ON THE FEASIBILITY OF TBM DRIVES IN SQUEEZING GROUND
AND THE RISK OF SHIELD JAMMING

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On the feasibility of TBM drives in squeezing ground and the risk of shield jamming

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Keywords
Tunnel boring machine, squeezing ground, TBM jamming, shield, tunnel support, numerical investigation, steady state method, design nomogram, advance rate, standstill, water-bearing ground, consolidation.

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Last but not least, I owe my loving thanks to my family – and particularly to Bettina – for their continuous support and encouragement.

Zurich, June 2010

Marco Ramoni
Summary

Squeezing ground represents a challenging operating environment as it may slow down or obstruct TBM operation. Due to the geometrical constraints of the equipment, relatively small convergences of 10–20 cm may lead to considerable difficulties in the machine area (sticking of the cutter head, jamming of the shield) or in the back-up area (e.g., jamming of the back-up equipment, inadmissible convergences of the bored profile, damage to the tunnel support).

Depending on the number and the length of the critical stretches, squeezing conditions may even call into question the feasibility of a TBM drive. On account of this, and bearing in mind the steady increase in the number of tunnels excavated with TBMs through so-called "difficult ground conditions", the topic investigated in this PhD thesis (which is structured in four parts) is of great practical relevance.

Based upon case histories reported in the literature, Part I sets out firstly to give an overview of the specific problems of TBM tunnelling under squeezing conditions. The factors governing TBM performance are then analysed by means of a structured examination of the multiple interfaces and interactions between the ground, the tunnelling equipment (TBM and back-up) and the support. Starting from the basic interactions, Part I also provides a critical review of the technical options already existing or proposed for coping with squeezing ground in mechanized tunnelling.

Planning a TBM drive in squeezing ground presents the tunnelling engineer with a complex problem where conflicting factors are present, each of them exerting an influence. In this respect, numerical analyses represent a helpful decision aid in support of engineering judgement, as they allow a quantitative assessment to be made of the effects of the key parameters. Part II presents a computational model which simulates accurately and efficiently the advancing TBM and the installed tunnel support in one single computational step applying the so-called "steady state method". Emphasis is placed on the boundary condition applied to model the interface between the ground and the shield or tunnel support. Furthermore, basic aspects of the interaction between the shield and the ground or tunnel support are analyzed by means of the computational results. Part II also discusses two application examples showing at the same time different methodical approaches applied assessing a TBM drive in squeezing ground. The first case history – the Uluabat Tunnel (Turkey) – mainly concerns the investigation of possible design measures aimed at reducing the risk of shield jamming. The second case history – the Faido Section of the Gotthard Base Tunnel (Switzerland) – deals with different types of tunnel support installed behind a gripper TBM.

Rapidly converging ground may exert such a high pressure on the shield that the available thrust force is no longer sufficient to overcome shield skin friction and the TBM becomes jammed. Part III advances a number of theory-based decision aids, which will support rapid, initial assessments to be made of thrust force requirements. A comprehensive parametric study has been carried out using the finite element method and, based on the numerical results, dimensionless design nomograms have been worked out that cover the relevant range of material constants, in situ stress and TBM characteristics. This is the first time that such a systematic and thorough investigation of the combined effects of the parameters governing shield loading has been attempted. The nomograms make it possible to assess the feasibility of a TBM drive in a given geotechnical situation and to evaluate potential design measures or operational measures such as reductions in shield length,
the installation of a higher thrust force, increases in the overcut or the lubrication of the shield surface, thus making a valuable contribution to the decision-making process.

From tunnelling practice, it is well-known that squeezing is a time-dependent process, which may take place over a period of days, weeks or months. The time-dependency can be traced back to creep or consolidation processes. Therefore, the risk of shield jamming depends essentially on the rapidness of ground deformation and thus on the creep or consolidation rate of the ground. For given geotechnical conditions and TBM characteristics, the load exerted by the ground upon the shield during continuous excavation depends on the gross TBM advance rate. During a break in operations, the ground pressure increases with time, thereby necessitating a higher thrust force in order to overcome shield skin friction and to restart the TBM. Part IV investigates the complex problem of the interaction between the advancing TBM, the consolidating ground and the lining. Emphasis is thereby placed on the effect of the gross advance rate and the effect of ground permeability on shield loading during regular TBM operation (the boring process including short standstills) and during a long standstill.
Zusammenfassung


Je nach Anzahl und Länge der kritischen Abschnitte können druckhafte Bedingungen sogar die Machbarkeit eines TBM-Vortriebs in Frage stellen. Demzufolge und unter Berücksichtigung der kontinuierlich zunehmenden Anzahl Tunnels, die mit TBM in sogenannten "schwierigen Baugrundverhältnissen" vorgetrieben werden, ist das in den vier Teilen dieser Forschungsarbeit untersuchte Thema von hoher praktischer Relevanz und sehr aktuell.


Schnell konvergierendes Gebirge kann einen so hohen Druck auf den Schild ausüben, dass die verfügbare Vorschubkraft nicht mehr ausreichend ist, um die Reibung zwischen Gebirge und Schild zu überwinden und die TBM blockiert wird. Der dritte Teil des vorliegenden Berichts enthält Hilfs-
mittel für die Entscheidungsfindung, welche die schnelle und einfache Berechnung der erforderlichen Vorschubkraft erlauben. Eine ausführliche Parameterstudie ist mit Hilfe der Methode der finiten Elemente durchgeführt worden. Basierend auf den numerischen Ergebnissen sind dimensionslose Nomogramme erarbeitet worden, welche die für die praktische Anwendung relevante Bandbreite der Materialparameter, des primären Spannungszustandes und der TBM-Eigenschaften abdecken. Es ist das erste Mal, dass eine so systematische und ausführliche Untersuchung der kombinierten Einflüsse der für die Schildbelastung massgebenden Parameter unternommen wurde. Die Nomogramme erlauben die Einschätzung der Machbarkeit eines TBM-Vortriebs in einer gegebenen geotechnischen Situation sowie die Beurteilung verschiedener technischer oder baubetrieblicher Massnahmen wie zum Beispiel eine Reduktion der Schildlänge, die Installation einer höheren Vorschubkraft, eine Vergrößerung des Überschnittes oder die Schmierung des Schildmantels.

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INTRODUCTION
The need for new underground structures is steadily increasing. In order to reduce construction time and, in some cases, to ensure the economic viability of the project, tunnels are being excavated ever more frequently with tunnel boring machines (TBMs). Although the experience gained over the years, along with the continuous technological advances, has made it possible to extend the range of applicability of the TBMs significantly, challenging operational conditions still exist where mechanized tunnelling encounters difficulties or even becomes impossible. Squeezing ground presents precisely such challenging conditions, as already relatively small convergences of 10–20 cm may lead to considerable difficulties in the machine area (sticking of the cutter head, jamming of the shield) or in the back-up area (e.g., jamming of the back-up equipment, inadmissible convergences of the bored profile, damage to the tunnel support).

Squeezing ground conditions may even cast doubt over the feasibility of a TBM drive. The topic is of great practical significance and topicality – particularly for (but not limited to) long, deep tunnels and it represents a major focus of tunnelling research activity at the ETH Zurich. The application-oriented part of the present PhD thesis was developed within the research project "Design aids for the planning of TBM drives in squeezing ground", which is supported by the Swiss Tunnelling Society (STS) and financed by the Swiss Federal Roads Office (FEDRO).

The present PhD thesis is structured in four parts.

Part I includes a qualitative discussion of the specific potential hazards as well as the complex interactions between the ground, the tunnelling equipment (TBM and back-up) and the support based upon both tunnelling experience and theoretical considerations. The peculiarities of the different TBM types are thereby highlighted and there is a brief description of the practical experience gained in mechanized tunnelling through squeezing ground. Furthermore, Part I deals with possible measures for coping with squeezing ground in TBM tunnelling.

Part II investigates the interaction between the shield, the ground and the tunnel support by means of numerical investigations. Particular emphasis is thereby placed on an adequate simulation of the interfaces between the ground and the shield or tunnel support. Furthermore, Part II illustrates the efficiency and the suitability of the computational model by means of two application examples: the Uluabat Tunnel (Turkey) and the Faido Section of the Gotthard Base Tunnel (Switzerland). At the same time, Part II shows different methodical approaches that can be applied in order to assess a TBM drive in squeezing ground.

Part III focuses on the risk of shield jamming and works out design aids that assist decision-making in the planning stage. The results of a comprehensive parametric study based upon numerical investigations and covering the relevant range of material constants, initial stress and TBM characteristics are presented in dimensionless 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. Using the nomograms it is also possible to evaluate the effects of potential design parameters and operational measures rapidly. Furthermore, Part III includes a large amount of TBM technical data that is helpful in assessing the technical feasibility of such measures.

Part IV deals with the time-dependency of squeezing with particular emphasis placed on the risk of shield jamming, on understanding the mechanisms governing the ground response to tunnelling operations and on some practical questions concerning mechanized tunnelling through water-bearing squeezing ground. The effects of the gross advance rate on shield loading as well as the
increase in ground pressure during TBM standstills are quantified by means of numerical investigations.

A large part of this report bases upon other works of the author that have been already published or have been submitted for publication. In fact, Part I represents an extended version of Ramoni and Anagnostou (2010d), Part II and Part III are based upon Ramoni and Anagnostou (2010b, 2010c) and Part IV refers to Ramoni and Anagnostou (2010a). Parts of this PhD research have also been presented on several other occasions (Anagnostou et al., 2009; Ramoni, 2007; Ramoni and Anagnostou, 2006, 2007a, 2007b, 2007c, 2007d, 2008, 2009).
PART I – EXPERIENCES AND BASIC CONSIDERATIONS ON
TBM TUNNELLING IN SQUEEZING GROUND

Squeezing ground represents a challenging operating environment as it may slow down or obstruct TBM operation. Due to the geometrical constraints of the equipment, relatively small convergences of 10–20 cm may lead to considerable difficulties in the machine area (sticking of the cutter head, jamming of the shield) or in the back-up area (e.g., jamming of the back-up equipment, inadmissible convergences of the bored profile, damage to the tunnel support). Depending on the number and the length of the critical stretches, squeezing conditions may even call into question the feasibility of a TBM drive. Based upon case histories reported in the literature, Part I sets out firstly to give an overview of the specific problems of TBM tunnelling under squeezing conditions. Secondly, this part of the report analyses the factors governing TBM performance by means of a structured examination of the multiple interfaces and interactions between ground, tunnelling equipment and support. Thirdly, Part I provides a critical review of the technical options existing or proposed for coping with squeezing ground in mechanized tunnelling.
1 Introduction

In recent years, the need for new infrastructure to handle the intercity transportation of people and goods has steadily increased. The construction of such facilities often requires the excavation of long, deep tunnels such as the two base tunnels of the Alptransit Project in Switzerland (Kovári, 1995), the Brenner Base Tunnel between Austria and Italy (Bergmeister, 2007), the Lyon – Turin Tunnel between France and Italy (Nasri and Fauvel, 2005) or the Gibraltar Strait Tunnel between Spain and Morocco (Pliego, 2005). In many cases the cost of such projects can be reduced to a justifiable level only by utilizing TBMs, because they allow significant savings in construction time and costs (Gerstner and Vigl, 1996).

Due to alignment constraints and the uncertainties of geological exploration (which may be large, particularly for long, deep tunnels), it is not always possible to find a route that will avoid the problem of excavating in difficult geological zones with a sufficient degree of certainty (Robbins, 1992). The extent and frequency of the difficulties encountered can be decisive in terms of economical viability or even in terms of the technical feasibility of a TBM drive. This is particularly true where special measures are needed in order to accelerate the TBM drive or to free the TBM in case of jamming. In some cases of very great potential damage, a single event can cast the entire project into doubt. Minor setbacks may also become relevant if occurring frequently. The length and the number of critical stretches are very important in this respect. Short tunnel stretches with unfavourable but well-known geological conditions are not particularly risky for the economic success of a TBM drive (Kovári, 1986a), provided that adequate countermeasures are planned in advance.

TBM performance can be affected by geological conditions in a great variety of ways (Barla and Pelizza, 2000). In hard rock, for example, boreability problems may occur such as a low penetration rate or excessive wear of the cutting tools, necessitating frequent cutter changes and other maintenance work. Mixed face or blocky rock conditions may cause steering difficulties or severe vibration of the cutter head, leading to considerable wear or even damage. Major water inflows may reduce the efficiency of the mucking system or affect the installation of segmental linings.

Another group of difficulties is associated with a low strength or high deformability of the ground and these fall, in a wider sense, under the heading of "stability and deformation problems". A low strength ground, such as highly fractured or weathered rock, may lead to cave-ins ahead of the tunnel face or to blockage of the cutter head. In open-type TBMs difficulties with the support installation or the gripper positioning may also occur in the case of unstable excavation walls. It is particularly challenging to cross fault zones with soil-like material under high water pressures (Anagnostou and Kovári, 2005). When such a zone is suddenly encountered, water and loose material flows into the opening. Both the timely identification and the treatment of such zones may decrease the degree of TBM utilization considerably.

On the other hand, when tunnelling through zones consisting of low stiffness ground, large long-term convergences of the opening may develop, destroying the tunnel support and, in extreme cases, completely closing the tunnel cross-section. Such, so-called "squeezing conditions" occur mostly in weak rocks (such as phyllites, schists, serpentinites and claystones) often in combination with a great depth of cover and a high pore pressure (Kovári, 1998; Vogelhuber et al., 2004).
Squeezing ground conditions may slow down or obstruct TBM operation (ITA, 2003) and sometimes even call into question the feasibility of a TBM drive. In fact, as described in Section 2, there have occasionally been some very negative experiences (including complete loss of the TBM) in the past and this has often lead to TBM drives in squeezing ground being classified as generally too risky and therefore not feasible. (A brief description of practical experience with mechanized tunnelling through squeezing ground, based upon a number of case histories reported in the literature, can be found in Section 2.) However, between the borderline cases of a heavily squeezing ground and a non-problematic competent rock, a wide range of conditions exist which neither exclude a priori mechanized tunnelling nor allow it without careful consideration. These cases call for a well-founded, thorough investigation of the risks, the technical feasibility and the cost of TBM application. So it is not surprising that the question of TBM applicability in squeezing conditions has kept engineers busy for more than 30 years. First remarks can be found already in Prader (1972), while more detailed conceptual considerations have been provided later by Lombardi (1981) and Robbins (1982). Other related works are, e.g., those of Kovári (1986a, 1986b), Amberg (1992), Gehring (1996), McCusker (1996) and Schubert (2000). As can be seen from recent publications (Downing et al., 2007; Herrenknecht, 2010; John and Schneider, 2007), the topic is particularly relevant today due to the increased economic importance of mechanized tunnelling associated with the demand for long deep tunnels.

The present part of the report presents a qualitative discussion of the complex interactions between ground, tunnelling equipment (TBM and back-up) and support based upon both tunnelling experience (Section 2) and theoretical considerations. Reference will be made to the peculiarities of the different TBM types and emphasis will be placed on the interfaces between the three essential system components: ground, tunnelling equipment and support (Section 3). Over the last decade, considerable research and development efforts have been made with the goal of widening the range of applicability for TBMs in squeezing ground either by improving established TBM types (i.e., gripper, single or double shielded TBM) or by developing new construction methods involving alternative machine designs or deformable lining systems. Reference to these works will be made in Section 4, which – starting from the basic interactions discussed in Section 3 – deals with possible measures for coping with large ground deformations or high ground pressures in mechanized tunnelling.

## 2 Practical experience and specific problems

### 2.1 Introduction

Table 1 summarizes the results of a comprehensive literature search on tunnelling experience involving TBMs under squeezing conditions. (As these case histories will be mentioned on several occasions throughout Part I, the references to the sources will, for the sake of convenience, be given only once, at the bottom of Table 1). According to Table 1, squeezing behaviour may become problematic at different distances behind the tunnel face. Therefore, the specific potential hazards concern both the machine area (sticking of the cutter head, jamming of the shield) and the back-up
Experiences and basic considerations on TBM tunnelling in squeezing ground – 23/212

area (e.g., jamming of the back-up equipment, inadmissible convergences of the bored profile, damage to the tunnel support).

In addition to the difficulties that are directly caused by squeezing behaviour, adverse events such as clogging of the cutter head, insufficient bracing of the grippers or instabilities of the working face or the tunnel wall may also occur when boring through weak ground. Often it is difficult or even impossible to distinguish the different phenomena from each other. For example, when driving through poor quality ground it may remain uncertain if the ground pressure acting upon the TBM is due to squeezing or ravelling behaviour. Furthermore, in several cases a feedback between the different problems may be observed (Kovári, 1986a).

2.2 Magnitude of relevant deformations

Concerning the magnitude of the potentially problematic deformations, a marked difference exists between conventional and mechanized tunnelling. Due to the geometrical constraints imposed by the equipment, even convergences as small as 10–20 cm may lead to difficulties in the machine or in the back-up area of a TBM drive. So, for example, Andraskay (1986) suggests that deformations in the bored profile of up to 10 cm have to be taken into account a priori in the selection of the boring diameter, while Kovári (1986b) and Barla (2004) consider convergences exceeding 5 % or 2–3 % of the tunnel radius, respectively, as problematic. Of course, these figures must be seen as very rough estimates as they do not account for the spatial distribution of the deformation along the tunnel axis and, furthermore, suggest that the absolute magnitude of the "critical" convergence decreases with the tunnel diameter, which is not necessarily true. A convergence of 3–4 % does not present serious problems for a small diameter tunnel as it corresponds to a radial displacement of only a few centimetres (see also Fellner et al., 2003). On the other hand, the values mentioned above indicate quite plainly the sensitivity of mechanized tunnelling to squeezing.

It should be noted that relatively moderate deformations of 10–20 cm, which may be problematic for a TBM (but could be easily dealt with by conventional tunnelling), are in no way limited to the typical squeezing formations of weak rocks such as phyllites, schists, serpentinites and claystones. Experience in some stretches of the Gotthard Base Tunnel (Switzerland) has shown that hard but highly fractured rocks may also exhibit relevant deformations and challenge TBM tunnelling, particularly if encountered at great depths.

2.3 The "time" factor

The case histories documented in the literature (Table 1) indicate that interruptions of the TBM drive may be unfavourable in squeezing ground, i.e., that the "time" factor may play an important role. In several cases, the TBM did not become jammed until there was a slowdown or standstill in the TBM drive, which suggests that maintaining a high gross advance rate and reducing standstill times may have a positive effect. The Yacambú – Quibor Tunnel (Venezuela, gripper TBM, D = 4.80 m) may be mentioned as a first example. During the holiday break of Christmas 1979
Table 1. Case histories (encountered problems and observed ground deformations).

<table>
<thead>
<tr>
<th>Project (country), Tunnel length [Reference]</th>
<th>TBM type, Manufacturer, Boring diameter, TBM operation time</th>
<th>Overburden, Geology</th>
<th>Sticking of the cutter head</th>
<th>Jamming of the shield</th>
<th>Inadmissible convergences</th>
<th>Damage to the tunnel support</th>
<th>Jamming of the back-up equipment</th>
<th>Observed deformations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yacambú – Quibor Tunnel (Venezuela), 24.3 km [1]</td>
<td>Gripper TBM, Robbins, 4.80 m, 1975–1980</td>
<td>350–400 m, graphitic phyllite</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>Along 250 m, bored profile practically closed within 30 days</td>
</tr>
<tr>
<td></td>
<td>Gripper TBM (2x) ?, Robbins, 4.80 m, 1975–1980</td>
<td>150–600 m, mudstone, sandstone</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Stillwater Tunnel (USA), 12.9 km [2]</td>
<td>Double shielded TBM, Robbins, 2.91 m, 1978–1979</td>
<td>600–800 m, sandstone, siltstone, blocky clayey schist, fault zones</td>
<td>x&lt;sup&gt;b,c&lt;/sup&gt;</td>
<td>x&lt;sup&gt;d&lt;/sup&gt;</td>
<td>x&lt;sup&gt;b,c&lt;/sup&gt;</td>
<td>x&lt;sup&gt;d&lt;/sup&gt;</td>
<td>x&lt;sup&gt;b,c&lt;/sup&gt;</td>
<td>Only small convergences</td>
</tr>
<tr>
<td></td>
<td>Walking Gripper Blade Shield, SNC Lavalin, 2.91 m, 1982–1983</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Convergences of 4 %; convergence rate of 0.4 %/d after 1 day and of 0.09 %/d after 10 days</td>
</tr>
<tr>
<td></td>
<td>Gripper TBM, Robbins, 3.20 m, 1982–1983</td>
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<td>x&lt;sup&gt;d&lt;/sup&gt;</td>
<td>x&lt;sup&gt;b,c&lt;/sup&gt;</td>
<td>x&lt;sup&gt;d&lt;/sup&gt;</td>
<td>x&lt;sup&gt;b,c&lt;/sup&gt;</td>
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<td>Convergences of 4 %; convergence rate of 0.4 %/d after 1 day and of 0.09 %/d after 10 days</td>
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<td></td>
<td>Gripper TBM, Robbins, 3.20 m, 1982–1983</td>
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<tr>
<td>Pueblo Viejo – Quixal Tunnel (Guatemala), 26.0 km [3]</td>
<td>Gripper TBM, Wirth, 5.64 m, 1978–1981</td>
<td>500 m, sandstone</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>Radial displacements of ≤ 50 cm</td>
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<td>Gripper TBM, Wirth, 5.64 m, 1979–1991</td>
<td>≤ 1500 m, marl</td>
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<tr>
<td>Tavanasa – Ilanz Tunnel, Strada Section (Switzerland), 4.2 km [4]</td>
<td>Gripper TBM, Demag, 5.20 m, 1985–1988</td>
<td>200 m, phylitic verrucano, 150–300 m long overthrust zone</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>Reduction of the bored profile by 25 cm within just a half day already in the machine area</td>
</tr>
<tr>
<td>Los Rosales Tunnel (Colombia), 9.1 km [5]</td>
<td>Double shielded TBM, Robbins, 3.54 m, 1987–1990</td>
<td>≤ 350 m, lutite</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
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<tr>
<td>Yindaruqin Irrigation Project, Tunnel 38 (China), 5.1 km [6]</td>
<td>Double shielded TBM, Robbins, 5.54 m, 1990–1992</td>
<td>200–430 m, clayey sandstone</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>Radial displacements of 3–5 cm at the cutter head and 5–8 cm at the location of support installation</td>
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<tr>
<td>Evinos – Mornos Tunnel (Greece), 29.4 km [7]</td>
<td>Gripper TBM (2x), Robbins, 4.20 m, 1993–1994</td>
<td>≤ 1300 m, flysch, thrust zones</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>Convergences of 15 cm within less than 1 hour at a distance of 1–2 m from the working face</td>
</tr>
<tr>
<td></td>
<td>Double shielded TBM, Robbins, 4.04 m, 1993–1994</td>
<td>700–950 m, flysch, 650 m long overthrust zone</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>Radial displacements of 3–5 cm at the cutter head and 5–8 cm at the location of support installation</td>
</tr>
<tr>
<td>Amsteg Power Plant Tunnel (Switzerland), 7.3 km [8]</td>
<td>Gripper TBM, Atlas Copco, 5.08 m, 1995–1996</td>
<td>500 m, cataclastic seritical phyllite</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>x&lt;sup&gt;a&lt;/sup&gt;</td>
<td>Only small convergences</td>
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</tbody>
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Table 1 (continuation).

<table>
<thead>
<tr>
<th>Project (country), Tunnel length [Reference]</th>
<th>Overburden, Geology</th>
<th>Sticking of the cutter head</th>
<th>Jamming of the shield</th>
<th>Inadmissible convergences</th>
<th>Damage to the tunnel support</th>
<th>Jamming of the back-up equipment</th>
<th>Observed deformations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Guadiaro – Majaceite Tunnel (Spain), 12.2 km [9]</td>
<td>150–400 m, sandy and clayey flysch, claystone</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
<td>Gap between shield and ground closed at a distance of 1 m from the working face; radial displacements of 5–6 cm of the damaged segmental lining</td>
</tr>
<tr>
<td>Vereina Tunnel, Nord Section (Switzerland), 11.6 km [10]</td>
<td>1250 m, heavy fractured crystalline rock</td>
<td>x</td>
<td>x</td>
<td>x</td>
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<td>Radial displacements of 20 cm at the cutter head during standstill due to ravelling or squeezing ground (the cause is unclear); execution of re-profiling works along 20 m</td>
</tr>
<tr>
<td>Umiray – Angat Tunnel (Philippines), 13.2 km [11]</td>
<td>1000–1200 m, basalt combined with other metamorphic volcanic rocks</td>
<td>x</td>
<td>x</td>
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<td>Convergence rate of 20 cm/h; in two cases reduction of the bored profile of 12 cm at a distance of 1 m from the working face, in a third case convergences of 37 cm within less than 2 hours in the machine area</td>
</tr>
<tr>
<td>Misicuni Tunnel (Bolivia), 19.8 km [12]</td>
<td>800–1200 m, cataclastic rocks within a 700 m long fault zone</td>
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<td>Convergences of 10–12 cm in the machine area with a convergence rate of 3 mm/min together with instabilities in the tunnel crown; the cause of the ground deformations is not clear (squeezing or ravelling)</td>
</tr>
<tr>
<td>Fujikawa Transport and Pilot Tunnels (Japan), 4.5 and 3.7 km [13]</td>
<td>250–300 m, 100 m long fault zone with clayey material</td>
<td>x</td>
<td>x</td>
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<td>Convergence rate of 15 cm/d</td>
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<td>500 m long fault zone with clayey material</td>
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<td>Reduction of the bored profile by 60 cm with a convergence rate of 29 cm/d; the gap between ground and tail shield became closed within 1 day</td>
</tr>
<tr>
<td>Nuovo Canale Val Viola Tunnel (Italy), 18.8 km [14]</td>
<td>200–800 m, pelitic and phyllitic rock</td>
<td>x</td>
<td>x</td>
<td></td>
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<td>The gap between shield and ground (8 cm in diameter) closed at a distance of 9 m from the working face (at the end of the shield) within 6 to 10 hours</td>
</tr>
<tr>
<td>Project (country), Tunnel length [Reference]</td>
<td>TBM type, Manufacturer, Boring diameter, TBM operation time</td>
<td>Overburden, Geology</td>
<td>Observed deformations</td>
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<td><strong>Salazie Aval Tunnel (France), 9.4 km [15]</strong></td>
<td>Double shielded TBM, Herrenknecht, 3.85 m, 1999–2005</td>
<td>900–1000 m, basalt</td>
<td>x</td>
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<tr>
<td><strong>Shanxi Wanjiazhai Yellow River Diversion Project, Connection Works Tunnel Nr. 7 (China), 13.5 km [16]</strong></td>
<td>Double shielded TBM, Robbins, 4.82 m, 2000–2001</td>
<td>300 m, marly rock</td>
<td>x</td>
<td>Convergence rate of 2–4 cm/h, gap between shield and ground (5–8 cm in diameter) closed within less than 2 hours</td>
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<tr>
<td><strong>Shanggongshan Tunnel (China), 13.8 km [17]</strong></td>
<td>Double shielded TBM, Robbins, 3.65 m, 2003–2005</td>
<td>200–250 m, alternating stratification of sandstones and clayey schists with cataclastic shear zones</td>
<td>x x</td>
<td>Gap between shield and ground (5–10 cm) closed practically instantaneously</td>
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<tr>
<td><strong>Gotthard Base Tunnel, Amsteg Section (Switzerland), 11.4 km [18]</strong></td>
<td>Gripper TBM (2x), Herrenknecht, 9.58 m, 2003–2006</td>
<td>700 m, cataclastic sericitical phyllite</td>
<td>x</td>
<td>Radial displacements of 15–30 cm in the back-up area</td>
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<tr>
<td><strong>Gotthard Base Tunnel, Bodio Section (Switzerland), 15.9 km [19]</strong></td>
<td>Gripper TBM (2x), Herrenknecht, 8.80 m, 2003–2006</td>
<td>1000 m, micaceous gneiss</td>
<td>x x x</td>
<td>Radial displacements of 7 cm at a distance of 4 m (at the end of the shield), of 10 cm at a distance of 8 m (after support installation) and of 14–22 cm at a distance of 55 m (back-up area) from the working face, respectively</td>
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<tr>
<td><strong>Arrowhead Tunnels East and West (USA), 9.3 and 6.1 km [20]</strong></td>
<td>Single shielded TBM (2x), Herrenknecht, 5.62 m, 2003–2008</td>
<td>≤ 650 m, hydrothermally altered granite, gneiss, fault zones with sandy and clayey material</td>
<td>x</td>
<td>Radial displacements of 6 cm within 2 days after installation of the segmental lining; considering that the TBM was already blocked for eight weeks 300 m before in the same geology due to instability of the working face and of the tunnel walls, it is possible that the ground deformations were due to ravelling and not to squeezing ground</td>
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<td><strong>Ghomroud Tunnel, Sections 3 and 4 (Iran), 16.5 km [21]</strong></td>
<td>Double shielded TBM, Wirth, 4.50 m, 2004–2008</td>
<td>≤ 650 m, graphitic schists and sandstones</td>
<td>x</td>
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<tr>
<td>Project (country), Tunnel length [Reference]</td>
<td>TBM type, Manufacturer, Boring diameter, TBM operation time</td>
<td>Overburden, Geology</td>
<td>Sticking of the cutter head</td>
<td>Jamming of the shield</td>
<td>Inadmissible convergences</td>
<td>Damage to the tunnel support</td>
<td>Jamming of the back-up equipment</td>
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<tr>
<td>Gilgel Gibe II Tunnel (Ethiopia), 25.8 km [22]</td>
<td>Double shielded TBM, Seli, 6.98 m, 2005–2009</td>
<td>670 m, weathered, brecciated and decomposed basalt</td>
<td>x</td>
<td>x</td>
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<tr>
<td>Pajares Tunnel, Section 4 (Spain), 10.3 km [23]</td>
<td>Single shielded TBM, Robbins, 9.88 m, 2006–2009</td>
<td>650 m, shale</td>
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<tr>
<td>Uluabat Tunnel (Turkey), 11.8 km [24]</td>
<td>Single shielded TBM, Herrenknecht, 5.05 m, since 2006</td>
<td>120 m, claystone</td>
<td>x</td>
<td>x</td>
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<tr>
<td>Gotthard Base Tunnel, Faido Section (Switzerland), 14.2 km [25]</td>
<td>Gripper TBM (2x), Herrenknecht, 9.43 m, since 2007</td>
<td>1600 m, micaceous gneiss</td>
<td>x</td>
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### Table 1 (continuation).

<table>
<thead>
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<th>Notes</th>
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<tbody>
<tr>
<td>a Loss of the TBM</td>
<td>h Sticking of the cutter head and jamming of the shield in sandy and clayey flysch one time avoided by applying the maximum possible torque and thrust force (overburden: 150 m)</td>
<td></td>
</tr>
<tr>
<td>b TBM drives abandoned</td>
<td>i Claystone (overburden: 400 m); damage to the segmental lining due to the application of a high thrust force (shield jamming could not be avoided)</td>
<td></td>
</tr>
<tr>
<td>c TBM jammed in a fault zone consisting of clayey schist (overburden: 150 m)</td>
<td>j Jamming of the shield two times due to rapid convergences and one time during a one-week holiday stop</td>
<td></td>
</tr>
<tr>
<td>d Due to insufficient backfilling of the segmental lining and to the combined action of ground pressure and thrust force</td>
<td>k In one case damage to the rear shield due to its lateral displacement</td>
<td></td>
</tr>
<tr>
<td>e TBM drive abandoned after loss of the TBM (ground collapse, major water inflow after sticking of the cutter head and jamming of the shield)</td>
<td>l Damage to the shield claimed by the contractor</td>
<td></td>
</tr>
<tr>
<td>f Jamming of the shield three times avoided by applying the maximum possible thrust force</td>
<td>m Due to the application of a high thrust force (shield jamming could not be avoided)</td>
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</tr>
<tr>
<td>g One TBM jammed in highly fractured radiolarit (overburden: 950 m)</td>
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</tbody>
</table>

### References

6. Andrea and Valenti (1993)
TBM operation was interrupted in graphitic phyllites encountered at a depth of 350–400 m. Within 30 days the rock mass converged unexpectedly to such an extent that the bored profile was closed and the TBM together with the tunnel support were destroyed over a length of 250 m. During the excavation of the Nuovo Canale Val Viola (Italy, double shielded TBM, $D = 3.60$ m) the TBM became trapped because of squeezing ground (pelitic and phyllitic rock) during a one-week holiday stop. In this case, it was possible to free the TBM by hand-mining. In the back-up area (over a length of 100 m), longitudinal cracks in the segmental lining were observed. In the Tunnel 38 of the Yindaruqin Irrigation Project (China, double shielded TBM, $D = 5.54$ m), the TBM was trapped in a clayey sandstone during a maintenance stop.

Standstills are unfavourable also with respect to cutter head operation. Depending on the rheological behaviour of the ground, high ground pressures acting upon the cutter head (Figure 1a) or an extremely high extrusion rate of the core (Figure 1b) may develop. In this respect, the Gilgel Gibe II Tunnel (Ethiopia, double shielded TBM, $D = 6.98$ m) may be mentioned. In this project, in one case the core extruded with a rate of 4–6 cm/h and pushed the TBM back more than 60 cm (with a lateral displacement of more than 40 cm). As a consequence, the shield and the segmental lining were damaged. However, if the TBM is boring, the excavation speed is normally high enough to avoid problems (Barla, 2001; Gehring, 1996; Hoek, 2001). The development of the ground pressure upon the cutter head is, as a rule, not fast enough to lead to an immobilization of the TBM. However, this may occur during a standstill (depending on the duration of the standstill and the deformation rate of the ground). During continuous excavation, i.e., when the TBM is boring, other aspects have also to be considered. For example, cutter head operation may be obstructed in blocky or soft rock as a result of damage caused to the cutters by falling blocks, mucking problems or excessive torque demand (Figure 1c). The installed thrust force and torque have to be sufficiently high to advance the TBM. In the case of a gripper TBM, the ground plays an additional important role, as it must provide a sufficient reaction force to the grippers. This was a problem, e.g., for all three TBMs in the Yacambú – Quibor Tunnel (the TBMs were also immobilized several times because of insufficient bracing of the grippers in very weak squeezing ground).

Maintaining a high advance rate is of course a major goal for any TBM drive. When tunnelling through squeezing ground it may also help to prevent the machine becoming trapped. Neverthe-
less, high gross advance rates should not be seen as a panacea for coping with squeezing. First of all, as can be seen from the last column of Table 1, the ground deformations may develop very rapidly and very close to the working face. In such a situation, the achieved gross advance rate would play a secondary role (the TBM would become jammed even if operated at the highest feasible speed). Furthermore, standstills of TBM operation cannot be completely avoided (Gehring, 1996; Lombardi, 1981). Besides adverse ground conditions, unpredictable stops due to technical problems (e.g., electric power stoppages, mechanical breakdowns of the TBM, problems in the back-up system) have to be considered. For example, during the excavation of the Evinos – Mornos Tunnel ( Greece) the cutter head of one of the gripper TBMs ($D = 4.20$ m) became jammed in highly fractured radiolarites at a depth of 950 m during an excavation standstill which was caused by an interruption of the electric power supply. The need to carry out regular maintenance work is also an important factor. This causes halts in excavation but it is at the same time important for reducing the risk of mechanical breakdown. Finally, it has also to be considered that a certain time is needed for support installation (a practically continuous excavation is possible only with double shielded TBMs advancing in the so-called "gripper mode").

In the case of time-dependent ground behaviour (which is characteristic for squeezing formations), the need for interruptions to allow support installation introduces important feedback effects and conflicting requirements. The maintenance of a sufficiently high gross advance rate is advantageous but difficult to achieve, especially in the case of poor quality ground. For example, if the tunnel is excavated with a gripper TBM, it becomes necessary to install a higher quantity of support and this lengthens standstill times. In extreme cases, the TBM becomes trapped and has to be freed with special measures that mostly require hand-mining. On the other hand, installing a lighter support for the sake of a higher advance rate may lead to inadmissible convergences in the back-up area. These aspects will be discussed in more detail in Section 3.1.3.

