Design Model"), tunnels in Japan—even those driven in very soft ground—are constructed by the use of a closed-type shield machine, without any ground improvement. In this connection, when Japanese engineers have attempted to adopt the ECL instead of the usual segmental lining, they have to start from a point that requires reinforcement of the ECL by steel bars.

Because the steel-bar reinforcement must be erected in a limited space, this type of system is unavoidably more complicated than an ECL system without steel-bar reinforcement. Various systems have been invented in Japan, and tests on these systems and on the concrete mix have been carried out both in laboratory setting and in work sites.

Since 1987, three sewerage or cable tunnels in Japan have been successfully constructed with steel-bar reinforced ECL. The outer diameters ranged from 2 m to 3 m; the length of ECL sections, from 50 to 200 m. The ground for these tunnels generally was so sound that the face was self-supporting, such as firm loam or mudstone; perhaps steel bars were not necessarily indispensable. However, these tunnelling projects were aimed at verifying the workability of the system, rather than at reflecting the mechanical behaviour of the lining in ECL design. The current state of ECL development for soft ground in Japan is summarized below.

Steel-Bar-Reinforced ECL System in Japan

The following problems must be solved for linking steel bars in the ECL system as it is used in Japan:

- The steel-bar reinforcement has to be erected and, in most cases, linked with the previous reinforcement, in a very limited space around the shield tail. Automatization of such work is difficult.
- There is some concern about whether or not the extruded concrete has filled up sufficiently around the steel bars.
- Unlike the segmental lining system, which provides a tail clearance, the ECL system has no margin for rectifying directional deviation of the excavation. Considerable irregularity in the lining thickness and in the cover depth of reinforcement should be avoided.

In addition to these technical issues, works necessary to link the reinforcement in the ECL influence the cycle-time and, consequently, the rate of advance of driving. Various systems are now under development in Japan. Some typical examples are described below:

Type A: The front end of the previous ring released. As shown in Figure 5(a), the reinforcement is supported at one end by a press ring, and linked with a part of previous reinforcement by means of hook-bolts. During this time, the front end of the previously extruded ring is released. The press ring is withdrawn after the front end becomes self-supporting, within several hours.

Table 3. Some tunnelling projectsworldwide that have used the Extruded Concrete Lining (ECL) System.

Date(s) of Construction	Location/Type of Tunnel	Type of Lining	Type of Ground	Literature Reference (see "References")
1967/68	Nagoya Drain	Steel-bar reinforced concrete	Sand and gravel layer	Suzuki (1969)
1976/77	Prague Metro	Plain concrete	Slate layer	Gran (1981), Novak (1984)
1978/79	Hamburg Sewer	Steel fiber concrete	Water-bearing sand layer	Babendererde (1984)
1980/82	Frankfurt Metro	Steel fiber concrete, partial secondary lining	Clay layer	Babendererde (1984)
1981	Tokyo Sewer	Stel-bar reinforced concrete	Alluvial sand layer	Hatada et al. (1983)
1984/87	Lyon Metro	Secondary lining, partially steel-bar reinforced	Sand and gravel layer	Babendererde (1984)
1985/86	Freudenstein Pilot Tunnel	Plain concrete	Gypskeuper	Babendererde (1984)
1988	Essen Metro	Steel fiber concrete, partial secondary lining	Silt layer and marl	Babendererde (1984)
1988/89	Shinanogawa Water Tunnel	Plain concrete (assisted by Hochtief AG)	Sand and silt layer	Kawana (1987)

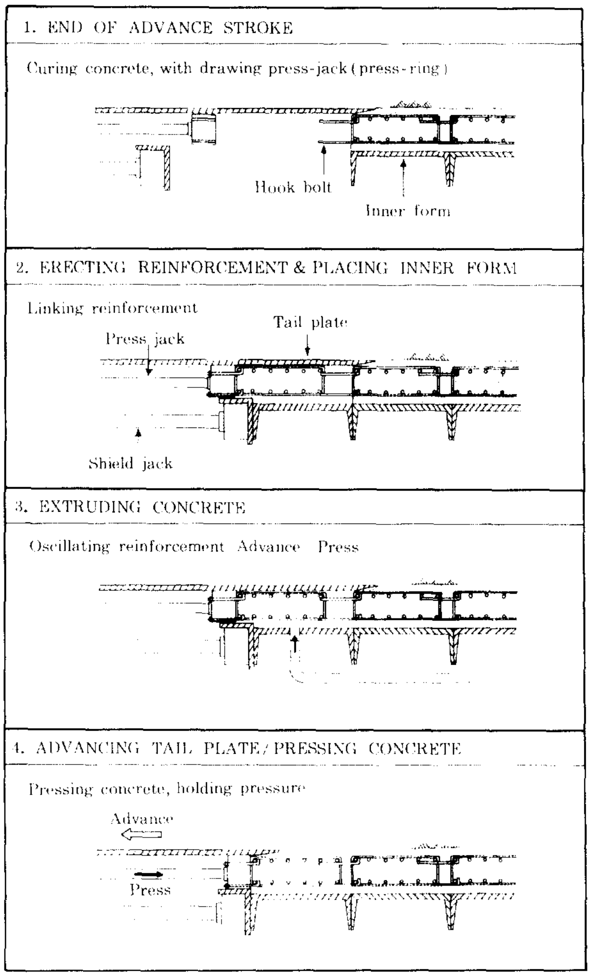

Figure 5(a). ECL system, Type A.

