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GROUND PROBING AND TREATMENTS IN ROCK TBM TUNNEL TO OVERCOME LIMITING CONDITIONS

Daniele Peila and Sebastiano Pelizza
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The negative consequences on TBM performances caused by an insufficiently complete and detailed "geo" characterization of the rock masses that have to be bored are discussed in the paper taking into account the possible ground probing operations in a TBM bored tunnel and the ground treatment techniques to be carried out ahead of the face that can be applied to make the construction of a bored tunnel feasible when limiting or prohibitive conditions for a TBM, must be faced. A description of some relevant and inspiring projects of TBM bored tunnels in rock where ground probing and treatments were used are also presented and discussed.tic approach and concept of physical volume of an inhomogenous medium. The numerical experiments on representative structures of rock masses have shown a good correspondence between the results obtained with the accurate and equivalent models

Rock, TBM, mechanized tunnelling, rock mechanics, probing

INTRODUCTION

Tunnel design and construction requires a complex set of decisions, which should be taken by the Owner, the Designer and the Construction Company that are conditioned by many factors both technical and economical and can vary in time in function of the preliminary knowledge and the development of the various design stages and construction [1 – 3]. The experience gained from various projects and the evolution of the methods and criteria for investigations, design and construction allow the following considerations to be made:

- The construction of tunnels in the past was based, more than any other construction method, on observational methods because the most important aspect in tunnelling is the changes in ground conditions;
- The observational methods are based on "experience" and they allow the design to be adapted to the geological-geotechnical conditions that are encountered during the excavation. These flexibility and adaptability in themselves go against the guarantee of the prevision of construction times and costs [4 – 6];
- The always increasing improvements in the feasibility of investigation techniques and elaboration of the preliminary "geo" data allow data to be obtained that are consistent and feasible. We should not, however, forget that the preliminary, complete and deterministic knowledge of the geological-geotechnical tunnel profile and the geotechnical data are but "a dream" in many cases and in particular in long and deep tunnels;
- The design technical and structural should, from necessity, evaluate the influence of the construction method which should be necessary based on the optimization of the various alternatives, since the instability aspects are influenced by the excavation operations (mechanized or traditional), by the types of supports , by the ground reinforcements and by the sequence of operations [7 – 11];

Politecnico di TorinoE-mail: daniele.peila@polito.it, Torino, Italy. Translated from Fiziko-Tekhnicheskie Problemy Razrabotki Poleznykh Iskopaemykh, No. 6, pp. 100-116, November-December, 2009. Original article submitted August 5, 2009.

• Full face TBM excavation is undergoing great development throughout the world and this sector is moving towards the use of machines:

— larger and more powerful, easier to install, disassemble and transport;

— able to operate in "mixed face" conditions or able to operate in different geological conditions;

— able to operate in tunnels with sections of different sizes;

— able to apply increasingly greater pressure to the face;

• Mechanised excavations require specific surveying, investigations and studies which must be precisely designed and calibrated due to the larger "rigidity" of the tunnel-machine system than with conventional tunnelling [7, 13 – 16];

— tunnel construction reality has always been of a not deterministic type due to the geological, hydrogeological, geomechanical uncertainties [8, 17 – 20].

The Designer must therefore find the best solution among the various construction hypothesis, each of which has different consequences in terms of technical, operational and economical risks [3, 17, 19, 21]. Today it is necessary to evaluate quantitatively using statistical tools and no longer just qualitatively as was normal procedure in the past. Besides, the baselines may not adequately describe the condition to be expected; are often indefinite, too broad, ambiguous or qualitative, resulting in disputes over what was indicated in the contract; may present conditions that are more adverse than indicated by data, or be just plain arbitrary and unrealistic, without discussion or explanations for such apparent discrepancies; the effect of means and methods of construction on ground behaviour are not well described [22].

The above discussed concepts are of great relevance for long and deep tunnels particularly when excavated by a TBM which are today under construction, to cross mountainous areas or relevant straits. In these conditions, very little is in fact known about the detailed geological, hydrogeological and geotechnical conditions that can be encountered: the deeper the tunnel, the greater are the uncertainties, the higher are the probability of encountering unforeseen and adverse conditions for tunnelling, the greater are the efforts and the costs for site investigations to reduce the uncertainties. The geological, hydrogeological and geotechnical local conditions could become unpredictable to a reasonable level, due to the extreme difficulties that can effect the drillability of deep boreholes or underground investigations.

The lack of "geo" knowledge is one of the most critical points for rock TBM tunnelling since the tunnel has to be excavated without precise knowledge of the type of ground that will be encountered and the various situations the excavation will have to deal with.

When using a rock TBM it is necessary to remember that its standard performance can be evaluated on the basis of rock mass properties [23]. A "limiting situation" for a TBM is when and where a machine cannot work in the way for which it was designed and manufactured, and the advance is significantly slowed down or even obstructed [14, 24 – 26]. A geological situation is therefore not "limiting" in an absolute sense, but only in relation to the type of the TBM being used, its design and special characteristics, and to possible operating errors. The most important and frequent limiting conditions that should be considered are: borability limits; instability of the excavation walls; tunnel face instability; fault zones or squeezing ground; strong inflow of groundwater; clay soil; occurrence of gas; rocks and water at high temperatures and karstic cavities [14].

In order to prevent these limiting conditions, escape a "TBM trap" and for the safe construction of a tunnel under "geo" conditions of great uncertainty, the following main requirements must be taken into account [27]

— systematic investigations ahead of the tunnel face for the preventive identification of water bearing geological structures and/or unstable or difficult geotechnical conditions (such as squeezing or swelling ground). These investigations could be carried out starting from the tunnel section or from lateral niches, excavated in order to permit long borehole drilling;

— pre-ground treatment, with the purpose of sealing the water around the tunnel, depressing the pore pressure, or improving the geotechnical characteristics of the ground, with the aim of making the stability of the tunnel compatible with the temporary and final lining. Therefore the TBM should be equipped to perform direct investigations such as mechanical drilling and/or indirect investigations.