### 2.4 Thrusting system

When tunnelling by a single shielded TBM (or a double shielded TBM in the so-called "auxiliary mode"), the tunnel support (lining by precast segments) forms part of the thrusting system. There have been negative experiences in cases where a proper backfilling of the segmental lining was not achieved. The double shielded TBM ($D = 2.91$ m) that excavated a part of the Stillwater Tunnel (USA) may be mentioned in this context. This TBM was abandoned in September 1979 after becoming jammed in a fault zone consisting of clayey schist at a depth of cover of 650 m. The TBM probably became trapped in squeezing ground because of the impossibility of fully utilising its installed thrust force. Firstly, it was not possible to drive the double shielded TBM in the gripper mode and secondly, the segmental lining was not able to withstand the thrust force generated in the auxiliary mode. Due to blocky, poor ground conditions with frequent instabilities of the tunnel wall and related over-excavation it was not possible to backfill the segmental lining properly with pea gravel, as planned. Complementary injections of cement grout were carried out only after the passage of the back-up trailers. On the one hand, the insufficiently embedded segmental lining was not uniformly loaded from the start and was partially damaged by the ground pressure. On the other hand, attempts to use the full installed thrust force caused additional damage to the segments. In this case, there was a further difficulty in relation to the telescopic part of the shield, which had a
smaller diameter than the front and the rear shield, favouring the accumulation of loose material in this area and thus leading to an increase in the friction that had to be overcome when moving the double shield. Similar problems also arose with gripper bracing, the backfilling of the segmental lining and the telescopic part of the shield during the excavation of the Los Rosales Tunnel (Colombia, double shielded TBM, $D = 3.54$ m).

### 2.5 Back-up area

Possible problems in the back-up area include inadmissible convergences of the bored profile or damage to the tunnel support. Such problems are basically the same as in conventional tunnelling, the main differences being that in conventional tunnelling, (i), there is the option of excavating a considerably larger profile in the critical stretches (in order to accommodate the deformations) and, (ii), there is also more flexibility concerning the location of support installation (stabilization measures can be taken practically wherever and whenever required). In TBM tunnelling, the space available for ground deformations and tunnel support is largely pre-determined by the fixed geometry of the excavated cross-section. The possibility of enlarging the boring diameter locally is very limited (up to 30 cm, if at all possible, see Section 4.3.2), while the design of the back-up equipment fixes the locations of the support installation and limits the scope for intervention in the back-up area.

Besides the typical problems mentioned above, jamming of the back-up equipment is an additional hazard scenario to be considered, particularly for gripper TBMs. For example, during the excavation of the Strada Section of the Tavanasa – Ilanz Tunnel (Switzerland, gripper TBM, $D = 5.20$ m) the convergences that occurred in the machine area were large enough to violate the clearance profile needed for the passage of the back-up trailers. Re-profiling works also became necessary along 20 m of the Northern Section of the Vereina Tunnel (Switzerland). In a heavily fractured zone in crystalline rock at a depth of 1250 m, the gripper TBM ($D = 7.64$ m) became blocked due to a cave-in above the cutter head. During the standstill, radial deformations of 20 cm developed behind the machine, making the passage of the back-up trailers impossible. Jamming of the back-up equipment has also been observed recently in the Faido Section of the Gotthard Base Tunnel (Switzerland). During excavation by a gripper TBM ($D = 9.43$ m) in micaceous gneiss at a depth of 1600 m, significant convergences occurred over a 250 m long stretch. The tunnel support was damaged although it was designed to be deformable and, in some cases, touched the main structure of the back-up equipment and it became necessary to remove or to dislocate part of the equipment in order to keep the back-up trailers moving.

### 3 Ground-equipment-support interactions

Identifying the relevant interfaces between the main system components and understanding their interactions is essential to an assessment of the critical situations which might affect the performance or even the feasibility of a TBM drive. The following sections shall discuss the interactions be-
tween ground, tunnelling equipment and support, taking into account the peculiarities of existing TBM types with respect to thrusting systems, tunnel support, the presence or absence of a shield (gripper TBMs are often also equipped with a canopy or a short cutter head shield with a length of about a half boring diameter) and the achievable gross advance rate, which is an influencing factor as well. The discussion starts with the case of gripper TBMs (Section 3.1) because their greater flexibility concerning tunnel support increases system complexity. The case of single or double shielded TBMs thrusting against a segmental lining will be discussed later in Section 3.2.

3.1 Gripper TBM

3.1.1 Overview of interactions

The large number of interfaces between ground, tunnelling equipment and support in combination with the possibility of conflicting requirements and feedback effects introduces a high level of complexity, which necessitates an efficient mapping of the system and of its interfaces in order to identify and analyze the relevant interactions. The so-called "$N^2$ chart" offers the possibility of a systematic approach and a neat analysis. This method was introduced by Lano (1990) and is a well-known diagramming technique in system engineering practice.

Following the description given by NASA (2007), an $N^2$ chart is an $N$-by-$N$ square matrix (also-called an "interaction matrix") containing the $N$ physical or functional entities of a system (also-called "system elements", "system components" or "functions") on the main diagonal and the interfaces between them in the remaining off-diagonal cells, while a blank off-diagonal cell means that there is no interaction between the respective system elements. As illustrated by the schematic example of Figure 2, the interactions have to be read directionally between the elements, i.e., first horizontally in the row and then clockwise in the column. For example, the fact that the cell at the intersection of Row $x$ with Column $y$ is non-empty indicates that entity $E_x$ has an effect on entity $E_y$ (the numbers $x$-$y$ within the cell represent the parameters involved as well as the "direction" of the interaction). The non-empty cells in Row $y$ show all of the entities that are influenced by entity $E_y$ (i.e., the "outputs" of $E_y$), while the non-empty cells in Column $x$ show which entities influence entity $E_x$ (i.e., the "inputs" to $E_x$). The usefulness of this representation technique is illustrated by the interaction loop {2-$x$-$y$-2} which indicates a feedback effect.

In order to condense more information into one $N^2$ chart, the diagrammatic language has been enhanced adding two mapping rules that exploit the shape and color of the cells: rhombuses (e.g., cell $y$-$N$ in Figure 2) indicate that an interaction exists only under certain conditions (for example, ground deformations may lead to TBM jamming only if a threshold value is exceeded), while circles (e.g., cell $x$-$y$ in Figure 2) denote unconditional interactions (for example, standstills always reduce the gross advance rate). As for the colours, green is used for interactions of the type "an increase of $E_x$ leads to an increase of $E_y$", red for interactions of the type "an increase of $E_x$ leads to a decrease of $E_y" and black for interactions where the effect of $E_x$ on $E_y$ may be either positive or negative.

Figure 3 shows the $N^2$ chart elaborated for the subject of the present report. The number of physical and functional entities represents the result of a trade-off, which has had to be made between
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As can be seen in Figure 3, the chosen entities include the physical components of the system (such as the ground or the TBM) as well as operational parameters (such as the cutter head rotational speed), the results of the boring process (such as the net advance rate) or events (such as problems in the back-up area or standstills due to TBM jamming). The interactions depicted have been classified as "relevant" based upon practical experience (cf. Section 2 and Table 1) and theoretical factors. In spite of their limitations, the latter are indispensable as there are hardly any systematic field investigations available.

The main inputs of the $N^2$ chart of Figure 3 are the ground (denoted by $G$), the TBM itself, the back-up equipment and the tunnel support (denoted by the entities $S_1$ and $S_2$ that will be explained later), while the gross advance rate ($v_g$) can be seen as the main output since it best represents TBM performance (construction cost could also have been considered as an output entity, of course). The gross advance rate depends on the net advance rate ($v_n$) achieved during the boring process and on the duration of the standstills (denoted by $t_1$, $t_2$ and $t_3$ depending on their cause). In this respect a distinction is made here between the regular operational standstills needed for the installation of the tunnel support or for the execution of maintenance works, etc. ($t_1$); standstills due to TBM jamming, where the TBM has to be freed and possibly repaired ($t_2$); and standstills due to other problems such as damage to the tunnel support or mechanical breakdown ($t_3$).

The dense population of the first two rows of the $N^2$ chart (Figure 3) reflects the paramount importance and the manifold effect of the ground and of the TBM (cutter head, shield, thrusting system). These entities summarize a large number of properties and features, which become relevant only for one or two interactions. Resolving the entities $G$ (ground) and TBM into their individual properties and features (strength, stiffness, permeability, ground heterogeneity, shield length, installed...
Figure 3. W-chart for a gripper TBM drive through squeezing ground.
Table 2. Comments on the interactions shown in Figure 3.

<table>
<thead>
<tr>
<th>Entities</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-5</td>
<td>The tunnel support to be installed in the machine area (1-5) or in the back-up area (1-6) is chosen on the basis of the ground conditions (although work safety must always be ensured).</td>
</tr>
<tr>
<td>1-6</td>
<td>The location of the first tunnel support installation depends on the length of the cutter head and of the shield. Furthermore, the design of the equipment installed on the TBM is relevant with respect to the type of tunnel support that can be installed and to the place available for the installation work.</td>
</tr>
<tr>
<td>16-5</td>
<td>The stiffness and the bearing capacity of the tunnel support may be time-dependent (shotcrete, grouted bolts). During a standstill, they increase with the elapsed time (17-4, 18-6, 19-4, 17-4, 18-4, 19-6). During ongoing excavation, the more rapid the TBM advance, the lower will be the stiffness and the bearing capacity of the tunnel support at a given distance from the tunnel face.</td>
</tr>
<tr>
<td>3-6</td>
<td>The location of the second tunnel support installation, the type of tunnel support and the space available for support installation all depend on the design of the back-up equipment.</td>
</tr>
<tr>
<td>7-7</td>
<td>During overboring, the cutter head rotational speed may have to be reduced (Section 4.3.2).</td>
</tr>
<tr>
<td>1-1</td>
<td>The choice of the overboring facility to be applied and the feasible increase in the boring diameter in particular will depend on the ground conditions (Section 4.3.2).</td>
</tr>
<tr>
<td>1-10</td>
<td>Overboring is only possible if this option has been catered for in the design of the cutter head and of the shield (Section 4.3.2).</td>
</tr>
<tr>
<td>1-11</td>
<td>Firstly, the dimensions (diameter, length) of the cutter head and the shield represent a “span” and therefore influence the amount of ground pressure acting upon the TBM. The ground pressure also depends on the stiffness of the different TBM components. The ground pressure acting upon the shield may be reduced, if the shield is shrinkable (Section 4.4). Furthermore, the conicity of the cutter head and of the shield as well as the overcut (i.e., the difference between the cutter head diameter and the shield diameter) play an important role as regards steering the TBM and allowing ground deformations without loading the TBM. During a standstill, the ground pressure acting axially upon the cutter head can be reduced, if the TBM design allows it to be pulled back. In order to overcome the friction caused by the ground pressure acting upon the cutter head and the shield, a sufficient pull force must have been installed and the ground must offer sufficient resistance to the grippers.</td>
</tr>
<tr>
<td>9-12</td>
<td>During overboring, the cutter head rotational speed may have to be reduced (Section 4.3.2).</td>
</tr>
<tr>
<td>10-10</td>
<td>The relevant influencing factors are the cutter head design (e.g., its form as well as number, spacing, shape, diameter and wear of the cutters) and the characteristics of the rock mass (intact rock and discontinuities).</td>
</tr>
<tr>
<td>11-12</td>
<td>A faster core extrusion leads to a greater depth of cut. This leads to an increase in the rolling force of each cutter and, therefore, in the rolling resistance of the cutter head.</td>
</tr>
<tr>
<td>12-12</td>
<td>The choice of the overboring facility to be applied and the feasible increase in the boring diameter in particular will depend on the ground conditions (Section 4.3.2). See Section 3.1.2.</td>
</tr>
<tr>
<td>13-12</td>
<td>The rolling force of each cutter increases with increasing normal cutter force (the boring thrust force acting upon a single cutter), because of the deeper cuts and of the higher rolling friction.</td>
</tr>
<tr>
<td>14-12</td>
<td>The choice of the overboring facility to be applied and the feasible increase in the boring diameter in particular will depend on the ground conditions (Section 4.3.2). See Section 3.1.2.</td>
</tr>
<tr>
<td>15-12</td>
<td>Besides stiffness and bearing capacity of the tunnel support, its thickness is also relevant, as it reduces the space available to ground deformations.</td>
</tr>
<tr>
<td>16-12</td>
<td>In the case of pronounced time-dependent ground behaviour, a reduction in the net advance rate or a standstill will lead to higher ground deformations or higher ground pressures at a given distance behind the tunnel face.</td>
</tr>
<tr>
<td>17-12</td>
<td>A large overboring allows more ground deformation to take place and thus leads to a lower loading of the TBM.</td>
</tr>
<tr>
<td>18-12</td>
<td>The relevant influencing factors are the cutter head design (e.g., its form as well as number, spacing, shape, diameter and wear of the cutters) and the characteristics of the rock mass (intact rock and discontinuities).</td>
</tr>
<tr>
<td>19-12</td>
<td>The relevant influencing factors are the cutter head design (e.g., its form as well as number, spacing, shape, diameter and wear of the cutters) and the characteristics of the rock mass (intact rock and discontinuities).</td>
</tr>
<tr>
<td>20-12</td>
<td>The relevant influencing factors are the cutter head design (e.g., its form as well as number, spacing, shape, diameter and wear of the cutters) and the characteristics of the rock mass (intact rock and discontinuities).</td>
</tr>
</tbody>
</table>
If during ongoing excavation the net advance rate becomes equal to zero, the TBM is jammed and has to be freed.

The ground conditions are relevant with respect to cutter wear (e.g., abrasivity of the rock) and more generally to the required maintenance work (as, e.g., in the case of cutter head clogging). Difficult ground conditions (e.g., major water inflows) may also slow down the installation of the tunnel support.

The frequency of standstills associated with mechanical breakdowns depends on the ground conditions (e.g., damage to the cutters or to the conveyor belt in blocky ground), on the robustness of the TBM (including the overboring equipment) and the back-up equipment as well as on the maintenance work carried-out on a regular basis.

By definition.

<table>
<thead>
<tr>
<th>Entities</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>7-16</td>
<td>See Equation 3.</td>
</tr>
<tr>
<td>11-16</td>
<td></td>
</tr>
<tr>
<td>15-16</td>
<td></td>
</tr>
<tr>
<td>1-17</td>
<td>The ground conditions are relevant with respect to cutter wear (e.g., abrasivity of the rock) and more generally to the required maintenance work (as, e.g., in the case of cutter head clogging). Difficult ground conditions (e.g., major water inflows) may also slow down the installation of the tunnel support.</td>
</tr>
<tr>
<td>2-17</td>
<td>The complexity of the TBM influences the frequency and duration of maintenance work.</td>
</tr>
<tr>
<td>8-17</td>
<td>Depending on the type of overboring equipment, the commencement of overboring operations may require a standstill (Section 4.3.2).</td>
</tr>
<tr>
<td>1-18</td>
<td>See Section 3.1.2 and Equations 1 and 2.</td>
</tr>
<tr>
<td>16-18</td>
<td>If during ongoing excavation the net advance rate becomes equal to zero, the TBM is jammed and has to be freed.</td>
</tr>
<tr>
<td>1-19</td>
<td>The frequency of standstills associated with mechanical breakdowns depends on the ground conditions (e.g., damage to the cutters or to the conveyor belt in blocky ground), on the robustness of the TBM (including the overboring equipment) and the back-up equipment as well as on the maintenance work carried-out on a regular basis.</td>
</tr>
<tr>
<td>2-19</td>
<td></td>
</tr>
<tr>
<td>3-19</td>
<td></td>
</tr>
<tr>
<td>8-19</td>
<td></td>
</tr>
<tr>
<td>17-19</td>
<td></td>
</tr>
<tr>
<td>16-20</td>
<td>By definition.</td>
</tr>
<tr>
<td>17-20</td>
<td></td>
</tr>
<tr>
<td>18-20</td>
<td></td>
</tr>
<tr>
<td>19-20</td>
<td></td>
</tr>
</tbody>
</table>
thrust force, etc.) is basically possible, but would increase the chart size beyond the limit that can be managed on paper.

The alternative possibility of including the lower-level entities, but grouping them into blocks – creating thus a hierarchy of $N^2$ charts that considers the major subsystems with appropriate resolution (cf. Lano, 1990) – was abandoned for reasons of complexity and because it would not add anything substantial. Instead, remarks concerning some of the relevant parameters under the headings “G” (ground) or “TBM” are given in Table 2 (which supplements the $N^2$ chart with comments) as well as in the remainder of this section, which addresses questions of boring, thrusting and support, including their interactions. For the sake of simplicity, pairs of numbers within curly brackets will be used for making reference to Figure 3 and denoting the interfaces of the respective entities (e.g., {9-15} denotes the effect of entity 9 on entity 15).

### 3.1.2 Boring and thrusting

The jamming of the TBM represents a major hazard as it may lead to serious damage, necessitating lengthy standstills for freeing or repairing the machine (standstill $t_2$, Figure 3). Besides being important from a practical point of view, this potential problem is also theoretically very interesting and has attracted several research efforts over recent years – recent works are, e.g., those by Graziani et al. (2007a), Ramoni and Anagnostou (2007b, 2007c) and Sterpi and Gioda (2007).

Part I of the present report limits itself to a qualitative discussion of the basic interactions and mechanisms underlying, (i), the inability to resume TBM operation after a standstill (which may be necessary, e.g., for support installation, re-gripping, etc.) and, (ii), the immobilization of a TBM during ongoing excavation.

#### Restart after standstill

If the ground behaviour is time-dependent, which is very common for squeezing conditions, a radial ground pressure may develop upon the machine during a standstill. In order to resume TBM operation, i.e., to move the TBM forwards and to rotate the cutter head, the thrusting system must be able to cope with the frictional forces acting upon the cutter head and the shield (Figure 4).

In order to move the TBM forwards, both the installed thrust force $F_i$ and the bearing capacity of the thrusting system $F_g$ must be higher than the frictional resistance $F_f$ (static friction): \[ \min(F_i, F_g) > F_f. \] (1)

The frictional resistance $F_f$ increases with the radial pressure acting upon the machine {10-18} (which may be high in the case of squeezing ground) and with the size of the loaded area (i.e., with the diameter and length of the cutter head and of the shield {2-18}). Furthermore, it depends on the type of the ground {1-18} and on the surface roughness of the cutter head and of the shield {2-18} as they are relevant with respect to the skin friction coefficient. The installed thrust force $F_i$ is a matter of TBM design {2-18}, while the bearing capacity $F_g$ of the thrusting system depends both on the TBM design (the number, dimensions and surface roughness of the grippers and the installed gripper force) {2-18} and on the stiffness and the strength of the ground {1-18}. If the ground is weak and offers only insufficient resistance to the grippers, the effectively available thrust force will be lower than the installed one.
In order to restart the rotation of the cutter head, the effectively available torque must be high enough to overcome the frictional resistance $T_f$ at the circumference of the cutter head (Figure 4) as well as its rolling resistance $T_r$:

$$\min(T_i, T_g) > T_f + T_r,$$  \hspace{1cm} (2)

where $T_i$ denotes the installed breakout torque (Figure 5) and $T_g$ is the maximum torque that can be applied when taking into account the limited bearing capacity of the ground next to the grippers. Concerning the installed torque \{2-18\} and the bearing capacity of the thrusting system \{1-18, 2-18\} the factors are similar to those for the thrust force. The frictional resistance depends in this case on the geometry of the cutter head and on the ground pressure acting upon it \{2-18, 10-18\} as well as on the type of ground \{1-18\} and on the characteristics of the cutter head surface \{2-18\}, while the rolling resistance increases with the depth of cut at the moment when the cutter head rotation restarts.

Normally the rolling resistance of the cutter head should be lower than during the boring process, as no (or only a low) thrust force is applied to the cutter head when restarting its rotation \{9-12\}. Under squeezing conditions, however, the ground at the tunnel face may deform axially and around the cutters. The core extrusion thus leads to an increase of the depth of cut and, therefore, of the rolling resistance of the cutter head \{1-12\}. Furthermore, since the cutter head hinders ground deformations, an axial pressure develops upon it (Figure 4). It is obvious that in this case the demand both for thrust force and for torque increases. The increased thrust force demand is not so problematic, because a thrust force reserve is available at this stage since the TBM is not engaged in boring. The torque demand may, nevertheless, be critical to the resumption of operations.

If the ground pressure acting upon the TBM reaches its bearing capacity, damage will occur and repair work will be needed \{2-18, 10-18\}. This is particularly true if the TBM is already immobilized and the ground pressure increases further. During a standstill, the possibility of the TBM being pushed back by the axial ground pressure acting on the cutter head has also to be considered. Finally, when restarting TBM operation (or during ongoing excavation) overstressing due to combined loading (thrust force, torque and ground pressure) is also possible.
Immobilization during ongoing excavation

TBM immobilization during the boring process can be seen as equivalent to the borderline case of a zero net advance rate \( \{16-18\} \) (Figure 3). Usually the net advance rate \( v_r \) is expressed as the product of the achieved penetration \( P \) \( \{15-16\} \) with the chosen cutter head rotational speed \( r \) \( \{7-16\} \).

In the case of intensively squeezing ground, however, one should bear in mind that before the machine moves forwards the extrusion of the core has first to be compensated and, consequently, the net advance rate

\[
v_n = \max(0, \frac{P}{r} - e),
\]

where \( e \) denotes the extrusion rate of the core \( \{11-16\} \). Under normal conditions (characterized by the usual values for penetration and rotational speed), the effect of the core extrusion rate is small, but it may become relevant in the case of a low penetration or a low rotational speed. In extreme cases, the cutter head penetrates and rotates without moving forward (the penetration is used-up just for removing the axially deforming ground at the working face). The circumstances leading to reduced values of penetration or rotational speed are outlined below.

The penetration rate depends on the rock mass characteristics (strength, discontinuities, etc.) \( \{1-15\} \), on the cutter head design (form and stiffness, cutter spacing, size, etc.) \( \{2-15\} \) and on the boring thrust force \( \{9-15\} \), i.e., on the force with which the cutter head is pushed against the working face. The boring thrust force \( F_b \) represents an operational parameter which can be chosen within certain limits that are imposed by the installed thrust force \( F_i \) \( \{2-9\} \), by the bearing capacity of the thrusting system \( F_g \) \( \{1-9, 2-9\} \) and by the frictional resistance of the cutter head and the shield \( F_f \) (in this case sliding instead of static friction has to be considered) \( \{2-9, 10-9\} \). The maximum possible boring thrust force

\[
F_{b,max} = \max(0, \min(F_i, F_g) - F_f).
\]
The analysis of TBM operational data and of field measurements, as done, e.g., from Farrokh and Rostami (2009) for the Ghomroud Tunnel (Iran, double shielded TBM, $D = 4.50$ m), confirmed that an increase of the frictional resistance $F_f$ can lead to a limitation of the net cutter load and, consequently, of the penetration $P$ (9-15). Of course, a further reduction of the boring thrust force may also be necessary for reasons not related to squeezing (for example, in order to limit vibrations that could damage the equipment or in the case of blocky ground or mixed face conditions).

Squeezing behaviour – particularly if encountered in combination with gripper bracing problems – may limit the boring thrust force to such an extent that penetration is no longer possible (9-15). Special problems may arise if squeezing weak ground alternates with hard rock. More specifically, when the TBM is exiting a weak zone and entering hard rock (Figure 6), a so-called "under-thrust situation" may occur (McCusker, 1996), which is characterized by the combination of several adverse factors: (i) low bearing capacity of the ground in the gripper area (1-9); (ii) the need to overcome high frictional resistance (2-9, 10-9); (iii) high resistance of the hard rock to the boring process (9-15).

The cutter head rotational speed is another operational parameter directly affecting the net advance rate (7-16) (Equation 3) and is chosen on the basis of several limiting factors such as the diameter and the robustness of the cutter head (2-7), the capacity of the mucking system and the torque demand $T_r + T_f$, where $T_r$ denotes the rolling resistance of the cutter head (12-7) and $T_f$ is the torque needed to overcome the frictional resistance (sliding friction) caused by the ground pressure acting axially and radially upon the cutter head (2-7, 10-7). The rolling resistance of the cutter head depends on the ground (1-12) as well as on the characteristics of the cutters (e.g., number, spacing, arrangement, shape, diameter, wear) (2-12). In weak ground, the torque demand may be very high and reduce the achievable rotational speed considerably, because rotational speed $r$ and installed torque $T_i(r)$ are interrelated and determined by the TBM design (2-7) (Figure 5). Additionally, the effectively available torque may be smaller than the installed one as the

![Figure 6. Gripper TBM leaving squeezing weak ground and entering hard rock.](image-url)
torque $T_g$ that can be reacted by the grippers may be low in poor quality ground \{1-7, 2-7\}. In general, the following condition must be satisfied in order to rotate the cutter head with a speed $r$:

$$\min(T_c(r), T_g) > T_r + T_g \quad (5)$$

Under given operational conditions it may be necessary to reduce the cutter head rotational speed to a value $r < r_{\text{max}}^*$ (Figure 5) in order for sufficient torque to be available for overcoming the frictional and rolling resistances. Sticking of the cutter head will occur if the torque is insufficient in spite of the reduction of the rotational speed of the cutter head up to $r = 0$ \{(1-7), (2-7), (10-7), (12-7), (7-16-18); please note, that the notation \{7-6-18\} summarizes the sequence of the interactions \{7-16\} and \{16-18\}\). Penetration $P$ and rotational speed $r$ determine the net advance rate $v_n$ (Equation 3). It should be noted, however, that the latter is not only the main output of the boring process but may also be an important influencing factor. In the case of pronounced time-dependent ground behaviour, a slower advance leads to a higher loading of the machine \{16-10\} and this reduces both the achievable penetration \{10-9-16\} and the maximum possible rotational speed of the cutter head \{10-7\}, thereby causing a further reduction in the net advance rate \{7-16, 15-16\}, i.e., the system response to the given "perturbation" is amplified. This so-called "positive feedback" is more pronounced where there is a high extrusion rate of the core \{16-11-16 or 16-11-12-7-16\}.

### 3.1.3 Tunnel support

The application of tunnel support usually takes place at two locations: in the machine area and later in the back-up area at a distance of 30–60 m behind the tunnel face (Maidl et al., 2001). The locations of the support installation are determined by the design of the tunnelling equipment. In the back-up area it is generally possible to install the tunnel support without slowing down the rate of TBM progress. Support application in the machine area, however, interferes considerably with TBM operation because, as a rule, it necessitates a halt of the machine. Furthermore, the support in the machine area may influence the ground pressure acting upon the shield (cf. Section 4.7) or may limit the possibility of retracting the cutter head if necessary (cf. Section 4.3).

Interventions outside the two sectors mentioned above are, as a rule, not possible. According to Schneider et al. (2007), particularly critical in this respect is the zone between the first and the second tunnel support installation points (zone A in Figure 7). In order to reduce the risk of problems in this area (e.g., jamming of the back-up equipment, inadmissible convergences of the bored profile, damage to the tunnel support) the installation of a higher quantity of tunnel support may be needed in the machine area and this, as said before, will affect general TBM performance.

For these reasons, the $N^2$ chart (Figure 3) was refined in order to take into account the specifics of the two locations for support installation. The two entities $S_1$ and $S_2$ denote the tunnel support applied in the TBM area and in the back-up area, respectively (Figure 7), and summarize all the relevant features of the support: the type (steel meshes, rock bolts, steel sets or shotcrete), the quantity (e.g., number of rock bolts per linear metre), the parameters (thickness, strength and stiffness) as well as the distance of the support installation point from the working face. (With respect to the resolution of these two entities, the same remarks apply as the ones made in Section 3.1.1 for the entities $G$ (ground) and $TBM$.)
Figure 7. Layout of a gripper TBM and hazard scenarios in squeezing ground.
Due to the impossibility of stabilizing interventions in given sectors of the back-up area, the two entities \( P_A \) and \( P_B \) have been introduced in order to summarize the problems that may occur in these zones (zones A and B, respectively, Figure 7). Similar problems to those depicted in Figure 7 may also occur of course behind the back-up area, i.e., after the passage of the tunnelling equipment. For example, in one of the drives of the Yacambú – Quibor Tunnel (Venezuela, gripper TBM, \( D = 4.80 \) m) major heave of the tunnel floor was observed starting 50–100 m behind the TBM. Such cases have not been included in Figure 3 although such a situation may also have an impact on TBM operation (for example, major heave or twisting of the tracks as well as the execution of repair works may impair rail operations and, thus, the supply of construction materials or the mucking).

The selection of the type, quantity and location of support application represents an important operational decision for a gripper TBM drive. As long as tunnel stability and working safety are not endangered, a trade-off between excavation progress and the quantity of support in the machine area is thoroughly conceivable. Deciding to apply as little as possible tunnel support in the machine area (in order to proceed more rapidly \( \{5-17-20\} \)) results, as a rule, in a lower support stiffness and bearing capacity and this may lead to problems in zone A (Figure 7), which may also reduce the gross advance rate \( \{5-13-19-20\} \). Decision-making has to take into account the potential consequences of problems in the back-up area. A jamming of the equipment leads anyway to a standstill in TBM operation, while repair works after the passage of the equipment may often be carried out without slowing down the TBM drive very much.

A standstill may lead to an additional and longer standstill. During support installation, for example, the ground pressure acting upon the machine will increase \( \{17-10\} \) and, if the duration of the standstill \( t_1 \) is long or the development of the ground pressure fast, the required thrust force or torque (Equations 1 and 2) may become so high that an excavation restart is no longer possible \( \{1-18, 2-18, 10-18\} \) with the consequence that costly, time-consuming and sometimes also dangerous work is needed in order to free the TBM (\( t_2 \)). Problems in the above-mentioned zones A and B may also cause a TBM standstill, during which the TBM may become trapped \( \{e.g., 14-19-10-18\} \).

### 3.2 Single and double shielded TBMs

A similar \( N^2 \) chart to the one shown in Figure 3 could also be drawn for the case of single shielded TBMs. For the sake of economy, however, only the main differences between the two machine types will be discussed here. These differences include the TBM length, the thrusting system, the tunnel support and the advance rate.

Single shielded TBMs are longer than gripper machines. As the area exposed to the squeezing pressure is larger, a higher frictional resistance has to be overcome and, consequently, all other parameters being equal, the risk of shield jamming is higher (cf. Section 3.1.2). The disadvantage of a longer shield is, nevertheless, not of absolute significance because single shielded TBMs usually have a higher installed thrust force than gripper TBMs.

Instead of being thrusted via grippers, single shielded TBMs are jacked against segmental linings. The shield as well as the segmental lining have to be designed of course for the combined action of
maximum jacking forces and ground pressure, in order to avoid overstressing or inadmissible ovalization. The structural design of the segmental lining plays an important role, as its bearing capacity limits the thrust force and torque that can be applied. This effect can be represented in the \( N^2 \) chart of Figure 3 by adding three interactions \({5-7, 5-9, 5-18}\). Although the effect of the ground on the available thrust force and torque is not as important as in the case of gripper TBMs, in this regard the interactions \(\{1-7, 1-9, 1-18\} \) do not disappear completely because the quality of the annulus grouting depends also on the ground and an improper backfilling of the segments may reduce the capacity of the segmental lining to handle the jacking loads.

As described in Section 3.1.3, the type, quantity and installation points of tunnel support represent important operational parameters for a gripper TBM drive. For shielded TBMs, however, these parameters are pre-determined (a segmental lining of given thickness is installed in the rear part of the shield). Therefore, the \( N^2 \) chart can be simplified by eliminating the differentiations made with respect to the tunnel support \( (S_1 \) and \( S_2 \)) and to the locations of problems in the back-up area \( (P_A \) and \( P_B \)). Furthermore, due to pre-fabrication, the stiffness and the strength of the tunnel support do not change over time and, therefore, do not depend on the gross advance rate (i.e., the interactions \(\{16-5, 17-5, 18-5, 19-5, 16-6, 17-6, 18-6, 19-6\} \) do not exist).

Single shielded TBMs offer the advantage of a higher advance rate in poor quality ground (Peila and Pelizza, 2009), although feedback effects are possible for these machines as well. For example, high water inflows or unstable tunnel walls may make lining installation or annulus grouting difficult and, therefore, may slow down the advance rate.

Double shielded TBMs operating in gripper mode install the lining simultaneously with the boring process and, all other parameters being equal, therefore achieve a higher performance than single shielded machines. In the case of time-dependent ground behaviour, a higher advance rate is also advantageous with respect to the amount of shield skin friction to be overcome. Comparative studies should, nevertheless, take account of the fact that double shielded TBMs are in general longer than single shielded TBMs. Potential differences may exist, furthermore, concerning machine availability as double shields are more complex than single shields and necessitate a particularly careful and robust design in order to reduce maintenance times or breakdown times.

In weak ground, bracing by the grippers may become impossible. In this case, the machine operates in so-called “auxiliary mode” jacking against the segmental lining with the consequence that it is no longer possible to install the segments simultaneously with the boring (unless a hexagonal segmental lining is applied). Depending on the machine design, unstable ground may also impair the extension and closure of the telescopic joint, thus necessitating machine operation in single shield mode. In both cases (auxiliary mode and single shield mode) the same remarks apply as for the single shielded TBM and the potential advantages of double shielded TBMs mentioned above are lost.
4 Countermeasures

4.1 Introduction

The basic interactions discussed in the last section suggest not only how a given parameter is involved in the performance of the entire system, but also where it is possible to intervene, i.e., to apply measures in order to influence system behaviour. Section 4 will review possible countermeasures to deal with the problems associated with squeezing. As in the last section, reference will be made to the interactions of Figure 3, reporting the corresponding numbers within curled brackets.

Over the years, technological improvements in various components of the TBMs have extended their range of applicability. The next sections aim to evaluate not only the well-established methods but also alternative lining and machine concepts, which have been proposed specifically for coping with squeezing ground. The order of this discussion reflects the location of the system components and, therefore, of the possible intervention points along the tunnel axis: the discussion starts with the ground ahead of the working face and in the machine area, continues with the TBM (cutter head, shield, thrusting system) and finishes with the back-up equipment and the tunnel support. Before doing so, however, some higher-level aspects, such as alignment, construction method and operational measures will be briefly addressed below.

The technical feasibility and the cost of a given measure (or package of measures) for dealing with the problems associated with squeezing depend on the number and the length of the tunnel stretches affected. If the geometry and the behaviour of critical geological zones are well-known, one would try first to reduce the length of the affected tunnel stretches by selecting another alignment in the planning phase. Such a route optimization presupposes knowledge of the geology with a degree of resolution that may be difficult to achieve particularly for long deep tunnels. Furthermore, it may lead to an unacceptably long tunnel or it may be impossible due to project constraints such as the location of the portals, access galleries or shafts, the minimum curve radiuses or the longitudinal gradients.

The conventional excavation of a critical section may lead to a reduction of the project schedule risks if it can be done in advance. However, this is only possible if the critical zone is well-known (position and length) and can be accessed via an auxiliary tunnel or a shaft. Switching to conventional excavation during a TBM drive is, as a rule, very difficult. Nevertheless, it represents an indispensable measure if the TBM is trapped and has to be freed. As a rule, this requires hand-mining over the shield or the construction of a by-pass tunnel – demanding operations, particularly for small boring diameters due to the very limited space available. Special measures have to be planned in advance, in order to reduce standstill time as much as possible. The potential delays and additional costs also have to be analyzed before construction and taken into account in the construction schedule and in the contractual regulations. This is particularly true in the case of a long drive through predominately competent rock, where the possibility of the TBM jamming in individual short fault zones may even be regarded as an acceptable risk.

A larger boring diameter offers more space for ground deformations, thus reducing the risk of a violation of the minimum clearance profile and, when combined with a yielding support (cf.
Section 4.7.2), will lead to lower ground pressures, thus reducing the risk of support overstressing as well \(2-13, 2-14\). The choice of a larger boring diameter also reduces the risk of shield jamming – this will be the case, however, only if it is combined with a larger overcut, i.e., with a larger gap between the tunnel wall and the extrados of the shield \(2-10\). Local enlargements of the boring diameter are often very problematic (cf. Section 4.3.2), but selecting a larger boring diameter for the entire tunnel may be a viable option particularly if squeezing conditions are expected to persist over a big percentage of the route. The financial viability of such a solution should be assessed case by case, since it will result in an unnecessarily large boring diameter in the tunnel stretches crossing competent rock \(\text{Amberg, 1992}\).