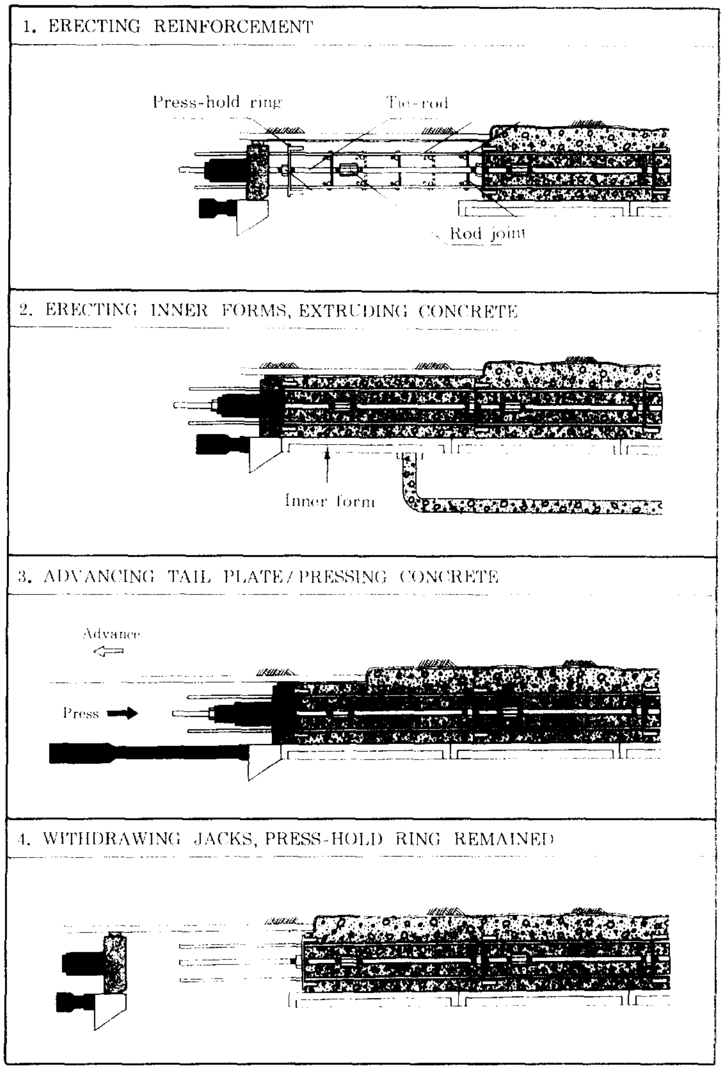

Figure 5(b). ECL system, Type B.

Type B: The front end of the previous ring is always retained. As shown in Figure 5(b), a tie-rod is provided to ensure the linking of each reinforcement. A press-hold ring is fixed to the tie-rod and buried in the extruded concrete, thereby serving as the hoop reinforcement.

Type C: Involves the reaction of pressing on the rear skin plate of the telescopic shield. The thrust of the shield is supported by the inner forms and exerted at a front skin plate. The press reaction is applied on a rear skin plate, as shown in Figure 5(c). Thus, the extruding and pressing occur independently of the advance of shield.

Type D: A two-step process, involving extrusion and pressing. This system operates on the same principle as Type B. As shown in Figure 5(d), an extrusion nozzle with a press ring is provided inside the tail plate towards the rear. Second-step extrusion and pressing take place for the tail void. A perforated steel sheet is used instead of steel bars.

In shield driving, a certain amount of directional deviation is unavoidable in spite of correcting devices. For each of the four types of ECL systems described above, some regulation devices are provided to assure a minimum effective thickness of lining and sufficient cover depth of reinforcement.

These types of systems have already been used at work sites for small-diameter tunnels. On the one hand, transporting the inner forms and assembling them in a limited space were difficult; on the other hand, the materials and mechanical parts were small and relatively lightweight. The application of steel-bar-reinforced ECL still has not been attempted on large-diameter tunnels. In such cases, different aspects of the problems described herein can be expected to occur.

Materials for ECL

Obtaining early strength of extruded concrete is one of the major tasks for the success of the ECL system. In the ECL method, the inner form furthest from the machine is disassembled after sufficient supporting strength of concrete has been ensured, after which the form is transported to the front and reassembled there. Accordingly, the longer the time required to obtain early strength of the concrete, the greater the number of expensive inner forms required.

The concrete should have sufficient fluidity during the extrusion process. The easiest way to achieve fluidity is to increase the water/cement ratio; however, an excess of water causes the cement to bleed. To achieve a balance between these two conditions, AE or plasticizer is mixed in without increasing the water/cement ratio.

For the ECL system, the early strength of concrete can be obtained through a process of press and dehydration. Niitsu et al. (1989) reported the results of laboratory experiments, as shown in Figure 6. These results show that the pressure holding time has a greater influence for early strength of concrete than does the magnitude of pressure. Meanwhile, it should be considered that the greater the quantity of plasticizer used, the slower the dehydration and, finally, the longer the time necessary to attain the early strength.