GROUND PROBING TECHNIQUES

Ground probing techniques ahead of the tunnel face both direct and indirect (Table 1, [28]) reduce the production. It is therefore necessary to find the optimal balance, using risk analysis techniques, between exploration costs that are mainly defined by lost production and the cost and time lost if the TBM is trapped and it is necessary to free it. This balance is mainly conditioned by the site geology but also by the type of drilling rig installed inside the TBM and its productivity; the possibility of installing a drilling rig so that it is independent of the TBM machine in such a way that it can be able to work while the TBM is working; the possibility of carrying out core drilling during the already foreseen stops, for example for maintenance (the length of the core drilling should be able to cover the length of the tunnel that has been bored by the TBM till the next stop [29]); the possibility of carrying out long core drilling from lateral niches; the choice of reducing the core drillings to the most critical tunnel sectors (these conditions should be known before starting the excavation and the risk of finding an anomalous condition should be considered right from the beginning of the works; the possibility of using geophysical methods which should be able to give information on time, with reference to the advancing speed of the machine; the possibility of managing the obtained data with tools able to update them and with a correct planning.

Direct Investigations

Direct investigations are usually carried out with boreholes ahead of the TBM head, with or without core recovering to investigate the rock mass quality, the position of a weak or critical zone and the presence underground water (Fig. 1). Direct investigations with core drilling ahead of the tunnel face have been carried out in many rock TBM tunnels [30 – 35].

The Piora exploration carried out from 1993 to 1997 did not consist of one single activity but was composed of an 5.5 km long exploration tunnel, driven in fairly good rock with a 5 m diameter TBM, extended accessory works at the end of the gallery, a series of long and deep investigation bores and related tests, and of a vertical 350 m deep shaft, with all the necessary accessory works and the subsequent exploration activities on the future tunnel level.

The critical and most interesting working phase was the approach to the Piora basin which was supposed to be formed by sugar-grained dolomite mixed with water at a pressure of up to 150 bar. This approach was adopted to avoid any uncontrolled encounter with the basin, especially while driving with the TBM. Among the different prediction methods, several geophysical and radar measurements were carried out.

A first campaign of advance bores from the TBM was started at a distance of 1 km from the supposed limit of the Basin and in the last 300 m before the predicted interface with the Piora basin was carried out with advance (80 - 120 m of length) bores from the machine with an overlapping of 11 - 20 m (Fig. 2) which were made just above the TBM crown, with variable inclinations of between 2 and 5º. The drilling installation mounted onto the TBM was equipped with a sophisticated preventer system.

TABLE 1. Ground Probing Ahead of a TBM Head [28]

Direct investigation type	Notes
Boreholes with core recovery	Horizontal boreholes are normally performed through the TBM cutting head; inclined boreholes are normally possible immediately behind the cutting head in open TBM, through the shield in shielded TBM. Radial boreholes are possible in all TBM types through the lining. The objective of boreholes is to: determine the lithological nature of the ground to be excavated through by the TBM; determine the presence of water; determine the presence of voids (karst) and/or decompressed zones. The drilling is realized with a rig positioned behind the TBM cutting head. In the case of shielded TBMs, it is also possible to utilize a "preventer" system to avoid the ingress of groundwater to the tunnel during execution of the drilling. Horizontal and/or inclined boreholes with core-recovery are not commonly used because the time and drilling diameter required.
Boreholes without core recovery	The method of no-core-recovery with registration of the following drilling parameters using a data-logger: drilling rate; pressure on drill bit; pressure of the drilling fluid; torque. It is possible to use either a drilling hammer or a tricone bit. The diameter of the drill hole may be limited to 75mm, whereas the drilling rods may be of the aluminum type in order to reduce potential problems associated with the advance of the TBM later in the case that the drilling rods might be completely lost in the drill hole.
Geostructural mapping of the face and/or of the sidewalls	The mapping must be performed using the same methodologies adopted for the face mapping in tunnels excavated by conventional methods. This type of investigation can be performed only when the TBM stops excavation and thus it can be executed at more or less regular intervals in function of the various construction needs. The mapping involves the collection of all geological, structural and geomechanical data of the soil/rock mass. The purpose of this kind of investigation is: direct characterization and classification of the soil/rock mass; calibration of all construction parameters which may permit indirect characterization of the rock mass.

The equipment was fitted to a water pressure of up to 150 bar, although regular hydrogeological tests showed real ground water pressures of between 70 and 100 bar.

In the Bodio section of the Gotthar-Base tunnel, the excavation is carried out in generally fairly good rock conditions. Nevertheless, after a first investigation phase based on seismic explorations completed with selected roto-percussion drillings, it was considered advisable to cover the whole length with advanced mechanical probing, in order to be prepared for the several forecasted but locally unknown faults, or groups of faults of modest to medium geotechnical importance. The drilling equipment is mounted onto the TBM and the bore hole is set in the TBM crown usually with 5° slope without preventer.

