3D simulation of mechanized excavation in squeezing ground along the Lyon-Turin Base Tunnel

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Abstract

With the great lengths of deep tunnels as the Alpine Base Tunnels, the worldwide trend in the last decades is towards an increased use of mechanized excavation by Tunnel Boring Machines (TBMs). This is the case of the 57km long Lyon-Turin Base Tunnel between Italy and France where squeezing conditions are anticipated in particular when crossing the Carboniferous Formation as experienced during the excavation of the Saint Martin La Porte access adit. Double Shield TBMs have been proven to be technologically advanced to deal with such conditions. Main features such as over-excavation technology, shield layout (i.e., stepwise conicity), thrust system, and easy access for pre-ground treatment are made available. This paper investigates the complex interaction between the rock mass, the tunnel machine, its system components, and the tunnel support by computational modelling. Emphasis is placed on a complete 3D simulator, recently developed to this end.

Keywords: Lyon-Turin Base Tunnel, Saint Martin La Porte access adit, Squeezing ground, Double Shield Tunnel Boring Machines, Tunnel support, 3D simulator

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<td>$A$</td>
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<td>$F_f^*$</td>
<td>normalized thrust force to overcome friction</td>
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\( F_N \) cutterhead thrust force
\( F_R \) cutterhead rolling force
\( G \) shear modulus
\( K \) bulk modulus
\( K_l \) stiffness of the lining
\( k_n \) elastic normal stiffness of the interface element
\( k_s \) elastic shear stiffness of the interface element
\( K_s \) stiffness of the shield
\( k_r \) elastic shear stiffness of the interface element
\( L \) shield length
\( N \) number of elements in the shield surface
\( n \) number of cutters
\( P \) cutting power
\( p \) ground pressure
\( \overline{p} \) cutterhead pressure
\( r \) rear shield radius
\( R \) excavation (tunnel) radius
\( RPM \) cutterhead rotational speed
\( t \) time
\( T_{\text{tot}} \) total torque
\( T_f \) torque to overcome friction
\( T_r \) torque due to the rolling forces
\( u \) final radial displacement
\( u_{\text{crown}} \) radial displacement at the crown
\( u_f \) displacement at the face
\( u_{\text{invert}} \) radial displacement at the invert
\( u_r \) radial displacement
\( UCS \) uniaxial compressive strength
\( w \) interspace between the rear shield and the segmental lining
\( \Delta D \) overcut
\( \Delta g \) gap size between the shields and the rock mass
\( \Delta r \) conicity
\( \Delta R \) radial gap size
\( \Delta z_{\text{min}} \) smallest width of an adjoining zone in the normal direction
\( \gamma \) unit weight; partial factor for the concrete for the ultimate limit state
\( \psi \) hardening/softening parameter
\( \varepsilon_{\text{pi}} \) principal plastic strain
\( \varepsilon_{\text{pl}} \) equivalent plastic strain
\( \mu \) friction coefficient
\( \nu \) Poisson’s ratio
\( \sigma \) stress
\( \sigma_0 \) initial stress
\( \sigma_c \) normal stress in the segmental lining
\( \sigma_{\text{cm}} \) rock mass strength
\( \sigma_n \) normal stress
\( \beta \) surface reduction coefficient
\( \tau \) shear stress
\( \varphi \) friction angle (Mohr-Coulomb)
\( \psi \) dilation angle (Mohr-Coulomb)
1. Introduction

A very significant length of the 57km long Lyon-Turin Base Tunnel between Italy and France is to be excavated by TBMs, as these may contribute significantly to savings in construction times and costs. However, during the excavation by the conventional method of the Saint Martin La Porte access adit in the Carboniferous Formation, a highly heterogeneous, overstressed and in cases anisotropic rock mass exhibiting a squeezing behaviour (Barla et al., 2010; Bonini and Barla, 2011) was encountered. Use of TBMs in squeezing ground conditions is yet under discussion due to the negative experiences which resulted in cases in very low rates of advancement and even in standstill with complete loss of the TBM. A comprehensive study in the adit was carried out over a period of five years (i.e., from January 2006 to December 2010) from chainage 1325 until the end of the excavation (Barla et al., 2010; Bonini and Barla, 2011). The available data are of great value in view of the decision to be taken in terms of feasibility and risk for the mechanized excavation of the Base Tunnel through the length where the Carboniferous Formation is to be met.

Double Shield TBMs have been proven to be technologically advanced to deal with such conditions. Main features such as overexcavation system, shield layout (i.e., stepwise conicity), thrust force and torque, and easy access for pre-ground treatment are made available. It is to be recognized however that at the design stage methods are needed to deal effectively with the complex interaction between the rock mass, the tunnel machine, its system components, and the tunnel support. A three-dimensional finite element simulator, which considers all the TBM components, has been recently developed by Zhao et al. (2011). This will be described in this paper with special reference to squeezing rock behaviour taking the Lyon-Turin Base Tunnel as a reference case study.

2 TBM tunnelling in squeezing ground

Debate has been going on for years on the applicability of TBMs for tunnelling through the characteristically complicated ground conditions found beneath high mountainous regions. Tunnelling in the Himalayas, the Andes and until recently the Alps, has taken away from machine’s use due to the perceived inflexibility, as a result of the likelihood of them getting trapped by squeezing ground and/or by adverse faults, or irreparably damaged by rock bursts or heavy water inflows. The fact that such conditions pose almost as many challenges for conventional methods as for a modern machine drive often gets ignored (Verman et al., 2012). The hazard scenarios as well as the countermeasures for coping with squeezing conditions with the modern machines are discussed in the following.
2.1 Squeezing and related hazard scenarios

The major hazard scenarios encountered in mechanized tunnelling under squeezing conditions relate to: (i) the significantly intensive convergence in the machine area and (ii) the highly time-dependent displacement either at a certain distance behind the face or during standstill of TBM drive.

In the first case, tunnelling experiences as well as theoretical considerations indicate that rapidly squeezing ground usually closes the gap along the shield and develops heavy loads on it as well as on the lining. When the available thrust cannot overcome skin friction, jamming of the shield occurs. Furthermore, the full use of the installed thrust force may induce additional damage in the segmental lining (mainly in the circumferential joint), besides the overstressing by the rapid ground pressure. In addition, the significant deformations (ovalization) or even horizontal or vertical shifting of the tunnel support probably lead to the jamming of the back-up equipment and inadmissible convergences of the bored profile (Ramoni et al., 2011). In case of rapid core squeezing, when the penetration rate is less than the advance rate, large ground pressures act on the cutterhead plate and may cause sticking of the cutterhead when the torque is not sufficient to overcome friction.