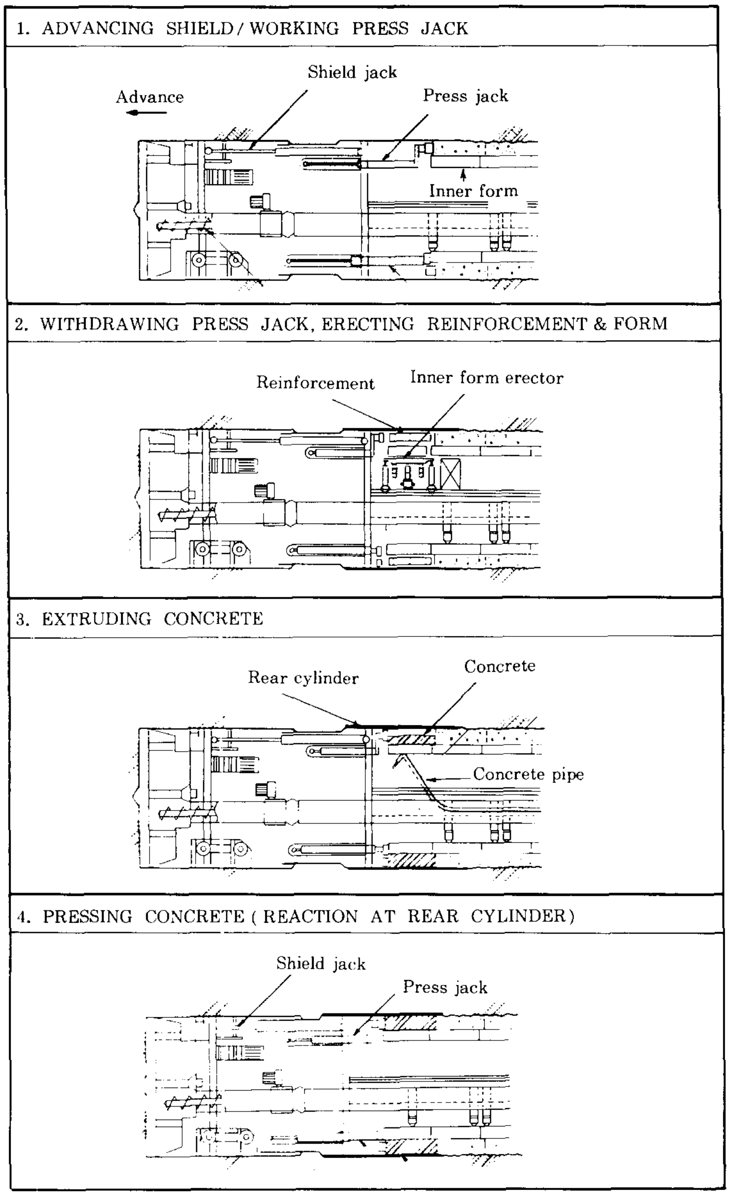

Figure 5(c). ECL system, Type C.

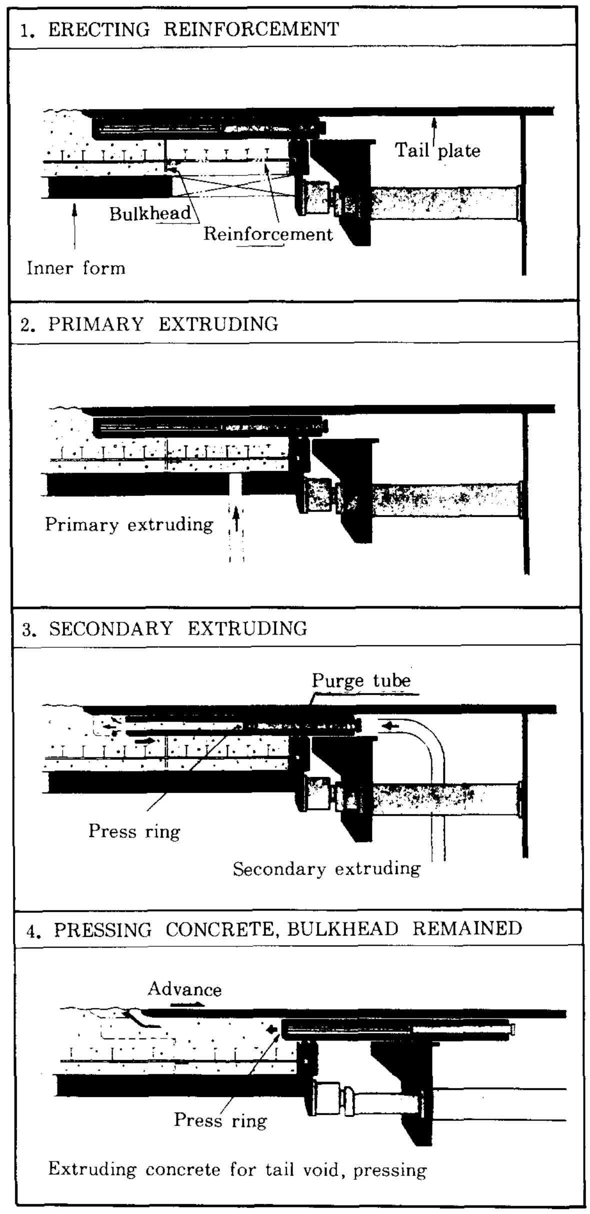

Figure 5(d). ECL system, Type D.

The ECL Subcommittee of the Japan Tunnelling Association conducted research by questionnaire on the development of the ECL system. Figure 7 shows the distribution of responses concerning the water/cement ratio and unit amount of cement. It seems that the water/cement ratio as a whole is slightly higher in Japan than that of the European ECL, while amount of cement is less. These results indicate the importance attached to better pumpability, transportability and filling-up of concrete, as well as to mitigation of thermal stress and drying shrinkage after concreting.