As mentioned in Sections 2 and 3, tunnelling practice as well as theoretical considerations indicate that maintaining a high overall advance rate may help with the problem of the TBM jamming due to squeezing ground. Operational measures and an appropriate construction site organisation are important for keeping the frequency and the duration of standstills low and thus the overall advance rate high \(4-17, 4-18, 4-19\). For example, if an identified critical zone has to be crossed, thorough maintenance work should be carried out in advance in order to reduce the risk of mechanical breakdowns \(17-19\) and the necessary logistical precautions should be taken to allow operations within the critical zone to be as continuous as possible \(4-17\). Such operational measures have been applied systematically, e.g., during the construction of the Wienerwald Tunnel \(\text{Austria, single shielded TBM, } D = 10.67 \text{ m}\) in order to reduce the risks in known fault zones \(\text{Matter et al., 2007}\).

Holiday periods (and, depending on local conditions, even perhaps the possibility and frequency of strikes) have also to be taken into account. In the case of an unexpected critical zone, reducing the amount of maintenance work may speed-up the TBM advance and help temporarily to avoid TBM trapping, but it will increase the risk of mechanical breakdown and thus the risk of an even longer standstill \(17-19\). Therefore, such a measure should not be applied as a matter of course.

Finally yet importantly, in addition to the technological and logistic aspects, the importance of the experience of the crew (the so-called “human factor”) should not be overlooked. This general truth is particularly relevant for dealing with adverse geotechnical conditions such as squeezing ground.

### 4.2 Pre-treatment or pre-support of the ground

The ground is (together with the TBM design, of course) the most important parameter for achieving a given gross advance rate. It represents in practical terms a purely input parameter because the possibilities of influencing its properties and behaviour are very limited in relation to the particular problem investigated in this report.

The pre-treatment of the ground, e.g., by grouting or drainage, can be carried out basically either before or during the TBM drive. The first solution is of course preferable (as the improvement work will not interfere with the TBM operation) but necessitates in nearly all cases the construction of an intermediate access tunnel or a pilot tunnel, which may also be technically demanding, costly and time-consuming. The pre-treatment of the ground from the TBM itself does not require auxiliary structures, but slows down the TBM drive considerably, which (as discussed in Sections 2 and 3) may also lead to critical situations (isolated short stretches with squeezing ground are more fa-
The applicability of grouting or drainage depends strongly on the ground characteristics. Squeezing grounds are unfavourable in this respect as they often have a high fraction of fines and, therefore, a low permeability. In addition, it has to be borne in mind that the cutter head and the shield, as well as the limited space available for the drilling equipment, impose geometric constraints on the layout of the boreholes. In the case of injection operations, the risk of cementing the cutter head has also to be investigated (Oreste and Peila, 2000). Chemical grout materials, if environmentally permitted, may be advantageous in this respect (Peila and Pelizza, 2009; Steiner, 2000). Another countermeasure would be to pull back the cutter head. This is, of course, only possible if the tunnel face is stable (Barla and Pelizza, 2000). Pore pressure relief by advance drainage is a highly effective measure for reducing deformations (Anagnostou, 2009a, 2009b) but investigations have to be performed in order to determine whether the technically feasible spacing and length of the boreholes are sufficient to achieve the necessary consolidation within an acceptable time period (Floria et al., 2008). In this respect, the heterogeneity of the ground permeability has also to be considered.

Concerning pre-support of the ground ahead of the working face, Einstein and Bobet (1997) suggested the application of jet grouting or of a steel pipe umbrella for TBM tunnelling through squeezing ground. Due to the limited space available for equipment, however, the execution of jet grouting is barely feasible and rarely used in mechanized tunnelling (Peila and Pelizza, 2009). The technical feasibility and effectiveness of pipe umbrellas are also questionable. Undoubtedly, forepoling would eliminate the risk of cave-ins, but would not reduce significantly the squeezing deformations or the load acting upon the shield (the stiffness of the steel pipes is very low).

Face bolts, which can be installed through openings in the cutter head (Lunardi and Focaracci, 2000), might limit core extrusion and represent a possible measure for overcoming short critical zones or for the case of long standstills (the time needed for the periodical installation of the bolts conflicts with the goal of maintaining a high advance rate and introduces a higher risk of TBM jamming). Furthermore, the effectiveness of such a measure is small because, as a rule, the openings in the cutter head do not allow for a close spacing of the bolts.

### 4.3 Cutter head

As discussed in Section 3.1.2, the design of the cutter head plays an important role with respect to the ground pressure acting upon it as well as the torque and thrust force required in order to keep the cutter head rotating and moving (during continuous excavation) or to get it restarted (after a standstill) {2-7, 2-9, 2-10, 2-12, 2-18).

The sticking of the cutter head during a standstill may be avoided by rotating it at regular time intervals. This should be regarded as an operational measure, which must be applied during longer standstills such as, e.g., holiday stops, as experience shows that these are particularly critical periods (cf. Section 2). In order to facilitate unlocking, the cutter head should be rotatable in both directions {2-12} (Güter, 2007).
Another measure for avoiding sticking of the cutter head where there is a major extrusion of ground at the working face is to pull back the cutter head or the TBM. For gripper TBMs this presupposes that the applied support does not impede a movement of the machine backwards (5-10, 5-12). Single or double shielded TBMs have to be designed so that the cutter head can be moved independently from the shield. Of course, a movement of the cutter head backwards is possible only if the friction caused by the radial ground pressure can be overcome, if the machine possesses a sufficient pull force and if the ground is able to provide a sufficient reaction force to the grippers (1-10, 2-10, 1-12, 2-12, 10-12, 10-18). Furthermore, during a standstill the cutter head may support the tunnel face. A careful evaluation of the face conditions is important, as moving back the cutter head may cause instability of the face (Barla and Pelizza, 2000).

The cutter head must also be stiff enough to guarantee an efficient boring process (Toolanen et al., 1993) and to allow for a full utilization of the installed torque (2-7, 2-9, 2-18). Such aspects are usually the responsibility of the TBM manufacturer. As a rule, it should be possible to assume that the related requirements are met.

4.3.1 Geometry

To reduce the friction between ground and cutter head it is first of all important that the cutter head does not have greatly protruding parts (Barla and Pelizza, 2000; Schmid, 2006). This applies also for the cutters which should protrude as little as necessary for the boring process (McCusker, 1996). Korbin (1998) recommends 80 mm cutter protrusion beyond the cutter head on the face and 50 mm on the gauge.

Keeping the size of the cutter head small in the axial direction reduces the area exposed to the ground pressure and is, therefore, favourable (Foster, 1997; Grandori and Antonini, 1994; Maidl et al., 2001). Moreover, the cutter head can be slightly "conical". A reasonable conicity according to Lovat (1997) would be about 13 % of the boring diameter.

Theoretical considerations show that a spherical shape of the tunnel face (which off course necessitates a spherical form of the cutter head) may reduce core extrusion and deformations in the machine area considerably (Moulton et al., 1995). On the other hand, curvature of the tunnel face is very unfavourable for its stability (Beckmann, 1984; Maidl et al., 2001; Steiner, 2000). Nowadays almost all TBMs have flat cutter heads.

4.3.2 Overboring

A moderate amount of squeezing can be accommodated by boring a larger profile, using one to three extendable gauge cutters (8-10, 8-13, 8-14). After Wolff and Goliashc (2003) and Toolanen et al. (1993), respectively, the first applications of an overboring system on full face hard rock TBMs were in the Shaft Project Lohberg (Germany, gripper TBM, \( D = 6.50 \) m) and in the Piedimonte Tunnel (Italy, gripper TBM, \( D = 5.86 \) m). The adaptation of the boring diameter in order to create space for expected ground deformations was proposed on a conceptual basis by Lombardi (1981).

The increase of the boring diameter \( \Delta D \) depends on the chosen system (2-8) and is, according to Wolff and Goliashc (2003), technologically limited to a maximum of 30 cm (Table 3a). Other au-
The authors suggest slightly different feasible values of $\Delta D$ (Table 3b). The amount of overboring does not depend on the boring diameter. Consequently, with increasing boring diameter, the ratio between overboring (allowed convergence) and boring diameter decreases and (since the ground pressure depends theoretically on this ratio) the efficiency of overboring also decreases.

Some authors suggest that allowing the occurrence of convergences may be counterproductive because the convergences cause softening or major loosening of the rock mass with a consequent increase in ground pressure or in the necessary support measures (Foster, 1997). As discussed by

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**Table 3. Technologically feasible overboring in diameter $\Delta D$.**

<table>
<thead>
<tr>
<th>(a) Overboring systems after Wolff and Goliash (2003)</th>
<th>$\Delta D$ [cm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fixed mounted gauge cutters</td>
<td>$\Delta D$ depends on the number of additional mounted gauge cutters</td>
</tr>
<tr>
<td>Manual extendable gauge cutters</td>
<td>Moving the gauge cutters outwards by introducing a plate under their mounting; after Herrenknecht (2003)</td>
</tr>
<tr>
<td></td>
<td>Moving the gauge cutters outwards by side-wise offset of their mounting; the final $\Delta D$ is achieved by moving the various gauge cutters stepwise; after Wirth (2003)</td>
</tr>
<tr>
<td>Hydraulic extendable gauge cutters</td>
<td>Rotatable, hydraulic movable gauge cutters (in their final position locked by pins); after Wirth (2003)</td>
</tr>
<tr>
<td></td>
<td>Telescopic, hydraulic movable gauge cutters (in their final position locked by pins); after Herrenknecht (2003)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>(b) Other sources</th>
<th>$\Delta D$ [cm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grandori (1993)</td>
<td>≤ 15–20</td>
</tr>
<tr>
<td>Barla (2001)</td>
<td>≤ 15–25</td>
</tr>
<tr>
<td>Voerckel (2001)</td>
<td>≤ 25</td>
</tr>
<tr>
<td>Vigl et al. (1999)</td>
<td>≤ 30</td>
</tr>
<tr>
<td>Hartwig (1995)</td>
<td>≤ 30–45</td>
</tr>
</tbody>
</table>
Kovári (1994), however, the load increase caused by loosening is of secondary importance for squeezing conditions. Consequently, a negative impact of the convergence on the ground characteristics (8-1) has not been introduced in the $N^2$ chart of Figure 3.

Wolff and Goliasch (2003) provided a detailed critical review of the different overboring systems and distinguished between the following types of additional gauge cutters (Table 3a): (a) fixed mounted; (b) manually extendable; (c) hydraulically extendable. The installation of the first type of gauge cutters requires a halt in excavation (8-17) and presupposes that appropriate, empty housings have been arranged in the cutter head (2-8), which are covered by steel plates during normal operation. The manual extension of the second type of gauge cutters – in steps of 25 mm (Downing et al., 2007) – also necessitates a standstill. The hydraulic extension of the third type of gauge cutters, however, can be carried out during excavation.

When applying overboring with shielded TBMs the centreline of the cutter head has to be lifted with respect to the centreline of the shield, in order to avoid sinking of the TBM (Rehm, 2005; Vigl and Jäger, 1997; Voerckel, 2001). The overboring can easily be handled by gripper TBMs (Voerckel, 2001). If the TBM is equipped with a cutter head shield, a re-positioning of its lower segment has to be carried out (Wolff and Goliasch, 2003). For all TBM types a re-positioning of the mucking buckets is also needed in order to ensure efficient muck removal during overboring (Schmid, 2008).

In order to avoid overstressing the gauge cutters (and, particularly, their supports), the rotational speed of the cutter head and the thrust force must be reduced during the overboring (8-7, 8-9) (Toolanen et al., 1993; Wolff and Goliasch, 2003) and this leads to a reduction of the net advance rate (7-16, 9-15-16). So, the overboring, if successful, leads on the one hand to a reduction of the ground pressure acting upon the cutter head and the shield (8-10). On the other hand, however, it causes a slow down of the TBM advance rate, which may lead to an increase of the TBM loading – either during the installation of the overboring facility (17-10) or during the boring process (16-10). Furthermore, it is particularly critical to ensure timely decision-making during construction. Determining the right point in time for initiating complicated overboring procedures is not easy, since the system has to be activated before encountering a critical zone and before the convergences become too large.

Overboring technology is not yet well developed and its value is very uncertain, at least for long reaches with squeezing conditions (ITA, 2003), and their successful application has yet rarely been achieved (Wolff and Goliasch, 2003). The trouble-free application of overboring only seems possible in rather soft rocks. The reliability of today’s overboring systems is in general critical (8-19), the ones with continuous adjustment of the boring diameter being, as a rule, the most sensitive (Gehring and Kogler, 1997; McCusker, 1996). The concentrated loads acting upon the extended gauge cutters (Schneider and Kapeller, 1995) or their abrupt loading due to falling blocks (Toolanen et al., 1993) are particularly critical. This was observed, for example, in the Northern Section of the Vereina Tunnel (Switzerland, gripper TBM, $D = 7.64$ m), where the overcutters were very susceptible to failure in hard or blocky rock (Hentschel, 1997). Concerning the reliability of overboring systems, the experiences from the Raron Section of the Lötschberg Base Tunnel (Switzerland, gripper TBM, $D = 9.43$ m) are interesting (Wolff and Goliasch, 2003). The boring diameter was increased by 20 cm in the Lias-limestones with a considerable effort. The enlargement of the bored profile happened within a 20 m long tunnel stretch by using two hydraulic extendable and two fixed mounted gauge cutters. Particularly critical was the steering of the TBM during the
enlargement phase as well as the very short lifetime of the four fixed mounted gauge cutters utilized to continue the excavation with overboring. Due to the considerable wear of the overboring system and the reduced penetration, one decided to reduce the overboring up to 10 cm in diameter and to realize it by shifting all gauge cutters and applying two fixed mounted extended gauge cutters. With this configuration, it was possible to drive about 860 m with a daily advance of 15.4 m.

Problems may also arise in heterogeneous rock masses where stretches with squeezing weak ground alternate with stretches of hard rock. The boring diameter should be enlarged before entering the critical squeezing zones, but this is only possible if the overboring system is able to bear the high load resulting from extending the gauge cutters within the hard rock stretch. Such a test has been carried out in the Bodio Section of the Gotthard Base Tunnel (Switzerland). In the initial phase of the TBM drive an enlargement of the boring diameter (gripper TBM, \(D = 8.80\) m) of 30 cm has been attempted within a stretch of 200 m of crystalline rock. The test has not been successful (Gollegger et al., 2009; Rehm, 2005; Vicenzi et al., 2007). The presence of mixed face conditions (hard and weak layers) was particularly unfavourable.

Another typical problem is represented by the blockage of the extendable gauge cutters in their start position due to their housings filling up with fine materials. In this case, it is still possible to extend them, but they do not rotate anymore and, therefore, wear faster and irregularly.

In the case of a large difference between the boring diameter and the diameter of the shield extrados, difficulties with the backfilling of the segmental lining may arise. On the one hand, the quantity of annulus grout is potentially larger. In this respect, questions arise concerning the costs and the supply. On the other hand, a grout flow towards the tunnel face is easier. After Gütter (2007), a solution might be to adapt the shield diameter to the overboring in such a way that the shield continuously supports the ground. In the same paper, however, this concept is rejected because of its technological complexity. Previous attempts to implement similar concepts have also been unsuccessful (cf. Section 4.4).

An alternative concept for a cutter head with a variable diameter was proposed by Baumann and Zischinsky (1993) of the company DMT (Deutsche Montan-Technologie). The first developed design was for applications in mining. The concept (Figure 8a) was later modified for a large cross-section \((D = 9.20\) m) and foresees a TBM with grippers placed near to the cutter head. The cutter head is composed of two radial adjustable arms (with four cutters each), is able to excavate non-circular profiles and allows for a continuous adjustment of the boring diameter depending on the thickness of the tunnel support. The boring process is carried out by applying the undercutting technique and requires, therefore, a pilot borehole to provide an additional free surface. The pilot borehole is bored, together with the enlargement of the main profile, by a small diameter "classical" cutter head (having seven cutters), which is located in the middle of the main cutter head. It is unknown whether such a machine has ever been applied. A first application was announced for a Canadian mine, but the applied machine was very different from the proposed one (Anonymus, 1993). Anyway, this concept seems to be very complex from the mechanical engineering point of view. The limited functionality and reliability of such a machine has to be expected.
4.4  Shield

The design of the shield, particularly its geometry, influences the thrust force that is required to overcome the skin friction (2-9, 2-10, 2-18).

The shield should be as short as possible (Lombardi, 1981). In order to reduce the surface of the TBM exposed to ground pressure to a minimum, Robbins (1997) proposed the use of gripper TBMs without either a canopy or a cutter head shield. Such a so-called "Open Top TBM" (Figure 8b) allows support installation immediately behind the cutter head and was applied (in non-squeezing conditions) in the Melbourne Underground Rail Loop (Australia, \(D = 6.90\) m) and in the Heitersberg Tunnel (Switzerland, \(D = 10.67\) m). The DMT-concept (Figure 8a) mentioned in Section 4.3.2 also represents an attempt in this direction. Anyway, gripper TBMs are generally equipped with a short shield having a length of about half a boring diameter. Improvements in TBM technology actually allow for a significant reduction in the machine length (to about 1–2 boring diameters) for single and double shielded TBMs as well. Such reductions in length can be realized more easily with larger tunnel diameters (see also the TBM technical data collected in Part III). For shielded TBMs, the shield length depends not only on the space needed for the equipment, but also on the size of the segments employed. After Foster (1997) an increase of the segment length necessitates an increase of double that value in the shield length. Bigger segments – nowadays with a width of up to 2.25 m (Stahn and Grimm, 2006) – allow for a higher gross advance rate because they reduce the operational standstill times (which is advantageous in squeezing ground (5-17-10)) but necessitate, at the same time, a longer shield (which is disadvantageous in squeezing ground (2-9, 2-10, 2-18)).

A slightly "conical" shield is also favourable (Lombardi, 1981). The conicity of today's single shielded TBMs amounts to 3–6 cm in diameter and is realised stepwise. Depending on their design, double shielded TBMs may have a slightly larger conicity of up to 10 cm in diameter. An increase in the conicity and, therefore, in the overcut as well, has a positive effect with respect to the risk of shield jamming as it provides more space for the deformations (Gehring and Kogler, 1997). The effectiveness of this measure increases with the ratio of overcut to boring diameter. This is a reliable measure, as it does not depend on the operability of mechanical parts. On the other hand, such a solution implies a larger boring diameter for the entire tunnel. In the stretches with non- or only slightly squeezing ground, a very wide annular gap may have to be filled (Dowden and Cass, 1991). Besides financial considerations, the potentially negative effect of the wide gap on the bearing capacity of the segmental lining has to be assessed from case to case. It should also be mentioned that the overcut and the conicity of the shield have first of all to permit the TBM to be steered (Amberg, 2008). If the gap between the shield and the ground becomes closed, steering the TBM may become difficult or even impossible. A change in the direction of the TBM will in this case cause a considerable constraint load on the shield, particularly if the latter is long.

Wittke-Schmitt et al. (2005) examined the feasibility of a TBM concept with a strongly conical single shield (conicity 60 cm in diameter, Figure 8c) in the planning phase of the Kallidromo Tunnel (Greece) and came to the conclusion that the risks and the additional costs were unacceptably high. More specifically, two aspects have been assessed as critical to the planning of single shielded TBMs \((D = 15.10\) m): the structural detailing and reliability of the tail-shield seal (due to its enormous thickness of 2.10 m) and the risk of the TBM trapping and the shield overloading in the case of standstills longer than 1–2 days (in spite of the strong conicity of the shield).
The surface of the shield should be as smooth as possible. A favourable construction, which avoids the stepwise increase of the rear shield diameter, is nowadays possible also for the telescopic part of double shields (Figure 9). Such a design may, nevertheless, present problems with the backfilling of the segments as it leads to a wide gap between the tunnel wall and the extrados of the rear shield (Burger, 2009).
The bearing capacity of the shield has to be high enough for the ground pressure \(10-18\), which acts in addition to the system loads (e.g., thrust force, torque, self weight). In this regard, structural reserves are recommended. The design load should be higher than the level of ground pressure that would lead to shield jamming. The probability of overloading with permanent damage to the shield can thus be reduced. Shield design should also take into account that the ground pressure may be markedly non-uniform both in the transversal and in the longitudinal direction.

An asymmetric loading may occur, for example, when approaching a weak zone, particularly if the latter crosses the alignment at a flat angle (Figure 10). This was observed, for example, in the Arrowhead Tunnels (USA, single shielded TBM, \(D = 5.82\) m). The shields employed in this project had been equipped with strain gauges in order to monitor their deformations and to back-calculate the ground pressure. This instrumentation made it possible to obtain useful information about the shield loading, for example during a three-day maintenance period, where the ground pressure increased so much that hand-mining was necessary in order to free the TBM \(17-10-18\). The standstill was due to maintenance work on the screw auger used for extracting the muck from the working chamber. On some occasions the efficiency of the mucking system was unsatisfactory, with the consequence that fine-grained muck became packed beneath the shield and the TBM began to climb. A closure of the radial gap between shield and tunnel wall may therefore also be caused by the packing of muck around the shield, if the material mucked out is less than the material produced by the excavation, i.e., if the capacity of the mucking system is not attuned to the TBM advance rate. An understanding of the TBM's operational parameters is also important in this regard. Packing of fines around the shield reduces the space available for deformations and may therefore lead to an earlier development of ground pressure upon the shield.

A higher deformability of the shield leads, as a rule, to a decrease of the ground pressure acting upon it. Lombardi (1981) has remarked that having the possibility to reduce the diameter of the shield might be helpful in restarting the TBM after a standstill \(2-10\). Grandori et al. (1995) proposed a flexible design of the rear part of the shield. The underlying idea was that, in the case of jamming, a small contraction of the shield should suffice in order to free it and continue the TBM drive. The shield should return back to its initial, undeformed condition as soon as possible in order to allow for any subsequent exploitation of its deformability as an unloading measure (Dowden and Cass, 1991). The application of this principle presupposes the choice of a bigger boring diameter or, at least locally, the acceptance of an under-profile with the associated re-profiling works. This

![Figure 9. Construction schemes for the telescopic shield of double shielded TBMs: (a) “classic” design; (b) modified design after Concilia and Grandori (2004).](image)
may be problematic particularly for segmental linings, as they impose a lower limit to the inner diameter of the shield.

Another TBM concept proposed by Robbins (1997) is characterized by a shield that is divided in several, 1.0–1.5 m wide "blades" (Figure 8d). This so-called "Walking Gripper Blade Shield" should be able to exert a support pressure on the surrounding ground of up to 2 MPa while advancing in steps of 10–15 cm. The blades form a deformable system that is able to contract inwards 7–10 cm. It is generally recognised that the complexity of such a machine has a negative impact of its functionality and reliability. The presence of several moving parts also increases the difficulties encountered when driving stretches in flowing or ravelling ground. The risk of the cutter head sticking or the shield jamming is still there, particularly in the case of a standstill. Despite the large surface of the grippers (i.e., of the blades), it is uncertain if the bracing of the TBM will be sufficient in weak ground. SNC Lavalin applied this concept by modifying a double shielded TBM ($D = 2.91$ m) for driving 1183 m of the Stillwater Tunnel in the USA in lightly squeezing ground. Problems have been reported with respect to blocky ground, steering of the machine and sealing of the main beam. Westfalia Lünen drove 1650 m of the exploratory tunnel ($D = 5.20$) of the Freudenstein Tunnel (Germany). In this case squeezing ground was neither expected nor encountered (Klonsdorf and Schaser, 1991), but the TBM drive was anyway unsatisfactory as the machine proved to be extremely sensitive to the presence of water (Maidl et al., 2001). Furthermore, the TBM drive was hindered by clogging of the cutter head (Babendererde, 1989) and suffered damage on several occasions (Klonsdorf and Schaser, 1991). A similar TBM concept was proposed, but not applied, by Moulton et al. (1995) for the drive of the Yacambú – Quibor Tunnel (Venezuela, $D = 5.00$ m).

Attempts to construct shrinkable shields for single shielded TBMs are still not satisfactory (Downing et al., 2007), although the implementation for gripper TBMs does seem to be possible. One example of a gripper TBM with a deformable shield is the gripper TBM ($D = 7.64$) that has driven the Zugwald Tunnel and the Northern Section of the Vereina Tunnel (Switzerland). This machine was equipped with a shrinkable cutter head shield, where hydraulic jacks allowed for a reduction of the shield diameter by up to 15 cm. Further examples include the gripper TBM ($D = 9.53$ m) of the
Tscharner Tunnel (Switzerland) – which is described in Hentschel (2000) – as well as the two gripper TBMs ($D = 8.80$) of the Bodio Section of the Gotthard Base Tunnel (Switzerland). The design of the Gotthard machines permitted a reduction of up to 20 cm in the shield diameter. The TBMs have been modified and are currently driving the Faido Section of the same tunnel ($D = 9.43$ m). Two further examples of such gripper TBMs – Tunnel Le Pougeot (France, $D = 5.05$ m) and Shaft Project Lohberg (Germany, $D = 6.50$ m) – can be found in Eistert (1982). Such a gripper TBM ($D = 3.20$) has also been used for one of the adits of the Stillwater Tunnel (USA).

Robbins (1997) proposed another gripper TBM concept (never used), which was the so-called "Low Pressure Walking Blade Canopy" (Figure 8e). In this concept the upper part of the cutter head shield of the gripper TBM would be deformable (a blade shield with the capability of exerting a support pressure) while the bottom part would be practically rigid.

For the friction between shield skin and ground a coefficient according to Table 4 can be assumed. In the Arrowhead Tunnels (USA), a field test along the first 15 m of the TBM drive (single shielded TBM, $D = 5.82$ m) has been carried out. The skin friction coefficients determined in situ amounted between 0.37 and 0.45 and agreed well with the values for rock-steel-contact reported in Table 4. As expected, the higher values have been registered when starting up the TBM (static instead sliding friction). The injection of lubricants such as bentonite can reduce considerably the friction between shield and ground (Table 4). The lubricant is injected through the shield (2-9, 2-18). A thin lubricant film suffices to reduce skin friction. Therefore the bentonite should be sprayed and not be pumped, in order to reduce the amount needed and the risk of an uncontrolled flow of the suspension towards the cutter head (Schmid, 2008).

Table 4. Skin friction coefficient for sliding and static friction (i.e., during TBM advance and for restart, respectively) with and without lubrication by bentonite after Gehring (1996), Herzog (1985) and Pohl (1979).

<table>
<thead>
<tr>
<th>State</th>
<th>Sliding friction (continuous excavation)</th>
<th>Static friction (restart after a standstill)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Lubrication</td>
<td>Not lubricated</td>
</tr>
<tr>
<td>Rock</td>
<td>0.25–0.30</td>
<td>0.10–0.15</td>
</tr>
<tr>
<td>Gravel</td>
<td>0.25–0.30</td>
<td>0.15</td>
</tr>
<tr>
<td>Sand</td>
<td>0.35–0.40</td>
<td>0.15</td>
</tr>
<tr>
<td>Silt</td>
<td>0.35–0.40</td>
<td>0.10</td>
</tr>
<tr>
<td>Clay</td>
<td>0.30–0.35</td>
<td>0.10</td>
</tr>
</tbody>
</table>

As a further measure for single and double shielded TBMs, Grandori (1996) as well as Vigl and Jäger (1997) remarked that it is advantageous to have the possibility of accessing the rock mass through the shield in order to be able to intervene (e.g., to free the TBM by hand-mining) whenever required. According to Grandori (2001) this is possible for double shielded TBMs through openings in the shield or through a complete opening of the telescopic shield. Before initiating such an operation, a careful evaluation should be made as to whether the stability of the rock mass is sufficient for ensuring working safety.
4.5 Thrust force and torque

Improvements in TBM technology allow the installation of higher thrust forces and torques.

For gripper TBMs, the achievable reaction forces have to be borne in mind when selecting the installed thrust force and torque. For given ground parameters \{1-7, 1-9, 1-18\}, the achievable reaction forces depend primarily on the dimensions of the grippers and on the installed gripper force \{2-7, 2-9, 2-18\} (cf. Section 3.1.2). The total gripper force (perpendicularly to the tunnel axis) is, as a rule, higher by a factor 2.0 to 3.5 than the installed thrust force. An actual gripper TBM with a boring diameter of about 10 m can dispose of an installed thrust force of about 30–35 MN and a breakout torque of about 10–15 MNm.

For shielded TBMs with a similar boring diameter, a thrust force of up to 150–200 MN and a breakout torque of up to 30–40 MNm are nowadays feasible. As a temporary solution, the thrust force can be increased on the construction site by installing removable auxiliary hydraulic jacks. This has been done, for example, in the Uluabat Tunnel (Turkey, single shielded TBM, \(D = 5.05\) m), in the Arrowhead Project (USA, single shielded TBM, \(D = 5.82\) m) and in the Section 4 of the Pajares Tunnel (Spain, single shielded TBM, \(D = 9.88\) m). The TBM of the Uluabat Tunnel had an installed thrust force of 27 MN, which has been temporarily increased to 46 MN. The TBMs of the Arrowhead Project had an installed thrust force of 29 MN for normal operation and a maximum possible thrust force of 37.5 MN. After a modification of the TBMs on site, the installed thrust force has been increased to 58.5 MN. Furthermore, a thrust force of up to 114 MN could be achieved temporarily by applying 11 removable auxiliary hydraulic jacks. In the Pajares Tunnel the TBM became trapped in squeezing ground in spite of the very high installed thrust force of 193 MN. The thrust force was temporarily increased to 225 MN in order to resume the excavation. In the first project, the segmental lining became damaged, while in the second and in the third project, it has already been able to bear the higher axial loads.

As reported by Iwasaki et al. (1999), Shimaya (2005) and Terada et al. (2008), a design combining the thrusting systems of gripper and single shielded TBM (Figure 8f) was developed and applied for the drive of the Hida Tunnel (Japan, \(D = 12.84\) m). Along the entire TBM drive, a 45 cm thick segment was installed in the tunnel floor. The upper part of the bored profile was supported, depending on the ground conditions encountered, either with rock bolts and shotcrete or with seven, 25 cm thick segments. The thrusting mode (i.e., grippers or hydraulic jacks) was chosen according to the support applied. The possibility of installing a segmental lining was foreseen in order to achieve a higher thrust force (55 MN in single shielded TBM mode vs. 33 MN in gripper TBM mode) and to achieve a faster (and, therefore, favourable \{16-10, 17-10\}) TBM advance in difficult ground conditions, including but not limited to squeezing ground. The papers referred to do not describe any practical experience with squeezing ground that might or might not confirm the suitability of this concept.
4.6 Back-up equipment

The production rate of the TBM may also be affected by the performance and layout of the back-up equipment. From a logistical point of view, it is very important that supplies to the machine area and mucking out, as well as all activities in the back-up area (e.g., casting of an invert arch), do not affect TBM performance (3-17). A robust design of the back-up equipment is important for reducing mechanical breakdowns, which might also affect TBM advance (3-19).

Problems in the back-up area may also slow down or even halt the TBM drive (13-19, 14-19). Jamming of the back-up equipment is particularly critical, as it always leads to the machine being stopped and, possibly, further problems as well (19-10, 19-13, 19-14). Therefore, it is a good idea to make the back-up trailers small enough relatively to the boring diameter to gain some space in case of unexpectedly high ground deformations in the back-up area (3-13, 3-14), i.e., if the support installed should not suffice to prevent ground deformations (5-13, 5-14, 6-14).

It has also to be borne in mind that the options for intervention in the back-up area are limited to certain areas only, which are pre-determined by the design of the back-up equipment. As discussed in Section 3.1.3, this is particularly important for gripper TBMs. These constraints must be considered when planning a TBM drive for squeezing ground and, where necessary, requirements must be formulated in the TBM specifications.

4.7 Tunnel support

Concerning conceptual design of tunnel support in squeezing ground, a distinction is usually made between two principles: the "resistance principle" and the "yielding principle" (Kovári, 1998). In the first case, one applies a practically rigid lining, which must be strong enough to bear the ground pressure developing when preventing ground deformations. In the second case, one avoids the development of excessive ground pressures by accepting a certain amount of ground deformation. In reality, the interaction between lining and ground leads as a rule to a situation that is somewhere between the two extremes. For the sake of simplicity, however, the following discussion will retain the distinction between a practically rigid and a yielding support.

It should be noted that the tunnel support – besides ensuring stability and limiting deformations of the tunnel – may also reduce shield loading. The installation of a stiff support immediately behind the shield improves load transfer by arching in the longitudinal direction (Figure 11), thus leading to a reduction of the ground pressure acting upon the shield (5-10) (Anagnostou and Ramoni, 2007). This effect is more pronounced for short shields and stiff linings according to the "resistance principle". On the other hand, the load developing upon a stiff lining installed immediately behind the shield is bound to be higher than the load upon a more deformable support installed at a greater distance behind the face.
4.7.1 Practically rigid supports

Segmental linings (the standard support with shielded TBMs) can be classified – on the basis of their high stiffness and strength – as tunnel supports which follow the "resistance principle". Prefabrication makes it possible to achieve high uniaxial compressive strengths in the concrete (up to 50 MPa or even more) with sufficient levels of reliability and thus the ability to cope with high ground pressures. In order to be able to utilize fully the installed thrust force and torque, the lining design has to take into account of course the combined action of ground pressure and TBM forces. A proper and fast-acting embedment of the segmental lining is very important for its bearing capacity. Rapidly setting, two-component grouts (applied through the shield tail as in soft ground TBMs) may be advantageous in this respect.

For shielded TBMs, it has also been proposed to use the segmental lining to support the tail shield in the case of major squeezing. For the Wienerwald Tunnel (Austria, single shielded TBM, \( D = 10.67 \text{ m} \)) this has been planned as a counter measure for unexpected stops of the TBM drive in squeezing ground (Matter et al., 2007). As a similar special measure, Gütter (2007) suggested inserting air cushions between the tail shield and the segmental lining in order to provide a temporary support to the tail shield, which, as a rule, is thinner and less stiff than the rest of the shield.

As an alternative for implementing the "resistance principle", Babendererde (1986) proposed the use of extruded concrete: fluid concrete is extruded at a constant pressure directly behind the TBM, thus filling the gap between the bored profile and a slipform continuously during TBM advance. The advance rate is regulated with the quantity of pumped concrete. An advantage of this solution is the lower cost of ordinary concrete compared to precast segments. Furthermore, the continuity of the concrete extrusion allows for a continuous and, therefore, fast advance of the TBM drive. Additionally, no backfilling material is required and a proper bedding of the lining is assured. Nelson et al. (1992) and Einstein and Bobet (1997) also emphasized the advantages of a machine employing extruded concrete, particularly in combination with a shorter shield than the one of Babendererde (1986), and proposed the so-called "MIT Continuous TBM" (Figure 8g). The papers referred to do not describe any case histories where these TBM concepts have been applied in squeezing ground. In spite of the practical applications reported in Babendererde and Babendererde (2001), extruded concrete has not been established in tunnelling practice. One reason for this seems to be the high mechanical complexity of these machines. Furthermore, it is questionable whether the

![Figure 11. Arch action in longitudinal direction around the shield in the presence of a rigid lining.](image)
thrust force requirements for squeezing ground can be satisfied. A high thrust force would not be possible, as it would increase the fluid concrete pressure and this would be problematic for the bearing capacity and the sealing of the slipform.

In the case of gripper TBMs, the applicability of the "resistance principle" is more limited than in shielded TBMs because it is difficult to achieve the same high quality and resistance for the shotcrete applied in the machine area as in the case of a prefabricated segmental lining. In order to achieve a comparably high support resistance, a large quantity of shotcrete must be applied and this slows down the advance rate considerably. Furthermore, the load on the tunnel support in the machine area increases with the advance of the tunnel face and this happens simultaneously with the hardening of the shotcrete. The higher the gross advance rate, the lower will be the strength and stiffness of the shotcrete in the machine area \(16-5, 17-5\) and, therefore, the less the shotcrete will contribute to the load bearing action in the longitudinal direction around the shield (Figure 11).

This is unfavourable for the shield \(5-10\). During long standstills, however, the longitudinal arch action between core and tunnel support can be enhanced by building a thicker shotcrete ring (30 cm or more) immediately behind the shield. This leads to lower shield loads, as discussed above. Such a precautionary measure was applied, e.g., in the Faido Section of the Gotthard Base Tunnel (Switzerland, gripper TBM, \(D = 9.43\) m) during the holiday break of Easter 2008.

4.7.2 Yielding supports

In tunnelling with gripper TBMs, the same yielding supports can be applied as for conventional tunnelling. The Northern Section of the Vereina Tunnel (Switzerland, \(D = 7.64\) m) and the Sections Amsteg \((D = 9.58\) m) and Faido \((D = 9.43\) m) of the Gotthard Base Tunnel (Switzerland) may be mentioned as practical examples.