In the second case, the rheological behaviour of the ground would cause overstressing of the lining at a certain distance behind the face. In extremely poor ground conditions, it is generally impossible to maintain a fast advance. The time-dependent (long term) deformation would thereby damage the lining at a short distance behind the machine associated with intensive convergences (short term). The worst hazard scenario occurs during machine standstill. Serious examples are the Yacambu-Quibor Tunnel (Venezuela, gripper TBM, $D = 4.80$ m) (Hoek and Guevara, 2009) and the Gilgel Gibe II Tunnel (Ethiopia, Double Shield TBM, $D = 6.98$ m) (Barla, 2010).

2.2 Tunnel Boring Machines and countermeasures to cope with squeezing

2.2.1 Machine types available

Gripper TBM with conventional adjustable roof (e.g., canopy or cutterhead shield) has great flexibility, and is less likely to get trapped when across squeezing ground. However, as the squeezing potential increases, the limitations of the machines become more significant, particularly due to the instability of the tunnel walls (Barla and Pelizza, 2000). The geometrical constraints in the area of the machine behind the cutterhead limit the possible thickness of the primary support and the admissible convergence for the back-up equipment (Ramoni and Anagnostou, 2010a). For example,
the Faido section of Gotthard Base Tunnel (Switzerland, gripper TBM, \(D = 9.43\text{m}\)), 50 % of total displacement occurred behind the finger shield in place, which is about 5m behind the face. It led to the destruction of the primary support and the jamming of tunnelling equipment. Furthermore, when the rock is very weak, the tunnel wall strength may even not be sufficient for the bracing of the grippers. In this situation, a gripper TBM is not able to operate.

**Single shield TBM** is longer than gripper machines. The bigger length increases the risk of getting trapped in squeezing ground. Single shield TBM is jacked against the segmental lining. The possible thrust force and torque depend not only on the design of the machine (installed thrust force and torque) but also on the structural design of the segmental lining and the quality of annulus grouting. Furthermore, the boring process is not continuous since a new ring of segments should be previously erected.

**Double Shield (DS) TBM** has come into fairly common use in long large diameter tunnels. The capability of the Double Shield TBM design was clearly demonstrated at the end of the 1980s when it was used in the Channel Tunnel. Variable rock formations can be bored, ranging from very hard quartzite and granite to shale, phyllite and fault zones with incompetent and squeezing ground. In good to medium quality rocks, as the DS TBM performs both the boring and the final lining of the tunnel at the same time (double shield mode), it can achieve better potential rates than gripper or single shield TBMs. Under squeezing conditions, it may be difficult to clamp radially through the grippers, so the necessary thrust forces can either be provided by the telescopic cylinders (stationary gripper shield) or by the auxiliary thrust cylinders. In this second mode (single shield mode), the machine will operate as a single shield TBM.

The traditional DS TBM has constraints and negative effects when extremely adverse ground conditions are encountered. It is sensitive to face instabilities. For example, in the Evinos-Mornos tunnel, severe squeezing ground was overcome without major stoppages, but the collapsing of the tunnel face created significant problems (Grandori et al., 1995). The inherently long shield is prone to being trapped in rapidly squeezing ground. Another potential problem is the muck jamming in the telescoping section, particularly at the crown and at the invert. The telescopic shield, with a smaller diameter than the front and the rear shield, favours the accumulation of loose material in this area, thus leading to an increase in the friction that has to be overcome when moving the machine. Last but not least, it is difficult to provide a seal on the telescoping joint, so that the machine is not able to operate under water inflows.

In order to overcome these limitations, the design has been completely reviewed and improved with the introduction of the so called **Double Shield Universal (DSU) TBM**. It was tested the first time by modifying an existing DS TBM which was jammed by the squeezing ground in a fault zone during the excavation of the Val Viola tunnel (Italy, \(D = 3.7\text{m}\)) (Grandori, 2006). Then, this new type of machine was adopted in the Abdalajis tunnel (Spain, \(D = 10\text{m}\)), where critical and variable rock conditions were anticipated. The remarkable achievements indicate that the machine characteristics are
2.2.2 Countermeasures to cope with squeezing

Overboring system

The bored diameter can be temporarily enlarged beyond the normal amount of overcut (e.g., approximately 6cm which is essential for steering) outside the shield by using one to three extendable gauge cutters. This can accommodate a moderate amount of convergence. Firstly, the larger profile leaves more space for ground deformations. It decreases dramatically the rock pressure on the shield (Ramoni and Anagnostou, 2011). Secondly, it increases the time period available before the gap closes. It reduces the contact length, or even completely avoids the occurrence of the contact if the advance rate is sufficiently high.

The amount of overboring depends on the chosen machine system and the specific ground conditions. From the experience of DS TBM, it is seen that the size reaches nearly 40cm (John and Schneider, 2007). However, 30cm is generally suggested as the limitation for overboring (Barla, 2001) and several difficulties in using this technology exist (Ramoni and Anagnostou, 2010a).

In order to realize the overboring, the cutterhead must be elevated so that the invert cut is not below the bottom of the shield resulting in the machine diving. Thus, the offset of the cutterhead axis with respect to the shields is necessary.

Shield layout

The layout of the shields is a decisive criterion for the proper function of a shielded TBM in weak rock conditions. The shield should be as short as possible. It influences skin friction on the shield more than the shield surface. Shorter shield length means that the stress redistribution is not yet developed completely and consequently the possible squeezing forces on the shield (and the risk of getting trapped) will be lower. The quadratic layout (i.e., \( L=D \)) is proven to be favourable for DS TBMs with large diameter (approximately 10m) (Güttter and Romualdi, 2003). This length also includes the available internal length for the installation of the segmental linings which is up to 2.25m wide (Stahn and Grimm, 2006).

The stepwise reduction of the rear shield (standing for inner telescopic, gripper and tail shield) diameter, also called conicity, is a further improvement, as illustrated in Figure 1 (Ramoni and Anagnostou, 2010a). It increases the difference
in diameter between the front and the rear shield and therefore provides more free space for deformation, reducing the risk of the jamming of the rear shield associated with the overboring. Furthermore, during regripping, the front shield sweeps the material accumulated in correspondence of the telescopic shield, which is then displaced to the rear of the TBM without interfering with the operation. Technically, the conicity of DS TBM nowadays is of the order of 3-6cm and up to 10cm in diameter.