It is worth mentioning an opinion *against* the use of press and dehydration to obtain early strength of concrete. Ozawa et al. (1989) have proposed instead a high-performance concrete, stressing that a concrete of good quality and durability with slow curing should be placed, without attempting to achieve early strength by press and dehydration. The question of choice of materials relates to the priority assigned to the economy of construction versus economy of maintenance, and raises a question to be answered in future.

Conclusion

This paper has emphasized urban tunnelling in soft ground in Japan, considering both design aspects, and the development and future prospects for the extruded concrete lining (ECL) system.

For shield tunnelling in soft ground, the importance of the rigidity of joints between segments and rings should be taken into account. It must be stressed that the mechanical behaviour of segmental lining may be considerably different from the calculated behavior, which represents accumulated measurement data taken from many tunnelling works performed in a wide range of ground and external conditions.

When the tail clearance, associated with shield tunnelling, is left as it is, some ground settlement occurs. In urban areas, backfill grouting is performed in almost all shield tunnelling for fear of giving rise to a serious environmental problem. The best method of backfill grouting involves filling up grout in the tail void without any delay. This "simultaneous grouting" technique should be distinguished from other delayed groutings. It is hoped that further improvements in this system and the materials for the simultaneous grouting will permit it to be used without delay.

Meanwhile, the ECL system is expected to provide relief from such troublesome works. In short, the ECL system is an alternative to the supporting faculty provided by segmental lining with backfill grouting. With the ECL system, there is no need to tighten many joints or to prepare skilled laborers for grout mixing and operation. It is for exactly that reason that the ECL system is thought to hold promise for reducing the work period, as well as for

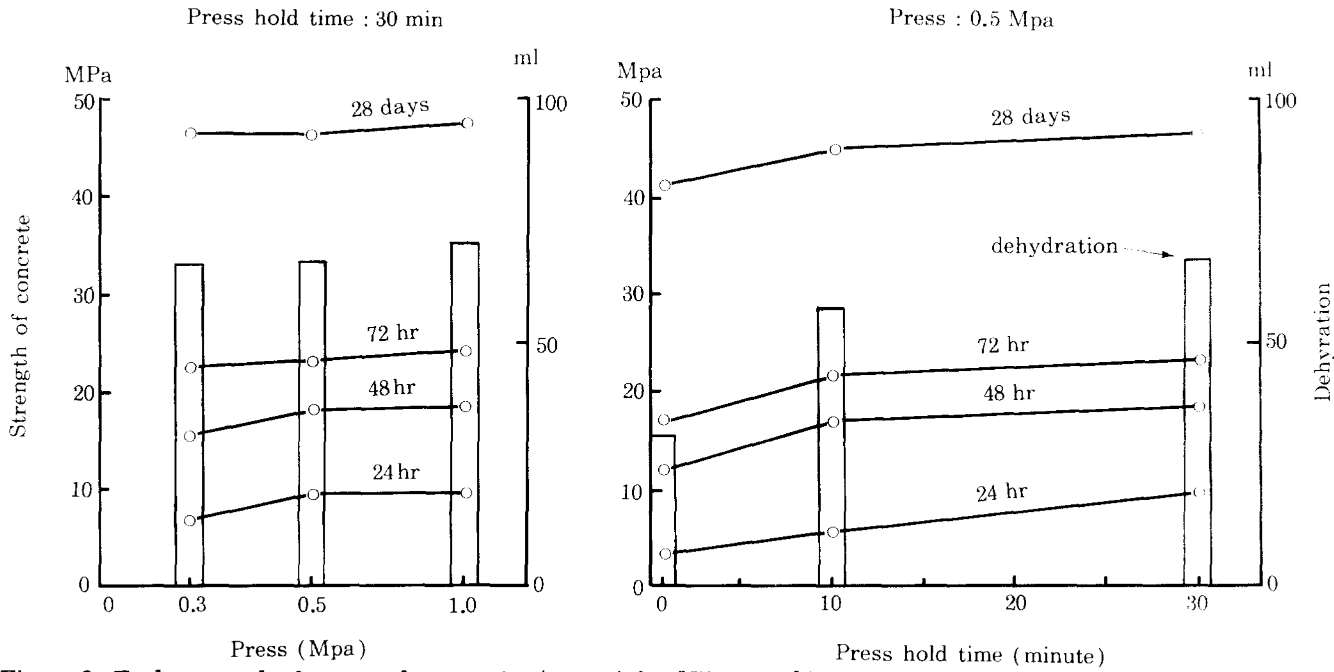

Figure 6. Early strength of concrete by extrusion/press (after Niitsu et al.).

lowering construction costs and, presumably, for automating the work.

The ECL system was used successfully on a tunnelling project in Japan in 1967, although the system was not very sophisticated. Recent activity in ECL development in Japan has certainly been stimulated by ECL tunnelling in Europe, especially in Germany.