The applicability range of the "yielding principle" is, nevertheless, strongly limited by the fixed geometry of the tunnelling equipment and the required clearance profile. The design of the yielding support must be considered when selecting the boring diameter and the dimensions of the back-up equipment in order to avoid costly (and sometimes dangerous) re-profiling works or jamming of the back-up equipment \(3-13, 5-13, 3-14, 5-14, 6-14\). The continuous adjustment of the boring diameter using overboring techniques is still not sufficiently reliable today (cf. Section 4.3.2) and, if at all feasible, the increase that can be achieved in the boring diameter is also limited (Table 3). On the other hand, the choice of a fixed, but larger boring diameter for the entire tunnel is often not economical.

It should be also noted, that the deformability of the lining leads to a reduction of its loading, but weakens the longitudinal arch action and thus leads to an increase in the ground pressure acting upon the shield \(5-10\). Furthermore, allowing larger deformations may lead to major loosening phenomena or to a softening of the ground. This has to be taking into account in the design of the deformable tunnel support. In addition to an appropriate structural detailing, a sufficiently high yield pressure is very important for safety (Anagnostou and Cantieni, 2007; Cantieni and Anagnostou, 2009b), but may be difficult to be achieved in combination with shotcrete because (in contrast to conventional tunnelling) the advance rates are high relative to the time needed for shotcrete hardening. Steel sets with sliding connections are advantageous in this respect. Furthermore, long bolts \((> 5–6\) m for common cross-sections of traffic tunnels) are indispensable, particularly for coping
with non-uniform rock convergences and in order to ensure the stability of deformable tunnel supports during the yielding phase. It should be noted that systematic bolting (in contrast to shotcrete or steel sets) does not consume bored space, thus leaving more space free for the ground deformations to occur. The support pressure achievable by bolts is, nevertheless, rather low (0.1–0.3 MPa).

In tunnelling with shielded TBMs, coping with squeezing pressure may necessitate very thick segments, which, besides increasing the required boring diameter, are difficult to handle. Deformable segmental lining systems specifically for shielded TBMs have therefore been the subject of intensive past and current research and development (e.g., Billig et al., 2007a; Schneider et al., 2005; Vigl, 2003). Such deformable linings could be applied either with a "conventional" TBM or, as proposed by Baumann and Zischinsky (1993), Robbins (1997) and Wittke-Schmitt et al. (2005), in combination with the alternative TBM concepts illustrated in Figure 8a–e. A deformable segmental lining can be realized basically in two ways: either, (i), by arranging a compressible layer between the ground and the lining (the ground experiences convergences, while the deformations of the lining remain small, Figure 12a); or, (ii), by arranging special deformable elements in the longitudinal joints of the lining that allow for a reduction of its circumference (Figure 12b).

**Yielding layers between segmental lining and ground**

The basic idea has been proposed and was patented in England in 1979 – J. Mowlem, UK Patent application GB 2013 757 A, cf. Schneider et al. (2005). For a TBM drive through swelling ground, Lombardi (1981) proposed the application of a compressible layer consisting of polyurethane foam. Wittke-Schmitt et al. (2005) investigated the possibility of using expanded clay as a backfilling material in combination with the alternative TBM concept of Figure 8c. (Note that in order to allow radial ground deformations of 1.20 m and considering the deformability of the expanded clay, the concept requires a radial annular gap of 2.40 m, thus leading to a boring diameter of 15.10 m.) Vigl (2003) presented a "convergence-compatible" segmental lining. The segments in this so-called "CO-CO-system" incorporate at their extrados supporting ribs which are in contact with the rock.
The ground is allowed to squeeze into the space between the ribs, which can be either empty or filled by a compressible material. Another possibility is given by the addition of a compressible layer fixed at the extrados of the segments in combination with a traditional annular grouting or a compressible grout (Billig et al., 2007a; Schneider et al., 2005).

A compressible annulus grouting material must have, with the exception of a high deformability of course, all of the other usual properties of gap grouting materials: easy processing, pumpability and high stability of the material. For these and for economic reasons light weight concrete is usually proposed (Strohhäusl, 1996). Schneider et al. (2005) developed the so-called "Compex", a compressible mortar with expanded polystyrene that can be compressed up to 50 %. Billig et al. (2007a) reported about the development of the so-called "DeCo Grout", a cement-based pumpable mortar with expanded polystyrene pearls and foam, which is also characterized by a maximum compression of about 50 %.

**Deformable longitudinal joint elements**

Wood was often used in the past as a compressible element in mining (Figure 13a) and it is interesting to note that it was also applied in combination with precast concrete elements many years ago (Lenk, 1931). Recent, mainly experimental, attempts to increase the flexibility of precast segmental linings utilize neoprene elements or hydraulic devices, which are arranged in the longitudinal joints (Figure 13b–c).

Brunar and Powondra (1985) reported on the development of the so-called "Meypo deformable elements", which should be placed in the longitudinal joints of a segmental lining and allow for a reduction of its circumference by 1.80 m (6 joints, each experiencing a compression by 30 cm). According to the authors, these elements have been designed with such a high yield load (3 MN) that the lining already offers a considerable support pressure at small ground deformations and this is important in order to avoid loosening or ravelling of the ground. The Meypo deformable elements have been applied in a tunnel in the Ibbenbüren Coalmine in Germany at a depth of about 1500 m (inner diameter of the segmental lining 9.47 m). In this case, however, the tunnel was driven conventionally, support during excavation consisted of shotcrete and rock bolts and the segmental lin-
ing was applied as a final tunnel support. The deformable elements fulfilled expectations, but it was also realized that the costs of such a solution would be too high for it to be systematically applied in tunnelling (Maidl et al., 2001). The suggestion was therefore made of deploying reusable deformable elements, which must be removed after the ground has deformed and before setting the system rigid by applying shotcrete into the longitudinal slots. Baumann and Zischinski (1993) proposed the use of hydraulic jacks for this purpose. These are also expensive, slowing down installation considerably and necessitating heavy reinforcement in order to overcome burst and shear forces.

As reported by Croci (1986), hydraulic jacks have also been applied earlier in the Tunnel Santomarco (Italy). In a section with squeezing crystalline and phyllitic rock, excavation was carried out mechanically under the protection of a shield (Caldarella, 1986). In order to reduce the load acting upon the segmental lining (45 cm thick, inner diameter 7 m) hydraulic jacks were introduced into the longitudinal joints in combination with deformable neoprene elements, which were glued onto the segments and allowed a convergence of 1%. This application had mainly an experimental character. After 2.5 months, it became necessary to strengthen the tunnel support by applying steel sets and shotcrete.

The technical literature also includes other types of deformable elements. So, for example, Strohhäusl (1996) proposed plastic bodies, which are made from the same material as sealing gaskets and have cavities which can be filled with more or less stiff materials like polyurethane or lightweight concrete. The mechanical properties of these deformable elements can be regulated by determining the volume and the infillings of the cavities. Another possibility is the application of steel or plastic pipes, which become ovalized at a certain hoop force (Maidl et al., 2001). According to a method patented by Tusch and Thompson (1996), the steel tubes (Figure 13d) should be filled by a fluid and, in order to control the hoop force, be equipped with pressure valves. Another technical option are the highly deformable concrete elements (Figure 13e) proposed by Kovári (2005), which are composed of a mixture of cement, steel fibres and hollow glass particles and collapse at a pre-defined compressive stress depending on the composition of the concrete.

4.7.3 On the appropriate support concept

Apart from a 20 m long successful test drive in no squeezing ground (Gamper et al., 2009; Schneider and Spiegl, 2009) accomplished with the "Compex" compressible mortar (c.f. Section 4.7.2) in the framework of the Jenbach Tunnel (Austria, mixshield, \(D = 13.00\) m), up until today, in spite of considerable research and development efforts, no successful implementation of the "yielding principle" has been achieved for a shielded TBM with a segmental lining (Schneider and Spiegl, 2008). The reasons for this can be traced back to some critical aspects, which are common to all proposed concepts.

There is, first of all, a fundamental difference between implementing the yielding principle in a gripper TBM (or in conventional tunnelling) and implementing the yielding principle in a shielded TBM. In the first case, the tunnel support (which consists usually of shotcrete, steel sets and bolts) has as a rule only a temporary function: it has only to ensure stability and to preserve the shape of the opening during construction. After construction of the final lining by cast-in-situ concrete, the condition of the temporary support (whether heavily cracked or deformed) is absolutely irrelevant. This is not true for a tunnel support by precast segments as this support represents in most cases (with
the exception of a few Swiss tunnels having a second inner lining of cast-in-situ concrete) the final lining as well and has, in general, to fulfil two requirements in addition to safety: the geometry of the clearance profile must not deviate too much from the theoretical one and, as a rule, the lining must be practically waterproof. The uncertainties are large with respect to these objectives, because a flexible lining adjusts its shape to the ground deformations and the distribution of the latter along the circumference of the opening cannot be predicted with sufficient reliability. This is true not only for segmental linings that stay in contact with the ground and consequently follow the ground deformations (i.e., linings incorporating yielding elements in the longitudinal joints), but also for the other type of yielding supports, i.e., for segmental linings that are embedded within the compressible material used for the annulus grouting. Such a system is very vulnerable to non-uniformly distributed pressures. An asymmetrically squeezing ground can displace the lining entirely.

In order to ensure that the clearance profile will not be violated, sufficient tolerances and thus a larger boring diameter are necessary. (According to Schneider and Spiegl (2008), the machine should allow for adjustments in the size of the annular gap.) Concerning continuous adjustments in the gap size, the same critical remarks apply as for the gripper TBM. Furthermore, a reliable forecast of ground deformations is often difficult. An underestimation of the gap size will cause over stressing of the support. This applies of course to gripper TBM as well. The difference is, again, that it may be acceptable to exploit a large percentage of the bearing capacity for a temporary structure (shotcrete shell, steel sets, bolts), but for a final structure it is not. A segmental lining has to fulfil the normal standards for permanent civil engineering works.

In addition to the fundamental issues addressed above, there is a series of specific disadvantages. So, for example, systems comprising deformable joints are very costly (in terms of materials and time), particularly if the tunnel lining must be waterproof and, consequently, the deformable elements need to provide a sealing function (another difficult problem, which is, nevertheless, irrelevant to the temporary support applied when employing a gripper TBM).

Questions also arise with respect to the other type of deformable segmental linings (employing a compressible layer between lining and rock). The characteristics of the backfilling material must be such that it is able to deform but at the same time it should be stiff enough to assure a proper bedding of the segmental lining. In general, the application of a compressible layer at the extrados of the segmental lining weakens its bedding (McCusker, 1996) and decreases, therefore, its bearing capacity with respect to non-uniformly distributed loads. A highly compressible backfill decreases the hoop forces but markedly increases the bending moments in the segments (cf., e.g., Graziani et al., 2007a). This problem is probably less important for the system proposed by Vigl (2003) which, as mentioned above, foresees ribs at the lining extrados (on the other hand, the concentrated loads arising from the ribs in this system need a high quantity of shear reinforcement). Billig et al. (2007b) acknowledged the "bedding problem" and proposed adjusting the time-dependent development of the stiffness and strength of the compressible mortar by using an appropriate mixture – a demanding task under construction site conditions.

Finally, a low stiffness of the annulus material may also present problems with respect to the jacking forces. Small, practically unavoidable deviations from the axial direction may push the lining aside. This is particularly true in the case of a rapidly squeezing ground that exerts a considerable load on the shield, thus necessitating a high thrust force in order to keep the machine advancing. In this respect, it is worth remembering that the ground pressure which develops upon the shield
when applying a yielding support is in general higher than in the case of a practically rigid lining (5-10), as the latter facilitates arch action in the longitudinal direction around the shield. A compressible layer around the segmental lining may therefore affect system behaviour in that, (i), it reduces the squeezing pressure acting upon the lining but, (ii), increases the ground loading of the shield and, (iii), this means that it may be necessary to increase the jacking forces, while, (iv), at the same time, the bearing capacity and the stiffness of the lining will in general be lower due to the weaker embedment.

It should be noted, furthermore, that it is questionable if a high deformability of the segmental lining is at all necessary. As indicated by numerical simulations, jamming of the shield (rather than over-stressing of the lining) is the relevant hazard scenario in many cases: under adverse conditions squeezing will halt the TBM advance before endangering the structural safety of the segmental lining.

Figure 14 shows computational results obtained for a 10 m long, single shielded TBM boring a 10 m diameter tunnel in weak sedimentary rock. The computational method is described in Part II.
of the present report and assumes axial symmetry and linearly elastic, perfectly plastic ground behaviour with the Mohr-Coulomb yield criterion. The diagram shows the effect of the depth of cover $H$ on the risk of shield jamming or lining overstressing (solid and dashed lines, respectively). The risk of shield jamming is expressed by the ratio of the installed thrust force $F_i$ to the force $F_f$ needed for overcoming friction. The risk of overstressing is expressed by the ratio of the allowable loading of the lining $\sigma_{l,max}$ to the ground pressure $\sigma_l$ developing far behind the face. For an installed thrust force $F_i = 150$ MN (a high but nevertheless achievable value, see also the TBM technical data collected in Part III) and a uniaxial compressive strength for the lining of $f_{c,l} = 25$ MPa (a design value already incorporating a safety factor), the diagram shows that the ground would immobilize the TBM at a depth of about 300 m, which is slightly lower than the depth at which overstressing of the 35 cm thick lining would occur. By installing a higher thrust force of 250 MN – which, according to Burger (2009), is technologically possible for common boring diameters of traffic tunnels applying high-pressure hydraulic systems – and by taking additional measures to reduce skin friction (lubrication), jamming would not occur until the depth of cover reached about 500 m, while the bearing capacity of the lining would become relevant at a depth of 300–350 m. Lining design for greater depths should also take into account the higher axial load caused by the thrust force. A solution based on the resistance principle would be, nevertheless, still possible in the present example: one could increase the thickness of the segments (which is feasible up to 70 cm) or apply segments made of high performance concrete (HPC) having a very high uniaxial compressive strength (a design value of up to 50–60 MPa).

Should it be necessary to further increase the resistance of the segmental lining (beyond the resistance offered by segments of manageable thickness and weight), it is possible to design a lining system consisting of two concentric segment rings (Figure 15). The inner segmental lining would be applied (by the same erector) only when required. The outer ring could be made of HPC, while normal concrete would be advantageous for the inner ring (on account of its higher fire resistance). Depending on the water pressures in the project area, it might be also possible for the inner ring alone to be waterproof. A solution with a double segmental lining makes sense only in heavily squeezing ground and should, therefore, be combined with a minimum possible shield length in or-

Figure 15. Double segmental lining.
order to reduce the surface exposed to ground pressure and to utilize the favourable longitudinal arching (Figure 11).

The above discussion suggests that shield jamming limits feasibility to a larger extent than lining overstressing. It could be argued, however, (i), that shield jamming, in contrast to segmental lining damage, may be regarded as an acceptable risk and, (ii), that if the deformations develop slowly, the shield may advance without any problems even if a high rock load develops far behind the face (in this case too, lining overstressing rather shield jamming would be the relevant factor). Concerning the first point, it is impossible to make a generally valid statement as the acceptance of jamming depends of course on the potential damage to the TBM as well as on the frequency of the events necessitating hand-mining for freeing the TBM. With regard to the second objection, it should be noted that estimating the intensity of squeezing is an extremely uncertain task in the planning phase and, consequently, decision-making should make some contingency for rapidly developing deformations (such as are occasionally observed, see Table 1). This is also why the computational methods employed in engineering practice (during design for the assessment of squeezing) usually do not take into account the time-dependency of ground behaviour.

In conclusion, the traditional, practically rigid segmental lining seems to be the appropriate solution for shielded TBMs crossing squeezing ground: precast segments can bear high ground pressures and, if properly bedded, high thrust forces as well. Indeed, it seems to be a contradiction to incorporate expensive, high-quality precast segments (which are constructed with tolerances of 1–2 mm) into a support system that may experience uncontrollable deformations of several cm – 10–20 cm radial convergence after Schneider et al. (2005) and Billig et al. (2007a). The large flexibility of deformable segmental linings is favourable with respect to their structural safety but (almost by definition) unfavourable as regards serviceability requirements, which as said above are particularly important given the permanent character of the structure.

For gripper TBMs, however, yielding supports appear to be more attractive: the serviceability requirements are of secondary importance for temporary supports; the implementation of the "yielding principle" in a gripper TBM bears less risks than in a shielded TBM because the thrusting system does not depend on the behaviour of the support and one can apply the same yielding elements as in conventional tunnelling. It should be noted, however, that the consequent application of the "yielding principle" is rarely possible as the boring diameter and the clearance profile determine the amount of permissible convergence. On the other hand, the implementation of the "resistance principle" may necessitate, depending on the ground pressure, such a thick shotcrete shell that the available space does not suffice. So, both design principles may require the choice of a bigger diameter in order to accommodate either a thicker lining or ground deformations. The scope of action is in fact fairly limited.

5 Closing remarks

TBM performance is the result of a complex interaction between the ground, the tunnelling equipment (TBM and back-up) and the support. The factors resulting from the three main components of the system affect the TBM drive simultaneously and are usually coupled with each other. Together
with "external" factors (such as the organization of the construction site) they ultimately result in the achievement of a gross advance rate, which represents the main outcome of the interaction and at the same time a factor influencing the interaction.

The behaviour of the ground and therefore its effect on TBM performance depends on its properties (e.g., stiffness, strength and, in the case of water-bearing ground, permeability) and on the initial conditions prevailing in the project area (i.e., initial stress and initial hydraulic head). The ground may be considered as "given", because improvement measures such as drainage or grouting are generally time-consuming and remain therefore in conflict with the goal of attaining a high gross advance rate. Furthermore, the feasibility and effectiveness of such measures depends to a large extent on the ground characteristics and has to be checked in each particular case. The TBM itself affects the progress of the work through its design (e.g., boring diameter, installed thrust force and torque, geometry of the shield, robustness). The characteristics of the tunnel support (e.g., stiffness and bearing capacity) also play an important role. In shielded TBMs, for example, the support is a component of the thrusting system. In gripper TBMs, the support introduces feedback effects and conflicting requirements as the installation of large quantities of support may slow down tunnelling progress (thereby increasing the probability of the TBM jamming), while an insufficient quantity of support will cause problems in the back-up area (which may also interfere with the TBM advance rate).

When planning a TBM drive in squeezing ground, the tunnelling engineer is confronted with a complex problem, where conflictive requirements may be present. In the tunnel engineering community there is a certain degree of controversy concerning the most appropriate machine type for coping with squeezing ground. The application of gripper TBMs is proposed on the grounds of their shorter length as well as their greater flexibility with respect to tunnel support. On the other hand, it is emphasized that single or double shielded TBMs allow for highly industrialized tunnel construction and this – in combination with the higher thrust force and torque that can be applied by these machines – results in a higher advance rate. The differing opinions can possibly be traced back to the different project-specific geological conditions and tunnelling experience of the authors. Consider, for example, a small diameter tunnel (say \( D = 5 \) m) and compare a gripper TBM (equipped with only a short cutter head shield) with a 10 m long double shielded TBM. The length of the double shield is potentially disadvantageous with respect to the risk of jamming. It cannot, however, be said that this disadvantage is really relevant in a particular geotechnical situation (it depends on how rapidly the ground converges). On the other hand, a gripper TBM also has potential disadvantages such as lower thrust (particularly in the case of gripper bracing problems) or lower advance rate (particularly if the support requirements are big). Again, these disadvantages may be relevant or not depending on the geology: on how high the bearing capacity of the ground is in the gripper area; on how rapidly the convergences or ground pressures develop; and on how much support has to be installed in the TBM area in order to stabilize the opening and control the deformations.

Without a thorough analysis, taking into account all the specifics of a given geotechnical situation, it is not easy to judge which factor will have the greater impact. Depending on the geological conditions, the advantages of one TBM type may or may not counteract the disadvantages of the other types. It is therefore impossible to provide universally valid recommendations. A systematic approach such as the one outlined in Section 3 is indispensable for the identification and assessment of the interactions between ground, tunnelling equipment and support.
PART II – THE INTERACTION BETWEEN SHIELD, GROUND AND TUNNEL SUPPORT IN TBM TUNNELLING THROUGH SQUEEZING GROUND

When planning a TBM drive in squeezing ground, the tunnelling engineer faces a complex problem involving a number of conflicting factors. In this respect, numerical analyses represent a helpful decision aid as they provide a quantitative assessment of the effects of key parameters. Part II investigates the interaction between the shield, ground and tunnel support by means of computational analysis. Emphasis is placed on the boundary condition which is applied to model the interface between the ground and the shield or tunnel support. Part II also discusses two real world applications, which illustrate different methodical approaches applied to the assessment of a TBM drive in squeezing ground. The first case history – the Uluabat Tunnel (Turkey) – mainly involves the investigation of TBM design measures aimed at reducing the risk of shield jamming. The second case history – the Faido Section of the Gotthard Base Tunnel (Switzerland) – deals with different types of tunnel support installed behind a gripper TBM.
1 Introduction

As discussed in Part I, squeezing ground represents a challenging condition for operating TBMs, because even relatively small convergences of up to 10–20 cm (that would not really be problematic in conventional tunnelling) may lead to difficulties in the machine area (sticking of the cutter head, jamming of the shield) or in the back-up area (e.g., jamming of the back-up equipment, inadmissible convergences of the bored profile, damage to the tunnel support).

When evaluating the feasibility of a TBM drive in squeezing ground, it is of paramount importance to understand the mechanisms governing the interaction between shield, ground and tunnel support. For the design of the TBM and the tunnel support, a series of issues must be investigated in relation to the ground pressure $p$ (acting upon the cutter head, the shield and the lining), the convergence of the tunnel wall $u$, the extrusion rate of the core $e$, the required thrust force $F$ and the torque $T$ as well as the resulting reaction forces $R_{F,T}$ (Figure 1). All of these parameters may also depend on the advance rate $v$ or on the duration of any excavation standstill that may take place.

A number of different analytical, empirical and numerical approaches have been proposed in the literature for the quantitative assessment of these parameters (Section 2). This part of the present report follows a numerical approach, illustrating the used computational model and addressing the question of the boundary conditions that need to be applied in order to simulate the interface between ground and shield or tunnel support adequately (Section 3) and discussing the structural interplay between these system components by means of computational results (Section 4). Part II also presents two examples of real world applications, which illustrate possible methodical approaches to the assessment of a TBM drive in squeezing ground (Sections 5 and 6). The first case history – the Uluabat Tunnel (Turkey) – mainly concerns the investigation of TBM design measures with the aim of reducing the risk of shield jamming. The second case history – the Faido Section of the Gotthard Base Tunnel (Switzerland) – deals with the different types of tunnel support installed.

![Figure 1. Critical parameters for a gripper TBM (a) and a single shielded TBM (b) in squeezing ground.](image-url)
behind a gripper TBM. A considerable degree of engineering judgement is required in cases such as this – in contrast to shielded TBMs where the support is, as a rule, pre-determined (precast segmental lining).

2 Design and analysis methods

2.1 Analytical solutions

The method of characteristic lines is the simplest and most widely used analysis method in tunnelling. It has also been used by Kovář (1986a, 1986b) with respect to some of the issues of TBM tunnelling in squeezing ground. Vogelhuber (2007) later applied the convergence-confinement method for investigating the crossing of a shear zone at great depth with a double shielded TBM of 10 m diameter. He was thereby able to differentiate between the short-term and long-term behaviour of the ground. The method of characteristic lines is still used today for analysing the interaction between ground and support also with regard to deformable segmental linings of shield-driven tunnels through squeezing rock (cf., e.g., Billig et al., 2007b; Schneider and Spiegl, 2008).

The main disadvantage of the method of characteristic lines is that it does not provide the longitudinal distribution of the ground pressure acting upon the shield and the lining. For this purpose, additional assumptions must be introduced. So, for example, Hisatake and Iai (1993) proposed a time-dependent (creep) non-dimensional displacement function for the longitudinal distribution of the radial ground displacements, while Moulton et al. (1995) and Feknous et al. (1996) introduced three-dimensional diagrams that show support pressure as a function of convergence and distance from the tunnel face. Making an a priori assumption about the distribution and magnitude of the ground pressure is an even stronger simplification. This approach was followed by Eisenstein and Rossler (1995), who developed design charts for the operability of double shielded TBMs in gripper mode, as well as by Vigl et al. (1999) in their discussion of the latest developments in double shielded TBMs. On the basis of numerical calculations, Gärber (2003) improved the convergence-confinement method, provided charts for the design of deep tunnels in low-permeability saturated porous media and applied the proposed semi-analytical solution method to the back-analysis of the segmental lining of the Nuclear Research Centre Connecting Gallery (Belgium), which was excavated by a single shielded TBM ($D = 4.81\,\text{m}$).

2.2 Empirical relationships

Other studies have attempted to get around the drawbacks of analytical solutions by introducing empirical functions based upon field measurements, which describe the longitudinal distribution of the radial displacement of the tunnel boundary. Schubert (2000) showed the effect of the advance rate on tunnel closure in a specific case using the relationships proposed by Sulem et al. (1987) and improved by Sellner (2000). Farrokh et al. (2006), Jafari et al. (2007) and Khademi Hamidi et
al. (2008) evaluated ground pressure and thrust force requirement in their empirical investigation into the double shielded TBMs of the Ghomroud Tunnel (Iran, $D = 4.50$ m) and the Nosoud Tunnel (Iran, $D = 6.73$ m).

The performance of TBMs in squeezing ground can also be assessed by evaluating and correlating the operational parameters of the TBM. This was done, e.g., by Kawatani et al. (1999) for the Takisato Tunnel (Japan, double shielded TBM, $D = 8.30$ m) and by Farrokh and Rostami (2008, 2009) for the Ghomroud Tunnel (Iran).

In spite of the applications mentioned above, one should bear in mind that the reliability of empirical methods is in general limited, as they are based upon correlations of field data obtained in specific projects with potentially different conditions.

### 2.3 Numerical investigations

Axially symmetric or three-dimensional numerical models pay due attention to the spatial stress redistribution in the vicinity of the advancing face, thus eliminating the errors introduced by the assumption of plane strain conditions (Cantieni and Anagnostou, 2009a) and providing information on the evolution of stresses and deformations in the longitudinal direction as well as allowing a more detailed modelling of the different system components (i.e., ground, TBM, tunnel support) and their interfaces.

The initial results of spatial numerical analyses have already been presented by Lombardi (1981), who discussed the influence of the advance rate on the lining loading for the simplified case of a lining that starts to become loaded 40 m behind the face. Lombardi’s (1981) work dealt with aspects of tunnelling in overstressed rocks from a fundamental point of view. In the majority of cases reported in the literature, however, the numerical investigations have been carried out in the framework of specific TBM projects. So for example, Lombardi and Panciera (1997) and Panciera and Piccolo (1997) analyzed the feasibility of a double shielded TBM drive for the Guadiaro – Majaceite Tunnel (Spain, $D = 4.88$ m) taking account of the effects of advance rate and of time-dependent ground behaviour. Matter et al. (2007) studied the crossing of shear zones by the Wienerwald Tunnel (Austria, single shielded TBM, $D = 10.67$ m) by means of axially symmetric numerical investigations. Fully three-dimensional computational models have been applied by Cobreros et al. (2005) and by Simic (2005) – a study which considers creep effects as well – for the Guadarrama Tunnel (Spain, double shielded TBM, $D = 9.51$ m) and by Graziani et al. (2007a; 2007b), who studied the planned Brenner Base Tunnel (Austria / Italy, double shielded TBM, $D = 11.00$ m) within the framework of the TISROCK research project (John and Schneider, 2007). Other project-related investigations include those of Wittke et al. (2007), who evaluated the stresses and deformations of the shield structure of the single shielded TBM of the Hallandsas Tunnel (Sweden, $D = 10.70$ m) taking account of seepage flow and dealing with the structural detailing of the shield by making a simplifying a priori assumption that the ground closes the steering gap at a distance of 4 m behind the working face.

Another group of papers involves numerical investigations which do not take specific account of the shield in the computational model. For example, Shalabi (2005) carried out a back-analysis of the
creep deformations and pressures of the Stillwater Tunnel (USA, $D = 3.06$ m) by assuming that the tunnel is lined up to the face. John and Mattle (2007) analyzed squeezing ground conditions for the Strenger Tunnel (Austria, $D = 11.00$ m) within the previously-mentioned TISROCK research project. Floria et al. (2008), Amberg (2009) and Lombardi et al. (2009) investigated the effect of advance drainage on ground response for the excavation of the service tunnel of the planned Gibraltar Strait Tunnel between Morocco and Spain ($D = 6.50$ m). In the first two studies (Floria et al., 2008; John and Mattle, 2007) the ground around the shield was regarded as being unsupported, while Amberg (2009) and Lombardi et al. (2009) simulated the shield by applying a support pressure of 1 MPa at the face and at the excavation boundary around the shield. All of these works assessed the feasibility of the TBM drive by comparing the computed radial displacements in the machine area with the size of the radial gap between shield and ground.

Research of a more general character, i.e., not related to a specific tunnel project, has been carried out by Ramoni and Anagnostou (2006) and by Schmitt (2009). Schmitt (2009) investigated the behaviour of single shielded TBMs by means of fully three-dimensional, step-by-step simulations of tunnel excavation, thus gaining a valuable insight into the effects of non-uniform convergence and of non-hydrostatic shield and lining loading, while Ramoni and Anagnostou (2006) employed axisymmetric numerical models in order to investigate the effects of thrust force, overboring, shield length and skin friction coefficient between the shield and the ground with respect to the problem of shield jamming.

The numerical solution method of Ramoni and Anagnostou (2006) simulated tunnel excavation through a stepwise unloading of the tunnel boundary (from the initial stress value $\sigma_0$ to zero) and the elastoplastic stress response was calculated by considering a monotonous stress path. This approach is identical to the one underlying the well-known ground reaction curve and is widely used for geomechanical calculations, but, as shown by Cantieni and Anagnostou (2009a), it may lead to an underestimation of the ground pressure and deformation. Ramoni and Anagnostou (2007b, 2007c, 2008) improved their model by implementing a stress-point algorithm (which accounts for the actual stress history of the ground) in accordance with the so-called "steady state method" of Nguyen Minh and Corbetta (1991), a numerical procedure for solving problems with constant conditions in the tunnelling direction by considering a reference frame which is fixed to the advancing tunnel face. A recent description of the computational method (including its further development for poro-elastoplastic materials) and numerical comparisons with the step-by-step simulation of an advancing tunnel can be found in Anagnostou (2007a, 2007b) and Cantieni and Anagnostou (2009a), respectively.

The steady state method makes it possible to solve the advancing tunnel heading problem in one single computational step, i.e., without the need to simulate several sequences of excavation and support installation. As shown by Cantieni and Anagnostou (2009a), the steady state method corresponds to the limit case of an excavation with zero round length. Therefore, it simulates TBM advance better than the commonly employed step-by-step method does, as the latter requires the arbitrary selection of a finite excavation round length, while TBM advance is actually a continuous process. For the commonly chosen, computationally manageable round length values of $s = 1–2$ m, the step-by-step method leads to a considerable underestimation of the shield and lining loading (Cantieni and Anagnostou, 2009a). The choice of a smaller round length ($s = 0.5$ m, cf. Section 5.3) improves accuracy but increases computer time.
The steady state method is particularly suitable for the purposes of the present report, not only because it produces an accurate simulation of the TBM advance, which occurs continuously rather than stepwise, but also because of its computational efficiency. Latter made it possible to carry out comprehensive parametric studies. In this respect, it is interesting to note that the large-scale parametric study underlying the design nomograms presented in Part III, which involved about 12000 numerical simulations, would be practically unfeasible with the step-by-step method.

In all of these investigations, the ground behaviour was considered as being time-independent. Time effects were taken into account by Sterpi and Gioda (2007), who highlighted the fundamental effect of creep, as well as by Einstein and Bobet (1997) and Ramoni and Anagnostou (2007b, 2007c), who studied the consolidation processes associated with the development and subsequent dissipation of excess pore pressures around the tunnel in a low-permeability water-bearing ground (cf. Part IV).

3 Computational model

The numerical computations carried out in the framework of this PhD thesis have been performed using the steady state method (cf. Section 2.3) and applying the finite element code HYDMEC of ETH Zurich (Anagnostou, 1992).

The investigations involve the axially symmetric modelling of the excavation of a deep cylindrical tunnel through a homogeneous and isotropic ground subjected to a uniform and hydrostatic initial stress (Figure 2). The assumption of a uniform initial stress field presupposes that the variation of the initial stress in the vicinity of the tunnel is small relative to the average stress prevailing at the tunnel axis. One can easily verify by means of comparative computations that this assumption is reasonable if the tunnel is deeper than $20R$, where $R$ denotes the tunnel radius. The boundary condition at the far field boundary (either fixed displacement or a traction according to the initial stress) is practically irrelevant if the far field boundary is far enough from the tunnel.

3.1 Constitutive model

The ground is modelled as an isotropic, linear elastic, perfectly plastic material obeying the Mohr-Coulomb yield criterion and a non-associated flow rule.

Creep or consolidation processes have been disregarded (for investigations concerning the effect of consolidation on shield loading the reader is referred to Part IV). The gradual increase of ground pressure and of ground deformations in the longitudinal direction is therefore considered to be due only to the spatial stress redistribution that is associated with the progressive advance of the working face (Lombardi, 1973).
3.2 Tunnel boundary conditions

According to Figure 2, the face is considered as being unsupported. The shield and the tunnel support are taken into account as a radial pressure, i.e., a shear stress between shield and ground is not considered in the numerical analyses.
An accurate simulation of the two elements "shield" and "tunnel support" must take into account,
(i), their different installation points \((y = 0\) and \(y = L\) in Figure 3, respectively) and, (ii), that the
shield and the tunnel support experience smaller displacements than the ground at any given
point \(y\) in the tunnel wall. This is due to the pre-deformation of the ground ahead of the tunnel
face \(u(0)\) and to the overcut \(\Delta R\), which is usually present between the shield and the excavation
boundary. In order to take these aspects into account, a mixed and non-uniform boundary condition
is introduced for the tunnel wall which in a general form reads as follows:

\[
p(y) = \begin{cases} f_s(u(y)) & \text{if } 0 \leq y \leq L \\ f_l(u(y)) & \text{if } y > L \end{cases}
\]

where \(p\) is the ground pressure developing upon the shield or the lining; \(u\) is the radial displace-
ment of the ground at the tunnel boundary; \(L\) denotes the shield length; and the functions \(f_s\) and \(f_l\)
describe the displacement-dependency of the resistance of the shield and of the tunnel support,
respectively.

### 3.2.1 Shield

The function \(f_s\) takes account of the fact that the ground starts to exert a load upon the shield only
after closing the radial gap around the shield, i.e., after experiencing an additional deformation
of \(\Delta R\) behind the face, where \(\Delta R\) denotes the size of the radial gap (Figure 4a). After the closure of
the gap, assuming that the shield is able to bear the load without being overstressed, there is a lin-
ear dependence between the developing ground pressure \(p\) and the radial displacement \(u\).

Shields may have a "conical" shape. This so-called "conicity" of the shield is realised with a step-
wise reduction of the shield diameter (Herrenknecht, 2010). In the computational model, this can be
taken into account defining a variable radial gap size \(\Delta R(y)\). For example, if the conicity of the
shield is realized in two steps (Figure 5), the non-uniform mixed boundary conditions of Equation 1
reads, in a general way, as follows:

\[
p(y) = \begin{cases} 0 & \text{if } 0 \leq y \leq L \text{ and } u(y) - u(0) \leq \Delta R(y) \\ K_s(u(y) - u(0) - \Delta R(y)) & \text{if } 0 \leq y \leq L \text{ and } u(y) - u(0) > \Delta R(y) \\ K_l(u(y) - u(L)) & \text{if } y > L \end{cases}
\]

where

\[
\Delta R(y) = \begin{cases} \Delta R_1 & \text{if } 0 \leq y \leq L_1 \\ \Delta R_2 & \text{if } L_1 < y \leq L_2 \\ \Delta R_3 & \text{if } L_2 < y \leq L_3 \end{cases}
\]

A numerical example illustrating the effect of the conicity of the shield will be discussed in
Section 4.3.