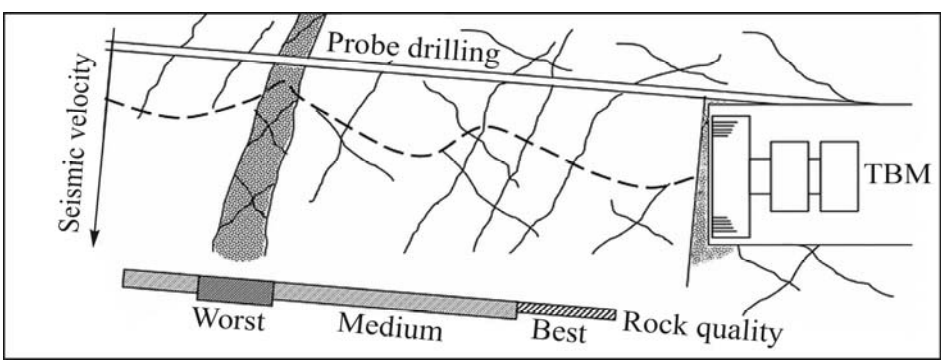

Fig. 1. Example of probe drilling and seismic velocity logging for preliminary rock class estimation [23]

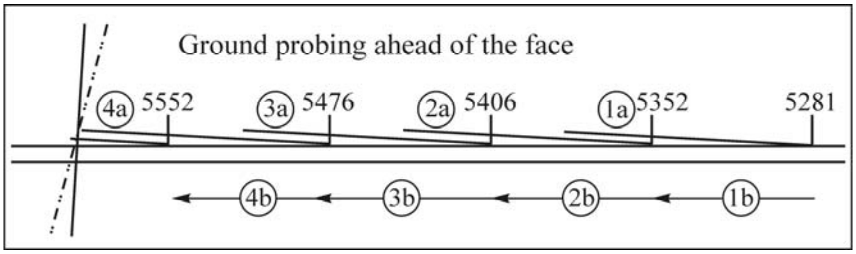

Fig. 2. Scheme of the investigation while approaching to the Piora syncline [33]. The figure illustrates the length of the various exploration drilling ahead of the tunnel face (1a – 4a) and the advancement steps of the machine (1b – 4b)

The length of the probing is decided according to the geological situation: it is usually 80÷100 m, which is compatible to the weekly advancement rate of the machine. The drillings are usually carried out during the maintenance shift and do not cause any delay of the construction program.

Indirect Investigations

The indirect investigations with geophysical methods ahead of the tunnel face can be subdivided into electrical/electromagnetic, sonic and seismic [36 – 38] (Table 2).

The Bore-Tunnelling Electrical Ahead Monitoring (BEAM) is based on the induced polarization method using the head of the TBM as an electrode and it allows the rock mass fractures to be found. This method was used for the probing in the excavation of the small diameter Ginori Tunnel, which is the safety tunnel of the Vaglia Tunnel of the high speed railway tunnel between Bologna and Florence (Italy) [39].

Various forms of sonic and seismic loggings performed in Japanese tunnels were described in [35 – 42]. These authors logged the velocities in the disturbed zone around a small diameter (2.6 m) TBM headrace tunnel for a hydroelectric project [43].

The most frequently used seismic methods today are: TSP 203 (Tunnel Seismic Prediction) and TRT (Tunnel Reflection Tomography). These allow evaluating the quality of the rock mass and the presence of faults. Sonic measurements are usually applied in the SSP method that investigates the difference of density of the rock mass using sonic waves. This system does not interfere with the excavation process and it was applied in front of a Slurry machine in the ground for the Elba tunnel.

GROUND TREATMENTS

The ground treatments that can be applied in a TBM excavated tunnel can be subdivided into methods which control the instability of the excavation walls; the water income; the tunnel face instability; the stability of local portion of the rock mass (Table 3).

TABLE 2. Geophysical Methods Used to Investigate Ahead of the Face of a Tunnel [36]

Method	Principle	Penetration ahead of the face	Interference with the excavation procedure	Easiness of data evaluation
BEAM	Electromagnetic	2.5 – 4 times the tunnel diameter	Nil	Medium
TSP-203	Seismic	10 – 20 times the tunnel diameter	High	Complex
TRT	Seismic	5 – 15 times the tunnel diameter	Medium	Complex
SSP	Sonic	30 times the tunnel diameter	Nil	Complex

TABLE 3. Summery of Ground Treatment Techniques in Rock TBM Tunnelling

Technology		Control of the instability of the excavation walls	Control of the water income	Control of the tunnel face instability	Control of local unstable portions of the rock mass
Rock mass improvement	Grouting	Intervention around the tunnel ahead of and behind the face	Intervention around the tunnel ahead of and behind the face And inside the core	Intervention around the tunnel ahead of and behind the face And inside the core	—
	Freezing	Intervention around the tunnel ahead of the face And inside the core	Intervention around the tunnel ahead of and behind the face And inside the core	Intervention around the tunnel ahead of and behind the face And inside the core	—
Rock mass reinforcement	Bolting with steel elements	Radial intervention behind the face	—	—	Spot or systematic interventions
	Bolting with fiber glass pipes	—	—	Intervention from nearby tunnels, from pilot bore or from the surface	—
Presupport	Forepoling or steel pipe umbrella	Intervention around the tunnel ahead of the face	—	Intervention around the tunnel ahead of the face	—
	Jet grouting columns	Intervention around the tunnel ahead of the face	—	Intervention around the tunnel ahead of the face	—
Drainage	Drains	—	Intervention around the tunnel ahead of and behind the face	—	—

The basic principle of pre-ground treatment is to use the ground surrounding the tunnel as a load bearing ring and make it an integral part of the primary support and lining. This concept can be reached by creating a tight ground ring around the lining so that the hydrostatic pressure in the outer permeable ground would act on the grouted ring instead of on the lining. The most used techniques are shortly described below.

Ground Freezing. The technique of freezing the water saturated ground is frequently used in tunnelling, generally associated with shallow tunnels, where it is possible to work from the surface. There are some cases of application for tunnels with depths of up to 80 m in Russia, but serious problems were encountered during excavation, due to the occurrence of strong water inflows through defects in the frozen ground. Freezing can be achieved by using either a large portable refrigeration plant or liquid nitrogen. The results of the treatment are influenced by the steering of the drill hole and the length of treatment should be kept short. This technique also requires the placing of the final lining before de-freezing of the ground.

Jet-Grouting. Jet-grouting technique is best applied in cohesionless soils, where the energy from high-pressure grouting breaks down the soil matrix efficiently and replaces it with a mixture of the grout mix and soil. The overall dimensions of a jet-grouting plant compared with the constrained space inside the shield, the need of cleaning of the mix flowing on the TBM head and the risk of cementation of the shield make this technique very rarely used in rock TBM tunnelling.