However, especially regarding the use of the ECL system in soft ground, there remain some concerns that the tunnel support has to depend on the reinforcement and inner forms until the concrete reaches full strength, whereas such support is available immediately with the segmental lining system. Therefore, substantial improvements in the ECL system will be required in order for it to be implemented in Japan as a reliable alternative to the segmental lining system. □

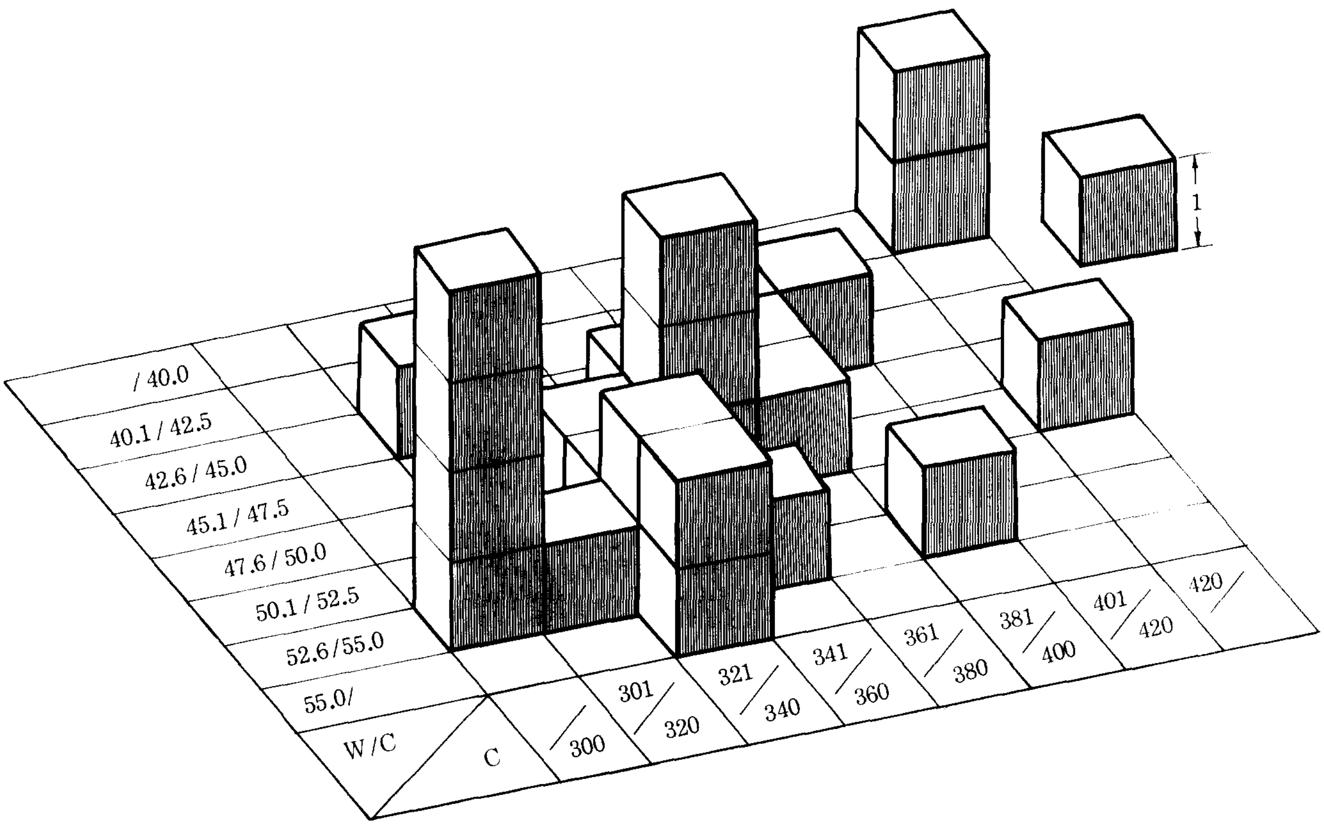

Figure 7. Results of questionnaire concerning water/cement ratios and unit weights of cement.

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Pilot Bore Excavation with TBM for the Design and Construction of Larger Tunnels

Sebastiano Pelizza

Abstract—*The author reviews the use of the tunnel boring machine (TBM) for pilot bore excavation in Italy, and summarizes the main functions of the TBM pilot bore with respect to large tunnel design and construction. Cost comparisons with traditional excavation methods are included.*

Résumé—*L'auteur évalue la machine à forer (TBM) utilisée pour un projet d'excavation par forage expérimental en Italie, et résume les fonctions principales de la machine en ce qui concerne la conception et la construction des grands tunnels. Les coûts par rapport aux méthodes d'excavation traditionnelles sont également mentionnés.*

The excavation of a 3.5- to 5-m-diameter pilot bore, using TBM, for the design and construction of larger highway, railroad, subway, and hydraulic tunnels, both in rock and in soft ground, is and has been a fairly common practice in Italy for the last seven years (see Table 1).

Since 1982, 30 pilot bores totaling about 103 km have been excavated or are in progress using TBMs; another 92 km of bores is forecasted, 17 km of which will start in the near future.

The use of the TBM pilot bore (see Fig. 1) has become so widespread because it can provide several functions, for both the design and the construction of larger tunnels.

For the purpose of design, the TBM pilot bore provides the following eight functions:

1. A continuous geological exploration of the rock mass at the horizon of the tunnel. This is a very important function because many other functions descend from it.

2. The geomechanical characterization of the rock mass, locally. An accurate and exact geological-structural surveying can be done on the bore walls, so that the rock can be read like an open book. All of the *in-situ* geomechanical measurements needed to characterize the rock mass can be easily executed.

The TBM can assume the role of a test apparatus to determine some of the geomechanical characteristics of the rock mass.

The "RS Method" (Becchi 1986; Lunardi 1986), which was developed in Italy, allows the inner rock mass strength to be estimated continuously (see Fig. 2). For this purpose, working and rating parameters of the TBM are used (especially the specific energy employed to crush the rock). The method can be used only in rock masses that can guarantee a minimum stability of the bore at a medium to high advance speed. The method cannot be used in poor ground, where the power of the machine is mostly required not for excavation, but for the auxiliary operations.