### 3.2.2 Tunnel support

The boundary condition \(f_l\) makes it possible to simulate each kind of tunnel support. Figure 4b
shows the boundary condition applied for practically rigid supports (for example, a shotcrete layer
or a segmental lining being immediately backfilled). Note that the assumption of a constant lining stiffness $K_l$ presupposes that the lining is not overstressed. On the other hand, Figure 4c shows, in general terms, a definition of the boundary condition that would simulate the non-linear behaviour of a yielding support (for details see the application example described in Section 6). As shown later in the Sections 4.4 and 6.3, a detailed simulation of the behaviour of the tunnel support is also important for analyzing its interaction with the shield. More specifically, a rigid support that is installed close to the shield tail facilitates load transfer in the longitudinal direction, thus reducing the ground pressure acting upon the shield. On the other hand, the tunnel support has to bear a higher load in this case.

3.2.3 Simplified model

As a simplified model for estimating shield loading, a uniform boundary condition (defined by the function $f_s$) can be applied to the entire tunnel boundary. In this case, the shield and the tunnel support are modelled as a unique body having the same stiffness and the same radial gap size $\Delta R$. This simplification has been made, e.g., by Ramoni and Anagnostou (2006) and Sterpi and Gioda (2007) and allows a faster investigation to be made of the effect of the shield length $L$ on the thrust force that is required to overcome shield skin friction because, in cases such as this, it is sufficient to integrate the function $p(y)$ over a tunnel sector $0 \leq y \leq L$ of arbitrary length $L$ without needing to carry out an individual numerical computation for each shield length. There are, however, several reasons for using the non-uniform boundary condition of Equation 1.

The simplified model disregards the fact that the installation of the tunnel support occurs later than that of the shield (Figure 3) and does not account for the radial unloading of the excavation boundary at the lining installation point. As shown later in Section 4.4, this leads to an overestimation of the lining loading. This is important not only with respect to the design of the lining, but also for the loading of the shield. In fact, the overestimation of the support pressure exerted by the lining behind the shield leads (due to the longitudinal arch action in the ground around the shield) to an un-
derestimation of the shield loading and, consequently, of the thrust force that is required in order to overcome shield skin friction.

Furthermore, a simplified boundary condition such as this presupposes that the gap size $\Delta R$ is constant along the shield and the lining and, consequently, it is not able to map neither shield conicity nor the perfect contact between lining and ground existing in some cases right from the start (for example, in the case of a gripper TBM with a stiff support or a shielded TBM with annulus grouting taking place simultaneously with TBM advance via the shield tail).

4 Basic aspects of the interaction between shield, ground and tunnel support

The interaction between the shield, the ground and the tunnel support will be analyzed by means of numerical examples for the hypothetical case of a 400 m deep tunnel with a boring diameter of 10 m. The tunnel is excavated by a TBM with a 10 m long single shield. The support consists of a 30 cm thick segmental lining being immediately backfilled. The material constants are according to Table 1 (Set 1).

The ground pressure developing upon the shield is of paramount importance both for the structural design of the machine and for the frictional resistance to be overcome when advancing the TBM. As the ground starts to exert pressure upon the shield only after a certain amount of deformation has occurred, this section starts with a discussion of the convergences and pressures developing along the tunnel (Section 4.1) and shows how much the geometrical parameters of the shield influence the degree of overstressing and the stress history of the ground (Section 4.2) as well as the thrust force required in order to overcome friction (Section 4.3). More specifically, this section will show that the ground at the excavation boundary experiences several unloading and reloading cy-

![Figure 5. Stepwise reduction of the shield diameter (conicity).]
cles and that a stepwise reduction of the shield diameter is very favourable with respect to the ground pressure. Furthermore, Section 4.4 discusses quantitatively the simplified model for the shield-ground interface mentioned in Section 3.2.3 and shows how important it is to take into account as realistically as possible the geometrical characteristics and the installation sequence of the shield and the tunnel support. In this respect it is important to note that the installation point and the stiffness of the tunnel support are essential not only for its loading but also for the pressure developing upon the shield (Section 4.5).

**Table 1. Assumed parameters values.**

<table>
<thead>
<tr>
<th>Set</th>
<th>Figures</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tunnel radius</td>
<td>(R) ([\text{m}])</td>
<td>5.00</td>
<td>5.00</td>
<td>2.50</td>
<td>2.50</td>
<td>4.75</td>
</tr>
<tr>
<td>Radial gap size</td>
<td>(\Delta R) ([\text{cm}])</td>
<td>0–20</td>
<td>5</td>
<td>3/6/9/12</td>
<td>3</td>
<td>12</td>
</tr>
<tr>
<td>Length of the shield</td>
<td>(L) ([\text{m}])</td>
<td>6–12/(\infty)</td>
<td>0–12</td>
<td>10/12</td>
<td>12</td>
<td>5</td>
</tr>
<tr>
<td>Stiffness of the shield</td>
<td>(K_s) ([\text{MPa/m}])</td>
<td>1008</td>
<td>1008</td>
<td>2688</td>
<td>2688</td>
<td>558</td>
</tr>
<tr>
<td>Stiffness of the lining</td>
<td>(K_l) ([\text{MPa/m}])</td>
<td>360</td>
<td>0/(\infty)</td>
<td>2688</td>
<td>2688</td>
<td>variable</td>
</tr>
<tr>
<td>Initial stress</td>
<td>(\sigma_0) ([\text{MPa}])</td>
<td>10</td>
<td>10</td>
<td>3</td>
<td>3</td>
<td>40</td>
</tr>
<tr>
<td>Young’s modulus of the ground</td>
<td>(E) ([\text{MPa}])</td>
<td>1000</td>
<td>2000</td>
<td>200–1000</td>
<td>2000*</td>
<td>3235</td>
</tr>
<tr>
<td>Poisson’s ratio of the ground</td>
<td>(\nu) ([-]</td>
<td>0.25</td>
<td>0.25</td>
<td>0.20</td>
<td>0.20</td>
<td>0.30</td>
</tr>
<tr>
<td>Uniaxial compressive strength of the ground</td>
<td>(f_c) ([\text{MPa}])</td>
<td>3.0</td>
<td>4.5</td>
<td>–</td>
<td>–</td>
<td>5.5</td>
</tr>
<tr>
<td>Cohesion of the ground</td>
<td>(c) ([\text{MPa}])</td>
<td>–</td>
<td>–</td>
<td>500–400</td>
<td>200*</td>
<td>–</td>
</tr>
<tr>
<td>Angle of internal friction of the ground</td>
<td>(\phi) ([^\circ])</td>
<td>25</td>
<td>25</td>
<td>20</td>
<td>20*</td>
<td>35</td>
</tr>
<tr>
<td>Dilatancy angle of the ground</td>
<td>(\psi) ([^\circ])</td>
<td>5</td>
<td>5</td>
<td>1</td>
<td>1*</td>
<td>5</td>
</tr>
<tr>
<td>Skin friction coefficient</td>
<td>(\mu) ([-]</td>
<td>0.15/0.25/0.30/0.45</td>
<td>0.45</td>
<td>0.25/0.50</td>
<td>0.50</td>
<td>0.30</td>
</tr>
<tr>
<td>Installed thrust force</td>
<td>(F_i) ([\text{MN}])</td>
<td>150</td>
<td>–</td>
<td>30/60</td>
<td>30</td>
<td>27.5</td>
</tr>
<tr>
<td>Boring thrust force</td>
<td>(F_b) ([\text{MN}])</td>
<td>0/18</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>17</td>
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<tr>
<td>Step length</td>
<td>(s) ([\text{m}])</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>0.5/1.0</td>
<td>–</td>
</tr>
</tbody>
</table>

**Notes**

* Competent rock
* Weak zone

**4.1 Shield-ground interaction**

Figure 6a shows the radial displacement \(u\) of the ground at the tunnel boundary for three values of the size \(\Delta R\) of the radial gap between shield and ground. The latter determines the amount of convergence that can occur freely. Figure 6b shows the convergence \(u - u(0)\) of the bored profile, i.e., the total radial displacement \(u\) less the so-called "pre-deformation" \(u(0)\) that occurs ahead of the tunnel face. In the case of a normal overcutting \((\Delta R = 5 \text{ cm})\) the ground closes the gap near to the face (at point A, Figure 6b). A larger gap \((\Delta R = 10 \text{ cm})\) remains open for a longer interval (up to point B, Figure 6b). After closing the gap, the ground starts to load the shield. Figure 6c shows the
Figure 6. Results of numerical computations for a 10 m long shield and for an overboring $\Delta R$ of 5, 10 or 15 cm: (a) radial displacement $u$ of the ground at the tunnel boundary; (b) convergence $u - u(0)$ of the bored profile; (c) ground pressure $p$ acting upon the shield and the lining; other parameters according to Table 1, Set 1.
distribution of the ground pressure $p$ acting upon the shield and the lining. The ground pressure increases with the distance from the tunnel face as the stabilizing effect of the core ahead of the face becomes less pronounced. The load concentration at the end of the shield can be traced back to the complete unloading of the tunnel boundary at the installation point of the lining.

As expected, the ground pressure $p$ decreases (both for the shield and the lining) when a larger overboring is provided. In the case of a very large overboring of $\Delta R = 15$ cm the gap between ground and shield would not close at all in this numerical example and the shield would remain unloaded. It should be noted, however, that the feasibility and the reliability of a large overboring have to be checked carefully, particularly for hard rocks (cf. Part I).

### 4.2 Overstressing and stress history of the ground

On the one hand, providing a larger overboring leads to a lower shield loading and therefore to a lower frictional resistance during shield advance. On the other hand, a larger radial gap allows for a larger deformation $u$ to occur and, as a consequence, there is a more extended zone of overstressed ground around the tunnel (Figure 7). In this numerical example, the thickness of the plastic zone increases from 2.8 to 5.5 m practically linearly with the size $\Delta R$ of the radial gap ($\Delta R = 5–15$ cm). In a ground exhibiting brittle behaviour, the deformations and the overstressing may enhance loosening and softening of the ground, thus favouring gravity-driven instabilities. This may lead to problems with the installation of the tunnel support (gripper TBMs) or the backfilling of the segmental lining (shielded TBMs). The issue of strength loss and loosening is particularly important for the design of a yielding support, because both strength loss and major loosening call for a higher yield pressure in the support system (Anagnostou and Cantieni, 2007). An example will be discussed in Section 6.3.

In the so-called "past-yield zone" (Figure 7), the deformations of the ground are partially irreversible but its stress state is located within the elastic domain. The ground has experienced plastic yielding in the past, but has become elastic again. The reason for this so-called "elastic re-compression"
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(Grünler, 2003) is the development of a radial pressure from the lining with increasing distance from the tunnel face. Figure 8 provides a complete picture of the stress history of the ground. With the approaching tunnel excavation, the axial stress $\sigma_{yy}$ decreases ahead of the tunnel face, while a stress concentration occurs in the radial and the tangential directions (Figure 8a). In the vicinity of the tunnel face, the stress deviator becomes so large that the core yields, plastic deformations start to develop and, due to Mohr-Coulomb yield condition, the radial and the tangential stresses $\sigma_{xx}$ and $\sigma_{tt}$ decrease together with the axial stress $\sigma_{yy}$. In the principal stress diagram of Figure 8b, the

**Figure 8.** Results of numerical computations for a normal overcut of $\Delta R = 5$ cm and a shield length of $L = 10$ m: (a) history of the radial ($\sigma_{xx}$), tangential ($\sigma_{tt}$), axial ($\sigma_{yy}$) and shear stress ($\sigma_{xy}$) along the tunnel boundary; (b) principal stress paths along the tunnel boundary; other parameters according to Table 1, Set 1.
onset of plastic yielding is indicated by the point C. At the tunnel face, the radial stress $\sigma_{xx}$ becomes equal to zero and the tunnel boundary remains unsupported as long as the gap between shield and bored profile is open. As a consequence of the Mohr-Coulomb yield criterion, the maximum principal stress over this unsupported span becomes equal to the uniaxial compressive strength $f_c$. (The stress state over the unsupported span is indicated by the point D in the principal stress diagram of Figure 8b.)

At a certain distance behind the face, the converging ground closes the gap and the shield starts to develop a radial support pressure $\sigma_{xx}$ upon the tunnel boundary with the consequence that the ground is able to sustain a higher stress level and the stress state returns to the elastic domain (Figure 8b, stress path DE). As can be seen in Figure 8a, the ground experiences two unloading (to $\sigma_{xx} = 0$) and reloading cycles, the first being near to the tunnel face until the ground closes the gap (Figure 8b, stress path ABCDE) and the second at the end of the shield, where lining installation takes place (stress path EFGH). As can be seen from Figure 9, which shows the radial stress $p$ at the excavation boundary for different shield geometries, unloading-reloading cycles occur several times if the shield has a stepwise decreasing diameter.

It is also interesting to note that the wider the radial gap, the later the ground closes the gap, leading to a later occurrence of elastic re-compression and a bigger plastic zone in the longitudinal direction (Figure 7). So, for example, if $\Delta R = 15$ cm the gap remains open over the entire shield

Figure 9. Ground pressure $p$ acting upon the shield and the lining for three different shield geometries having the same average radial gap size (shield length $L = 10$ m, radial gap size $\Delta R = 4$–6 cm; other parameters according to Table 1, Set 1).
length $L$ (which can be seen as a free span between the tunnel face and the lining) and the plastic zone extends up to the end of the shield ($y = L = 10$ m, Figure 7c).

### 4.3 Thrust force

The thrust force $F_f$ required to overcome shield skin friction can be calculated by integrating the ground pressure $p$ over the shield surface and multiplying the integral by the skin friction coefficient $\mu$ (cf. Part III). Figure 10a shows the effect of the radial gap size $\Delta R$ on the required thrust force $F_f$ for two operational stages ("ongoing excavation" and "restart after a standstill"). During ongoing excavation, the TBM has to overcome sliding instead of static friction, but, on the other hand, an additional thrust force $F_b$ is needed for the boring process ($F_f = F_f + F_b$, $F_b$ was taken to 18 MN in this example). Following Gehring (1996), the skin friction coefficient was taken to be $\mu = 0.15$–0.30 for sliding friction and $\mu = 0.25$–0.45 for static friction, where the lower friction coefficient values aim to illustrate the positive effects of lubrication of the shield extrados, e.g., by bentonite. The line marked by $F_i$ denotes a high, but still feasible thrust force of 150 MN (see also the TBM technical data collected in Part III).

Figure 10b shows the required thrust force $F_f$ as a function of the shield length $L$ for the two operational stages and an overcut of $\Delta R = 5$–15 cm. The diagram illustrates the positive effect of a shorter shield. It has to be noted that the dependency of thrust force on the shield length is in general non-linear.

![Figure 10](image1.png)

**Figure 10.** Required thrust force $F_f$ during ongoing excavation (thrust force for the boring process $F_b = 18$ MN) and for the restart after a standstill with (skin friction coefficient $\mu = 0.15$ or 0.25, respectively) or without (skin friction coefficient $\mu = 0.30$ or 0.45, respectively) lubrication of the shield extrados: (a) as a function of the overboring $\Delta R$ for a 10 m long shield; (b) as a function of the shield length $L$ for an overboring $\Delta R$ of 5, 10 or 15 cm; other parameters according to Table 1, Set 1.
As shown in a condensed form by Figure 10, the required thrust force $F_r$ depends strongly on the shield length $L$, on the skin friction coefficient $\mu$ and on the overcut $\Delta R$. Another important TBM design parameter is the so-called "conicity" of the shield, i.e., the variation $\Delta R(y)$ of the radial gap size along the shield (cf. Section 3.2.1). Figure 9 shows the ground pressure $p$ acting upon the shield and the lining for three different shield geometries having the same average radial gap size of $\Delta R = 5$ cm. The positive effect of a "stepwise" construction of the shield becomes evident when comparing the average ground pressure $p_s$ (which governs the required thrust force) acting upon the shield (Figure 9). It decreases by 16 or 28 % respectively where there are two or three steps in the construction of the shield. A wide gap is more important for the rear part of the shield because the convergence of the ground increases with the distance behind the face.

4.4 A simplified model of shield-ground interaction

As already mentioned (cf. Section 3.2.3), the computational model can be simplified by modelling the shield and the lining as a single infinitely long cylindrical body of constant stiffness and radial gap size.

Figure 11a compares the longitudinal distribution of the ground pressure $p$ of the simplified model with a pressure distribution based on the more accurate model discussed in the previous sections (for a 10 m long shield). The simplified computational model overestimates the ground pressure developing upon the lining (by 56 % in the final state developing far behind the face) and, consequently, the supporting effect of the lining in the area immediately behind the shield (the diagonally dashed region in Figure 11a). Due to the load transfer in the longitudinal direction, this leads to a lower shield loading (the vertically dashed region in Figure 11a), thereby underestimating the thrust

Figure 11. Distribution of the ground pressure $p$ acting upon the shield and the lining (a) and required thrust force $F_r$ as a function of the shield length $L$ (b) based upon the simplified model as well as for a model employing a non-uniform boundary condition (radial gap size $\Delta R = 5$ cm, skin friction coefficient $\mu = 0.45$; other parameters according to Table 1, Set 1).
force required to overcome friction. Figure 11b shows the thrust force $F_r$ required to restart TBM advance after a standstill as a function of the shield length $L$ in the simplified model and also based upon the more accurate model with the non-uniform boundary condition of Equation 1. The latter requires a numerical solution of the finite element equations for each value of the shield length $L$, while the simplified model enables the curve $F_r(L)$ to be computed on the basis of a single numerical calculation. This advantage, however, is achieved at the expense of accuracy. The simplified model underestimates the thrust force by about 40 MN in this example. It is therefore important to model the characteristics and the installation point of the tunnel support as accurately as possible, not only from the perspective of structural assessment but also with respect to the design of the TBM.

### 4.5 Shield-support interaction

The installation of a stiff support close to the shield reduces the shield loading and the thrust force requirement because it improves load transfer in the longitudinal direction. (The case of yielding supports will be discussed later in Section 6.) The stiffer the lining and the shorter its distance from the face, the more pronounced will be the longitudinal arching effect and the bigger will be the reduction of the shield load.

The upper part of Figure 12a illustrates this effect by presenting the thrust force $F_r$ (required for restarting TBM advance after a standstill) as a function of the shield length $L$ for two borderline cases with respect to support stiffness: a rigid support ($K_l = \infty$) and an unsupported tunnel ($K_l = 0$). As expected, the unloading effect is more pronounced for short shields. A stiff support that is installed close to the face is favourable with respect to the shield but, nevertheless, attracts a higher ground load. In fact, for short shields (where the longitudinal arching effect is particularly pronounced) the final ground pressure $p$ developing upon the rigid support ($K_l = \infty$) reaches values that cannot be sustained by the usual linings (Figure 12a, lower part). As expected, the load of the tunnel support $p$ decreases with increasing shield length $L$, i.e., with decreasing arching effect.

It should be noted that the case of an unsupported tunnel is not only theoretically possible. As a matter of fact, in shield tunnelling through rock, backfilling of the segmental lining by pea gravel is carried out at a certain distance behind the shield with the consequence that the rock behind the shield actually remains unsupported (Figure 12b). There is no unloading effect in this case, of course (point A in Figure 12a). Shield load reduction (point B in Figure 12a) via longitudinal arching between the face and the segmental lining presupposes annulus grouting simultaneously with TBM advance via the shield tail (Figure 12c). The peculiarities of segmental lining installation have been analyzed by Lavdas (2010).
5 An example of a single shielded TBM

5.1 Introduction

The first application example concerns the 11.8 km long Uluabat Tunnel in Turkey (about 100 km south of Istanbul). A 12 m long single shielded TBM with a boring diameter of 5.05 m and an installed thrust force of about 30 MN started work in 2006. The overcut \( \Delta R \) was 3 cm at the front part of the shield and increased to 4 cm at the shield tail. The ground is of Triassic origin and consists of a claystone matrix containing 1–50 cm big sandstone lenses. The claystone fraction amounts to about 80%. The claystones are intensively sheared, have several slickensides and disintegrate quickly under the action of water. Laboratory results revealed an angle of internal friction of about

Figure 12. (a) Required thrust force \( F_r \) and final ground pressure \( p \) acting upon the lining as a function of the shield length \( L \) for a rigid support \( (K_i = \infty) \) as well as for an unsupported tunnel \( (K_i = 0) \); radial gap size \( \Delta R = 5 \) cm, skin friction coefficient \( \mu = 0.45 \); other parameters according to Table 1, Set 2; (b) single shielded TBM in rock with delayed backfilling of the segmental lining (case A of Figure 12a); (c) single shielded TBM with annulus grouting via the shield tail (case B of Figure 12a).
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$\varphi = 20^\circ$, strongly variable cohesion values ($c = 50–400$ kPa) and a Young's modulus of $E = 200–1000$ MPa. During the first 3 km of TBM operation, squeezing caused jamming of the shield on several occasions although the depth of cover was rather moderate (about 120 m, i.e., an estimated initial stress of $\sigma_0 = 3$ MPa). The available monitoring results were sparse, but indicated a large variability in squeezing intensity with maximum deformation rates of up to 6 cm/h.

A comprehensive parametric study was carried out in order to gain a better understanding of the observed phenomena, analyze the factors influencing the jamming of the TBM and evaluate the effectiveness of possible TBM improvements (Kovári and Ramoni, 2008; Ramoni and Anagnostou, 2008). On account of the variability of the squeezing phenomena, attention was paid to the specific situation prevailing in certain critical zones. The following sections outline the most important results.

5.2 Investigations on TBM optimization

The numerical investigations were carried out on the basis of an axially symmetric model with the simplified boundary condition of Section 3.2.3 (see also Section 4.4) and a constant overcut $\Delta R$ along the shield. The calculations disregarded a possible time-dependency of the ground behaviour – a reasonable assumption considering the high convergence rates observed in situ. Furthermore, on account of the low strength of the ground, the thrust force calculations neglected the boring thrust force. The skin friction coefficient was taken to $\mu = 0.50$ after Gehring (1996). This value is relevant for the static friction conditions prevailing when attempting to restart excavation after a stop for the installation of the segmental lining.

For a given depth of cover and for given TBM parameters, a critical range of rock mass parameters can be determined beyond which the required thrust force is higher than the installed one, thus indicating that shield jamming may occur. In the present case, the critical range was defined in terms of the Young's modulus $E$ and of the cohesion $c$ of the rock mass (all other ground parameters being kept constant). The reason for this choice was the large uncertainty concerning these two parameters in combination with the great sensitivity of the ground response with respect to their variations.

Figure 13a shows the critical ground conditions for a depth of cover of $H = 120$ m and an overcut $\Delta R$ of 3, 6 or 9 cm (the 6 to 9 cm applies to the case of an increased overcut). The points of each curve (hereafter referred to as "critical curve") fulfil the condition that the required thrust force (for the specific value of $\Delta R$) is equal to the installed thrust force. The lower the Young's modulus $E$, the higher the cohesion $c$ must be in order that the average ground pressure acting upon the shield remains equal to a given value. Value-pairs ($c$, $E$) above a certain curve characterize subcritical ground conditions (i.e., the installed thrust force is sufficient for overcoming the frictional resistance of the ground). On the other hand, the region below the critical curve indicates ground conditions that may trap the TBM. The grey box shows the actual range of the two ground parameters based upon the results of laboratory tests. By considering the position of the critical curve relative to this box an optical assessment can be made of TBM operating conditions. The fact that the critical curves diagonally cross the box representing the laboratory values points to a pre-
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diction uncertainty and agrees with the variability experienced during the TBM drive: depending on the variation of the ground conditions the shield may or may not be jammed.

In order to reduce the risk of major delays in the completion of the project, the option of an additional TBM drive from the other portal of the tunnel was investigated (but finally rejected for contractual reasons). The second TBM would cross the same formation, where the first TBM experienced difficulties, but under an even higher overburden (up to 240 m in the deepest portion of the alignment). A main goal of the investigations was, therefore, the optimization of the second TBM in order to cope with squeezing ground. Figure 13b shows the critical curves for a depth of cover of 120 m (curves A and B) or 240 m (curves C and D). The critical curves for the first machine are denoted by A and C, while the curves B and D apply to a new machine implementing a series of technical improvements: a higher thrust force $F_t$ (60 instead of 30 MN), a 2 m shorter shield length $L$, a bigger overboring $\Delta R$ and reduced skin friction $\mu$ (achieved by lubrication of the shield extrados). The combination of all these measures would shift the critical curve from curve A to curve B in the bottom-left corner of the diagram and this means that an improved TBM would be able to cope with adverse conditions such as those encountered by the first TBM at a depth of cover of $H = 120$ m. However, the possibility of shield jamming would persist in the deepest portion.

Figure 13. (a) Critical ground conditions for an installed thrust force of $F_t = 30$ MN (radial gap size $\Delta R = 3, 6$ or 9 cm, shield length $L = 12$ m, skin friction coefficient $\mu = 0.50$, depth of cover $H = 120$ m, safety factor for the required thrust force $SF = 2.0$); (b) effect of a combination of several technical improvements (installed thrust force $F_t = 30 \rightarrow 60$ MN, radial gap size $\Delta R = 6 \rightarrow 12$ cm, shield length $L = 12 \rightarrow 10$ m, skin friction coefficient $\mu = 0.50 \rightarrow 0.25$, depth of cover $H = 120$ or 240 m, safety factor for the required thrust force $SF = 2.0$); other parameters according to Table 1, Set 3.
of the alignment \( H = 240 \text{ m} \). According to curve D of Figure 13b, the improved TBM would perform at a depth of 240 m similarly to the current TBM at a depth of 120 m (curve A), while operation of the current TBM at the maximum depth would be possible only in the case of considerable improvements to ground strength and stiffness (curve C).

### 5.3 Effect of short weaker zones

The numerical computations presented in the last section were based on the assumption of homogeneous ground in the longitudinal direction. During the TBM drive in Uluabat, however, the ground behaviour, as reflected by the thrust force needed in order to keep the TBM advancing, changed within short intervals, thus indicating a succession of weak zones with stretches of more competent ground.

According to past research on the mechanics of deformation in short geological fault zones, the adjacent competent rock also has a stabilizing effect with respect to the fault zone (Cantieni and Anagnostou, 2007; Kovári and Anagnostou, 1995). When crossing a single weak zone, shear stresses are mobilized at its interface with the adjacent competent rock, because the latter experiences smaller deformations. The shear stresses reduce the convergences within the weaker zone, particularly when its length is small. As shown, e.g., by Matter et al. (2007) and in more detail by Graziani et al. (2007a), this so-called "wall effect" is also favourable with respect to the risk of TBM jamming.

Due to these considerations, an examination was performed as to whether the variability of the ground behaviour observed in Uluabat could be explained by the existence of weak zones of variable extent and an analysis was conducted into the effect of the length of a weak zone on the required thrust force. Figure 14 and Table 1 (Set 4) show the layout of the problem and the assumed parameters, respectively. The main assumptions are the same as in the last section. In order to reduce the computational cost, the numerical investigations assumed that the behaviour of the competent rock before and after the weaker zone is linearly elastic. Due to the non-uniformity of the

![Figure 14. Layout of the short fault zone problem investigated with step-by-step calculations.](image-url)
conditions in the longitudinal direction, it was not possible to apply the steady state numerical solution method and, therefore, the tunnel excavation and support installation were modelled step-by-step on the basis of a step length \( s \) of 0.5 or 1.0 m.

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**Figure 15.** Required thrust force \( F_r \) as a function of the position \( y' \) of the tunnel face (cf. Figure 14) for different lengths \( l \) of the weak zone (step length \( s = 1 \) m, radial gap size \( \Delta R = 3 \) cm, shield length \( L = 12 \) m, skin friction coefficient \( \mu = 0.50 \), safety factor for the required thrust force \( SF = 1.0 \); other parameters according to Table 1, Set 4).

**Figure 16.** Maximum required thrust force \( F_r \) in the weak zone as a function of their length \( l \) (radial gap size \( \Delta R = 3 \) cm, shield length \( L = 12 \) m, skin friction coefficient \( \mu = 0.50 \), safety factor for the required thrust force \( SF = 1.0 \); other parameters according to Table 1, Set 4).
Figure 15 shows the required thrust force $F_r$ as a function of the tunnel face position $y'$ (the latter refers to the onset of the critical zone, cf. Figure 14). The curves apply to weak zones of different lengths $l$. As expected, the required thrust force increases when the TBM enters into the weak zone and decreases when the TBM leaves it. Assuming that practically the entire installed thrust force of $F_i = 30$ MN is available for overcoming skin friction, the TBM would be able to cope with a 5–10 m thick weak zone. In the case of a weak zone longer than about 10 m, however, the TBM might become trapped. The observed variability might therefore be associated with a sequence of weaker and stronger rock zones.

Figure 16 shows the maximum required thrust force $F_r$ as a function of the length $l$ of the weak zone for step lengths $s$ of 0.5 or 1 m. The shorter the weak zone, the more pronounced will be the wall-effect and the lower will be the risk of shield jamming. In the example of Figure 16, the wall-effect is remarkable for critical zones shorter than about 10–15 m, i.e., two or three tunnel diameters. For long fault zones and a step length of $s = 0.5$ m, the results of the step-by-step solution agree well with those obtained by the steady state method. On the contrary, for the reasons mentioned in Section 2.3, adopting a longer step length ($s = 1$ m) leads to an underestimation of the required thrust force $F_r$ (by 15 % in this example).

6 An example of a gripper TBM

6.1 Introduction

Other than in the case of shielded TBMs, where the support characteristics are largely pre-defined (precast segments, maybe of variable reinforcement content), a certain degree of flexibility exists in gripper TBMs with respect to the means and quantities of support. Due to the largely predefined boring diameter, decision-making is nevertheless constricted within a relatively narrow space as the geometrical constraints of the tunnelling equipment limit both the admissible convergence and the possible thickness of the tunnel support. Decision-making may also be particularly challenging because of the conflictive criteria often existing in squeezing ground: stabilizing interventions behind the machine are generally possible only in some locations that are dictated by the TBM design. In order to avoid problems such as a violation of the clearance profile, a high quantity of support may have to be installed shortly after excavation, i.e., behind the cutter head. This, however, slows down TBM advance and, in the case of time-dependent ground behaviour, increases the risk of the machine becoming trapped.

The present section discusses the effect of different support types based upon the results of numerical investigations carried out by Anagnostou and Ramoni (2007) for the 14.2 km long Faido Section of the Gotthard Base Tunnel in Switzerland. The tunnel is currently under construction by means of two gripper TBMs ($D = 9.43$ m) having 5 m long cutter head shields and installed thrust forces of 27.5 MN. The TBM drives in the Faido Section started in July and October 2007, respectively, with manually shifted gauge cutters ($D = 9.50$ m). Squeezing related phenomena were observed in the so-called "Lucomagno-Gneiss" – a metamorphic, micaceous, crystalline rock – at a
depth of 1600 m (estimated initial stress \( \sigma_0 = 40 \) MPa). In a 250 m long stretch, convergences in the roof (of up to 5–10 cm in the eastern tube and of up to 25 cm in the western tube) and heave of the tunnel floor (of up to 30 cm in the eastern tube and of up to 75 cm in the western tube) caused damage to the tunnel support and jamming of the back-up trailers (Böckli, 2008; Boissonnas, 2008; Flury and Priller, 2008; Gollegger et al., 2009; Herrenknecht et al., 2009). Deformations of up to 10 cm occurred within the short interval between the working face and the shield tail, thus using up most of the convergence margin offered by the shield articulation, without however to immobilize the TBM.

6.2 Investigations

The aim of the investigations was to find out which support type would present the lowest risks (with respect to a series of squeezing-related hazard scenarios), thereby maximizing the range of manageable squeezing conditions. For this purpose, the hazard scenarios, (i), jamming of the shield, (ii), overstressing of the tunnel support and, (iii), violation of the clearance profile have been analyzed for a series of hypothetical rock mass constants covering a wide range of squeezing intensity.

![Figure 17. Range of the ground response curves (a) and uniaxial compressive strengths \( f_c \) and Young’s moduli \( E \) (b) of the “rock mass types” considered in the numerical computations (initial stress \( \sigma_0 = 40 \) MPa; other parameters according to Table 2).](image-url)
The parameterization of the squeezing intensity was done on the basis of the free convergence \( u/R \) (i.e., the convergence that would occur in the theoretical case of an unsupported opening) and of the non-linearity of the ground response curve. Figure 17a shows the range of ground response curves for the rock types under consideration: the free convergence \( u/R \) amounts to 2–9 %, while, for a given free convergence, the ground response curve may be more or less curved depending on the uniaxial compressive strength \( f_c \) and on the Young's modulus \( E \) of the ground. The values of the friction angle, the dilatancy angle and the Poisson's ratio were fixed to \( \phi = 35 \, ^\circ \), \( \psi = 5 \, ^\circ \) and \( \nu = 0.30 \), respectively. Figure 17b shows the value pairs \( (f_c, E) \) considered in the numerical analyses (different markers are used in order to show the corresponding free convergence values \( u/R \)). In the present report, only the results for the material constants from Table 2 will be presented, as the curvature of the ground response curve does not significantly affect the main conclusions in the present case. The term "rock mass type" used hereafter refers to the parameters of Table 2.

The numerical calculations were based upon an axisymmetric model with uniform support characteristics over the tunnel cross-section. The assumption of rotational symmetry represents a strong simplification in the present case in view of the observed asymmetric ground deformations. Table 3 shows the investigated tunnel support types. The systems \( RS15 \) and \( RS25 \) are practically rigid supports including a 15 or 25 cm thick shotcrete ring, respectively. The systems \( YS15/S5, YS15/C5, YS15/C15 \) and \( YS25/C15 \) are yielding supports with a 15 or 25 cm thick shotcrete ring incorporating either 5 cm thick Styrofoam plates (which can be compressed completely) or 15 cm thick high ductility concrete elements which can experience a yield strain of about 50 % (Solexperts, 2007). All of the support types include steel sets (TH 36) at 1 m spacings and with sliding connections in the case of the yielding support systems. Additionally to the support types of Table 3, the hypothetical case of an unsupported tunnel was also considered for comparison purposes and as a simplified model of very light tunnel support.

For all support cases the assumption was made that the tunnel support is installed immediately behind the shield, i.e., at a distance of 5 m from the working face. Concerning overcut, two radial gap
The sizes of $\Delta R = 6$ or 12 cm have been considered, taking into account the shifting of the gauge cutters and the kinematics of the articulated shield.

The computational model simulates the support types described above by mixed, non-linear boundary conditions according to the different characteristic lines of Figure 18. The latter take due account of the characteristic lines of the different support components as well to the sizes and the number of the yielding elements, including the sliding connections of the steel sets. The time-dependency of the shotcrete stiffness was taken into account in a simplified way by adopting a re-

![Table 3. Investigated support systems.](image)

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<td>$Y_{S15/C15}$</td>
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</tr>
<tr>
<td>$Y_{S25/C15}$</td>
<td>$d_1 = 25$ cm</td>
</tr>
</tbody>
</table>

![Figure 18. Characteristic lines (ground pressure $p$ as a function of the radial displacement of the lining $u$) of the investigated support systems (cf. Table 3).](image)
duced Young's modulus of $E_{sc} = 10$ GPa. The Appendix shows the detailed computation for the example of support type YS25/C15.

6.3 Discussion of the results

The numerical analyses have been carried out for all combinations of rock and support types (Figure 17b and Table 3, respectively). The main results of each numerical analysis are the ground pressure distribution along the shield and the tunnel and the deformations of the bored profile. These results were evaluated with respect to the following criteria: (i) Is the installed thrust force sufficient to overcome frictional resistance? (ii) Is the structural safety of the tunnel support sufficient? (iii) Do the rock mass convergences violate the clearance profile (under-profile)?

Concerning the thrust force requirements, the investigations considered the conditions both during ongoing excavation and for restart after a standstill. These are different with respect to the skin friction coefficient – $\mu = 0.30$ or 0.45, respectively, cf. Gehring (1996) – and to the thrust force needed for boring ($F_o = 17$ MN). Furthermore, the evaluation disregarded possible limitations of the available thrust force due to problems with the gripper bracing – a reasonable assumption considering the crystalline character of the rock. The operational stage "ongoing excavation" is the relevant one in the present case. This is due to the fact that the thrust force needed for boring (which in the present example amounts to 62 % of the installed thrust force) overweighs the positive effect of having a lower skin friction coefficient.

In order to evaluate structural safety, the lining hoop stress was compared to the shotcrete strength not only at the final state (assuming $f_{c,sc} = 25$ MPa) far behind the TBM, but also at a section located at 2 m behind the shield in order to check overstressing of the green shotcrete. The early strength of the shotcrete at this section was taken to $f_{c,sc} = 10$ MPa according to lab tests on 8–10 hours old specimens. This age is relevant for the shotcrete loading taking into account the actual gross advance rate of $v = 5–6$ m/d.