The shield should be structurally strong to provide a temporary support to the tunnel wall when deformation capability of overboring and conicity are no longer active.

In probe drilling configuration, the machines are equipped so that an exploration range (40-100m) as well as an action (e.g., ground pre-treatment) range (1.5-2D) can be developed where crossing adverse geological conditions (Vigl et al., 1999). Due to the fact that the TBM has an inherently poor way to the face, special holes are generally set through the shield.

Moreover, one should be able to shift double shield to single shield mode immediately to seal the telescoping joint when unanticipated prevailing poor rock mass conditions are met associated with water or silt inflows.

**Thrust force and torque**

High thrust force and torque are essential to reduce the risks of jamming of the shield and sticking of the cutterhead and to achieve an appropriate advance rate.

Today, the maximum (i.e., auxiliary) thrust force for DS TBMs with a boring diameter of approximately 10m has increased up to 150MN and the maximum torque up to 30MNm (Barla et al., 2011). Furthermore, the temporary installation of removable hydraulic jacks on site would contribute more thrust force if necessary. It should be kept in mind that the jacks work against the segmental lining so that the structural design of the segmental lining and the quality of backfilling constraints the possible thrust force despite the thrust system itself. For gripper TBMs with similar boring diameter, the installed thrust force is about 30MN at most and a torque about 15MNm (Ramoni and Anagnostou, 2010a).

In case that the core squeezing rate is higher than the penetration rate, the cutterhead torque required is not only for rock cutting but for overcoming the rolling resistance of ground friction on the plate. High torque is common on EPB TBMs as they always operate with a face full of material against the cutterhead (Home, 2009). Therefore, rock TBMs are currently designed to extend torque to equivalent level of EPB TBMs, especially for machines with a large diameter.
2.3 Tunnel support

Among the options for tunnel support in squeezing ground, two basic concepts are available (Kovari, 1998): the “resistance approach” and the “yielding approach”. However, the yielding approach has been rarely used for shield-driven tunnels with segmental lining (Schneider and Spiegl, 2008; Ramoni and Anagnostou, 2010a).

2.3.1 Segmental lining

The segmental lining is generally the standard support with shielded TBMs. Lining design for intensive squeezing conditions should basically take into account the combined loading of the high ground pressure (including water pressure in water-bearing layers) and the great thrust force during the construction phase rather than in the utilization stage (Herrenknecht and Bappler, 2003; Caratelli et al., 2010).

In order to withstand these actions, high concrete strength is compelling regarding load transfer in the joints (which are the critical sections from the structural design point of view) and crack propagation caused by the thrust force. Generally, the prefabricated segments can be produced in concrete strength up to C55 (according to Eurocode 2). The equivalent solution is to increase the thickness of the segments. At present is thought to be feasible up to 70cm (Ramoni and Anagnostou, 2010a).

As far as the jack forces, the last lining ring is subjected to point load conditions from the jack rams and load transfer through the circumferential joints. When an extremely high thrust force is needed to keep the machine advancing, steel fibre reinforcement concrete segments (SFRC) may be suitable to limit concrete cracking (by improving the tensile performance considerably) (Caratelli et al., 2010). Moreover, load transfer plates are usually inserted in the circumferential joints.

2.3.2 Support installation (Backfilling)

The prefabricated segments are installed under the protection of the shield. Consequently, a radial gap remains behind the rear shield, which is limited on the inside by the segmental ring and on the outside by the surrounding ground. It is created by overexcavation ($\Delta D$) (i.e., sum of overboring and overcut), conicity ($\Delta r$) of the shield and interspace ($w$) between the rear shield and the segmental lining. Nevertheless, the rapid radial displacement ($u$) would reduce the gap considerably for the intensive squeezing cases. The gap size ($\Delta R$) at the shield tail is the sum of all these factors as follows:
\[ \Delta R = \begin{cases} \Delta D + \Delta r + w - u & \text{if } u < \Delta D + \Delta r \\ w & \text{otherwise} \end{cases} \] (1)

Once the ground closes the gap in the shield area, the shield starts to confine the ground. A circular annular gap with width \( w \) is thereby formed and will be backfilled after the emplacement of the segmental lining.

The instantaneous filling of the radial gap during the TBM advance is an operation of paramount importance for difficult ground conditions. It is essential to activate the support system right behind the machine and to improve the longitudinal arching effect on the shield loading (Ramoni and Anagnostou, 2010a). Furthermore, the material backfilled (e.g., mortar) can penetrate into the surrounding ground and help to roll the uneven surface caused by slabbing and cracking.

Two typical approaches (i.e., grouting via the shield tail and pea gravel with mortar) have been widely used to ensure good embedment of the segmental lining (Herrenknecht and Bappler, 2003). The continuous grouting of either mortar or two-component grout (Peila and Borio., 2011) enables the backfilling to occur right behind the shield. It is noted that the fast hardening material is preferable as the transient (softening) phase may lead to an unloading process of the tunnel wall. Backfilling with pea gravel followed by grouting is performed at a certain distance behind the shield with the consequence that an unsupported span exists as shown in Figure 2.

2.3.3 TBM-Support interaction (Load transfer along the longitudinal direction)

Combining the support of the core ahead of the face, the shield (after closing the gap) and the backfilled segmental lining, three arching actions typically develop between each other, which improve the load transfer in the longitudinal direction significantly.

The action between the core and segmental lining leads to a reduction of the ground deformations in this area and further reduces the ground pressure acting on the shield. Then, the thrust force required for overcoming shield skin friction decreases. Therefore, the stiffer the lining and the shorter its distance from the face, the more pronounced will be the arching effect and the less will be the thrust force required (Ramoni and Anagnostou, 2011). In this respect, backfilling with continuous grouting is more advantageous than pea gravel and mortar with an unsupported span behind the shield (for the machine with a given length). On the other hand, a stiff support with embedment right behind the machine is favourable for the shield but, inevitably, attracts a higher ground load. If the rock mass behind the shield is left free, the ground would experience an unloading process with a smaller load on the lining but also with a stress concentration at the shield tail (Ramoni and Anagnostou, 2011).
2.4 Evaluation of the thrust force

When designing a new TBM and assessing the feasibility of a proposed TBM drive, the knowledge of the frictional force is essential. Ramoni and Anagnostou (2010b) developed dimensionless design nomograms that allow a quick preliminary assessment to be made of the thrust force required in order to overcome shield skin friction and avoid jamming of the shield. These nomograms have been developed on the basis of finite element modelling in axisymmetric condition. The excavation of a deep cylindrical tunnel through a homogeneous and isotropic ground subjected to a uniform and hydrostatic initial stress has been considered.