Drainage. Drainage ahead of the face is frequently used in TBM tunnelling (often associated with grouting) to cross water-bearing zones in order to reduce the water pressure. The technique of long drainage ahead of the face allows the hydrostatic pressures to be reduced in advance, and thus significantly improve the stability of the crown and the face [44].

Steel Pipe Umbrella or Forepoling. Steel pipe umbrella is obtained by installing steel pipes ahead of the tunnel face with a dip of 5 - 10° (with reference to the horizontal) in such a way as to form an umbrella with a truncated cone shape up to 15 length [45]. It is also possible to use shorter steel elements, usually self drilling bolts or bars, which are installed with a larger dip than the steel pipes.

Grouting. The grouting of soil or rock masses with cement slurries or chemical mixtures to improve their mechanical and hydraulic properties is a well-established practice in engineering [46]. Grouting standards and procedures for rock masses are well established [47]. Very interesting developments concerning the application of dam grouting techniques to tunnelling are at present being investigated [48]. Pressure grouting (or injection) in rock is carried out by drilling boreholes of a suitable diameter, length and direction into the rock material, placing packers near the borehole opening, connecting the ground conveying hose or pipe between a pump and the packer and pumping a prepared grout into the cracks and joints of the rock surrounding the boreholes. Two fundamentally different approaches may be applied in tunnelling grouting:

1) Pre-grouting: holes are drilled from the excavation face into the rock mass ahead of the tunnel face and the grout mix is injected into the ground from the advancing tunnel, from the surface or from an existing nearby tunnel or from lateral niches or drifts (Fig. 3). In all the cases the grouting must be located in the portions of the rock mass where the water is quiet to avoid that the moving water could wash out the mix;

2) Post-grouting: drilling for ground holes and injection take place along the excavated part of the tunnel mainly with a radial shape.

USE OF THE VARIOUS GROUND TREATMENT TO OVERCOME LIMITING CONDITIONS

In the following chapters the most important limiting conditions which can be encountered during the TBM excavation and the various ground treatments which can be used to overcome them are analyzed and discussed.

Instability of Excavation Walls

The instability of excavation walls is a limiting characteristic for open type rock TBMs. The problem manifests when the instability phenomena occur immediately behind the support of the cutterhead, making both the installation of the supports and the correct positioning of the grippers difficult [25, 26].

The consequences of the instabilities on the production and on the methods employed to overcome the instabilities vary enormously in function of: the magnitude and the type of the instability phenomena; the type of the TBM used (a simple or a double system of grippers); the design and characteristics of the TBM; the tunnel size; the system installed inside the TBM for the installation of the tunnel supports, and the type of supports itself.

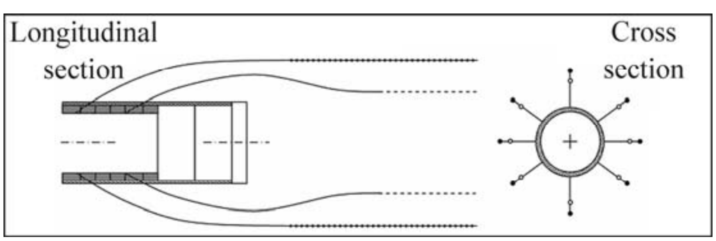

Fig. 3. Boring layout from the back of the shield\ using ahead directional drilling techniques: *G* — grout holes; *D* — drain holes [43]

Single or double shielded rock TBMs are not very sensitive to the instability phenomena of the excavation walls since it is possible to install a precast-concrete or steel lining inside and under the protection of the shield. It is thus possible for the TBMs to advance by pushing against the lining.

In the case of medium to large diameter tunnels (from 6 to 12 m) the difference in the behavior and productivity between open and shielded TBMs, under excavation wall instability conditions, increases considerably with the shielded TBM being more advantaged.

With open TBMs, the possibility of counteracting against the instability phenomena effectively at the excavation walls depends on the following interventions: 1) stabilization and reconstruction of the walls, carried out immediately behind the support of the cutting head using steel arches, wood lagging and shotcrete. The installation of these supporting elements, particularly the shotcrete, in this delicate zone of the machine, requires a long time and there is also the risk of the excavation equipment being damaged; 2) traditional excavation ahead of the TBM head, often top heading; 3) pre-treatment ahead of the tunnel face with injection at low pressure; 4) forepoling or steel pipe umbrella.

Instability of the Tunnel Face

If the fracturing and/or alteration or the rock mass is such that important instabilities of the excavation face can occur, with the detachment of rock, there is the risk that the machine could become entrapped, blocking the head, or that the over-excavation created by the slide reaches such dimensions that it can no longer be controlled. This problem involves all types of TBMs (open or shielded). In this case the most frequently used reinforcements are as follows.

Preventive Injections of the Fractured Portions of the Rock Mass with Mixtures of Cement and Resins

With this intervention it is necessary to find a design equilibrium between the possibility of intensive reinforcement, before advancement but with the consequent high costs and long times, and the use of limited interventions for which it is however difficult to define the acceptable lower limit. In this latter case serious consequences can in fact occur whenever the reinforcement has not been carried out properly and the problem of stability again appears [15]. In the Kunming Zhangjiuhe Water Diversion and Water Supply Project [33], the excavation of the Shanggongshan tunnel was carried out using a Robbins Double Shield TMB with a diameter of 3.65 m and an overall cutterhead power of 1300 kW. Unfortunately, the actual geomechanical conditions encountered were different than those forecast. There were 24 faults encountered along the first 4000 km of the tunnel for a total thickness of approximately of 402 m. The TBM advancing rate when the faults were encountered, was very slow due to the collapses and excavation face instability often in conjunction with significant water inflow. The advancement of the TBM proved to be very problematic in alignment sections where faults or deformed rock exceeded 10 – 15 m. Where these faults were encountered, the instability of the excavation face with the consequent formation of chimneys, frequently hampered the smoothness of the excavation.