3. The study and measurement of *in-situ* stress in the geologic media.

4. The study of the behavior of the bore after excavation. It should be noted that it is difficult to make geotechnical measurements while the TBM is operating; this is true in particular for convergence and extension of the rock, which are interesting phenomena to be pointed out when one starts with measurements very close to the face. Therefore, it is necessary to plan for the stoppage of the TBM for the periods of such measurements; these stoppages have to be anticipated in the contract, and the cost must be paid to the contractor.

Figure 1. A TBM pilot bore (10m²) in a railroad tunnel (95m²) in a flysch rock mass (Leonardi 1987).

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Table 1. TBM pilot bores in Italy since 1982.

Tunnel	Client	Pilot Bores	
		Number	Length (km)
Road	A.N.A.S. (Autonomous National Board for Roadways)	6	12.01
Highway	Autostrade S.p.a. RAV—Raccordo Autostradale Valle d'Aosta S.p.a.	6	12.66
Railroad	F.S.-Ente Ferrovie Stato (State Railroad Organization)	12	69.95
Subway	Metropolitana Milanese S.p.a.	various stretches	4.51
Hydraulic	ENEL (National Board for Electric Power) Ministry of Civil Defense	2	3.84
	TOTAL	30	102.98
	Further Planned TBM Pilot Bores	11	92

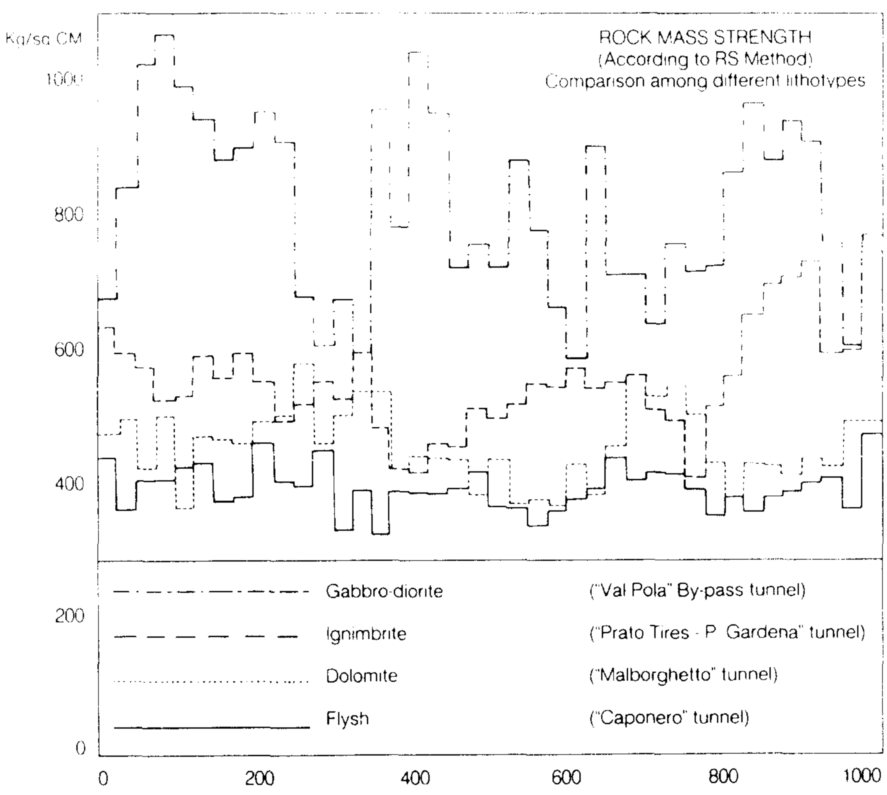

Figure 2. RS Method: rock mass strength in different tunnels and rock types (Italstrade 1989).

5. The continuous design of the support systems. The project can be planned with precision because the geomechanical characteristics of the rock, such as the *in-situ* stress field and behavior of the bore are already well known.

Another procedure can also be followed for evaluating the supports for the stability of large tunnels. When the TBM is working at the fastest possible speed, the minimum number of supports is used: this number represents the minimum level required for the stability of tunnel. Therefore, the support requirements for the pilot tunnel can be used as a measure for the design of support for the larger opening. The link between the two requirements is made analytically by means of the confining pressure necessary for the stability of the two tunnels.

6. To study, in a precise and continuous way, the excavated material and to determine the possibility of employing it as a concrete material, or earthwork, or disposing of it as waste.

7. Planning and scheduling the excavation of the larger tunnel.

8. Making a correct evaluation of the time and cost of the enlargement.

For the purpose of construction, the TBM pilot bore provides the following functions:

9. Preventive and continuous drainage of water and gas.

10. Preventive improvement of the rock mass before excavating the larger opening.

This operation can be performed when it is necessary: i.e., in zones of weak rock, or in failed and fractured zones. In this way, the enlarging excavation can proceed without obstacles which, in the absence of the pilot bore, could occur suddenly and could require long interruptions of the work.

A classic example of this system is the construction, in sand and gravel, of some tunnels of the Milan subway. Another example is a railway tunnel passing through Milan (see Fig. 3), where the preconsolidation with radial-ring injections was systematically executed by pilot-bore (Picco 1986).

11. Preventive de-stressing of high stresses in the rock mass.

12. Easing the enlargement excavation. The pilot bore is an additional free surface that eases the rock crushing, either by explosives or by machine. In particular, when explosives are used, the pilot bore represents the "opening bore" (the cut, which is no longer to be executed for the enlargement. Consequently, the amount of explosives and the number of blast holes is reduced, with a resulting decrease in the vibration-damage to the surrounding rock mass.