The required thrust force $F_f$ generally depends on: the material constants of the ground (Young’s modulus $E$, Poisson’s ratio $\nu$, uniaxial compressive strength $UCS$, friction angle $\phi$ and dilatancy angle $\psi$), the initial stress $\sigma_0$, the characteristics of the TBM (tunnel radius $R$, radial gap size $\Delta R$, shield length $L$, and shield stiffness $K_s$), the skin friction coefficient $\mu$ and the stiffness of the lining $K_l$. By fixing the values of some of these parameters, the normalized required thrust force $F_f^*$ is shown to depend on the four following parameters:

$$F_f^* = \frac{F_f}{\mu 2 \pi R L \sigma_0} = f\left( \frac{E}{\sigma_0}, \frac{\Delta R}{R}, \frac{UCS}{\sigma_0}, \phi, \frac{L}{R} \right)$$

(2)

3 Project description

3.1 Base tunnel: general features

The new Lyon-Turin rail connection (a priority project for the European Union) lies at the intersection between the North-South and East-West axes of communication, helping to complete the European rail network. It will be the key link of the Mediterranean Corridor n°3 connecting South of Spain (Algeciras) to the Hungary-Ukrainian border (European Commission, 2011). The line is divided in three sections: a French section from the urban area of Lyon to Saint-Jean-de-Maurienne; a joint Franco-Italian section from Saint-Jean-de-Maurienne to Chiusa San Michele in Susa Valley; and an Italian section from Chiusa San Michele to the urban area of Turin.

The Base Tunnel will be 57km long (from Saint-Jean-de-Maurienne to Susa, where “international” train stations will be built) and will comprise two parallel tubes connected by cross passages at 333m intervals. The maximum overburden is 2500m and a length of more than 10km will be excavated at 2000m of depth.

Along the French side, three access adits have already been completed: Saint Martin La Porte, La Praz and Modane.
(following the tunnel in the direction from Lyon to Turin). On the Italian side, the preliminary works for the Maddalena exploratory tunnel started in June 2011. The excavation of the Base Tunnel is to be initiated in the following years.

### 3.2 Construction history of the Saint Martin La Porte access adit

The excavation of the Saint Martin La Porte access adit started in March 2003 with a design length of about 2000m and was completed at the end of 2010. Following the large convergences occurred between chainage 1200 and 1550m, the alignment of the tunnel initially chosen was modified in order to better accommodate the rock mass anisotropic conditions (with significant asymmetric deformations of the tunnel section) and the junction with the Base Tunnel.

After chainage 1000m approximately, the Carboniferous Formation, “Zone Houillère Briançonnaise-Unité des Encombres”, which is composed of black schists (45 to 55%), sandstones (40 to 50%), coal (5%), clay-like shales and cataclastic rocks, was encountered. It appeared to be increasingly affected by faulting and shear zones and resulted in a significant worsening of the rock mass conditions during excavation.

The excavation of the adit took place by the conventional method first with a stiff support system. However, it soon became apparent that the resistance solution was not feasible in the severe squeezing conditions encountered between chainage 1190 and 1267m. Since then, the yielding principle was adopted, where large deformations were allowed to develop around the tunnel with the expectation that rock pressure would decrease with increasing deformation. In particular, a novel “yield-control” support system, known as DSM, was designed and tested after re-mining of the tunnel section from chainage 1230 to 1325m (Barla, 2009).

### 3.3 Characterization of the Carboniferous Formation in the Base Tunnel excavation

The characterization described in the following, based on the Saint Martin La Porte adit, may in due right be adopted for prediction of the rock mass conditions to be encountered along the Base Tunnel where the excavation in the Carboniferous Formation is anticipated. Rock mass characterisation studies (including the assessment of deformability, strength, and time dependent properties for different models of behaviour) were carried out systematically based on monitoring of the tunnel response and a detailed mapping of the geotechnical and geomechanical conditions at the face (Barla et al., 2010; Bonini and Barla, 2011).

Figure 3, taken from Bonini and Barla (2011), shows 4 plots with related photographs of the conditions at the face, where the rock mass is noted to gradually improve. Starting from a very disturbed rock mass (chainage 1325m) including
significant portions of weak rocks, the face at first remains rather heterogeneous with a smaller presence of weak rocks (1670m). Then, more homogeneous conditions are met where the schist content gradually reduces and the presence of sandstone increases (2001 and 2330m).

The rock mass strength $\sigma_{cm}$, resulting from the GSI index and obtained by weighting the contribution of each rock type based on its percent presence at the tunnel face, was found to be rather insensitive to the changing conditions actually met during excavation. As a consequence, $\sigma_{cm}$ was instead back-calculated from the actual convergences measured in situ through the well known relationship between the normalised tunnel displacement to tunnel radius ($u/R$) and the ratio ($\sigma_{cm}/\sigma_0$) of rock mass strength to in situ stress ($\sigma_0$), as proposed by Hoek (2001).

Given the available geological and geomechanical data along the access adit (Figure 3), including the overburden depth, the type of support installed, the advancement sequence adopted, and the geological profile, the rock mass has been divided into 4 zones, corresponding to rock mass classes A to D as described below (Figure 4) (Bonini and Barla, 2011):

- Class A (chainage 2150 to 2330m): nearly homogeneous face with meta-siltstones and sandstones, with subordinate interbedded decimetric/metric pelitic and/or graphitic schists, with rare levels of coal;
- Class B (chainage 1850 to 2150m): mixed face formed by alternations of decimetric/metric levels of meta-sandstones, meta-siltstones and shales in varying proportions, with some decimetric/metric levels of coal;
- Class C (chainage 1550 to 1850m): mixed face with pelitic and/or graphitic schists, decimetric/metric levels of coal and subordinate intercalations of meta-sandstones;
- Class D (from 1200 to 1550m): tectonically sheared zone.