In order to check the clearance profile, the space used up by the actual thickness of each support system was taken into account. (A thicker shotcrete lining needs more space but leads to smaller deformations on account of its higher stiffness.)

Table 4 shows the combined evaluation of the criteria mentioned above. The yielding support systems in combination with the bigger overcut ($\Delta R = 12$ cm) cover the widest spectrum of geological conditions (this conclusion is also true for the other parameter combinations of Figure 17b).

The rigid support systems (RS15 and RS25) have a positive effect with respect to the thrust force requirements because they rapidly offer a high resistance to the ground deformations close to the shield (cf. Section 4.5 and Figure 12). This becomes evident by comparing the distribution of the ground pressure $p$ acting upon the shield and the lining for the support systems RS15 (rigid) and YS15/S5 (yielding). As shown in Figure 19, the shield remains practically unloaded in the first case, while a high load develops at the shield tail in the second case and may immobilize or even damage the shield. Nevertheless, in the most of the cases that were investigated, the load developing upon the rigid support systems is much higher than their bearing capacity. Even a simplified estimation of their bearing capacity – disregarding possible bending moments (axial symmetry) –
shows an insufficient level of structural safety. Applying a thicker shotcrete layer \( (d_t = 25 \text{ instead of } 15 \text{ cm}) \) does not improve matters substantially. At this point, it has also to be noted that the load developing upon the rigid support systems close to the shield depends strongly on the assumed stiffness of the shotcrete. If the assumed "average" value of \( E_{sc} = 10 \text{ GPa} \) overestimates the actual Young's modulus of the green shotcrete, the computations overestimate the ground pressure acting upon the lining near to the shield and, consequently, the positive longitudinal arching effect.

Limitations also exist, however, for yielding supports. Taking into account the boring diameter, the clearance profile and the space needed for the final lining, 40 cm in the tunnel radius were available for the thickness of the tunnel support and for admitting some load-reducing convergences without violating the clearance profile.

A very light support (a practically unsupported tunnel) is theoretically satisfactory. Assuming that the tunnel support would use 10 cm of the tunnel radius, the calculated deformations would violate the clearance profile only for "rock mass types" 7 and 8. However, such a solution would not allow ground deformations to be controlled and would be unacceptable with regard to possible gravity-driven instabilities.

---

Table 4. Combined evaluation of the hazard scenarios "shield jamming", "support overstressing" and "under-profile" for ongoing excavation (support systems according to Table 3, ground parameters according to Table 2).

<table>
<thead>
<tr>
<th>Overboring</th>
<th>Support system</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>6 cm</td>
<td>Unsupported</td>
<td>B</td>
<td>B</td>
<td>B</td>
<td>B</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>RS15</td>
<td>0 C A A A A A</td>
<td>B</td>
<td>B</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>RS25</td>
<td>0 C A A A A A</td>
<td>B</td>
<td>B</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>YS15/S5</td>
<td>0 B B A A A A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
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<td>A</td>
</tr>
<tr>
<td>YS15/C5</td>
<td>0 B A A A A A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
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</tr>
<tr>
<td>YS15/C15</td>
<td>0 B B B A A A</td>
<td>A</td>
<td>A</td>
<td>A</td>
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<tr>
<td>YS25/C15</td>
<td>0 B B B B A A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>12 cm</td>
<td>Unsupported</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>B</td>
<td>B</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>RS15</td>
<td>0 C C C C C C</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>C</td>
<td>C</td>
<td>C</td>
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<tr>
<td>RS25</td>
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<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>C</td>
<td>C</td>
<td>C</td>
<td>C</td>
</tr>
<tr>
<td>YS15/S5</td>
<td>0 0 0 0 C A A</td>
<td>0</td>
<td>0</td>
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<td>0</td>
<td>C</td>
<td>C</td>
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</tr>
<tr>
<td>YS15/C5</td>
<td>0 0 0 0 0 C A</td>
<td>0</td>
<td>0</td>
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<td>0</td>
<td>C</td>
<td>C</td>
<td>C</td>
<td>C</td>
</tr>
<tr>
<td>YS15/C15</td>
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<td>0</td>
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</tr>
<tr>
<td>YS25/C15</td>
<td>0 0 0 0 0 0 A</td>
<td>0</td>
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<td>0</td>
<td>C</td>
<td>C</td>
<td>C</td>
<td>C</td>
</tr>
</tbody>
</table>

Legend

D Installed thrust force sufficient
Structural safety of the tunnel support warranted, no under-profile
A Installed thrust force not sufficient
Structural safety of the tunnel support not warranted and/or under-profile
B Installed thrust force not sufficient
Structural safety of the tunnel support warranted, no under-profile
C Installed thrust force sufficient
Structural safety of the tunnel support not warranted and/or under-profile
After Table 4, the support systems YS15/S5 and YS15/C15 employing 15 cm shotcrete in combination with Styrofoam or high ductility concrete elements are the most advantageous. (The first one has been successfully applied, while the second one has only been tested along 30 m of the TBM drive shortly before the end of the critical stretch.) However, when comparing these systems, it has to be borne in mind that the admission of ground deformations may cause more loosening of the rock. The thickness of the plastic zone (3.7 m), which results from the calculations, provides a rough indication of the extent of the loosened zone and thus of the possible loosening pressure. Assuming a unit weight of the rock of 25 kN/m$^2$, the resulting loosening pressure amounts to about 90–100 kPa. This value is lower than the yield pressure of the support system with the high ductility concrete elements (YS15/C15), but exceeds the resistance of the support system with the Styrofoam elements (YS15/S5). Consequently, the latter might start to deform under the action of the loosened rock mass (Figure 18) and would use up its deformation capacity with the consequence that it would behave like the rigid support RS15 during the squeezing phase. The support system YS15/S5 would therefore be equivalent to YS15/C15 only if combined with 5–6 m long rock bolts at the crown that would bear the loosening pressure of about 100 kPa. On the other hand, and as shown by tunnelling experience, the application of the support system YS15/C15 presupposes – due to the relatively high stiffness of the high ductility concrete elements – that the shotcrete develops a sufficiently high early strength (i.e., at least the same compressive strength as the deformable concrete elements). This may be a problem if the time-dependent development of the resistance of the shotcrete is too slow relative to the TBM advance (this aspect is in general less relevant in conventional tunnelling because of the lower advance rates).

Figure 19. Ground pressure $p$ acting upon the shield and the lining ("rock mass type" 6 according to Table 2, shield length $L = 5$ m, radial gap size $\Delta R = 12$ cm, support system RS15 or YS15/S5 according to Table 3; other parameters according to Table 1, Set 5).


7  Closing remarks

The numerical solution method presented and applied in this report represents a powerful tool for the simulation of a TBM drive in homogeneous squeezing ground. The mixed boundary condition developed for the ground-support interface allows an accurate simulation of the shield and of any kind of tunnel support. The application of the steady state method makes it possible to solve the advancing tunnel heading problem in one single computation step with a major reduction of the computation time, thus allowing comprehensive parametric studies to be performed at a justifiable cost. The effects of changes in ground conditions as well as the suitability of modifications to the TBM layout and the tunnel support can therefore be investigated easily and quickly.

For the investigation of heterogeneous ground conditions, the commonly used step-by-step method still remain to be applied. A comparative analysis involving a short critical zone (striking orthogonally to the tunnel axis) has shown that a reduction of the step length improves accuracy with respect to the required thrust force – although this comes, of course, at a higher computational cost.
PART III – THRUST FORCE REQUIREMENTS FOR TBMs IN SQUEEZING GROUND

Rapidly converging ground may exert such a high pressure on the shield that the thrust force is no longer sufficient to overcome shield skin friction and the TBM becomes jammed. Part III advances a number of theory-based decision aids, which will support rapid, initial assessments to be made of thrust force requirements. A comprehensive parametric study has been carried out using the finite element method and, based on the numerical results, dimensionless design nomograms have been worked out that cover the relevant range of material constants, in situ stress and TBM characteristics. The nomograms make it possible to assess the feasibility of a TBM drive in a given geotechnical situation and to evaluate potential design measures or operational measures such as reductions in shield length, the installation of a higher thrust force, increases in the overcut or the lubrication of the shield surface, thus making a valuable contribution to the decision-making process.
1 Introduction

Squeezing ground may lead to inadmissible deformations of the tunnel, damage to the tunnel support or – in the case of mechanized tunnelling – to the immobilization of the TBM (cf. Part I). Squeezing conditions may even put in question the feasibility of a TBM drive – an aspect that is particularly serious for (but not limited to) long, deep tunnels.

A detailed analysis of the potential hazards and the numerous interfaces and complex interactions between the ground, the tunnelling equipment and the tunnel support can be found in Part I. Part III of the present report focuses on the risk of shield jamming and particularly on the thrust force required in order to overcome the skin friction developing when attempting to advance a loaded shield. An insufficient thrust force may make it impossible to restart the TBM after a standstill. The TBM may also become trapped during the actual excavation. In this case, if the installed thrust force is insufficient, the friction generated by the squeezing pressure will reduce the penetration rate, thus slowing the TBM advance rate down to a complete standstill (cf. Part I). In both cases, the TBM will have to be freed by hand-mining around the machine or by the less commonly used method of installing auxiliary thrust cylinders.

It is essential to have information on the frictional force when designing a new TBM and when assessing the feasibility of a proposed TBM drive. Concerning the utilization of a second-hand TBM, checks have to be made as to whether the installed thrust force is sufficiently high or whether the TBM has to be refurbished. Furthermore, the planner has to consider that it may be impossible to utilize the full installed thrust force because the bearing capacity of the thrusting system may place an upper limit on the feasible thrust force. In the case of gripper TBMs, the thrust force may be limited by the bearing capacity of the ground around the grippers. With single or double shielded TBMs (the latter being operated in the so-called "auxiliary mode"), the actual thrust force is limited by the bearing capacity of the segmental lining. In this case, the combined action of the thrust force and of the ground pressure must be taken into account. It should additionally be noted that an insufficient backfilling of the segmental lining may reduce its bearing capacity.

For the majority of practical tunnelling issues, computational investigations are – together with experience gained from projects with comparable geological conditions – indispensable for evaluating potential hazards, as they provide indications as to the magnitude of the key parameters. This is particularly true for assessments of thrust force requirements, because such assessments rely on a detailed knowledge of the distribution of the ground pressure acting upon the shield. Plausibility considerations and engineering judgement, although indispensable, offer limited help in this respect. Empirical methods – such as the ones mentioned in Part II – are based upon correlations of field data obtained in specific projects with potentially different conditions. The widely used simplified closed-form solutions are also of limited value for the problem under consideration as they provide only a rough assessment of the squeezing potential without providing any information concerning ground pressure distribution in the longitudinal direction. Furthermore, the assumption of plane strain conditions, which underlies the closed-form solutions, introduces large errors in the case of heavily squeezing ground (Cantieni and Anagnostou, 2009a). Three-dimensional numerical models take account of the spatial stress redistribution in the vicinity of the advancing face and thereby remedy, at least in principle, these difficulties.
Part III presents the assumptions, the method and the results of a comprehensive parametric study into the thrust force required for overcoming friction. The numerical investigations cover the relevant range of material constants, initial stress and TBM characteristics and have been carried out for many different combinations of these parameters. One of the important components of this parametric study was the development and implementation of efficient (in terms of computer time and stability) numerical solution methods specifically for this purpose (cf. Part II). Based upon the results of the parametric study, dimensionless design nomograms have been developed 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. The nomograms also make it possible to evaluate rapidly the effect of potential design parameters and operational measures such as reductions in the shield length, the installation of a higher thrust force, increases in the overcut or the lubrication of the shield surface. Part III also includes a large amount TBM technical data that is helpful in assessing the technical feasibility of such measures, i.e., for checking whether a particular technical measure is within the normal range of present-day TBMs.

This is the first time that such a systematic and thorough investigation of the combined effects of the parameters governing shield loading has been attempted. The present part of the report extends and improves previous research work by the author (Ramoni and Anagnostou, 2006) with respect to several points: (i) a more realistic modelling of the tunnel boundary (cf. Part II); (ii) a more realistic integration of the elasto-plastic stress-strain relations (cf. Part II); (iii) reasonably conservative assumptions concerning the stiffness of the shield and of the lining (Sections 4.1 and 4.2); (iv) use of a more appropriate normalizing factor for the required thrust force (Section 3); (v) a far more comprehensive parametric study (Sections 4 and 5). From the practical point of view, it is important to note that the assumptions of Part III lead to higher shield loadings than the initial model by Ramoni and Anagnostou (2006).

2 Computational model

The numerical investigations of this part of the PhD thesis are based upon the axially symmetric computational model presented in Part II. In the present section, Part III discusses the underlying simplifying assumptions, which must be borne in mind when using the nomograms presented in Section 5.

2.1 Axial symmetry

The assumption of axial symmetry presupposes, in addition to the obvious condition of a cylindrical tunnel, that, (i), the initial stress field is hydrostatic, (ii), the material behaviour is isotropic, and, (iii), the tunnel support is uniform. The first two conditions are usual in the analysis of overstressed rock masses (cf., e.g., Hoek and Marinos, 2000a; 2000b; Kovári, 1998), while condition (iii) implies that the effect of the TBM weight can be neglected and, in the case of a segmental lining, that annulus grouting is uniform along the tunnel periphery.
The most significant consequence of the assumption of axial symmetry is that only normal forces are seen developing in the shield and in the lining, while in reality bending may also occur. Given the goal of the design nomograms (which is to assess thrust force requirements), however, it is sufficient to determine the overall ground pressure acting upon the shield. As a consequence of the assumption of axial symmetry, the pressure obtained is "homogenized" over the tunnel cross-section due to the fact that the model assumes an overcut that is constant around the circumference of the shield, while in reality the shield slides along the tunnel floor, which means that the overcut is bigger above the centre than in the lower portion of the tunnel cross-section.

Another consequence of the assumption of axial symmetry is that the response of the model shield and lining tends to be stiffer than it is in reality and this should lead to an overestimation of the ground pressure and of the necessary thrust force.

2.2 Variability of the ground

The nomograms have been worked out assuming a homogeneous ground. Although this assumption is standard, it should be borne in mind that the intensity of squeezing may vary considerably within short stretches along the tunnel alignment (Cantieni and Anagnostou, 2007) and, as pointed out by Kovári (1998), this variability is often responsible for the setbacks often experienced when tunnelling in squeezing ground.

The assumption of homogeneity presupposes that uniform ground conditions persist along the alignment and may be conservative if the TBM crosses a single short geological fault zone. Past research (Cantieni and Anagnostou, 2007; Kovári and Anagnostou, 1995) has shown that in this case the adjacent competent rock has a stabilizing effect as shear stresses are mobilized at its interface with the weaker zone. The shorter the fault zone, the more pronounced the favourable effect of the adjacent competent rock (see also Part II). This so-called "wall effect" also has a positive influence in TBM tunnelling (cf., e.g., Graziani et al., 2007a; Matter et al., 2007; Ramoni and Anagnostou, 2008).

2.3 Time-dependency of ground behaviour

In order to reduce complexity, time-dependent ground response – due to creep or consolidation, cf. Anagnostou and Kovári (2005) – is not taken into account here. According to this simplifying assumption, all plastic deformations occur instantaneously. As the actual behaviour of squeezing ground is often time-dependent (i.e., the deformations develop with a delay with respect to excavation), the model tends to overestimate deformations in the vicinity of the advancing face and the pressure acting upon the shield during continuous excavation. Consequently, with respect to the shield loading, a safety margin exists in this case, which may be bigger or smaller depending on how quickly the ground responds to the excavation. This safety margin can be taken into account qualitatively in the evaluation of the results, but this presupposes that sufficient knowledge exists as to the behaviour of the ground over time. In most cases there is a large uncertainty about the
time-development of squeezing and it may be unwise to rely upon the hypothesis that deformations will develop slowly far behind the face. It should be noted that rapidly developing convergences have been observed in a number of tunnels in the past (cf. Part I).

2.4 Constitutive model

For the mechanical behaviour of the ground, the standard linearly elastic, perfectly plastic material model with the Mohr-Coulomb yield criterion and non-associated flow rule has been adopted. The implementation of a more complex material law would be possible and might lead to a better representation of phenomena such as softening. However, such models contain a larger number of parameters which cannot be determined with sufficient reliability and therefore necessitate additional assumptions, which themselves introduce further uncertainties. Furthermore, there are in practice many relevant materials – e.g., intensively sheared or kakiritic rocks – which exhibit a material behaviour that can be mapped reasonably well by the assumed constitutive law (Vogelhuber, 2007). In the case of a non-linear yield condition, equivalent Mohr-Coulomb parameters can be used (cf., e.g., Sofianos and Nomikos, 2006).

The material model, which has been employed, is familiar in tunnel engineering practice and it offers the additional advantage of having a relatively small number of easily interpretable parameters (Kovári, 1977): the Young's modulus $E$, the Poisson's ratio $\nu$, the uniaxial compressive strength $f_c$, the angle of internal friction $\phi$ and the dilatancy angle $\psi$. It is, therefore, a reasonable model at least for making an initial assessment of a TBM drive (which is the objective of the design nomograms).

Even with this relatively simple constitutive model, the number of problem parameters is still large and, therefore, a trade-off has had to be made between the completeness of the parametric study and the cost of computation and data processing. For this reason, the Poisson's ratio was kept constant to $\nu = 0.25$, while the dilatancy angle $\psi$ was not treated as an independent parameter but was taken as a function of the angle of internal friction $\varphi$ (Vermeer and de Borst, 1984):

$$\psi = \begin{cases} 1^\circ & \text{for } \phi \leq 20^\circ \\ \phi - 20^\circ & \text{for } \phi > 20^\circ \end{cases}$$

Equation 1 leads to dilatancy angles of up to $5^\circ$ for friction angles of $\phi = 20$–$25^\circ$. This prediction is reasonable according to laboratory results obtained by ETH Zurich with weak squeezing rocks (Vogelhuber, 2007). In the case of a more pronouncedly dilatant behaviour of the ground, however, the assumption made (Equation 1) would underestimate the squeezing pressures and deformations.

As for every geomechanical analysis, the estimation of representative ground parameters (based on experience, laboratory tests, field tests or back analysis of field measurements) is demanding.
The inherent uncertainties associated with the material constants of the ground should be kept in mind when using the nomograms. In this respect, sensitivity analyses are indispensable.

3 Dimensionless parameters

With respect to the main goal of the present investigation, which is to determine the required thrust force, the main computational result is the spatial distribution of the radial pressure acting upon the shield. Figure 1 shows a typical load distribution $p(y)$. This example concerns a 400 m deep, 10 m diameter tunnel that is driven by a 10 m long shielded TBM and supported by a 30 cm thick segmental lining. The load concentration in the rear part of the shield is a consequence of the complete unloading experienced by the tunnel boundary at $y = L$ (cf. Part II). The load concentration is nevertheless of minor importance for the required thrust force, as it affects only a narrow area.

The thrust force $F_t$ required to overcome shield skin friction can be calculated by integrating the ground pressure $p$ over the shield surface:

\[ F_t = \mu 2\pi R \int_0^L p(y) dy = \mu 2\pi RL \int_0^L \frac{dy}{L} = \mu 2\pi RL p_s , \]

where $\mu$ denotes the skin friction coefficient, $R$ is the tunnel radius and $p_s$ the average ground pressure acting upon the shield.

The required thrust force $F_t$ generally depends on all of the parameters of the problem under consideration, which are the material constants of the ground (Young’s modulus $E$, Poisson’s ratio $\nu$, uniaxial compressive strength $f_c$, angle of internal friction $\varphi$ and dilatancy angle $\psi$), the initial

![Figure 1. Ground pressure $p$ acting upon the shield and the lining.](image-url)
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 \( K_l \):

\[
F_f = f (E, \nu, f_s, \varphi, \psi, \sigma_0, R, \Delta R, L, K_s, \mu, K_l),
\]

where the radial gap size \( \Delta R \) is assumed to be constant. The number of parameters can be reduced by performing a dimensional analysis and taking into account Equation 2:

\[
F_f^* = \frac{F_f}{2\pi RL \sigma_0} = f \left( \frac{E}{\sigma_0}, \nu, f_s, \frac{\varphi}{\sigma_0}, \psi, \frac{\Delta R}{R}, \frac{L}{R}, \frac{K_s}{E}, \frac{K_l}{E} \right).
\]

(4)

A further reduction in the number of parameters is possible by taking into account that the displacements \( u \) of an elasto-plastic medium depend linearly on the reciprocal value of its Young’s modulus \( E \), i.e., the product of \( E \) by \( \sigma_0 \) is constant (Anagnostou and Kovári, 1993). Thanks to this general property of elasto-plastic materials, it is reasonable to expect that the product of \( E \) by \( \Delta R \) (rather than the individual parameters \( E \) and \( \Delta R \)) will be significant and, consequently, the normalized required thrust force \( F_f^* \) depends on the product of the two normalized parameters \( E/\sigma_0 \) and \( \Delta R/R \):

\[
F_f^* = \frac{F_f}{2\pi R L \sigma_0} = f \left( \frac{E}{\sigma_0}, \nu, f_s, \frac{\varphi}{\sigma_0}, \psi, \frac{\Delta R}{R}, \frac{L}{R}, \frac{K_s}{E}, \frac{K_l}{E}, \frac{\Delta R/\sigma_0}{E} \right).
\]

(5)

This theoretical hypothesis was tested specifically for the problem under consideration by means of a series of numerical calculations, which were carried out for the three TBM types (gripper TBM, single shielded TBM and double shielded TBM) and for the entire range of material constants and initial conditions. In order to illustrate the approach to testing the hypothesis, consider the example of a single shielded TBM with a normalized shield length of \( L/R = 2.0 \). Figure 2a shows the normalized required thrust force \( F_f^* \) as a function of \( E/\sigma_0 \). Each curve applies to another value of \( \Delta R/R \). The correctness of the normalization of Equation 5 can be tested by plotting the results of Figure 2a as a function of the product of \( E/\sigma_0 \) and \( \Delta R/R \). As can be seen from Figure 2b, all points fall on one single curve.

As Equation 5 assumes a constant radial gap size, a slightly different expression applies to the case of double shielded TBMs having a bigger gap in the rear shield than in the front shield:

\[
F_f = \frac{2\pi RL \sigma_0}{F_f} = f \left( \frac{E}{\sigma_0}, \nu, f_s, \varphi, \psi, \frac{L}{R}, \frac{K_s}{E}, \frac{K_l}{E}, \frac{\Delta R_f}{R}, \frac{\Delta R_r}{L} \right),
\]

(6)

where \( \Delta R = \Delta R_f \) denotes the overcut of the front shield; \( L \) is the total shield length; and \( \Delta R_r \) and \( L_r \) denote the radial gap size and the length of the rear shield, respectively.

The normalizing factor \( 2\pi R L \sigma_0 \) introduced by Equation 4 is better than the factor \( \mu \pi R^2 \sigma_0 \) initially proposed by Ramoni and Anagnostou (2006), because the normalized force \( F_f^* \) has a clear physical meaning: it is equal to the ratio of average ground pressure \( p_s \) acting upon the shield to the initial stress \( \sigma_0 \) (cf. Equation 2), i.e.,

\[
p_s = F_f/\sigma_0.
\]

(7)

The design nomograms presented in this report are also useful, therefore, with respect to the structural design of the shield. Although the distribution of the ground pressure \( p(y) \) is non-linear
4 The parameter range covered

In order to reduce computational effort, the numerical analyses have been carried out only for selected machine and lining parameters.
4.1 Shield stiffness

The stiffer the shield, the higher will be the ground pressure upon it. All calculations have been carried out for a normalized shield stiffness of $K_s R/E = 10$. If the actual stiffness $K_s R/E$ is lower than this value, the required thrust force will be overestimated by the nomograms. The value of $K_s R/E = 10$ corresponds to a rather stiff shield and has been chosen in order to be on the safe side for most cases. Figure 3 shows the influence of this dimensionless parameter on the normalized required thrust force $F_f^*$ for the given ground conditions and typical geometrical data of single

Figure 3. Effect of the normalized stiffness of the shield $K_s R/E$ and of the normalized stiffness of the lining $K_l R/E$ on the normalized required thrust force $F_f^*$ for single shielded TBM with a normalized shield length of $L/R = 2.0$ (a) and $L/R = 5.0$ (b).
shielded TBMs. It can be observed that the overestimation of $F_f^*$ can be quite relevant in the case of thin shields or a ground with a high Young's modulus $E$.

### 4.2 Lining stiffness

Figure 3 also shows the effect of the normalized stiffness of the lining $K_{LR}/E$. It is interesting to note that the stiffer the lining, the lower the required thrust force. As can be observed by comparing Figure 3a, which applies for a normalized shield length $L/R = 2.0$, with Figure 3b ($L/R = 5.0$) this effect is more pronounced for short than for long shields. The reason for this behaviour is that a stiff lining, which is installed close to the face, facilitates arch action in the longitudinal direction and thus reduces shield loading (cf. Parts I and II).

The calculations for the shielded TBMs have been carried out assuming a normalized lining stiffness of $K_{LR}/E = 0.5$. As this value corresponds to a rather soft lining, the nomograms are on the safe side for most cases. This conservative assumption is also justified by the fact that the annulus backfill has a lower stiffness than the segmental lining, thus reducing the overall stiffness of the system (the stiffness $K_i$ introduced in the computation has to be seen as an "average" one). On the other hand, if the actual normalized lining stiffness $K_{LR}/E$ is lower than 0.5, the required thrust force will be underestimated by the nomograms.

In the case of gripper TBMs, the installation of a stiff lining in the machine area is difficult. It has also to be considered that the development of stiffness in the shotcrete takes time and is, in general, slow relatively to the advance rate. On account of the uncertainties existing with respect to the support stiffness immediately behind the shield, the decisions has been taken to work out the gripper TBM nomograms without taking into account a tunnel support (i.e., $K_{LR}/E = 0$).

It should be noted that in the previous work by Ramoni and Anagnostou (2006) the assumption of a practically rigid segmental lining was made and the local unloading of the ground at the installation point of the lining was neglected (cf. Figure 1). These simplifying assumptions overestimate the effect of the arch action in the longitudinal direction (cf. Part II), thus leading to lower shield loads than the nomograms of the present report.

### 4.3 Machine layouts

The calculations have been carried out for a limited number of normalized shield length values $L/R$, which however are typical for the different TBM types and have been selected on the basis of the technical data of geometrical nature collected in Figure 4.

The normalized shield length is shorter for gripper TBMs ($L/R = 1.0$) than for single shielded TBMs ($L/R = 2.0$ to 5.0, depending on the tunnel diameter, the larger values applying to small diameters). This is also true for double shielded TBMs, which show, however, a wider variation of the normalized shield length. In order to cover the relevant range better, the calculations have been carried out for three values ($L/R = 2.0$, $L/R = 3.5$ and $L/R = 5.0$). Furthermore, as different components of
the thrusting system are employed for advancing the front and the rear shield, the ground pressure and the frictional resistance of the two shields have to be considered separately and this necessitates assumptions as to their lengths. On account of Figure 4c, the ratio between the length of
the rear shield \( L_r \) and the length of the front shield \( L_f \) was taken as 1.5 in the numerical analyses (i.e., the length of the rear shield \( L_r \) amounts to 60% of the total length \( L \)). The overcut \( \Delta R \) is also bigger for the rear shield \( (\Delta R_r) \) than it is for the front shield \( (\Delta R_f) \). A typical ratio of \( \Delta R_f / \Delta R_r = 1.5 \)
was assumed (cf. Figure 4a) and separate nomograms have been worked out for the front and the rear shield. The thrust force required for moving the entire double shield can be determined easily by adding the forces necessary for overcoming the skin friction of its two parts.

Figure 5 enhances the TBM data collection mentioned above by presenting data about thrust force and torque. Figure 5 allows, e.g., to evaluate quickly whether the required thrust force determined with the nomograms is feasible or not. The data has been collected based upon a comprehensive literature study concerning a number of projects. The detailed data underlying Figures 4 and 5 can be found in Table 4 (located at the end of Part III).

5 The nomograms

Bearing in mind that the values of some of the parameters have been fixed as discussed above, the normalized required thrust force depends on the four remaining parameters:

\[
\frac{F_f}{\mu 2\pi RL \sigma_0} = f \left( \frac{E}{\sigma_0 R}, \frac{f_u}{\sigma_0}, \phi, \frac{L}{R} \right).
\]

Equation 8 is represented graphically in the diagrams of Figures 6 to 14. Each figure applies to a different TBM type and normalized shield length \(L/R\), while each diagram applies to a different value of the angle of internal friction \(\phi\) and includes a band of curves (each curve corresponding to another value of the normalized uniaxial compressive strength \(f_u/\sigma_0\)) showing the normalized thrust force \(F_f^*\) required to overcome shield skin friction as a function of the dimensionless product of \(E/\sigma_0\) by \(\Delta R/R\). Table 1 contains an overview of the nomograms and of the parameter ranges under consideration.

6 Model behaviour

According to the numerical results of Figures 6 to 14, the shorter the shield and the larger the radial size of the gap, the lower will be the required thrust force. The effect of the uniaxial compressive strength of the ground is, however, not as straightforward as one might expect. At low values of \((E/\sigma_0)R\), an increase in the normalized uniaxial compressive strength \(f_u/\sigma_0\) leads to an increase in the normalized required thrust force \(F_f^*\), i.e., for a given value of \((E/\sigma_0)R\), a better quality ground requires a higher thrust force. This counter-intuitive behaviour is well-known in the literature. Other authors (Boldini et al., 2000; Graziani et al., 2005; Matter et al., 2007) obtained similar results from numerical calculations carried out using the classic step-by-step method. This effect can also be observed when applying the convergence-confinement method and, as pointed out by Graziani et al. (2005), it is due to the pre-deformations of the ground ahead of the face. In a lower strength ground, the core ahead of the tunnel face plastifies more and this promotes stress-relief.
7 Application examples

The use of the design nomograms will be demonstrated by returning to the example of Figure 1 (a 400 m deep, 10 m diameter tunnel, which is driven by a 10 m long single shielded TBM). Table 2 contains the input parameters as well as detailed, step-by-step calculations. Columns 1 to 5 show how to compute the required thrust force for given ground conditions, TBM parameters and operating states, i.e., these columns deal with dimensioning, while Columns 6 to 9 address the inverse problem and show how to estimate the feasibility limits of a given TBM (in terms of the ground quality and the given thrust force, shield length, overcut, etc.).
7.1 Determining the required thrust force

Columns 1 to 3 calculate the force required during the boring process for normal, medium and major overcuts $\Delta R$ (of 5, 10 or 15 cm, respectively, see Row 10), while Columns 4 and 5 concern the thrust force requirements for restarting the machine after a standstill. As no time-dependency is considered in respect of the ground behaviour, the two operational states differ only with respect to the skin friction coefficient $\mu$ (static vs. sliding friction, Row 15) and to the force $F_r$, required for the boring process (Row 19), on the assumption that the boring thrust force is equal to zero at the moment of restarting the TBM (i.e., with a retractable cutter head).

Considering the present normalized shield length of $L/R_0 = 2.0$ (Row 26), Figure 7 applies, while the appropriate curve has to be selected according to the actual values of the angle of internal friction ($\phi = 25^\circ$, Row 4) and the normalized uniaxial compressive strength ($f_c/\sigma_0 = 0.3$, Row 24). The dimensionless gap size (Row 29) is calculated on the basis of the parameters of Rows 1, 8, 9 and 10 and is entered into the nomogram in order to depict the value of the normalized force $F_*^r$ (Row 30) and to calculate the required thrust force $F_r$ (Row 32). For a normal overboring of $\Delta R = 5$ cm, the required thrust force amounts to $F_r = 263$ MN (Row 32, Column 1). This value exceeds the installed thrust force ($F_i = 150$ MN, Row 16), thus indicating that the shield may become jammed. As the value of 263 MN is far beyond the normal range (cf. Figure 5a), it is questionable whether a higher thrust force can be installed at all. However, a larger overboring ($\Delta R = 10$ cm) leads to a considerable reduction in the required thrust force to a feasible value of $F_r = 93$ MN (Row 32, Column 2). In the case of a major overboring ($\Delta R = 15$ cm), the shield would remain unloaded ($F_r = 0$, Row 31, Column 3) and a thrust force would be needed only for the boring process ($F_r = 18$ MN, Row 32, Column 3).

The thrust force required in order to restart the TBM can be calculated analogously (Columns 4 and 5) and amounts to 367 and 113 MN, respectively, i.e., it is higher than the thrust force required during the boring process. This is due to the change in the friction regime (static instead of sliding friction), which overweighs the advantage of not having to take into account a boring thrust force. The contrary is also possible, particularly for short shields, where the friction may become less relevant relative to the boring thrust force.

The results obtained by applying the nomograms agree well with the results of the numerical analyses that were carried out specifically for the example of Figure 1 and lead to the same basic conclusions. For this example, the design nomograms overestimate the required thrust force by about 15–25%. According to Figure 3a this deviation can be traced back to the overestimation of the shield stiffness and to the underestimation of the stiffness of the segmental lining. A single shielded TBM typically has a 12 cm thick shield (Maidl et al., 1995). For the example under consideration, the actual normalized stiffness of the shield would be $K_{sR}/E \approx 5$ (Row 27) instead of the value of 10 which underlies the nomograms, while the actual normalized stiffness of a 30 cm thick segmental lining would be $K_{lR}/E \approx 1.8$ (Row 28, neglecting the compressibility of the backfill) instead of 0.5.

The results of the examples discussed above suggest that, for the ground conditions under consideration, both continuous excavation and a restart after a standstill are possible with a medium size overboring ($\Delta R = 10$ cm). As discussed in Part I, the reliability of overboring may nevertheless be
limited (particularly for hard rocks). It should be noted, furthermore, that the overcut (together with the conicity of the shield) must first of all allow the TBM to be steered. If the ground deforms and closes the gap, any attempt to steer the TBM will lead to the development of constraint loads. In this respect, it is necessary to build in some reserves when applying the nomograms (i.e., a reduced radial gap size $\Delta R$ should be introduced into the calculations).

Table 2. Application examples.