Another hindrance consisted of the convergences that derived from brittle and plastic deformations which caused an almost instantaneous filling of the gap between the excavation section and the external profile of the shield. These rendered the pea-gravel backfilling nearly unfeasible; the TBM was often trapped. Different types of solutions have been implemented, depending on the kind of fault that was, on the quantity of water and on the status of the TBM. This ranged from polyurethane foam injection used to consolidate the ground in front of the TBM when faults of up to a few meters of thickness and without any remarkable presence of water were crossed, to the excavation by traditional method of a room all around the TBM and ahead of it for several meters when the TBM was stacked in squeezing ground. Rock grouting ahead of the face has proven to be ineffective and very problematical in the schist since the absorption of the grout was, in all the test holes, very low with no significant improvement of the rock characterization.

In the Hsuehshan Tunnel (Taiwan) [13, 49, 50], the tunnel was constructed using three shielded rock TBMs: a 4.8 m diameter Robbins machine for the pilot tunnel and two 11.74 m diameter Wirth machines for the main tunnels. Many problems were encountered in the first 2 – 3 km on the eastern tunnel due to the presence of a quartz sandstone rock mass. This sandstone was not correctly investigated in the preliminary phase and, therefore, was faced with good machines which were however inadequate to pass through this zone. The critical rock mass was a very hard quartz sandstone with a compressive strength of up to 350 MPa and a quartz content of 98 %.

Referring to the excavation of the pilot bore it can be highlighted that: the used TBM was an excellent machine able to give high productivity in good rock but it proved unable to pass through the described critical rock mass and in little more than 1 km face collapses have on 10 occasions caused the TBM to jam since the following problematic conditions occurred:

— instability of the face which was difficult to control considering that it was impossible to apply a sufficient high thrust due to the extremely high abrasive properties of the rock that caused a quick wear of the cutter head;

— instability of the tunnel boundary (crown and walls), which made it difficult to grout the annular gap between the lining and the tunnel profile;

— the ground water which was linked with very important reservoirs was increasing the instability conditions with water pressure and large water incomes. In this example it is important to highlight that due to the fact that the machine was not equipped from the beginning for drilling ahead of the face, to investigate and to grout the rock mass in this case it was necessary to use lateral excavations and small tunnel (Fig. 4).

Forepoling or Steel Pipe Umbrella

A metallic pipe umbrella intervention, through holes in the shield of a shielded TBM, to control the instability of the excavation face proved not to be effective in that the holes were usually too distant from each other (obviously due to the presence of the technological structures of the machine) [25, 26].

In the Presenzano hydroelectric tunnel (Caserta, Italy) which has a diameter of 6.65 m and had to pass through a massif made up of blocks due to karsism, reinforcements were performed with steel pipes thanks to the presence of a down-hole hammer on the head of the open TBM.

In the Abdalajis Tunnel (Spain) a prototype for a new type of TBM was used with good performances [51]. The work is a 7.1 km long tunnel with 10 m diameter which is part of the new high speed railway line form Cordoba to Malaga. This machine is defined as a Double Shield Universal (DSU) TBM and it was designed as an evolution of the Double Shield TBM, to cope with rapid squeezing ground. It is able to treat and to stabilize the rock ahead of the face through a combination of piles, special grouting and dewatering. In this tunnel, when the over excavation at the face in the argillite formation increased above a couple of meters in front and above the cutterhead, the TBM advancement was stopped, the void filled with resin foams and the collapsed material in front of the face was consolidated with a chemical grout mix. The TBM was then advanced for a few strokes until the treatment had to be repeated (Fig. 5). In the most critical sections the weak argillite had the behavior of a flowing gravel and it was necessary to install a pattern of fiber glass pipes in front of the machine in order to stabilize the crown of the tunnel. These pipes were installed using through specific holes in the rear shield of the TBM and were grouted with chemical grout mix and this treatment was repeated each 3 – 5 m in the worst tunnel sections.

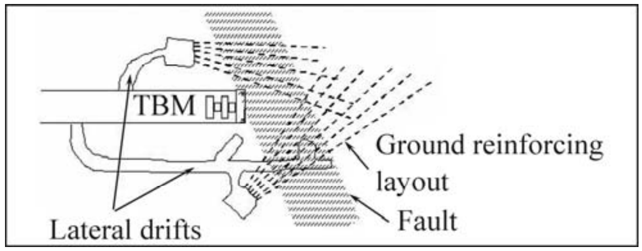

Fig. 4. Example of the ground reinforcing used in the Hsuehshan tunnel at the TBM stop at the chainage 39 km + 079. The grouting boreholes were executed from lateral drifts to treat all the fault rock mass. (Redrawn from Prof. Pelizza personal information)

Injections of the Collapsed Material and Filling of the Voids with Resins of Foams (*Alternative to Grouting*)

In the Javanon tunnel which is part of the Buech hydroelectric plant (France) and which was excavated with an open TBM, an interesting systematic intervention was performed with grouted pipes in advancement injected with polyurethane resins, to solve problems connected both to the stability of the face and the walls and to the control of high water inflows [52]. The ground reinforcements were carried out from an "orange slice shaped" over-excavation around the TBM head and where an important collapse of about 100 m^3 occurred and was stabilized using acrilic phenol expansive foam.

In the case of the raised tunnel excavation for the hydroelectric Power Plant in Maen (Aosta, Italy) ground injections and forepoling were used to pass through a collapse [53]. In this case the TBM was stopped by the collapse of the face and by a "flow" of very altered schist through the opening of the cutterhead. The phenomenon occurred in two phases: initially a fall of completely disconnected rock boulders blocked the cutterhead and then during an attempt to free the TBM, a cavern of several cubic meters formed above the tunnel with the resulting debris resting directly on the cutterhead (Fig. 6).