The four tunnel lengths corresponding to the rock mass classes A to D are associated with different convergence levels and rock mass strengths, as summarised in Table 1 (Bonini and Barla, 2011). It is of interest to underline that the zone with the largest convergences, between chainage 1200 and 1550m, is characterised by extensive shearing of tectonic origin and resulted to be overlain by a deep-seated gravitational slope deformation, with a limited thickness of in situ rock. It is expected that the probability of crossing class D rock masses along the Base Tunnel is very limited, as the conditions encountered in the access adit were very particular and also because it is assumed that the rock mass quality increases with increasing the effective cover, as observed in the adit. Therefore, it is assumed that the Base Tunnel where mechanized excavation will be used is to cross mainly the classes A, B and C. The rock mass class adopted in the following simulation is therefore class C.
3.4 A TBM of reference

For the purpose of the following computations, the tunnel internal diameter has been fixed to 8.1m. The segmental lining is assumed to be loaded due to the rock mass interaction only, as in zones with high water pressure durable drainage systems need be adopted. It is assumed that concrete class C45 for the segmental lining may conveniently be chosen. The thickness of the precast segments has been set to 45cm.

Given the rock mass conditions, the single-shield operation mode is assumed as a likely choice with the front and gripper shield forming a stiff unit and the auxiliary cylinders producing the forward thrust required (Maidl et al, 2008). The maximum cutterhead thrust has been set to 17MN, in order to achieve the necessary penetration rates also for hard rock. 64 conventional 17-inch diameter cutters (with a single load of 267kN) can be used. Five additional 17” cutters are provided for the max radial overboring of 70mm. Table 2 shows the main features of the TBM for overcoming squeezing, whereas Figure 5 gives the schematic view of the assumed TBM arrangement.

4 Computational model

Several computational models in one (e.g., closed-form solution as with the convergence-confinement method), two (e.g., numerical two-dimensional plane strain or axisymmetric models) and three spatial dimensions are available today. However, only axisymmetric or three-dimensional models can be used to consider the spatial stress distribution in the vicinity of the face as they can deal effectively with the complex interaction between the rock mass, the tunnel machine, its system components, and the tunnel support (Cantieni and Anagnostou, 2009).

A novel and complete 3D simulator, recently developed by the authors (Zhao et al., 2011), is briefly described in the following and will be used to analyze TBM excavation in the Carboniferous Formation along the Lyon-Turin Base Tunnel.

4.1 Model construction

The Base Tunnel is a twin tunnel. It is considered that the distance between the two tubes is large enough so that the excavation process has no influence on each other. Therefore, only one tube is taken into account. Under the assumption of symmetry with reference to the vertical plane through the tunnel axis, only half of the entire domain is modelled. A “cylindrical domain” is used and 8-nodes hexahedron solid elements are chosen as appropriate.
Given the large plastic zones expected, the external boundary is extended to a distance of $15D$ in the transversal plane to minimize boundary effects. A very refined region around the tunnel contour and a coarser zone away from it has been used as shown in Figure 6. Element size of 30cm x 30cm near the tunnel boundary has been adopted to capture high strain gradients. In the longitudinal direction, a total length of $20D$ is suggested to provide a sufficient distance between the rear boundary and the last excavation face. The mesh discretisation is chosen to be equal to the excavation length (1m); then, ahead of the final face position, it is gradually increased.

As the model refers to deep tunnels, all the outer boundaries (4 faces) are fixed along the perpendicular directions and the in situ state of stress is applied as a uniform initial stress $\sigma_0$ (26MPa) without consideration of the free surface and of the stress gradient due to the gravity.

The main TBM components are considered:

- **shields** (front and rear), which are modelled with plate elements applied at the excavation boundary and with the stiffness properties of steel;
- **cutterhead**, which is modelled with plate elements (with the stiffness properties of steel) at the current excavation face and where a pressure is applied;
- **tail gap grouting**, which is modelled by solid elements with two phases (i.e., softening and hardening phase);
- **lining**, which is modelled with plate elements (with the stiffness properties of concrete); no joints are introduced and the lining is considered to be continuous;
- **thrust jacks**, which are applied, in the form of an edge pressure, on the last lining ring installed.

All these components are modelled with a linearly elastic isotropic law. The self-weight is applied to all of them. The simulation parameters are given in Table 3.

### 4.2 Rock mass model

The difficulties to account for time dependence through elasto-viscous plastic models (Debernardi and Barla, 2009) lead to the adoption of simplified models where the rock mass is linearly elastic perfectly plastic, with strength and deformability properties estimated in “short” and “long term” conditions (Loew et al, 2010; Hoek and Guevara, 2009).

Given that the interest of this paper is to study the interaction between the rock mass and the TBM and the support system near the face and during excavation, the use of short-term parameters can be acceptable. The gradual increase of ground pressure and of ground deformations in the longitudinal direction is therefore considered to be only due to the spatial stress redistribution that is associated with the progressive advance of the working face (Lombardi, 1973).
The rock mass is considered to be continuous, homogeneous and isotropic. A linear elastic, perfectly plastic Mohr-Coulomb material with a non-associated flow rule is adopted to represent the squeezing behaviour. The assumed short-term parameters for the rock mass class C (§ 3.3) are listed in Table 4.

4.3 TBM advancement and simulation procedure

The model simulates the ongoing TBM excavation by a step-by-step method. The excavation length is taken to be equal to 1m. The construction stages and the total number of steps to be performed allow for a representative steady state condition to be reached, also considering the boundary effects (the first few meters of the excavation containing these effects will be removed from the results).

With respect to the Carboniferous Formation, the single-shield operation mode is considered as described previously. The construction stages (35 stages in this case) are defined as follows:

- in the first step, initialisation takes place;
- in the second step, the TBM enters the model and the cutterhead is activated;
- in the third step, the first slice of the shield is activated;
- in the thirteenth step, the grouting with a softening phase, the relevant pressure, the lining and the jack pressure are activated concurrently with the excavation, assuming a stroke of 1 m;
- since the fifteenth step, the properties of the grouting are changed into the hardening phase;
- the stages proceed until a steady-state condition is reached.

The simulation process is illustrated in Figure 7. The parts of the shields in contact with the rock mass (according to § 4.4.2) are also shown.