<table>
<thead>
<tr>
<th>Application example</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
</tr>
</thead>
<tbody>
<tr>
<td>State</td>
<td>continuous excavation</td>
<td>restart after a standstill</td>
<td></td>
<td></td>
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<td>Thrust force requirements</td>
<td>$F_r = ?$</td>
<td>$F_r = F_f$</td>
<td>$F_r = 0$</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Ground</td>
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<td></td>
</tr>
<tr>
<td>1 Young's modulus</td>
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<td>1000</td>
<td>1000</td>
<td>1000</td>
<td>9000</td>
<td>1000</td>
<td>1425</td>
</tr>
<tr>
<td>2 Poisson's ratio</td>
<td>$\nu$ [-]</td>
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<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3 Uniaxial compressive strength</td>
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<td>3.0</td>
<td>3.0</td>
<td>3.0</td>
<td>3.0</td>
<td>3.0</td>
<td>2.4</td>
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<tr>
<td>4 Angle of internal friction</td>
<td>$\varphi$ [']</td>
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<td></td>
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<tr>
<td>5 Dilatancy angle</td>
<td>$\psi$ [']</td>
<td>5</td>
<td></td>
<td></td>
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<tr>
<td>6 Unit weight</td>
<td>$\gamma$ [kN/m$^2$]</td>
<td>25</td>
<td></td>
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<td></td>
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<tr>
<td>7 Initial stress</td>
<td></td>
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<td></td>
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<td>8 Depth of cover</td>
<td>$H$ [m]</td>
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<tr>
<td>9 Boring diameter, boring radius</td>
<td>$D, R$ [m]</td>
<td>10, 5</td>
<td></td>
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<tr>
<td>10 Radial gape size</td>
<td>$\Delta R$ [cm]</td>
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<td>10</td>
<td>15</td>
<td>5</td>
<td>10</td>
<td>10</td>
<td>10</td>
<td>10</td>
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<tr>
<td>11 Length (shield)</td>
<td>$L$ [m]</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>12 Young's modulus (shield)</td>
<td>$E_s$ [MPa]</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>13 Thickness (shield)</td>
<td>$d_s$ [cm]</td>
<td>12</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>14 Stiffness (shield)</td>
<td>$K_s$ [MPa/m]</td>
<td>1008</td>
<td></td>
<td></td>
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<tr>
<td>15 Skin friction coefficient</td>
<td>$\mu$ [-]</td>
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<td>0.30</td>
<td>0.30</td>
<td>0.30</td>
<td>0.30</td>
<td>0.30</td>
<td>0.30</td>
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<tr>
<td>16 Installed thrust force</td>
<td>$F_i$ [MN]</td>
<td>150</td>
<td></td>
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<td></td>
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<tr>
<td>17 Maximum cutter force</td>
<td>$F_c$ [kN]</td>
<td>267</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>18 Number of cutters</td>
<td>$n_c$ [-]</td>
<td>67</td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>19 Thrust force (boring process)</td>
<td>$F_f$ [kN]</td>
<td>18 $^a$</td>
<td>18 $^a$</td>
<td>18 $^a$</td>
<td>18 $^a$</td>
<td>18 $^a$</td>
<td>0</td>
<td>0</td>
<td>0</td>
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<tr>
<td>20 Young's modulus</td>
<td>$E_l$ [MPa]</td>
<td>30000</td>
<td></td>
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<tr>
<td>21 Thickness</td>
<td>$d_l$ [cm]</td>
<td>30</td>
<td></td>
<td></td>
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<tr>
<td>22 Stiffness</td>
<td>$K_l$ [MPa/m]</td>
<td>360 $^b$</td>
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<tr>
<td>Dimensionless products</td>
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<tr>
<td>23 $E/\sigma_0$ [-]</td>
<td>100.00</td>
<td>100.00</td>
<td>100.00</td>
<td>100.00</td>
<td>100.00</td>
<td>90.00</td>
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<tr>
<td>24 $f_u/\sigma_0$ [-]</td>
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<td>0.30</td>
<td>0.30</td>
<td>0.30</td>
<td>0.30</td>
<td>0.30</td>
<td>0.30</td>
<td>0.24 $^i$</td>
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</tr>
<tr>
<td>25 $\Delta R/R$ [-]</td>
<td>0.01</td>
<td>0.02</td>
<td>0.03</td>
<td>0.01</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>26 $L/R$ [-]</td>
<td>2.00</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>27 $K_s R/E_s$ [-]</td>
<td>5.04</td>
<td></td>
<td></td>
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<td></td>
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<tr>
<td>28 $K_l R/E_l$ [-]</td>
<td>1.80</td>
<td></td>
<td></td>
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<td></td>
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</tr>
<tr>
<td>29 $(E/\sigma_0) (\Delta R/R)$ [-]</td>
<td>1.00</td>
<td>2.00</td>
<td>3.00</td>
<td>1.00</td>
<td>2.00</td>
<td>1.80 $^i$</td>
<td>2.00</td>
<td>2.85 $^i$</td>
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</tr>
<tr>
<td>30 Normalized required thrust force</td>
<td>$F_f^*$ [-]</td>
<td>0.26 $^i$</td>
<td>0.08 $^i$</td>
<td>0 $^i$</td>
<td>0.26 $^i$</td>
<td>0.08 $^i$</td>
<td>0.11 $^i$</td>
<td>0.11 $^i$</td>
<td>0 $^i$</td>
</tr>
<tr>
<td>31 Required thrust force (friction)</td>
<td>$F_i$ [MN]</td>
<td>245 $^j$</td>
<td>75 $^j$</td>
<td>0 $^j$</td>
<td>367 $^j$</td>
<td>113 $^j$</td>
<td>150 $^m$</td>
<td>150 $^m$</td>
<td>0 $^m$</td>
</tr>
<tr>
<td>32 Total required thrust force</td>
<td>$F_r$ [MN]</td>
<td>263 $^n$</td>
<td>93 $^n$</td>
<td>18 $^n$</td>
<td>367 $^n$</td>
<td>113 $^n$</td>
<td>150 $^o$</td>
<td>150 $^o$</td>
<td>0 $^o$</td>
</tr>
</tbody>
</table>

Notes

- $\sigma_0 = \gamma H$
- $K_s = E_d R^2$
- After Gehring (1996)
- Assumed
- After Sänger (2006)
- $n_c = 6.7 D$, after Vigl et al. (1999)
- $F_f = F_f^* / \mu^2 \pi R L / \sigma_0$
- $F_i = F_i / \mu^2 \pi R L / \sigma_0$
- $F_r = F_f^* / \mu^2 \pi R L / \sigma_0$
- "Given", in this column the calculation proceeds from down to top
7.2 Vulnerability with respect to ground variations

The variability of squeezing along a tunnel alignment may be significant (Cantieni and Anagnostou, 2007). It is therefore important to investigate the influence of variations in the ground conditions on the required thrust force. For the sake of simplicity, one assumes that the ground can be described by combinations of its Young's modulus $E$ and its uniaxial compressive strength $f_c$ and that all other ground parameters can be kept constant. Combinations of these two parameters would thus describe the "quality" of the ground.

The reference ground for the investigations in this section of the report is that of Column 5 of Table 2, i.e., a Young's modulus of $E = 1000 \text{ MPa}$ and a uniaxial compressive strength of $f_c = 3 \text{ MPa}$. Assuming a medium overboring ($\Delta R = 10 \text{ cm}$), a thrust force of 113 MN is required in order to restart TBM advance (Row 32, Column 5). This value is high but thoroughly feasible and lower than the installed thrust force of 150 MN.

How much worse would have to be the ground in order for the TBM to become trapped? In order to answer this question the required thrust force $F_r$ is made equal to the installed thrust force of $F_i = 150 \text{ MN}$, the uniaxial compressive strength $f_c$ is kept equal to the reference value of 3 MPa and the Young's modulus $E$ is calculated back (down to top). According to the calculation in Column 6, a decrease in the Young's modulus $E$ of just 10% may endanger the TBM drive. A similar result can be achieved keeping the Young's modulus $E$ equal to its reference value of 1000 MPa and calculating the critical reduction of the uniaxial compressive strength $f_c$: according to Column 7, a reduction of 20% would lead to the jamming of the shield.

In the interests of a stricter set of design criteria, this section will now address the other borderline case, where the required thrust force $F_r$ is equal to zero. How much better should the ground be in order that no squeezing pressure develops upon the shield? This condition is true for the intersection points of the curves with the $x$-axes of the nomograms (i.e., for $F_f^* = 0$). According to Columns 8 and 9, the Young's modulus $E$ should be higher by about 40%, while the necessary uniaxial compressive strength $f_c$ amounts about 1.5 time the reference value of Column 5.

The examples in this section of the report demonstrate that the design nomograms allow a quick sensitivity analysis to be made, while also identifying the critical geotechnical conditions with respect to a given set of design criteria. The nomograms also make it possible to assess planned TBM drives, at least on a preliminary basis, as well as the effects of other possible design measures such as lubrication of the extrados of the shield or reductions in shield length.

8 Analysis of case histories

In this section, the suitability of the proposed design nomograms will be shown by means of a simplified back-analysis of seven case histories. Table 3 summarizes the project data, the TBM data, the geology of the critical stretches, the ground parameters and the references. Furthermore, in its
Table 3. Analysis of case histories.

<table>
<thead>
<tr>
<th>Project (country)</th>
<th>TBM type, Manufacturer</th>
<th>Start year of TBM drive [Reference]</th>
<th>TBM</th>
<th>Depth of cover</th>
<th>Ground</th>
<th>Thrust force</th>
<th>F1 [MN]</th>
<th>F2 [MN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gotthard Base Tunnel, Bodio Section</td>
<td>Herrenknecht</td>
<td>2003 [1]</td>
<td>D [m] = 8.80</td>
<td>H [m] = 1000</td>
<td>$E$ [MPa] = 4400–18000</td>
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<td>0–117 b,c</td>
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<td>$\Delta R$ [cm] = 6 a</td>
<td>$f_{ci}$ [MPa] = 5.2–34.5</td>
<td>$\phi'$ = 32–49</td>
<td>$\gamma$ [kN/m²] = 26</td>
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<td></td>
<td>$\Delta R$ [cm] = 12 d</td>
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<td></td>
<td>$L$ [m] = 5.00</td>
<td>$\Delta R$ [cm] = 4 a</td>
<td>$f_{ci}$ [MPa] = 2.5</td>
<td>$\phi'$ = 32</td>
<td>$\gamma$ [kN/m²] = 26</td>
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<td>$\Delta R$ [cm] = 4</td>
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<td>Ulubat Tunnel (Turkey)</td>
<td>Single shielded TBM, Herrenknecht</td>
<td>2006 [3]</td>
<td>D [m] = 5.05</td>
<td>H [m] = 120</td>
<td>$E$ [MPa] = 200–1000</td>
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<td>0–85 b,g</td>
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<td>$f_{ci}$ [MPa] = 0.2–1.2</td>
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<td>$\gamma$ [kN/m²] = 25</td>
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<td>$L$ [m] = 11.00</td>
<td>$\Delta R$ [cm] = 7 a</td>
<td>$f_{ci}$ [MPa] = 1.5–3.0</td>
<td>$\phi'$ = 23</td>
<td>$\gamma$ [kN/m²] = 26</td>
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<td>$\Delta R$ [cm] = 4</td>
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<td>$\Delta R$ [cm] = 7 a</td>
<td>$f_{ci}$ [MPa] = 0.5–1.0 a</td>
<td>$\phi'$ = 15–20</td>
<td>$\gamma$ [kN/m²] = 25 a</td>
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<tr>
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<td>$\Delta R$ [cm] = 4 a</td>
<td>$f_{ci}$ [MPa] = 0.6–1.0</td>
<td>$\phi'$ = 23</td>
<td>23.3 b, 26–38 i,k,m</td>
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<tr>
<td>Guadarrama Tunnel (Spain)</td>
<td>Double shielded TBM, Wirth-NFM</td>
<td>2002 [7]</td>
<td>D [m] = 9.46</td>
<td>H [m] = 300</td>
<td>$E$ [MPa] = 300–700</td>
<td>89.0</td>
<td>35–73 i,j,n</td>
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<td>$L$ [m] = 15.24</td>
<td>$\Delta R$ [cm] = 4</td>
<td>$f_{ci}$ [MPa] = 0.5–7.5</td>
<td>$\phi'$ = 20</td>
<td>108.0 b, 57–135 i,k,a</td>
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<td>$\Delta R$ [cm] = 4</td>
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</tbody>
</table>

Legend

- D: Boring diameter
- E: Young's modulus of the ground
- $f_{ci}$: Uniaxial compressive strength of the ground
- $F_i$: Installed thrust force
- $F_r$: Required thrust force
- $H$: Depth of cover
- $L$: Length of the shield
- $L_f$: Length of the front shield
- $L_r$: Length of the rear shield
- $\Delta R$: Radial gap size
- $\Delta R_f$: Radial gap size (front shield)
- $\Delta R_r$: Radial gap size (rear shield)
- $\phi$: Angle of internal friction of the ground
- $\gamma$: Dilatancy angle of the ground

Notes

- a: Assumed
- b: Skin friction coefficient $\mu = 0.45$, after Gehring (1996)
- c: Figure 6
- d: With overboring
- e: Tail shield
- f: By installing removable auxiliary hydraulic jacks
- g: Figure 7 and Figure 8 (linear interpolation)
- h: Maximum possible thrust force
- i: Skin friction coefficient $\mu = 0.25$ (with lubrication of the shield mantle), after Gehring (1996)
- j: Main system
- k: Auxiliary system
- l: Figures 11 and 13 (linear interpolation)
- m: Figures 12 and 14 (linear interpolation)
- n: Figures 9 and 11 (linear interpolation)
- o: Figures 10 and 12 (linear interpolation)

References

right part, Table 3 compares the installed thrust force $F_i$ of the used TBMs with the required thrust force $F_r$, calculated using the design nomograms. For the sake of simplicity, Table 3 reports only the results concerning thrust force requirements for restarting the machine after a standstill.

8.1 Bodio Section of the Gotthard Base Tunnel (Switzerland)

The Bodio Section of the Gotthard Base Tunnel (Switzerland) was excavated by two gripper TBMs ($D = 8.80 \text{ m}$). Squeezing ground conditions were not anticipated in the planning phase. During construction, however, the shield of the western tube TBM was jammed in March 2006, the bored profile experienced a convergence of 7–10 cm in the machine area and of 14–22 cm in the back-up area and the tunnel support became damaged. At this time, the TBM drive was proceeding in strong anisotropic micaceous gneiss under a depth of cover of 1000 m approaching at a small angle a fault zone with a thickness of 3–5 m. The ground parameters reported in Table 3 are based upon laboratory tests carried out after the TBM became trapped. Depending on the ground parameters that are introduced, the required thrust force computed with the design nomograms varies from zero to very high values (Table 3), which, of course, are clearly not feasible with a gripper TBM. The large uncertainty – depending on the sort of ground encountered the TBM may or may not become trapped – was confirmed by what actually happened: the western TBM ground to a halt and its shield was damaged, while the following eastern TBM only slowed down without becoming trapped. The damage to the shield claimed by the contractor suggests that a high ground pressure did develop and, therefore, that the computed high values of the required thrust force are not unreasonable.

8.2 Faido Section of the Gotthard Base Tunnel (Switzerland)

This section of the Gotthard Base Tunnel is currently under construction by the refurbished gripper TBMs ($D = 9.43 \text{ m}$) that excavated the Bodio Section. The TBM drives started in July and October 2007, respectively, in micaceous gneiss under a depth of cover of 1600 m. After what happened in the Bodio Section and in the Faido Multi-function Station, squeezing ground conditions were expected. The Bodio TBMs were refurbished with a bigger boring diameter ($D = 9.43 \text{ m}$ instead of 9.30 m) and the TBM drive started with shifted gauge cutters (so-called "overboring", $D = 9.50 \text{ m}$). During excavation, convergences of up to 5–10 cm occurred in the shield area (observed at the shield tail about 5 m behind the face) but shield jamming was avoided. However, convergences in the machine and in the back-up area occurred over a length of 250 m (of up to 5–10 cm in the roof and of up to 30 cm in the floor of the eastern tube as well as of up to 25 cm in the roof and of up to 75 cm in the floor of the western tube). The yielding tunnel support thereby became damaged and the back-up locally jammed. The nomograms (applied with the design ground parameters) return a large range of values for the required thrust force depending on the variation of the Young's modulus $E$ (Table 3). The fact that the TBM did not become jammed does not mean that the nomograms cannot be applied but only suggests that the "quality" of the ground
was better than the expected "worst case" (in this regard, see also the investigations presented in Part II).

8.3 Uluabat Tunnel (Turkey)

The TBM drive (single shielded TBM, $D = 5.05$ m) started in April 2006 in a ground consisting of a claystone matrix (80 %) containing 1–50 cm big sandstone lenses (20 %). The claystones are intensively sheared, have several slickensides and disintegrate quickly under the action of water. During the first 3 km of TBM operation, squeezing caused jamming of the shield on several occasions although the depth of cover was rather moderate (120 m). An increase in the installed thrust force by applying additional thrust cylinders was not successful as it caused damage to the segmental lining. The available monitoring results are very sparse, but some observations indicated a large variability of squeezing intensity and maximum deformation rates of up to 6 cm/h. The range of ground parameters given in Table 3 is based upon laboratory tests. The latter are representative for the ground, given its very weak character up to the scale of the specimen. The computed thrust force requirements (Table 3) agree very well with what happened during the TBM drive: depending on the variation of the ground conditions, the shield may or may not become jammed (in this regard, see also the investigations presented in Part II).

8.4 Section 4 of the Pajares Tunnel (Spain)

The tunnel excavation started with a single shielded TBM ($D = 9.88$ m) in August 2006 and finished in July 2009. Heavily squeezing ground was expected in the so-called "Formigoso Formation", particularly in sections exhibiting a high degree of tectonization. In fact, the TBM became trapped in November 2007 in spite of the very high installed thrust force ($F_i = 193$ MN). The excavation was resumed with an increase of the thrust force ($F_i = 225$ MN) by installing removable auxiliary hydraulic jacks (this operation also required a reinforcement of the shield) and lubricating the shield mantle with bentonite in order to reduce shield skin friction. The thrust force resulting from the application of the design nomograms and computed with the design parameters (the "worst" ones in Table 3) is higher than the installed one ($F_r = 381$ MN vs. $F_i = 225$ MN), which was sufficient for resuming excavation. This can be explained by a number of factors, which concern the uncertainties related to the structure and the parameters of the ground. Firstly, as only the tail shield was blocked, it seems that the squeezing ground was localized in a short fault zone. In such a case, the design nomograms overestimate the required thrust force as they disregard the "wall effect" (cf. Section 2.2). Secondly, detailed field investigations showed that the rock mass was better than expected. In fact, the assumption of a "better" combination of ground parameters ($E = 4500$ MPa and $f_c = 3$ MPa, Table 3) leads to a lower required thrust force, which is near to the installed one ($F_r = 201$ MN vs. $F_i = 225$ MN).
8.5 Wienerwald Tunnel (Austria)

This tunnel was excavated by two single shielded TBMs ($D = 10.67$ m) between October 2005 and August 2007. Squeezing ground conditions did not occur. In the planning phase, however, attention was paid to the potential hazard of shield jamming with respect to the crossing of several long fault zones with poor ground conditions. For this case history, the required thrust force has been computed with a reduced skin friction coefficient (Table 3) in order to take account of the lubrication of the shield mantle carried out on the construction site as a preventive counter measure before entering the fault zones. The thrust force resulting from the application of the design nomograms is only slightly higher than the installed one (Table 3). This result can be traced back to the fact that the nomograms are on the safe side in most cases (cf., e.g., Sections 4.1 and 4.2).

8.6 Guadiaro – Majaceite Tunnel (Spain)

The TBM drive (double shielded TBM, $D = 4.88$ m) started in November 1995 and finished in February 1997. Squeezing ground conditions were expected in the planning phase and occurred during construction. The TBM was trapped when excavating flysch consisting almost entirely of claystone under a depth of cover of 400 m. The TBM drive was restarted after freeing of only 10% of the shield surface. This suggests that a slightly higher thrust force would be sufficient in order to avoid shield jamming. The required thrust force computed with the design nomograms (Table 3) agrees very well with what happened during construction.

8.7 Guadarrama Tunnel (Spain)

This tunnel was driven between October 2002 and June 2005 by four double shielded TBMs ($D = 9.46$ m and $D = 9.51$ m). Squeezing ground conditions were expected in the 600 m long fault zone "La Umbria" under a depth of cover of 300 m but did not occur under construction. This confirms the suitability of the result returned by the design nomograms (also in this case the computations of Table 3 take into account the lubrication of the shield mantle carried out during construction).

9 Closing remarks

Planning a TBM drive in squeezing ground is a complex problem. The tunnelling engineer is faced with a series of conflicting factors. A simplified assessment based only on plausibility considerations or on experience from projects with comparable geological conditions will not be sufficient. In
this respect, numerical analyses are helpful for evaluating potential hazards, as they provide indications of the magnitude of the key parameters.

The design nomograms presented in this report show in a condensed form the statical conditions that have to be fulfilled in the design of a TBM (with respect to the thrust force requirements) in order to avoid jamming of the shield when crossing squeezing ground. They cover the relevant range of material constants and TBM characteristics. When applied together with the extensive collection of TBM technical data, which reviews the state of present-day TBMs, they enable quantitative statements to be made concerning the feasibility of a TBM drive as well as the effectiveness of other measures, thus making a valuable contribution to decision-making in the design process.

The simplified back-analyses of the case histories (cf. Section 8) confirm the usefulness of the design nomograms for better understanding of the fundamental mechanisms of interaction between the ground and the TBM. Of course, the design nomograms cannot eliminate the uncertainties associated with ground parameters. Such uncertainties are intrinsic to every geomechanical calculation. In the planning phase, the tunnelling engineer deals with this uncertainty and utilises the computational results as a decision aid for his risk analysis. In this respect, the design nomograms represent a powerful tool.

As with every geomechanical computation, the underlying simplifying assumptions must be borne in mind when using the nomograms presented in the current report. The nature of these assumptions and the associated limitations of the nomograms have been discussed in Section 2. It should be noted, however, that most of the assumptions are the same as those employed in the widely used convergence-confinement method and, consequently, the results of the computations are subject to the same limitations. However, the computational model developed in the current report has the clear advantage of providing the longitudinal distribution of the ground pressure acting upon the shield (cf. Figure 1) while taking due account of the stress history, thus avoiding the errors introduced by plane strain analyses (Cantieni and Anagnostou, 2009a) and allowing for a reliable computation of the thrust force requirements.

However, in view of the difficulties associated with the initial stresses and the ground parameters, improving understanding of the relevant mechanisms and enabling comparative analyses of their impacts (rather than making apparently exact predictions) is perhaps the most important value of the presented nomograms.
Figure 6. Nomograms for gripper TBM (normalized shield length $L/R = 1.0$).
Figure 7. Nomograms for single shielded TBM (normalized shield length $L/R = 2.0$).
Figure 8. Nomograms for single shielded TBM (normalized shield length $L/R = 5.0$).
Figure 9. Nomograms for double shielded TBM (normalized shield length $L/R = 2.0$) – front shield.
Figure 10. Nomograms for double shielded TBM (normalized shield length $L/R = 2.0$) – rear shield.
Figure 11. Nomograms for double shielded TBM (normalized shield length $L/R = 3.5$) – front shield.
Figure 12. Nomograms for double shielded TBM (normalized shield length $L/R = 3.5$) – rear shield.
Figure 13. Nomograms for double shielded TBM (normalized shield length $L/R = 5.0$) – front shield.
Figure 14. Nomograms for double shielded TBM (normalized shield length $L/R = 5.0$) – rear shield.
### Table 4. Technical data of: (a) gripper TBMs; (b) single shielded TBMs; (c) double shielded TBMs.

#### (a) Gripper TBMs

<table>
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<tr>
<th>Project (country), Manufacturer, Start year of TBM drive</th>
<th>$D$ [m]</th>
<th>$L$ [m]</th>
<th>$F$ [MN]</th>
<th>$F_g$ [MN] ($p_g$ [MPa])</th>
<th>$T$ [MNm] ($r$ [rpm])</th>
</tr>
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<tbody>
<tr>
<td>Stillwater Tunnel (USA), Robbins, 1982</td>
<td>3.20</td>
<td>3.30</td>
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<td>3.40</td>
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<td>Älpe Devero Tunnel (Italy), Atlas Copco, 1989</td>
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<td>(2.9)</td>
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<td>6.4</td>
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<td></td>
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<td>6.3</td>
<td>18.7</td>
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<td>9.0</td>
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<td>22.0 (3.5)</td>
<td>1.3 (0–11.2)</td>
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<td></td>
<td>12.3</td>
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<td>1.9 (10.8), 3.5 (0–6.2)</td>
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Table 4 (continuation).

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<th>Project (country), Manufacturer, Start year of TBM drive</th>
<th>$D$ [m]</th>
<th>$L$ [m]</th>
<th>$F$ [MN]</th>
<th>$F_g$ [MN] ($p_g$ [MPa])</th>
<th>$T$ [MNm] ($r$ [rpm])</th>
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<tbody>
<tr>
<td>Brooklyn Tunnel (USA), Robbins, 1994</td>
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(b) Single shielded TBMs

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(b) Single shielded TBMs

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(c) Double shielded TBMs
Table 4 (continuation).

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<th>Project (country), Manufacturer, Start year of TBM drive</th>
<th>$D$ [m]</th>
<th>$L$ [m]</th>
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<th>$D_l$ [cm]</th>
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## Table 4 (continuation).

### (c) Double shielded TBMs

<table>
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<tr>
<th>Project (country), Manufacturer, Start year of TBM drive</th>
<th>$D$ [m]</th>
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<th>$D_s$ [cm]</th>
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<th>$D_t$ [cm]</th>
<th>$\Delta D_t$ [cm]</th>
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<th>$T$ [MNm] ($r$ [rpm])</th>
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<td>11.65</td>
<td>11.65</td>
<td>34</td>
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</table>

### Legend

- $D$: Boring diameter
- $D_s$: Diameter of the shield
- $D_t$: Outside diameter of the segmental lining
- $F$: Thrust force (installed)
- $F_m$: Thrust force (main system, installed)
- $F_a$: Thrust force (auxiliary system, installed)
- $F_g$: Gripper force (installed)
- $L$: Length of the shield
- $\rho_g$: Gripper pressure
- $r$: Cutter head rotational speed
- $T$: Torque (installed)
- $\Delta D_s$: Overcut (in diameter)
- $\Delta D_t$: Annular gap (in diameter)

### Notes

- $^a$: Thrust force at cutter head
- $^b$: Breakout torque
- $^c$: Maximum possible thrust force
- $^d$: Length of the TBM
- $^e$: Front part of the shield
- $^f$: Middle part of the shield
- $^g$: Tail shield
- $^h$: After modification of the TBM
- $^i$: By installing removable auxiliary hydraulic jacks
- $^j$: With closed telescopic shield
- $^k$: Front shield
- $^l$: Telescopic shield
- $^m$: Rear shield

### pg

- $p_1$: 2.8 MPa
- $p_2$: 3.3 MPa
- $p_3$: 6.0 MPa
PART IV – THE EFFECT OF CONSOLIDATION ON TBM SHIELD LOADING IN WATER-BEARING SQUEEZING GROUND

Jamming or overstressing of the shield due to ground pressure are potential problems for TBM tunnelling in squeezing ground. The risk of shield jamming depends essentially on the deformation rate of the ground in the vicinity of the working face. The time-dependency of the ground response to the excavation is associated with its rheological properties as well as with the transient consolidation process that takes place around the opening in the case of a low-permeability saturated ground. Part IV focuses on the second mechanism and investigates the interaction between the advancing shield, tunnel lining and consolidating ground by means of transient numerical analyses. For a given set of geotechnical conditions and a given TBM configuration, the load exerted by the ground upon the shield during TBM operation decreases with increasing gross advance rate. During a long break in operations, the ground pressure may increase significantly, thereby necessitating a higher thrust force to overcome shield skin friction and restart the TBM. It is interesting to note that a high advance rate reduces the risk of shield jamming not only during TBM advance, but is also favourable with respect to any subsequent long standstills.
1 Introduction

Although cases of rapidly converging ground are known in the literature (cf. Part I), squeezing ground usually exhibits a markedly time-dependent behaviour. Depending on its characteristics, the ground may respond to the excavation with some delay and may continue to deform over a period of days, weeks or even months (Barla, 2001; Kovári and Staus, 1996). The delay in the ground response is favourable for a TBM because a TBM can accommodate only relatively small convergences in the machine area and the back-up area without running into problems. However, such favourable time effects can be considered in the planning phase only if there is sufficient advance knowledge of how the ground will behave over time. Failing this, it would be unwise to rely upon the assumption that deformations will develop only slowly and far behind the machine.

As a consequence of the time-dependency of ground behaviour, the overall advance rate not only represents the main outcome of the complex interaction between the ground, the TBM and the tunnel support, but also exercises a decisive influence over this interaction (cf. Part I). More specifically, as emphasized repeatedly in the literature, a rapid excavation rate (involving high net advance rates and short standstills) reduces the risk of the shield or back-up jamming (e.g., Herrenknecht and Rehm, 2007; Kovári, 1986a, 1986b; Lombardi, 1981; McCusker, 1996; Robbins, 1982). The frequency and duration of standstills can be reduced through appropriate operational measures and construction site organisation. For example, necessary logistical precautions should be taken in order to allow operations within critical zones to proceed as continuously as possible. However, in spite of every effort, it is not always possible to avoid long interruptions (Gehring, 1996; Lombardi, 1981). Sudden changes in ground conditions, technical problems (e.g., electric power stoppages, mechanical breakdowns of the TBM with consequent repair work, problems in the back-up system), holiday periods, strikes and, of course, a TBM jamming during regular operation can cause unpredictable stoppages. Major maintenance operations are also an important factor as they may lead to an unfavourably long standstill. At the same time, however, they are important for reducing the risk of mechanical breakdown. The conflict in priorities here can be resolved by carrying out any lengthy maintenance operations before entering a critical zone, provided of course that the location of the critical zone is known and its length is sufficiently short that it can be crossed practically non-stop. In this respect, the timely identification of critical zones by means of reliable advance ground probing is very important (Anagnostou et al., 2010b; Peila and Pelizza, 2009).

The time-dependency of ground behaviour is due to the creep and consolidation processes taking place around the tunnel (Anagnostou and Kovári, 2005). In the vicinity of the tunnel face, these processes develop simultaneously with the spatial stress redistribution caused by the face advance. Creep is associated with the rheological behaviour of the ground, becoming evident particularly when the ground is highly stressed. It is therefore very important where squeezing conditions prevail. Part IV will focus on the consolidation-induced time-dependency of ground response. This mechanism comes into play when tunnelling through water-bearing ground. From tunnelling experience, it is well-known that pore water under high pressure encourages the development of squeezing (Kovári and Staus, 1996). Consolidation represents a source of time-dependency in the case of a low-permeability ground. It is associated with the transient seepage flow process that is triggered by the tunnel excavation. Understanding the role of consolidation is important for deep
alpine tunnels (Vogelhuber, 2007) as well as current or planned subsea tunnel projects crossing weak rocks, such as the Lake Mead Intake No 3 Tunnel in the USA (Anagnostou et al., 2010b) or the planned Gibraltar Strait Tunnel between Spain and Morocco (Pliego, 2005).

The present part of the report investigates the interaction between shield and ground by means of hydraulic-mechanical coupled numerical analyses which account for the highly complex transient process of consolidation around the advancing tunnel heading. The first results of this research have been presented by Ramoni and Anagnostou (2007b, 2007c), while a comprehensive review of the literature on analytical methods used for TBM tunnelling in squeezing ground can be found in Part II, which investigated the interplay between TBM, ground and tunnel support under the simplifying assumption of time-independent ground behaviour.

Part IV starts with a qualitative discussion of the mechanisms underlying the ground response to tunnelling operations and continues with a quantification of the identified effects. Section 2 sketches out the most important interrelations among the operational parameters (advance rate, standstill duration) and the deformations associated with the development and subsequent dissipation of excess pore pressures around the tunnel. Sections 3 and 4 present the modelling assumptions and outline the numerical solution method, respectively. Section 5 deals with conditions during continuous excavation and analyzes the effect of the advance rate on shield loading, while Section 6 investigates ground pressure development during a standstill.

2 The consolidation mechanism

Squeezing is associated with overstressing and plastic yielding of the ground. Squeezing ground generally experiences an increase in volume (plastic dilatancy). If the ground is saturated, its water content also increases during squeezing. This occurs more or less rapidly depending on the permeability of the ground. In a low-permeability ground, the water content remains constant in the short term. Since the pore water hinders dilatancy, negative excess pore pressures are generated by to the excavation work. As these are higher in the vicinity of the tunnel than further away, a transient seepage flow process starts to develop towards the tunnel. The negative excess pore pressures dissipate over time, thus changing the effective stresses and leading to additional, time-dependent deformations (Anagnostou and Kovári, 2005). When a shield or a lining hinders ground deformations, the load acting on it will increase over time.

With respect to the transient process, two important states can be distinguished: the state immediately after excavation (i.e., at time $t = 0^+$) and the long-term state ($t = \infty$). The first state is characterized by the condition of constant water content (so-called "undrained conditions"). The second state is governed by the steady state pore pressure distribution (so-called "drained conditions"). As in other geotechnical problems involving "unloading" of the ground (e.g., deep excavations), the short-term behaviour is more favourable than the long-term behaviour. In fact, according to theoretical and experimental investigations (Anagnostou, 2007c; Vogelhuber, 2007), the negative excess pore pressures developing under undrained conditions strengthen the ground temporarily, as they increase the effective stress and thus the resistance to shearing. This so-called "dilatancy hardening" is temporary because the excess pore pressures dissipate with time.
The time-dependent development of ground deformations is governed by the ratio of advance rate $v$ to ground permeability $k$ (Anagnostou, 2007a). If this ratio is high (as in the case of rapid excavation through a low-permeability ground), undrained conditions, which are more favourable, will prevail in the machine area. On the other hand, if the excavation proceeds slowly or the ground permeability is high (low $v/k$-ratio), unfavourable drained conditions will set in almost immediately after excavation. The advance rate $v$ means the gross advance rate resulting from the boring process and including regular short standstills for the installation of the tunnel support or for the execution of inspections and minor maintenance work. The effects of advance rate and permeability will be investigated quantitatively in Section 5.

Major maintenance or repair work (planned or not) or other problems may cause longer standstills, which cannot be classified among regular TBM operations and have to be investigated separately. The ground behaviour during such a standstill is governed by the mechanisms described above. If drained conditions have not already been reached during the preceding regular excavation (i.e., if the ratio $v/k$ was high enough), consolidation will continue during the standstill period until the steady state pore pressure distribution is reached. Again, due to the change in the effective stresses, the ground will deform and the ground pressure will increase over time. For a given ground permeability, the higher the advance rate during the preceding excavation, the more the conditions prevailing at the beginning of the standstill will deviate from the drained conditions and, consequently, the more time must elapse before the steady state is reached. Rapid excavation is therefore also advantageous with respect to subsequent standstills. The conditions prevailing during standstills will be investigated quantitatively in Section 6.

The risk of the TBM jamming depends on the ratio $v/k$ as this governs the intensity of the deformations in the machine area. In general, the less permeable the ground, the more rapid the excavation and the shorter the standstills, the closer conditions will be to a favourable undrained state. The range of feasible advance rates is relatively narrow (i.e., $v = 30$ m/d – in difficult ground conditions $v = 5–10$ m/d), but the ground permeability $k$ may vary over several magnitudes, thus playing a more important role with respect to the risk of the TBM jamming. As reliable estimations are particularly difficult for heterogeneous ground, permeability introduces a prediction uncertainty, which has to be borne in mind in the design phase (Anagnostou and Kovári, 2005).

Similar considerations also apply to the creep-induced time-dependency of the ground response. As shown by Sterpi and Gioda (2007), a high advance rate is favourable as it leads to lower deformations in the machine area. In the borderline case of a very high advance rate, only small, elastic deformations develop in the vicinity of the tunnel face. However, as was the case with permeability, a reliable estimation of the ground creep parameters before construction may be very difficult to achieve.

3 Computational model

The interaction between the advancing TBM and the consolidating ground will be investigated on the basis of an axially symmetric model (Figure 1). The underlying simplifying assumptions are the same as in the Parts II and III of the present report: deep, cylindrical tunnel; hydrostatic and uni-
form initial stress field; homogeneous and isotropic ground; constant overcut around the circumference of the shield; negligible TBM weight; and uniform tunnel support (and uniform annulus grouting along the tunnel periphery in the case of a segmental lining). Again, the mechanical behaviour of the ground is taken to be linearly elastic and perfectly plastic with the Mohr-Coulomb yield criterion and a non-associated flow rule. As stated in the Introduction, creep has been disregarded.

The mechanical boundary conditions at the far field boundary and at the tunnel face can be seen from Figure 1. The tunnel face is considered as being unsupported (the effect of a face support is discussed briefly in Section 6). The tunnel wall is modelled by the mixed non-uniform boundary condition proposed in Part II:

$$p(y) = \begin{cases} 0 & \text{if } 0 \leq y \leq L \text{ and } u(y) - u(0) \leq \Delta R \\ K_s (u(y) - u(0) - \Delta R) & \text{if } 0 \leq y \leq L \text{ and } u(y) - u(0) > \Delta R \\ K_l (u(y) - u(L)) & \text{if } y > L \end{cases}$$ (1)

where $p$ is the ground pressure developing upon the shield or the lining, while $u$ denotes the radial displacement of the ground at the excavation boundary, $K_s$ and $K_l$ the stffnesses of the shield and of the lining, respectively, $L$ the shield length and $\Delta R$ the difference between shield and boring radius. This condition assumes that the backfilling of the segmental lining takes place immediately behind the shield according to Figure 2a (the effect of a delayed backfilling will be discussed briefly in Section 6). As illustrated in Part II, Equation 1 takes due account of the different installation points (at $y = 0$ and $y = L$, respectively, cf. Figure 1), stiffnesses ($K_s$ and $K_l$, respectively) and radial gap sizes ($\Delta R$ and 0, respectively) of the shield and of the segmental lining, thus allowing a more
The effect of consolidation on TBM shield loading in water-bearing squeezing ground – 147/212

realistic simulation than the model by Ramoni and Anagnostou (2007b, 2007c), which considered the simplified model of an infinitely long, practically rigid shield having a constant radial gap size $\Delta R$. (The same assumption was made by Sterpi and Gioda (2007) in their analysis of the effect of creep.)