13. Facilitating better control of vibrations by blasting (Leonardi 1987). This control is related to the following factors:

- The elimination of the "opening bore", which usually causes the most intense vibrations; and
- The possibility of designing the explosive charges for enlarging as desired.

Figure 4 compares controlled blasthole patterns for traditional excavation with those for a pilot tunnel enlargement.

14. Easing the ventilation during enlargement (full-length pilot bore).

15. Easing the introduction of intermediate entrances for multi-point excavation of the enlargement, thereby reducing the excavation time for the larger tunnel.

16. Elimination of geological surprises. This is very important aspect, both in the planning phase of the work and in the contract management. Geologic surprises, and the related possible claims, are limited to the execution of the pilot bore, which, in terms of economics, represents less than 20% of the total cost of the work. This may be a good thing for the client—but is it so good for the contractor?

17. Routine progress of the enlargement excavation. This is a direct consequence of elimination of the geologic and hydrogeologic uncertainties,

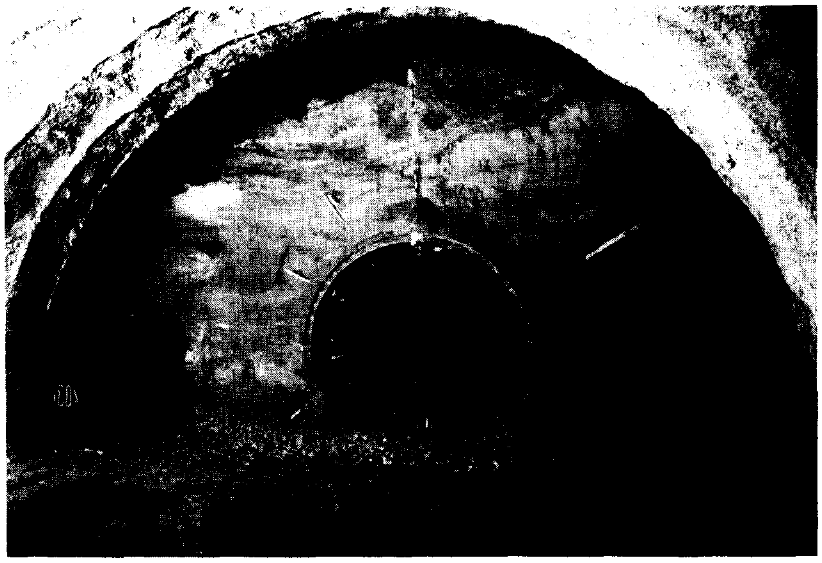

Figure 3. Preventive radial consolidation from TBM pilot bore in soft ground (Giemme 1989).

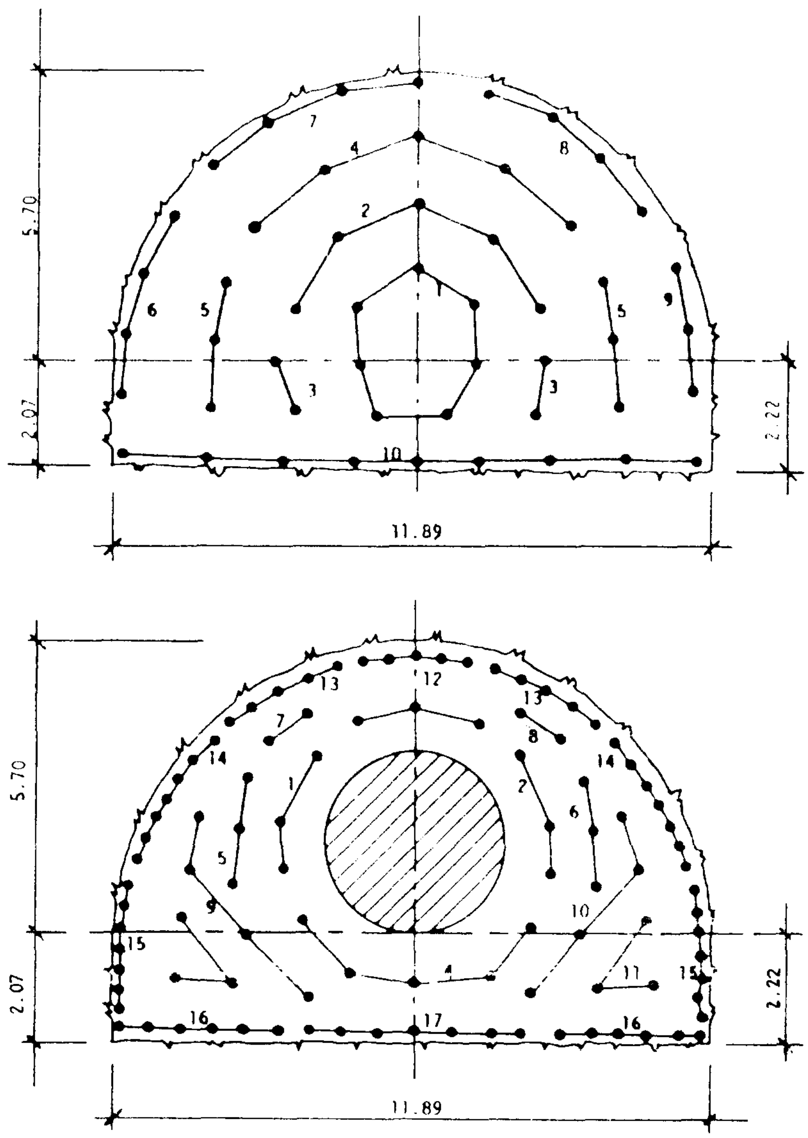

Figure 4. Controlled blast hole pattern. Above, for traditional excavation; below, for pilot tunnel enlargement (Leonardi 1987).

as well as of acquiring precise knowledge of the larger tunnel behavior and knowing its needs for support.