The remedial measurements were subdivided into several phases: 1) protecting the roof area by means of an umbrella of steel pipes grouted at the pressure of 5 bars; 2) filling of the cavern above the pipe umbrella with expansive resin; 3) consolidation of the debris by means of injections of polyuretanic resin and filling the voids in the debris with cement mix. At the end of the grouting of the debris fiberglass bolts were inserted to nail the blocks immediately in front of the cutterhead; 4) final grouting of the cavern with cement mix.

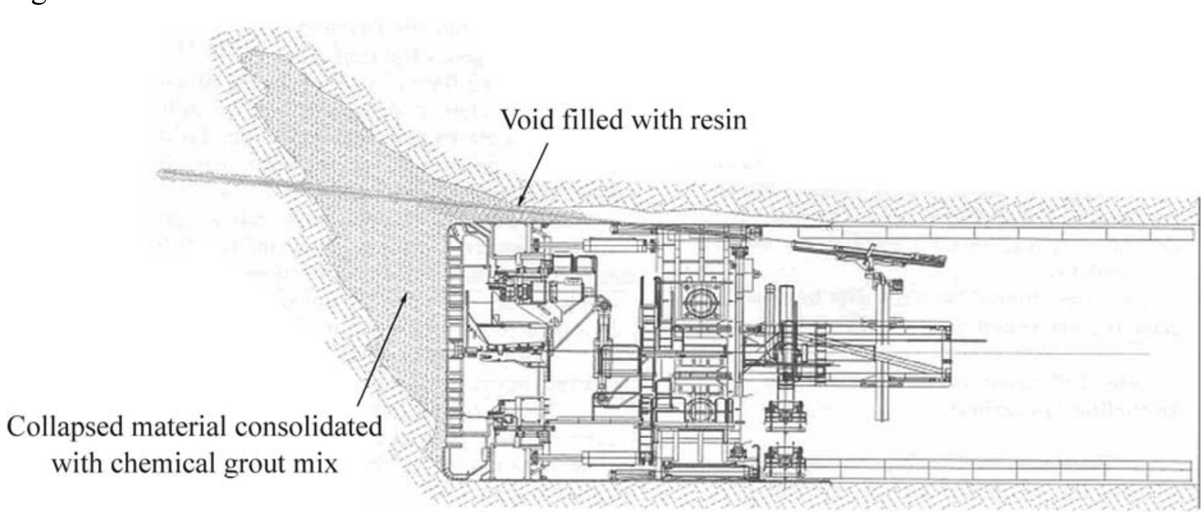

Fig. 5. Face stabilization treatment used in the Abdalajis tunnel (Courtesy SELI S.p.A.)

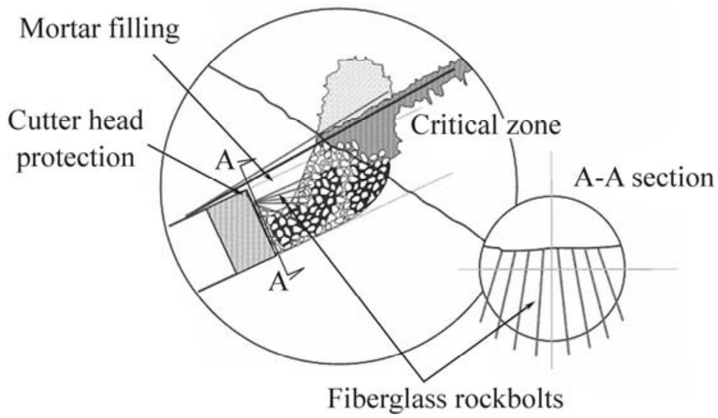

Fig. 6. Ground reinforcements carried out in Maen tunnel (Italy) [52]

In the supply tunnel of the Stariano Enel plant (Catanzaro, Italy), a reinforcement intervention was carried out with pipes with un-injected metallic elements which were installed from a pilot tunnel that was excavated above the TBM machine to surpass a large sized rise in strongly deteriorated granite and with strong water flows [54]. Grouted pipes were not used due to the presence of large quantities of water which tended to wash away the injections and it was decided against the possibility of using columns of reinforced jet grouting because of the typology of the material made up of mylonite with blocks of granite of variable sizes (Fig. 7).

In the Frasnadello road tunnel which was excavated using an 11.8 m diameter shielded TBM, close to the inhabited area of San Pellegrino Terme (Italy) instability developed when going through a thrust zone and the TBM head was blocked [15].

The shielded 11.8 m diameter TBM was used after a 3.90 m diameter pilot bore was excavated inside the tunnel cross section two years in advance of the main tunnel with an open TBM. The instability occurred with a sudden inflow of rock blocks, clay and water into the pilot tunnel. The water percolated through the thrust zone with a flow rate that ranged from 6.6 l/s minimum to a maximum of 10 l/s. In this case the solution was to use a combination of various ground reinforcing techniques and the main working stages were as follows: 1) creation of a consolidated ground arch around the tunnel using resin and silicate injections with drainage holes above performed from the back of the TBM, just behind the shield; 2) creation of a working access chamber (approximately 8 m in length) starting from the pilot tunnel, in order to allow the launching of pipe pipes (length of 22 m) ahead of the tunnel face; 3) anchoring sound rock above the shield with the pipes spaced every 60 cm (to compact the loose zones and to reduce the risk of water percolating during freezing); 4) execution of ground freezing by using liquid nitrogen: a frozen vault was formed with a minimum thickness of 80 cm at the crown and 100 cm at the foot wall; 5) excavation of the access chamber to its full length to reach the TBM head; 6) driving the TBM through the thrust zone.