4.4 Modelling the interaction between the machine components and the rock mass

4.4.1 Interfaces

The simulation of the interaction problems requires a special attention to the discontinuous behaviour at the following interfaces: (i) rock mass-shield and (ii) grouting-lining. This behaviour involves frictional sliding, possible closure of the gap between the shield and the rock mass and possible contact/surface separation due to the self-weight of the structural components.
In order to simulate the relative movement, zero thickness interface elements are used to represent the frictional shearing mechanism (Day and Potts, 1994). Since the support and the ground cannot overlap, the normal stiffness is set to be very high. Furthermore, no slip before yielding of the interface is assumed (Fakharian and Evgin, 2000; Cai and Ugai, 2000). The interface is stiff and shear takes place within the elastic region. As a good rule-of-thumb the normal and shear stiffness are set equal to ten times the equivalent stiffness of the softer neighbouring zone, which is given by:

$$\frac{K + 4/3G}{\Delta z_{\text{min}}}$$

where:

$K$ and $G$ are the bulk and shear moduli, respectively, and $\Delta z_{\text{min}}$ is the smallest width of an adjoining zone in the normal direction.

When the shear stress exceeds the assigned threshold value, plastic slip deformation occurs at the interface. Based on no cohesive bond between the shield and the rock as well as between the lining and the rock, this threshold value can be expressed by the Coulomb friction law as follows:

$$\tau = \mu \sigma_n$$

where $\tau$ is the shear stress, $\sigma_n$ is the normal stress and $\mu$ is the skin friction coefficient.

According to Gehring (1996) and Ramoni and Anagnostou (2011), the skin friction coefficient $\mu$ of the rock mass on the shield is taken as $\mu = 0.15 - 0.30$ for the kinetic coefficient during the ongoing excavation and $\mu = 0.25 - 0.45$ for the static coefficient used for restart after a standstill. The coefficient of the rock mass on the lining is usually taken as $\mu = 0.30 - 0.40$ for static conditions (ACI 318, 2002). Therefore, for simplicity, the value of 0.3 is adopted for all the interfaces.

Since the numerical formulation used is based on the small strain /displacement assumption, the shields have been modelled right on the tunnel boundary even if there is a gap in between. Specifically, in order to simulate the gap between the rock mass and the shield at the crown and at the sidewalls (while at the invert contact is allowed), a special interface element is used, in which the elastic stiffnesses are set to zero, i.e., $k_n = k_s = k_t = 0$. No stress can thereby transfer from the rock mass elements to the shield elements. The stiffness of the shield is unrelated to the deformations of the rock masses. Thus the gap is correctly simulated since the presence of the shield has no confinement effect on the tunnel boundary.
4.4.2 Shield-rock mass interaction

The accurate simulation of the interaction between the shield and the rock mass has to take into account both (i) the geometry update in order to consider the deformation ahead of the face and (ii) the gap due to the overcut, the overboring and the conicity of the machine, as the closure of the gap determines the amount of unloading of the tunnel boundary and allows the following contact and slippage between the rock mass and the shield.

In Double Shield TBMs, the gap between the shields and the rock mass assumes a finite value and, furthermore, as shown in Figure 8, its width $\Delta g$ is not uniform in the cross-section and has a stepwise increase due to the conicity $\Delta r$, as follows:

\[
\Delta g = \begin{cases} 
\Delta D & \text{for the front shield} \\
\Delta D + 2\Delta r & \text{for the rear shield}
\end{cases}
\]

where $\Delta D$ is the overexcavation.

When a large convergence of the bored profile takes place, the closure of the entire gap may occur either along the front shield or along the rear shield or along both of them. The convergences can be easily obtained by monitoring the longitudinal displacement profiles (LDP) at the crown and at the invert after each step. When the convergence is such that the gap is closed, the shields start to support the excavation walls. Due to the in-situ state of stress and the large convergence which will take place, it is necessary to consider the possible uplift of the machine itself which may lead to a reduction of the free gap at the crown (as illustrated in Figure 8 by comparing the cross-section A-A with B-B).

For the front shield, the entire gap thus closes where the sum of the radial displacement at the invert $u_{\text{invert}}$ and at the crown $u_{\text{crown}}$ exceeds the gap size $\Delta g$ as in the cross section B-B shown in Figure 8:

\[
u_{\text{invert}}(B) + u_{\text{crown}}(B) \geq \Delta D
\]

Before the closure of the gap, only the invert of the front shield is in contact with the rock mass (as shown in Figure 8 A-A). The gap is simulated by the interface element described in § 4.4.1. Then, when the gap is closed, the entire shield starts to support the rock mass. This is carried out in the relevant step just by modifying the properties of the interface element of interest.

When the entire front shield is in contact with the rock mass, the gap around the rear shield is uniform and only due to the conicity. Because of the three-dimensional geometry being considered, the closures of the gap at the invert and at the crown however occur in different cross sections such as D-D and E-E shown in Figure 8.

Closure occurs at the invert where the relative radial displacement between the cross sections D-D and C-C exceeds the conicity $\Delta r$: 
\[ u_{\text{invert}}(D) - u_{\text{invert}}(C) \geq \Delta r \]

(7)

and at the crown where the relative radial displacement between the cross sections E-E and C-C exceeds the conicity \( \Delta r \):

\[ u_{\text{crown}}(E) - u_{\text{crown}}(C) \geq \Delta r \]

(8)

If the entire front shield is in contact with the rock mass only at the invert, the contact point D at the invert of the rear shield should satisfy the same condition as before:

\[ u_{\text{invert}}(D) - u_{\text{invert}}(C) \geq \Delta r \]

(9)

while for the point E at the crown the following should hold true:

\[ u_{\text{crown}}(E) + u_{\text{crown}}(E) \geq \Delta D + 2\Delta r \]

(10)

As a slice of the rock core ahead of the face is excavated, the displacement \( u_f \) at the face is removed. To consider the geometry update, the final radial displacement \( u \) of equations (6), (7), (8), (9) and (10) is \( u = u_r - u_f \), where \( u_r \) is the radial displacement. In order to find the exact positions where the shields gets into contact with the rock mass (which is actually the nearest point in the 1m discretisation), one is to note that, at the first excavation steps, the presence of the shield improves the behaviour of the ground by a longitudinal arch effect. Hence, the LDPs have to be checked at every step to find out where the shield gets in contact with the rock mass, as the shape of the LDPs changes from the previous step. However, after few steps (generally, 3 steps), such span reaches a constant value.

4.4.3 Lining-rock mass interaction (backfilling)

The load transfer between the rock mass and the lining, which occurs through a backfilling layer, is simulated by assuming that a grout annulus is injected via the shield tail with a very fast hardening mortar and simultaneously with the shield advance. As shown in Figure 2a, the tail gap grouting is modelled in two phases as follows:

1. in the softening (transient) phase, right behind the shield, the grouting is simulated by activating solid elements with a low modulus (0.5GPa) and by the application of a grouting pressure to the excavation boundary and to the lining;

2. in the hardening phase, 2m behind the shield (a very fast hardening grout is considered), a higher modulus (1GPa) is activated in the grouting elements and the grouting pressure is deactivated.