18. Improving the rate and reducing the cost of the larger tunnel construction.

The 18 primary functions of the TBM pilot bore are summarized in Table 2.

Because the pilot bore is generally an exploratory tunnel, its construction essentially is based on preliminary geological data. Therefore, it is necessary to select a type of TBM that will bear the impact of geological surprises while at the same time allowing the maximum excavation speed, in order to minimize the execution time and cost.

In Italy, both open TBMs and TBMs with shields are employed. Although the experience generally has been successful, some failures have occurred for both types of TBM, always due to the very poor properties and behavior of the rock mass. The problem is still unresolved for hard rock: some people maintain that the simpler, open TBM works more favorably because it is equipped for drainage or consolidation operations that may be required to overcome locally unfavorable geological conditions. Others prefer the shielded TBM, which is more complicated and harder to maintain, but more sure in handling.

The costs mainly depend on the effective behavior of the rock mass. Italian experience indicates that a TBM pilot bore represents 15–20% of the cost of the larger tunnel (without special consolidation operations; see Table 3). Nevertheless, it must be considered that the effective cost of the TBM pilot bore is decreased by several advantages related to lower costs in the phase of enlargement:

- Rock volume already excavated by the TBM.
- Reduced specific explosive consumption, because an "opening charge" is not necessary.
- Reduced extent of geological testing and of preliminary investigation.
- Easier dewatering and elimination of natural gas.
- Easier ventilation.

Table 2. Summary of function of main pilot bores excavated by TBM.

Main Pilot Bore Functions for the Design of Larger Tunnels

1. Continuous geological exploration at the horizon of the tunnel.
2. Local geomechanical characterization of the rock mass.
3. Measurements of *in-situ* stresses.
4. Study of the behavior of the bore.
5. Continuous design of the support system.
6. Define piling of excavation waste for further usage or disposal.
7. Planning and scheduling of the larger tunnel.
8. Correct evaluation of the time and cost of enlargement.

Main Pilot Bore Functions for the Construction of Larger Tunnels

9. Providing preventive and continuous drainage of water and gas.
10. Providing preventive improvement of rock mass.
11. Providing preventive de-stressing of high stresses in the rock mass.
12. Facilitating the enlargement excavation.
13. Facilitating better control of vibrations from blasting.
14. Facilitating the ventilation during enlargement (full-length pilot bore).
15. Facilitating intermediate entrances for multi-point excavation of the enlargement.
16. Eliminating geologic surprises.
17. Industrialization of the enlargement excavation.
18. Improving the time and reducing the costs of larger tunnel construction.

Table 3. General indications of cost of TBM pilot bores.

Quality	TBM Pilot Bore (10 sq. m)		Larger Tunnel: Average cost of construction (M lit/m)	Incidence of Costs TBM P.B./L.T. (%)
	Average advance rate (m/day)	Average cost of supported bore (M lit/m)		
Very Good	40	0.9	6	15
Medium	20 to 8	1.8 to 2.8	9 to 12	20
Poor (incohesive soil)	15–20	6.8	40 (Plus 14 for radial preconsolidation in subway)	15 (10 with preconsolidation, which can increase costs to 30%)

Reduction in cost of enlargement excavation after pilot bore: 15%

Current Italian cost index:
- Manpower (miner) 30,000 Lit/h
- Excavation of a large tunnel (average) 50,000 Lit/cu. m

- Better organization of the work in the phase of enlargement.

All of these lower costs can reduce the fractional cost of the TBM pilot bore to 6–7% of the total cost of the larger tunnel (Casale 1987). To these lower, direct costs must be added the following indirect benefits:

- Better management of contracts.
- Absence of work interruptions due to lack of geological surprises.
- Shorter global execution time.

With regard to this final feature, we must remember that the execution time for the TBM pilot bore, with just a small area needed, for a 5- to 7-km-long tunnel is the same as that which would be needed for the general works to prepare the excavation of the larger tunnel (area availability, track and road construction, auxiliary plants and service, adit preparation, etc.).

The decision of whether or not to use of a pilot bore with TBM is not easily made. It is, of course, always necessary to define the purposes of the pilot bore and then to make the decision about how and where it should be executed. In good rock, the TBM pilot bore is executed very fast and at low cost.

During the enlargement, the presence of the pilot bore permits the maximum depth per round to be achieved when explosives are used. Thus, it is possible to obtain remarkable advantages in both time and cost. For example, in one tunnel in good rock, an advance rate for the enlargement from 16 to 25 m/day was realized, saving costs of up to 50% (Ceppi 1986). On the other hand, when the rock is poor, the advance rate of a TBM is generally low and the cost of pilot bore is high.

The enlargement excavation is difficult and slow. It would also be difficult without a pilot bore because the advance rate remains low and the direct advantages in terms of the costs are limited.

Of course, a pilot bore eliminates the geological surprises so that the enlargement excavation can be planned exactly, together with preventive improvement of the rock mass where necessary. □

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