In the water-supply tunnel of Agri-Sauro (Basilicata, Italy) which was excavated with a 4.08 m diameter shielded TBM, the alignment crossed limes and clays with water reaching a pressure of 8 bars at a depth of about 150 m [54]. After 2600 m of excavation, there was a sudden inrush of water which was also carrying limes for about 2600 m^3. In order to stop the water it was necessary to create a dam inside the tunnel about 100 m behind the face. It was necessary to grout the ground from the surface in order to reach the machine again using cement and chemical mixes integrated with ground freezing, in a conic shape, carried out from inside the tunnel. After that a new TBM in a closed mode was used integrated with a systematic grouting and draining ahead of the tunnel face with a conic shape.

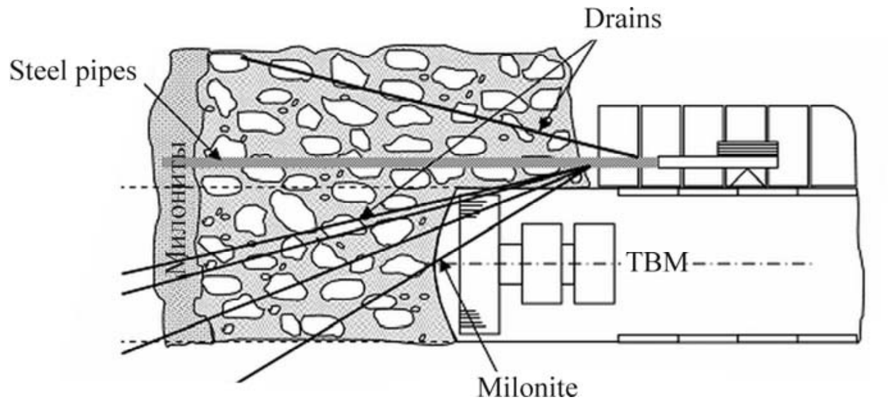

Fig. 7. Scheme of the intervention in the Stariano Enel plant (Catanzaro, Italy)

Strong Water Inflow

A sudden inrush of water can create an adverse internal environment, leading a TBM to the functioning limit when its flow is higher than the natural evacuation capacity of the tunnel and/or the pumping capacity of the dewatering plant but also a sudden water inrush can create an unacceptable impact on the external, surrounding environment since it may lower the groundwater table, cause settlements of the buildings and other surface structures, interfere with lakes and rivers and with the clean water wells. When crossing faults and mylonitised areas, the rock mass is very often completely altered and with water under pressure.

In these situations the instabilities that can occur are: collapse of the face and above portions with the formation of voids and large entity of water flows with transport of the fine elements of the soil. If a condition of this type is encountered by an inadequate machine and without any pre-warning, it is possible that the situation becomes critical and could cause severe delays, increases in costs and, sometimes, in risks for the workers.

Open TBMs are particularly subject to this problem while, with shielded TBMs, even though it might be necessary to interrupt the advancement, it is possible to perform the treatment from the inside of the shield in safe conditions. The technical choices which can be used are limited: draining the rock mass lowering the water table by controlling the water inflow; making the tunnel watertight.

On the basis of many different examples it is possible to say that in the case of high hydrostatic pressures and very difficult geotechnical conditions, or when the basin which feeds the underground water is very important, as is the case of the see, rivers and lakes, the water must be kept outside the tunnel by forming with grouting techniques an impermeable and resistant shell around the tunnel profile and ahead of the face.

This is practically the scheme that was successfully adopted for the Seikan tunnel, where grouting ahead of the tunnel face was used to reduce the water inflow risk during excavation and to strengthen the loose ground around the excavation, aiming at crating a long-term bearing ring and also the solution which was designed by Lombardi (1997) to go through the Piora fault when it was supposed that the tunnel had to face a milled dolomite and the high water pressure [55, 56]. A similar solution was also used to go through the important fault of Valle Fredda crossed by the Gran Sasso Tunnel (Italy) with a water pressure till 60 bar coupled with weak rock mass. Water-tightness can be achieved by the use of probe-drilling ahead of the face followed by pre-grouting of the rock mass. The primary purpose of a pre-grouting scheme is to establish an impervious zone around the tunnel periphery by reducing the permeability of the most conductive fractures in the rock mass and also improving the rock mass stability.

Due to its importance pre-grouting must be efficient and correctly carried out: during the TBM advancement no un-grouted joint should be encountered and the coverage of the grout fan ahead of the TBM face (before a new grouting stage) should be of a sufficient length to ensure that the tunnelling face does not recall water from ahead. As a consequence, the design should be focused on the creation of a continuous grouting curtain around the tunnel and a very good quality control (on the injection procedure, injected quantities, geometry of the perforation etc.) should be set up in the job site.

Probe-drilling and pre-grouting may be performed continuously along the tunnel advance and the length of the ground holes can vary form 15 m to 35 m with an overlap of 6m to 10 m between each grouting round [57]. The use of a systematic grouting scheme (drilling pattern and injection procedure) should also reduce the possibility of an erroneous evaluation of the grouting requirements in the site. It is also important to remark that if the pre-grouting is carried out in a correct way and with a sufficient quantity of grout, the post grouting can be reduced in quantity.

The other possible scheme (to be used in conjunction with pre-grouting) to control and minimise water inflow and post grouting is to use a shielded TBM with pre-cast lining segments and install the lining just behind the face. If an accurate grouting annulus is made with this scheme, it is possible to prevent limited inflow. (Obviously the lining must be designed to be able to stand up to the water pressure) [58, 59].

The problem of water inflow and pre-ground treatment is of great importance when an under sea tunnel has to be excavated. The analysis of relevant examples of probing and the pre-ground treatment selected applied during the design or the construction of some deep tunnels are discussed and analysed with the purpose of defining some basic rationales for the techniques to be applied for new deep tunnels in the following [60].