As the gap in squeezing rocks is intended to leave more free space for the ground deformations, the nominal thickness of the gap grouting is not respected and the actual thickness during construction depends on the LDP at the shield tail.
5. Results

5.1 Rock mass and machine components

Figure 9 shows the plastic strain contours in a 3D view. The degree of squeezing is expressed through the equivalent plastic strain, defined as:

$$\varepsilon_{pl} = \frac{2}{3} \left( \varepsilon_{p1}^2 + \varepsilon_{p2}^2 + \varepsilon_{p3}^2 \right)$$

with $\varepsilon_{p1}$, $\varepsilon_{p2}$, $\varepsilon_{p3}$ the principal plastic strains.

This definition comes from the most commonly used expression for the softening/hardening parameter (based on incremental plastic strain) which is (Vermeer and De Borst, 1984):

$$\frac{\Delta \varepsilon}{\Delta t} = \sqrt{\frac{2}{3} \left( \dot{\varepsilon}_{p1} \dot{\varepsilon}_{p1} + \dot{\varepsilon}_{p2} \dot{\varepsilon}_{p2} + \dot{\varepsilon}_{p3} \dot{\varepsilon}_{p3} \right)}$$

It is to note however that this softening parameter has not been introduced in the constitutive law. The equivalent plastic strain is thus only a variable describing the degree of failure in the rock mass.

The plastic zones are quite wide around the excavation (the plastic radius is 10m), even if the shield and the lining provide a support to the rock mass. It is also noted that the plastic strain at the invert is smaller than that at the roof (there is a reduction of 29% at the front shield invert). It is clear that the presence of the shield invert (in this case, both front and rear) before the gap is closed, associated with the self-weight of the machine, provides an additional confinement near the tunnel face.

Figure 10 depicts the results in terms of (a) plastic strain in a longitudinal section, (b) LDP and (c) contact pressure on both the shields and the lining (taken by the normal stress in the interface elements). The closure of the entire gap between the front shield and the ground is shown to occur 4m behind the face. For the rear shield this occurs at a distance of 9m at the invert and 10m at the crown, where complete closure takes place. It is of interest to note that the shapes of the LDP and the contact pressure distribution are very similar to those of another typical squeezing case studied by Ramoni and Anagnostou (2011).

Figure 11 shows the maximum principal stress in the shields and the cutterhead. Given this value, one is in position to carry out the structural design of the shields (and therefore of the machine).

By analyzing the pressure in the lining it is seen that, when the grout reaches the hardening phase, the stresses due to the excavation advancement are gradually transferred to the lining (i.e., the radial pressure on the lining increases), while the rock mass does not experience further plasticity (so-called “past-yield” zone, as described by Ramoni and Anagnostou,
The so-called “elastic re-compression” on the tunnel boundary occurs (Gärber, 2003) and finally, the steady state condition is reached.

Figure 12 shows the minimum principal stress contours around the excavation. The values of these stresses at the crown and at the invert are highlighted. It is noted that the rock mass experiences three unloading processes during the excavation.

Firstly, as the tunnel sidewall and crown remain unsupported, the tunnel boundary experiences the first unloading process, while the invert is confined by the shield at 1 m behind the face. Then, as soon as the entire gap is closed, loading of the shield takes place. Secondly, the entire tunnel boundary is unloaded at the rear shield due to the conicity of the machine.

Finally, as the annulus is injected via the tail shield, the last unloading process occurs.

As shown in Figure 13, it is also possible to check the normal stresses in the lining in short-term conditions in the following way:

$$\sigma_c = 21.4 < f_{cd} = \frac{f_{ck}}{\gamma} = 30 \text{ MPa}$$ (13)

where $\gamma$ is the partial factor for the concrete for the ultimate limit state equal to 1.5 in the case of persistent load, according to the Eurocode 2 (1992).

### 5.2 Thrust force

By integrating the ground pressure $p_i$ (contact pressure) over the shield surface ($N$, number of elements on the surface) and by multiplying the integral by the skin friction coefficient $\mu$ and the reduction coefficient $\beta$ which is the ratio $\beta = \frac{r}{R}$ of the real shield radius $r$ over the tunnel radius $R$ (as the shield is modelled on the tunnel boundary and therefore, considering the overcut and the conicity, it is bigger than the real shield), the thrust force required to overcome friction has been calculated:

$$F_f = \beta \cdot \mu \sum_{i=1}^{N} p_i A_i = 0.985 \times 0.3 \times 186 = 54.9 \text{ MN}$$ (14)

This result is compared with the value obtained by the nomograms for evaluating the thrust force recently developed by Ramoni and Anagnostou (2010b), as described in § 2.4. The nomogram specific for this case is shown in Figure 14, based on a normalized shield length $L/R=2$ and on a normalized rear shield length $L_r/R=1.2$ (the closest to the real situation). The dimensionless parameters $\frac{E}{\sigma_0} \frac{\Delta R}{R}$ and $\frac{UCS}{\sigma_0}$ have been entered in the nomogram (as shown) and a value for the normalized required thrust force $F_f^*$ equal to 0.025 is obtained. This leads to a thrust force $F_f$ equal to 63.6MN.
This value is in good agreement with the results obtained by the 3D simulator; it is slightly greater as a consequence of the assumption of axial symmetry (Ramoni and Anagnostou, 2010b) and of the other simplifications.

The maximum total thrust force by the auxiliary thrust cylinders is the sum of the maximum cutterhead thrust \( F_N \) and the thrust to overcome friction as follows: \( F = F_N + F_f = 17 + 54.9 = 71.9 \) MN. In order to reduce this quite high value of the auxiliary thrust force, it is possible to augment the gap corresponding to the rear shield: the stepwise increase between the rear shield and the front shield can pass from 3cm to 5-6cm, so that the contact between the rock mass and the shields would occur only in correspondence of the front shield and the required thrust force would be significantly lowered.