Japan and Norway undoubtedly hold the record in undersea tunnels constructed over the past 30 years. The Seikan Tunnel [61], which was broken through in 1985, 18 years after the pilot bore excavation was started still represents a milestone in deep tunnelling in terms of length and challenges, through it was excavated with conventional Drill and Blast method. The Seikan tunnel testifies that the art of tunnelling can overcome very serious problems such as seawater flooding into a tunnel. This tunnel was excavated through complex geological volcanic and sedimentary rocks, and very severe problems such as water inflow, ground squeezing and unconsolidated soil, under 170 m of water and 100 m maximum of overburden were resolved. Advance boring and ground treatment played a key role in completing the excavation. Systematic pre-grouting was performed in particular at the face of the tunnel (or from a side-drift that preceded the excavation) along difficult zones with the aim of creating a water-seal ahead of the tunnel and improving the geotechnical characteristics of the ground in the short and the long term [61 – 64].

The grouting procedures followed the evolution of the grouting agents: grouting started using Portland cement, that was rapidly changed to Portland blast furnace slag cement and water glass admixtures which responded to the need for sealing off the high-pressure seepage water and of improving the soft rock properties. In order to respond to these requirements, the grouting materials had to be characterized by high strength, excellent permeability in the ground and high durability.

Several other undersea tunnels were constructed in Japan through soft soils at shallow depths, using mechanized shields, from the late sixties onwards, mainly without the need of pre-treating the ground.

In Norway, about 30 undersea tunnels have been constructed in the last 20 years. Practically all of these have been excavated by D&B, mainly in hard Precambrian bedrock, and only in few cases less competent Paleozoic rocks have been traversed (e.g. the North Cape Tunnel, 6.8 km long). Nevertheless, difficult excavation conditions have been experienced in some sections of several of these tunnels that were generally related to limited extent faults or weakness zones of heavily crushed rock and gouge.

TABLE 4. Influence of Pre-Grouting on Rock Mass Properties [66]

Rock mass property	Effect of pre-grouting	Example of increment or decrement induced by pre-gouting, %
RQD	Increases	30 – 50
Jn	Reduces	9 – 6
Jr	Increases – (Probable effect)	1 – 2
Ja	Reduces – (Probable effect)	2 – 1
Jw	Increases	0.5 – 1.0
SRF	Reduces	2.5 – 1.0

The authors [65] reported that the ground treatment, which consisted in grouting and in one case of ground freezing, was applied in ten of the eleven recorded cases. This significant experience of pre-grouting in the Scandinavian geo-environment permitted to evaluate the effect of grouting on rock mass quality through an improvement of the Q Index parameters (Table 4) [66, 67].

It appears quite clear, both from the Seikan and Norwegian experience, that pre-grouting is more than just a water control method but it allows to achieve an improvement of the rock mass [68].

Subsea projects in other countries where pre-ground treatments were used include the Strategic Sewage Disposal Scheme in Hong Kong (Fig. 8) and a couple of undersea galleries in the Paluel and Flamanville nuclear plants in France [69]. In the first case, advance probing and pre-grouting were used to seal off water leakage from the saturated man-made embankments laying on the seabed that are now densely urbanized while in the French tunnels localized pre-grouting was performed after important water and sand inflows into the tunnels .

Chemical grouts, such as waterglass and acrylates, have very high penetrability compared to the Portland based grouts, and can therefore ensure sufficient sealing (also as far as gas is concerned, as documented by Balossi Restelli [70] for the Capo Calavà tunnel in Sicily, Italy) and improve the properties of the rock mass without the necessity of high grouting pressure. Nevertheless, chemical grouts are associated with lower elastoplastic properties and low strength, so the effective rock mass improvement is not very significant.

A good compromise between penetrability and final strength is offered by microcements based grouts, which also offer good durability and no toxicity.

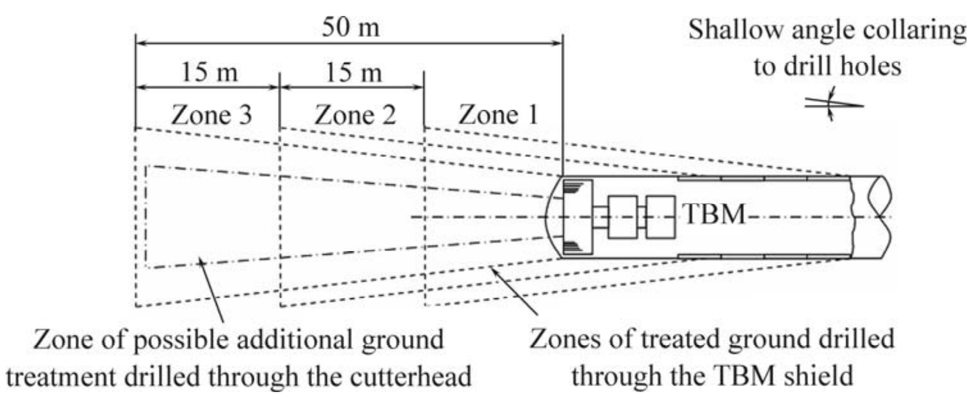

Fig. 8. Grouting ahead the TBM in the Strategic Sewage Disposal Scheme in Hong Kong (Redrawn from Prof. Pelizza personal information)

CONCLUSIONS

Probing, pre-ground treatments and drainage are the most important methods of making the crossing of unpredicted, difficult geotechnical conditions by a rock TBM tunnel feasible. Therefore these operations should be considered as an integral part of the tunnel advancement technique already at the design stage both in the time evaluation and in the structural design since the possibility of an easy ground treatment of the core can "turn the light from red to green" as far as a rock TBM tunnel is concerned.

The manufacturers of TBMs are also given the difficult task of studying machines capable to apply increasingly higher counter-pressure to the face, in order to reduce the necessity of pre-grouting, at least ahead of the tunnel face, as well as to reduce the impact on the tunnel advancing rate of the pre-ground treatment. These important technological improvements must first of all be encouraged and supported with an adequate financial aid by the Employers as their main interest is that a tunnel should be constructed in the safest possible way and in the shortest possible time and, as a consequence, with the lowest possible costs.

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