5.3 Torque

Also the torque and the consequent cutterhead installed power can be calculated. The total torque is the sum of maximum torque due to the rolling forces of the cutters and the torque needed to overcome the frictional resistance (sliding friction) caused by the ground pressure acting axially and radially upon the cutter head. The maximum rolling force \( F_r \) of a single cutter can be estimated simply equal to \( 0.1 F_N \), according to the average cutting coefficient \( C_c \left( \frac{F_N}{F_r} \right) \) obtained from tests and machine parameters (Balci et al., 2009; Rostami, 2008; Farrokh and Rostami, 2009; Yagiz, 2006). The maximum torque due to the rolling forces is (Balci et al., 2009):

\[
T_r = \frac{n \cdot F_r \cdot D}{4} = 4675 \text{ kNm}
\]  

where \( n \) is the number of cutters (69).

The torque needed to overcome the frictional resistance has the following expression:

\[
T_f = \mu \cdot \int_{0}^{\infty} \bar{p} \cdot 2\pi r dr \cdot r = \frac{2}{3} \mu \cdot \bar{p} \cdot \pi \cdot R^3 = 16826 \text{ KNm}
\]

where \( \bar{p} \) is the cutterhead pressure.

The total torque is thus equal to \( T_{\text{tot}} = T_r + T_f = 21.6 \) MNm.

The relationship between the torque and the power \( P \) is the following (Balci et al., 2009):

\[
P = \frac{2\pi \cdot \text{RPM} \cdot T}{60}
\]

where \( \text{RPM} \) is the rotational speed of the cutterhead. The power considered in table 2 is thus sufficient for the TBM advancement, even if the cutterhead rotational speed has to be reduced to 2.1rpm.


6 Conclusions

Squeezing conditions are anticipated during excavation of the Lyon-Turin Base Tunnel when crossing the Carboniferous Formation, as experienced during the recent excavation of the Saint Martin La Porte access adit. The research and characterization studies carried out from 2006 to 2010 (Bonini and Barla, 2011) were taken as the basis for the analyses described in this paper, in view of the mechanized excavation of the Base Tunnel.

Double Shield TBMs have been proven to be technologically advanced to deal with such conditions. Main features such as overexcavation technology, shield layout (i.e. stepwise conicity), thrust system, and easy access for pre-ground treatment are available to deal with such conditions.

A recently developed 3D simulator, which represents a powerful tool for the analysis of TBM excavation in squeezing ground, has been applied. The results obtained show that the 3D model is highly effective in reproducing the interaction between the TBM system components and the surrounding rock mass and may contribute to the evaluation of the feasibility of TBM excavation in terms of key parameters (plastic zone, shield structural design, necessary thrust force and torque, etc.).

It should be noticed that only the conditions near the face have been considered. Further improvements of the 3D simulator could take into account the time dependent behaviour by implementing the advanced constitutive models of the elasto-viscous plastic type (e.g., SHELVIP model) (Debernardi and Barla, 2009).

References

ACI 318 (2002) Building Code Requirements for Reinforced Concrete, American Concrete Institute, Detroit, Michigan.


Table captions

Table 1: Subdivision of the Carboniferous Formation in rock mass classes A, B, C and D as observed during excavation of the access adit (Bonini and Barla, 2011)

Table 2: Main features of the TBM to be considered

Table 3: Parameters for the components of TBM excavation

Table 4: Rock mass parameters in short term condition
<table>
<thead>
<tr>
<th>Zone</th>
<th>Chainage (m)</th>
<th>Average overburden (m)</th>
<th>Average convergence (mm - %)</th>
<th>$\sigma_{cm}$ (MPa)</th>
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<tr>
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<td>640</td>
<td>11 – 0.09</td>
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<td>B</td>
<td>1850-2100</td>
<td>550</td>
<td>52 – 0.45</td>
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<td>114 – 0.88</td>
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<td>476 – 3.98</td>
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<td>2.01 MPa</td>
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<tr>
<td>Friction angle $\varphi$</td>
<td>24°</td>
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<tr>
<td>Dilation $\psi$</td>
<td>4°</td>
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Figure captions

Figure 1: Construction schemes for the telescopic shield of Double Shield TBMs: (a) “classic” design; (b) modified DSU design (Ramoni and Anagnostou, 2010a)

Figure 2: Backfilling of the segmental lining: (a) with pea gravel; (b) with annulus grouting via the shield tail

Figure 3: Mapping of the geological conditions at the face at chainage (clockwise) 1325, 1670, 2001 and 2300 m (a=clay-like shales, b=schists, b-a=silty schists, b-a-c=silty schist with coal, b-e=grey schists, c=coal, d=cataclastic rocks, e, e-b, g, gps=sandstones, q=quartz lens). The lines in colour represent ductile or fragile schistosity and discontinuities (Bonini and Barla, 2011)

Figure 4: Convergences measured 15 m behind the face and indirect rock mass strength with indication of the zones of homogenous behavior (Bonin and Barla, 2011)

Figure 5: Schematic overview of the TBM arrangement (with maximum overboring)

Figure 6: 3D mesh at the initial stage (only rock mass elements) with detail

Figure 7: Illustration of the 3D model at different simulation steps. The two gradations of blue differentiate the parts of the shield in contact with the rock mass (dark-blue) with the parts which are not in contact (light-blue).

Figure 8: Illustration of the rock-mass shield interaction: (a) longitudinal section and (b) several cross-sections

Figure 9: Plastic strain contours along the tunnel

Figure 10: Results of the model: (a) Longitudinal displacement profile (LDP); (b) contact pressure developing on the shield and on the lining

Figure 11: Maximum stress in the shield and cutterhead

Figure 12: Minimum principal stress contours around the excavation (the gradations of red denote the degree of unloading)

Figure 13: Maximum stress in the lining

Figure 14: Nomogram for the specific case with determination of the normalized required thrust force; in the right side the assumptions at the basis of this nomogram (taken by Ramoni and Anagnostou, 2010b) are shown
Figure 1

(a) front shield  rear shield  telescopic shield  segmental lining

(b) front shield  rear shield  telescopic shield  segmental lining
Figure 3
Figure 4
Figure 5
Figure 6
Figure 7

(a) 12th step

(b) 13th step

(c) Final step
Figure 10

Equivalent plastic strain contours

(a)

(b)
Figure 11
Figure 13
Double shielded TBM
L/R = 2.0
Rear shield (L_f/R = 1.2)

\[ \nu = 0.25 \]
\[ K_s R/E = 10 \]
\[ K_f R/E = 0.50 \]

\[ K_s = E_s d/R^2 \]
\[ \Delta R_f = 1.5 \Delta R_f \]
\[ L_f = 1.5 L_f \]

Figure 14