More information on the computational model as well as a detailed discussion of the major assumptions can be found in the Part II and III, respectively. The rest of this section will deal only with assumptions which specifically involve a water-bearing ground.

Pore pressures were taken into account by modelling the ground as a saturated porous medium according to the principle of effective stresses. Seepage flow was modelled by Darcy's law. Cavitation effects have been neglected. Incompressible ground constituents have been assumed (a reasonable simplification for weak rocks). In order to preserve the condition of axial symmetry, the contribution of the geodetic height to the hydraulic head was disregarded, i.e., the hydraulic head $h$ was taken to be equal to $p_w/\gamma_w$, where $p_w$ and $\gamma_w$ denote the pore pressure and the unit weight of the water, respectively. The error introduced by this simplification is discussed later in this section.

Pore pressure at the excavation boundary (tunnel wall and face) can be considered as atmospheric. As a consequence of this boundary condition, a flow towards the ground through the excavation boundary would occur if the pore pressure within the ground were negative (suction). This means that the ground would be watered from the tunnel and presupposes the existence of free water along the tunnel boundary. As this assumption is in most cases unrealistic (Anagnostou, 1995), a mixed hydraulic boundary condition was adopted in order to avoid ground watering via the tunnel:

$$q_n = \begin{cases} -\frac{k}{\gamma_w} \frac{\partial p_w}{\partial n} & \text{if } \frac{\partial p_w}{\partial n} \leq 0 \\ 0 & \text{if } \frac{\partial p_w}{\partial n} > 0 \end{cases},$$

where $k$ is the ground permeability, $n$ denotes the boundary surface normal (outwards positive) and $q_n$ is the boundary flux. This condition is equivalent to a common Dirichlet condition, if positive pore pressures prevail within the ground, and to a Neumann condition (no flow) in the case of suction.

Figure 2. (a) Single shielded TBM with annulus grouting immediately behind the shield via the shield tail; (b) single shielded TBM in rock with delayed backfilling of the segmental lining.
At the far-field boundary, the hydraulic head is fixed to $h_0^*$ (Figure 1), where $h_0^*$ takes into account the initial hydraulic head $h_0$, the finite size of the computational domain and the deviation of the

Figure 3. Distribution of the hydraulic head $h$ (a) and of the pore pressure $p_w$ (b) above, alongside and below a circular tunnel (radius $R = 5 \text{ m}$) located 100 m below a constant remaining water table.
simplified axially symmetric head field from the actual one. The boundary value $h_0^*$ was estimated by means of preliminary two-dimensional seepage flow analyses.

Figure 3a is based upon such an analysis and shows the distribution of the hydraulic head $h$ along three lines for the example of a circular tunnel (radius $R = 5$ m), which is located 100 m below the water table, under the assumption that, due to a sufficiently high groundwater recharge rate, the water table remains constant in spite of the drainage action of the tunnel. Figure 3a makes clear the asymmetry of the hydraulic head distribution. At a given distance (e.g., at $20R = 100$ m) from the centre of the tunnel, the hydraulic head is higher above the tunnel ($h = 100$ m, see point A in Figure 3a) than lateral to ($h = 78$ m, point B) or below the tunnel ($h = 70$ m, point C). It should be noted that the asymmetry of the actual pore pressure field is much bigger than the asymmetry of the hydraulic head field: Figure 3b shows the distribution of the pore pressure head $p_w/\gamma_w$ along the three lines mentioned above and illustrates how big the asymmetry becomes far away from the tunnel. The deviation is, however, relatively small in the vicinity of the tunnel, where the biggest deformations occur and the hydraulic-mechanical coupling is important.

A reasonable value for the boundary head $h_0^*$ of the simplified axially symmetric model can be obtained by considering the middle distribution (point B along the horizontal axis in Figure 3a) and taking into account the outer radius $R_m$ of the computational domain. Figure 4 is based upon the results of a comprehensive parametric study and shows the normalized hydraulic head $h_0^*/h_0$ to be prescribed at the outer boundary of the axially symmetric computational model as a function of the normalized initial hydraulic head $h_0/R$ for different sizes $R_m/R$ of the computational domain. Due to the finite size $R_m$ of the computational domain, the head at its outer boundary is lower than the initial hydraulic head as the latter prevails theoretically at an infinite distance lateral to and below the tunnel (cf. Figure 3).
4 Numerical solution method

The analysis of an advancing tunnel heading in water-bearing ground, where excess pore pressure dissipation takes place simultaneously with the spatial stress redistribution around the working face, is a time-dependent, three-dimensional problem with a moving boundary. This problem was solved numerically by the so-called "steady state" finite element method, which was proposed by Nguyen Minh and Corbetta (1991) for solving elastoplastic and elasto-viscoplastic tunnelling problems and extended by Anagnostou (1993, 1995, 2007a, 2007b) for seepage flow or poro-elastoplastic tunnel analyses. The steady state method solves the advancing tunnel heading problem in just one computational step, thus avoiding the extremely high computational cost and the numerical accuracy and stability problems of step-by-step tunnel advance and support installation simulations (see also Part II).

The basic idea of the steady state method is to reformulate and solve the equations underlying the transient poroplastic problem in a frame of reference which is fixed to the moving face (coordinate y in Figure 1). It is interesting to note, that under steady state conditions (with respect to the moving face) the objective rate of any arbitrary field function $A^*(y^*, t)$ in the spatially fixed coordinate system (coordinate $y^*$ in Figure 1) is related to its spatial derivative in the face-fixed coordinate system (Figure 5):

$$\frac{\partial A^*}{\partial t}_{y^*} = v \frac{\partial A}{\partial y},$$

where $v$ denotes the advance rate. Equation 3 allows for the elimination of the time-coordinate from the governing equations and, therefore, the need for integration in the time-domain. It will be used later in this report for the interpretation of the numerical results.

Figure 5. Geometrical illustration of the transformation of Equation 3 (at a steady state, the curve $A^*(y^*)$ moves with the rate $v$ in the direction of tunnelling); after Anagnostou (2007a).
The role of excess pore pressure dissipation and the effect of the ratio of advance rate $v$ to ground permeability $k$ (cf. Section 2) will be analyzed by means of numerical computations concerning the hypothetical case of a 400 m deep tunnel crossing weak ground at a depth of 100 m beneath the water table. The tunnel has a diameter of 10 m and is excavated by a 10 m long single shielded TBM. The applied segmental lining is 30 cm thick and supposed to be rigid. Table 1 summarizes the material constants and the other model parameters.

### Table 1. Assumed parameter values.

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<tr>
<th>Ground</th>
<th>Traffic tunnel</th>
<th>Service tunnel</th>
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<tbody>
<tr>
<td>Young's modulus $E$ [MPa]</td>
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<td>Poisson's ratio $\nu$ [-]</td>
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<td>Unit weight of water $\gamma_w$ [kN/m$^3$]</td>
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<td>Advance rate $v$ [m/d]</td>
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<td>Reduced initial hydraulic head $h_0^*$ [m]</td>
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**Notes**

- With the exceptions of Figures 16 and 18, where $\Delta R_l = \Delta R + 7.5$ cm

### 5 Conditions during regular TBM operation

The role of excess pore pressure dissipation and the effect of the ratio of advance rate $v$ to ground permeability $k$ (cf. Section 2) will be analyzed by means of numerical computations concerning the hypothetical case of a 400 m deep tunnel crossing weak ground at a depth of 100 m beneath the water table. The tunnel has a diameter of 10 m and is excavated by a 10 m long single shielded TBM. The applied segmental lining is 30 cm thick and supposed to be rigid. Table 1 summarizes the material constants and the other model parameters.

#### 5.1 Distribution of the pore water pressures

As discussed qualitatively in Section 2, the distribution of the pore pressures is of paramount importance for the deformations of the ground and, therefore, for its interaction with the shield and for the thrust force that is required for overcoming friction. During regular TBM advance, the ground
behaviour will be undrained, drained or somewhere in-between depending on the ratio of advance rate $v$ to ground permeability $k$.

Figure 6 shows the contour lines of the pore pressure $p_w$ for different ratios $v/k$, thus illustrating the transition from undrained conditions to less favourable drained conditions occurring when the advance rate decreases (or the ground permeability increases). Under practically undrained conditions (i.e., when the $v/k$-ratio is high), negative excess pore pressures develop within an extended region around the tunnel and dissipate far behind the working face. With a decreasing advance rate (or with increasing ground permeability), the dissipation of the excess pressures takes place faster, i.e., closer to the tunnel heading. Of course, the predicted high suctions can occur only in the absence of cavitation (i.e., in extremely fine-grained, clayey ground).

5.2 Interaction between shield, ground and support

The higher the ground permeability $k$ (or the slower the TBM advance rate $v$), the quicker will be the development of the consolidation process and the larger will be the deformations in the machine area.

Figure 7a shows the radial displacement $u$ of the ground at the tunnel boundary for different ratios $v/k$ and a radial gap size $\Delta R$ of 5 cm (left side) or 10 cm (right side). The radial gap size is equal to the difference between the radius of the bored profile and the outer radius of the shield and represents the space which is available for accommodating ground deformations without development of a ground load upon the shield. The radial displacement $u$ according to Figure 7a includes the pre-deformation of the ground, i.e., the deformation $u(0)$ that occurs ahead of the face. The risk of shield jamming depends on the convergence $u - u(0)$ of the bored profile (Figure 7b). Figure 7b shows that the higher the ground permeability $k$ and the lower the advance rate $v$, the faster will the convergence develop and the nearer to the tunnel face will be the closure in the radial gap (compare, e.g., point B with point A in Figure 7b, left side). After closing the gap, the ground starts to develop a load upon the shield (Figure 7c). The ground pressure $p$ increases with the distance $y$ behind the face and drops to zero at the installation point of the lining. Assuming that the annulus grouting is carried out via the shield tail with a very fast hardening mortar (cf., e.g., Pelizza et al., 2010) and simultaneously with the shield advance, a ground pressure starts to develop upon the lining immediately after its installation at the shield tail. It should be noted that the simplified model of Ramoni and Anagnostou (2007b, 2007c) cannot reproduce the stress relief at the installation point of the lining and leads to an underestimation of the shield loading – see Part II for a detailed discussion of this point.

According to Figure 7c, the ratio of advance rate $v$ to ground permeability $k$ is decisive for the shield loading. Consider, for example, the case of a normal overcutting ($\Delta R = 5$ cm, left side). If $v/k \geq 3 \times 10^5$, the ground does not close the gap and the shield remains unloaded, while for $v/k \leq 3 \times 10^3$ a considerable load develops, which may immobilize the shield (due to skin friction) or even endanger its structural safety. It should be noted that the $v/k$-ratio has the opposite effect on the final pressure developing upon the lining far behind the face. The higher this ratio, the smaller will be the deformations developing prior to lining installation and, consequently, the higher will be
Figure 6. Contour lines of the pore pressure $p_w$ for different values of the ratio of advance rate $v$ to ground permeability $k$ (radial gap size $\Delta R = 5$ cm, uniaxial compressive strength of the ground $f_c = 3.0$ MPa; other parameters according to Table 1).
the final load (see Figure 8 for a numerical example). So, high ratios of $v/k$ are favourable with respect to the risk of shield jamming but not with respect to a possible overstressing of the lining. A larger radial gap $\Delta R$ has, as one might expect, a positive effect with respect to the shield loading.
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as it remains open for a longer interval behind the face (compare, e.g., point C with point B in Figure 7b, right and left side, respectively) and widens the subcritical range. In order that the shield remains unloaded in this example, the $v/k$-ratio should be higher than about $3 \times 10^5$ if $\Delta R = 5$ cm, while a $v/k$ of $2 \times 10^4$ would be sufficient if $\Delta R = 10$ cm. For a radial gap of $\Delta R = 15$ cm, the shield would remain unloaded even for $v/k \to 0$ (i.e., standstill or a very rapid, practically time-independent ground response). The feasibility and the reliability of such a large overboring is nevertheless questionable and has to be checked carefully for given project conditions (cf. Part I).

5.3 Calculation of thrust force

The thrust force $F_f$ needed to overcome shield skin friction can be calculated by integrating the ground pressure $p$ over the shield length $L$ (cf. Part III):

$$F_f = \mu \int_0^L p(y)dy,$$

where $\mu$ is the skin friction coefficient. Two operational stages have to be considered with respect to the thrust force requirements: (i) ongoing excavation; (ii) restart after a regular short standstill (e.g., for the installation of the tunnel support or for the execution of routine maintenance work). For stage (i) the thrust force $F_b$ needed for boring must also be taken into account (cf. Part I). Therefore, the required thrust force

$$F_f = F_f + F_b,$$

where

$$F_b = \begin{cases} F_c n_c & \text{for ongoing excavation} \\ 0 & \text{for restart after a standstill} \end{cases}$$
$F_c$ is the maximum cutter force and $n_c$ is the number of cutters. For cutters with a diameter of 17 inches $F_c$ can be taken as equal to 267 kN (Sänger, 2006), while for $n_c$ the following empirical expression applies (Vigl et al., 1999):

$$n_c = 6.7D,$$

(7)

where $D$ is the boring diameter. The two operational stages mentioned above are also different with respect to the skin friction coefficient $\mu$. During ongoing excavation, the TBM has to overcome sliding friction, while static friction has to be taken into account when restarting the TBM. Here the skin friction coefficient $\mu$ was taken to be 0.30 and 0.45 for sliding and static friction, respectively (Gehring, 1996).

### 5.4 Effects of overboring and lubrication

Figure 9 shows the required thrust force $F_r$ as a function of the $v/k$-ratio for a radial gap size of $\Delta R = 5$ cm (normal overcutting) or 10 cm (overboring). The dashed lines apply to ongoing excavation ($F_c$ was taken to 18 MN in this example), while the solid lines apply to a restart.

Assuming that the installed thrust force $F_r$ amounts to 150 MN – a high, but feasible value provided that the bearing capacity of the segmental lining is sufficient (cf. Part III) – and that the radial gap size $\Delta R$ is equal to 5 cm, the TBM would be trapped if $v/k \leq 2.5 \times 10^4$ (point A in Figure 9), i.e., if $v = 1$ m/d and $k \geq 4.6 \times 10^{-10}$ m/s or $v = 10$ m/d and $k \geq 4.6 \times 10^{-9}$ m/s (see axis at the bottom of Figure 9). Of course, the required thrust force can be reduced considerably by overboring (to less than 100 MN in this example) – this comes, however, at the cost of possible steering difficulties and reduced production rates (cf. Part I).

Besides overboring, another possible countermeasure for coping with squeezing is the lubrication of the shield extrados (e.g., by bentonite). Lubrication reduces the shield skin friction and thus the required thrust force (compare curves for $\mu = 0.15$ or 0.25 with curves for $\mu = 0.30$ or 0.45 in Figure 9). The effect of lubrication is smaller than that of overboring in this example. However, as overboring may be not sufficiently reliable, a combination of these two measures would be required in order to mitigate the risk of shield jamming.

### 5.5 Effects of permeability

Figure 9 also shows that the consolidation process is practically irrelevant if $v/k \leq 10^3$ or, taking into account an assumed maximum feasible advance rate of $v = 10$ m/d, if the permeability $k$ is higher than about $10^{-7}$–$10^{-8}$ m/s. There is no observable time-dependency in the ground response in such a case. Unfavourable steady state conditions apply continuously during excavation right from the start and may, depending on the countermeasures applied, affect the feasibility of the TBM drive. On the other hand, if $v/k \geq 10^6$ favourable undrained conditions would prevail and the required thrust force would be considerably lower. In a low-permeability ground ($k < 10^{-10}$–$10^{-11}$ m/s) the shield would remain unloaded even at moderate advance rates of $v = 1$–10 m/d.
5.6 **A counter-intuitive aspect of model behaviour**

According to Figure 9, the required thrust force \( F_r \) in the \( v/k \)-range between \( 10^2 \) and \( 10^4 \) is slightly higher than the force needed for \( v/k \to 0 \). This behaviour is counter-intuitive because one would expect conditions to become increasingly unfavourable with a decreasing advance rate.

As can be seen from the additional computational results of Figure 10, this effect is more pronounced in the case of a low strength ground – i.e., at higher levels of plastification. This has also been observed by other authors (Boldini et al., 2000; Graziani et al., 2005; Matter et al., 2007) and

![Figure 9. Required thrust force \( F_r \) during ongoing excavation (including the thrust force needed for boring \( F_b = 18 \) MN) and for restart with lubrication of the shield extrados (skin friction coefficient \( \mu = 0.15 \) or 0.25) or without lubrication of the shield extrados (skin friction coefficient \( \mu = 0.30 \) or 0.45) as a function of the ratio of advance rate \( v \) to ground permeability \( k \) (radial gap size \( \Delta R = 5 \) or 10 cm, uniaxial compressive strength of the ground \( f_c = 3.0 \) MPa; other parameters according to Table 1).](image)
is associated with the pre-deformations of the ground ahead of the face. (The bigger the pre-deformation, the more pronounced the stress relief.) The counter-intuitive behaviour occurs at \( \frac{v}{k} \)-ratios, for which the pore pressures are negative in the core ahead of the face and in the front part of the machine, but positive behind the cutter head (see, e.g., the contour lines for \( \frac{v}{k} = 2000 \) in Figure 6). The conditions along the shield are therefore close to the steady state, but the strengthening effect of the negative pore pressures ahead of the face is still there and keeps the pre-deformations small. (A similar effect might also occur in the case of creep, when the core is still behaving elastically while the rest of the tunnel is already experiencing plastic deformations.)

Another possible explanation could be the observation made by Anagnostou (2009b) that intermediate transient states may exist being more unfavourable than the steady state. Anagnostou (2009b) traced this effect back to the fact of consolidation processes occurring faster in the vicinity of the tunnel than in the far-field. Therefore, approaching the steady state, hydraulic gradients higher than at the steady state may occur. As can be seen from Figure 11, a similar effect does not occur in the example investigated in this report. Figure 11a shows the distribution of the pore pressure \( p_w \) in the radial direction \( x \), while Figure 11b depicts the pore pressure gradient \( \frac{dp_w}{dx} \) at the tunnel boundary (at a distance of \( y = L/2 = 5 \) m from the tunnel face) for different ratios \( v/k \). As can be seen from Figure 11b, hydraulic gradients higher than at the steady state do not occur in this example. However, this topic has not been investigated exhaustively. Further research work is needed in the future. For example, the effect of the dimensions of the computational domain on the distribution of the hydraulic gradients should be analyzed in more detail.
5.7 Extrusion rate of the core

"Extrusion of the core" means the axial displacement of the working face towards the tunnel. During the boring process, some of the TBM penetration is used up simply to compensate the extrusion, i.e., to excavate the ground that squeezed into the opening from the face. If the extrusion rate of the core $e$ is very high, the penetration effort will be entirely used up in excavating the axially deforming ground at the tunnel face, i.e., the cutter head will penetrate and rotate without moving forward (cf. Part I).

In general, the advance rate $v$ actually achieved is the advance rate that would occur in the absence of relevant face deformations minus the extrusion rate of the core $e$:

$$v = \max(0, \alpha Pr - e),$$

Figure 11. Distribution of the pore pressure $p_w$ in the radial direction $x$ (a) and pore pressure gradient $dp_w/dx$ at the tunnel boundary (b) as a function of the ratio of advance rate $v$ to ground permeability $k$ (distance from the tunnel face $y = 5$ m, radial gap size $\Delta R = 5$ cm, uniaxial compressive strength of the ground $f_c = 1.5$ MPa; other parameters according to Table 1).
where $\alpha$ is the utilisation degree of the TBM (i.e., the fraction of the total time used for boring), $P$ is the penetration rate (i.e., the TBM advance relative to the ground at the working face per revolution of the cutter head) and $r$ the cutter head rotational speed.

The extrusion rate of the core $e$ can be computed from the longitudinal gradient of the axial displacements $u_e$ by applying the transformation rule according to Equation 3:

$$ e = \frac{\partial u_e}{\partial t} = v \frac{\partial u_e}{\partial y} . $$ (9)

Figure 12 shows the extrusion rate $e$ as a function of the ground permeability $k$ for the numerical example considered throughout Part IV. The extrusion rate is small relative to commonly achievable advance rates. Since this numerical example concerns rather adverse conditions (in terms of ground strength and depth of cover), the results support the hypothesis that the excavation speed is normally high enough to avoid problems with deformations of the working face (Barla, 2001; Gehring, 1996; Hoek, 2001).

### 5.8 Role of tunnel diameter

An issue sometimes arising in the design phase concerns the option of constructing a smaller diameter tunnel (which may be part of the final structure – e.g., the service tunnel of a twin railway tunnel) for exploration or ground improvement in advance of the main, larger diameter tunnel. The underlying idea is that the potential geomechanical problems are less serious in the case of a smaller cross-section. It is a fact that there are many statical problems where the size of the opening represents a relevant parameter (e.g., the height and the width of the tunnel face may be decisive for its stability or the stability of the tunnel crown may depend on the round length).
Within the context of the present report, a question arises as to whether a smaller diameter tunnel offers advantages with respect to the risk of shield jamming. This question cannot be answered on the basis of qualitative considerations because some of the differences between small and large diameter tunnels are in favour of small cross-sections and others in favour of large diameter TBMs. More specifically, a smaller diameter TBM will be able, under normal operational conditions, to proceed faster (but the limited space available may present greater difficulties in the case of adverse conditions). Furthermore, a given amount of overboring $\Delta R$ (say 5–10 cm) will be more effective in reducing the ground pressure in the case of a small diameter machine (because the load reduction is governed by the ratio of radial gap size $\Delta R$ to tunnel radius $R$). On the other hand, as can be seen from the TBM technical data collected in Part III, smaller diameter TBMs are relatively longer (the ratio of shield length $L$ to tunnel radius $R$ is higher than for traffic tunnel TBMs — typical values are $L/R = 5.0$ and 2.0, respectively) and have a lower installed thrust force (reported values — at the upper limit of the proven range — are $F_i = 50$ and 150 MN, respectively). This results in a slightly lower ratio of installed thrust force $F_i$ to shield mantle surface $2\pi RL$ (i.e., the surface exposed to the ground pressure) for the smaller diameter TBMs than for the larger ones ($F_i/2\pi RL = 0.40$ and 0.48 MPa, respectively).

Figure 13 compares a "traffic tunnel" TBM (assumed to advance at a rate of $v = 10$ m/d, solid curves) with a faster advancing "service tunnel" TBM ($v = 25$ m/d, dashed curves). The diagram shows the utilization degree of the thrust force as a function of the ground permeability $k$. The utili-
zation degree of the thrust force is defined as the ratio of the thrust force $F_r$, required in order to overcome the frictional resistance of the ground, to the installed thrust force $F_i$. An utilization degree of more than 100% means that the machine will be jammed. A low utilization degree indicates that the machine has a large reserve against jamming. The diagram deals with the thrust force requirements for TBM restart, i.e., for overcoming static friction (and, according to Equations 5 and 6, $F_r = F_i$). It contains three curves for each TBM in order to show the effect of the overcut (radial gap size $\Delta R = 5$ or 10 cm) and the ground quality (uniaxial compressive strength $f_c = 1.5$ or 3.0 MPa).

In a low strength and high permeability ground ($f_c = 1.5$ MPa, $k > 10^{-8}$ m/s) the thrust force requirements would be critical for both machines in the case of a normal overcut ($\Delta R = 5$ cm). For the reasons mentioned above, the benefit of increasing the overcut (from $\Delta R = 5$ to 10 cm in this example) is greater in the case of the smaller diameter tunnel. Furthermore, it is interesting that in the case of a better quality ground ($f_c = 3.0$ instead of 1.5 MPa) the situation would improve significantly for the smaller diameter TBM but not for the large TBM. The results of Figure 13 indicate that a smaller diameter TBM would cope slightly better with squeezing than the larger diameter machine under the conditions of this specific example.

6 Conditions during standstills

6.1 Dissipation of the excess pore water pressures

The conditions during a standstill will be discussed by means of numerical results relating to the example introduced in Section 5. The pore pressure distribution at the beginning of a standstill depends on the ratio of the advance rate $v$ of the preceding excavation to the ground permeability $k$. As can be seen from Figure 6, the higher this ratio, the more the pore pressure field prevailing at the start of the standstill will deviate from the steady state. During the standstill, excess pressure dissipation and consolidation of the ground will continue until the steady state is reached.

The transition from favourable undrained conditions to unfavourable drained conditions can be observed in Figure 14, which shows the contour lines of the pore pressure $p_w$ at different times $t$ (since the start of the standstill) assuming that the ground has a permeability of $k = 10^{-9}$ m/s and that the preceding excavation proceeded at a rate of $v = 10$ m/d. In this example, the steady state is reached after about 10 days.

6.2 Longitudinal arching in the shield area

The dissipation of excess pore pressures leads to a time-dependent increase in ground deformations and, as the shield and lining allow only a limited amount of deformation, an increase in the ground pressure as well.
Figure 14. Contour lines of the pore pressure $p_w$ at different times $t$ during standstill (permeability of the ground $k = 10^{-9}$ m/s, advance rate of the preceding excavation $v = 10$ m/d, radial gap size $\Delta R = 5$ cm, uniaxial compressive strength of the ground $f_c = 3.0$ MPa; other parameters according to Table 1).
Figure 15. Numerical results for a radial gap size of $\Delta R = 5$ or 10 cm (left and right side, respectively): (a) convergence $u - u(0)_{t=0}$ of the bored profile as a function of the standstill time $t$; (b) ground pressure $p$ acting upon the shield and the lining as a function of the standstill time $t$; ground permeability $k = 10^{-9}$ m/s, advance rate of the preceding excavation $v = 10$ m/d, uniaxial compressive strength of the ground $f_c = 3.0$ MPa; other parameters according to Table 1.

Figure 15a shows the increase of the convergence $u - u(0)_{t=0}$ over the time $t$ (that has elapsed since the start of the standstill) for two values of radial gap size ($\Delta R = 5$ or 10 cm, on the left and right side, respectively), where $u$ denotes the total radial displacement of the ground at the excavation boundary, while $u(0)_{t=0}$ is the pre-deformation of the ground (i.e., the deformation which occurred ahead of the face during excavation). Figure 15b shows the time-development and spatial distribution of the ground pressure $p$ acting upon the shield and lining. As in Section 5, the simplifying assumption is made that the backfilling of the segmental lining is carried out immediately after
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installation of the segments (Figure 2a) and this is why a load starts to develop upon the lining immediately behind the shield. The effect of a delayed annulus backfilling (Figure 2b) will be discussed in the next section.

In the case of normal overcut ($\Delta R = 5\, \text{cm}$), the ground already closes the gap around the rear part of the shield during TBM advance and, consequently, a load $p$ already acts upon the shield at the start of the standstill (see curve for $t = 0$, left side of Figure 15b). During the standstill, the load act-

Figure 16. Numerical results for the borderline case of a segmental lining without backfilling for a radial annulus of $\Delta R_l = 12.5$ or $17.5\, \text{cm}$ (left and right side, respectively): (a) convergence $u - u(0)_{t=0}$ of the bored profile as a function of the standstill time $t$; (b) ground pressure $p$ acting upon the shield and the lining as a function of the standstill time $t$; ground permeability $k = 10^{-9}\, \text{m/s}$, advance rate of the preceding excavation $v = 10\, \text{m/d}$, radial gap size $\Delta R = 5$ or $10\, \text{cm}$, uniaxial compressive strength of the ground $f_c = 3.0\, \text{MPa}$; other parameters according to Table 1.
ing upon the shield and the lining increases and reaches its final value after about 10 days (when the pore pressure distribution reaches the steady state, cf. Figure 14).

The convergence and pressure distribution are completely different in the case of the bigger overcut (right side of Figure 15). The 10 cm wide radial gap under consideration remains open during TBM advance. During standstill, the ground takes about 9 days to establish contact with the shield (and to start developing a load upon it) and another 10 days to reach the steady state. (The longer duration of the consolidation process is due to the lower system stiffness associated with the bigger overcut.) During the first phase of the standstill, where the ground remains unsupported over the shield area, a considerable load develops upon the front part of the lining. It is also interesting that the ground first makes contact with the shield in its middle part. The front and the rear parts of the shield remain unloaded due to the support action of the lining and the core, respectively. The observed pressure distribution illustrates a pronounced load transfer via arching in longitudinal direction between the lining and the ground ahead of the face. This effect becomes less pronounced with time due to the plastic yielding of the ground surrounding the middle part of the shield.

On the one hand, the arching effect is favourable with respect to shield loading and to the thrust force that is required in order to overcome the frictional resistance of the ground during TBM restart. On the other hand, arching increases ground pressure behind the shield and may endanger the structural safety of the lining. The practical conclusion is that a reinforcement of the tunnel support close to the shield may be necessary in the case of longer standstills and, as mentioned above, of complete backfilling of the segmental lining immediately behind the shield.

The arching effect discussed above depends on the resistance offered by the lining behind the shield and the stiffness of the core ahead of the shield. The influence of these two factors will be discussed in Sections 6.3 and 6.4, respectively.

### 6.3 Effect of backfilling of the segments close to the shield

A significant load transfer in the longitudinal direction (which is favourable for the shield, but unfavourable for the lining) is possible only if the segments are "perfectly" backfilled directly behind the shield, i.e., if annulus grouting is carried out simultaneously with TBM advance via the shield tail (Figure 2a). This is normally the case with closed shields, where the counter-pressure offered by the supporting medium prevents the flow of grout around the shield towards the cutter head. In rock TBMs, however, where pea gravel is used for backfilling, it is not possible to achieve a contact between ground and segmental lining right from the start and, as sketched in Figure 2b, a certain "span" behind the shield remains unsupported (Lavdas, 2010). Shield load reduction via longitudinal arching is practically impossible in this case.

For the sake of comparison, consider the borderline case of a segmental lining without backfilling. In this case, the ground starts to develop a pressure \( p \) upon the lining only after experiencing sufficient deformation and closing the lining annulus gap \( \Delta R_l \). The latter denotes the difference between the boring radius and the outer radius of the segmental lining and is taken to be \( \Delta R + 7.5 \) cm, where \( \Delta R \) is the radial gap size of the shield.
The lining annulus gap $\Delta R_l$ is taken into account by prescribing the following boundary condition instead of Equation 1:

$$p(y) = \begin{cases} 
0 & \text{if } 0 \leq y \leq L \text{ and } u(y) - u(0) \leq \Delta R_l \\
K_l(u(y) - u(0) - \Delta R_l) & \text{if } 0 \leq y \leq L \text{ and } u(y) - u(0) > \Delta R_l \\
0 & \text{if } y > L \text{ and } u(y) - u(0) \leq \Delta R_l \\
K_l(u(y) - u(0) - \Delta R_l) & \text{if } y > L \text{ and } u(y) - u(0) > \Delta R_l 
\end{cases} \quad (10)$$

Figure 16 presents the numerical results in the same way as Figure 15 did in the last section, i.e., it shows the spatial distribution and time-development of ground convergence and pressure for two values of the radial gap size $\Delta R$ of 5 or 10 cm (on the left and right side, respectively). As expected, the ground starts to develop a pressure $p$ upon the lining later (after closing the lining annulus gap) and the final load is significantly lower than for the case with perfect backfilling of the segments – this, however, at the cost of a much higher shield load. (In this computational example, the load developing on the rear part of the shield would endanger its structural safety.)

The comparison of Figures 15 and 16 shows that conflicting requirements may have to be met. On the one hand, backfilling of the segments immediately behind the shield facilitates longitudinal arching and reduces shield load. On the other hand, ground deformation should be allowed in order to reduce lining load to a manageable level.

For fast excavation through a low-permeability ground (i.e., for a high $v/k$-ratio), where undrained conditions prevail in the shield area and the ground pressure $p$ acting upon the shield at the beginning of the standstill is very low (see curves for $t = 0$ in Figure 16b), it seems advantageous to delay the backfilling of the segments. It should be noted, however, that improper backfilling of the segments may reduce the bearing capacity of the lining and may therefore limit the effectively available thrust force (it may be impossible to utilize the installed thrust force).

In the other borderline case of a slow excavation or a high permeability ground (i.e., low $v/k$-ratio), where almost drained conditions prevail in the shield area and the shield loading is significant even during continuous excavation, a careful backfilling of the segments close to the shield may help avoid TBM jamming. Nevertheless, it should also be noted that the backfilling work may slow down the TBM advance rate, thus leading, as a rule, to a less favourable situation.

6.4 Load transfer to the core ahead of the face

Longitudinal arching presupposes, in addition to the development of lining resistance close to the shield tail, a sufficiently stiff core ahead of the face. The radial resistance of the core depends in general on the face support pressure (as mentioned in Section 3, the computational model applied in this report regards the tunnel face as being unsupported). The lower the face support pressure, the more the core yields and extrudes in the axial direction, the more the radial stress ahead of the face decreases and the more load is transferred to the shield. This is why the ground pressure in the front part of the shield increases with time during the standstill (see pressure peak at $y = 1.5$ m behind the face in Figure 15b, left side).
Figure 17. (a) Ground pressure $p$ developing at $y = 1.5$ m during a standstill as a function of time $t$ (unsupported or fixed face); (b) core extrusion $u_e$ developing during a standstill as a function of time $t$ (unsupported face); (c) axial loading $p$ developing upon the cutter head as a function of time $t$ (fixed face); ground permeability $k = 10^{-9}$ m/s, advance rate of the preceding excavation $v = 10$ m/d, radial gap size $\Delta R = 5$ cm, uniaxial compressive strength of the ground $f_c = 3.0$ MPa; other parameters according to Table 1.
This effect becomes clearer from Figure 17a, which shows the ground pressure $p$ at $y = 1.5$ m behind the face as a function of standstill time $t$ (the inset in Figure 17a shows the final pressure distribution along the shield). The solid line applies to an unsupported face, while the marked line applies to the case of zero core extrusion (where axial displacement is constrained by the cutter head). In the first case the face experiences an additional axial displacement $u_a(t)$ during the standstill (Figure 17b), while in the second case an axial pressure develops upon the cutter head (Figure 17c).

The ground pressure developing upon the shield is lower when the core cannot extrude. Face support therefore has a positive effect on the shield load during standstill. The cutter head provides a certain degree of face support – this however at the cost of higher thrust force and torque requirements (cf. Part I). Of course, the extrusion of the core $u_a$ also plays an important role with respect to face stability, as excessive deformations may lead to collapse. The linear increase in axial displacement over time (Figure 17b) indicates such a collapse scenario.

6.5 Thrust force needed for restarting TBM operation

Due to the increasing shield load (Figures 15, 16 and 17), the thrust force $F_r$, which is required in order to overcome skin friction during a TBM restart, increases as well. Figure 18 shows the time-development of the required thrust force $F_r$ (as mentioned in Section 5.3, the thrust force calculation takes into account static friction, but not the boring thrust force).

The solid lines presuppose a complete backfilling of the segments immediately behind the shield and an unsupported face. In the case of an installed thrust force of $F_i = 150$ MN and a normal overcutting of $\Delta R = 5$ cm, the TBM can be restarted if the standstill is shorter than about 2.5 days (see point A in Figure 18). In the case of a larger radial gap size ($\Delta R = 10$ cm), it would always be possible to restart the TBM in this example. This could be achieved also by lubricating the shield extrados (i.e., reducing the skin friction coefficient $\mu$ from 0.45 to 0.25).

The dashed lines in Figure 18 concern the borderline case without backfilling of the segments (Figure 2b). The positive effect of backfilling with respect to the risk of shield jamming can be seen clearly by comparing these dashed lines with the solid lines. Without backfilling, the critical standstill duration decreases from 2.5 days to about 1.5 day in the case of a normal overcut of $\Delta R = 5$ cm (see points B and A in Figure 18). For an overboring of $\Delta R = 10$ cm, the shield becomes loaded earlier (after 5 instead of 9 days, see points D and C) and the required thrust force increases considerably over time.

The marked lines in Figure 18 apply to the case of a fixed face, thus illustrating the positive effect of a face support (cf. Section 6.4). This effect is, however, of minor relevance in the numerical example under consideration.

As shown in Figure 18, the required thrust force $F_r$ may increase to the level of the installed thrust force $F_i$ after a certain period of time $t$ (e.g., 2.5 days for $\Delta R = 5$ cm and complete backfilling of the segmental lining). Figure 19a shows the influence of ground permeability $k$ on the critical standstill duration $t_{crit}$ under the assumption that the advance rate during the preceding excavation was $v = 10$ m/d and the installed thrust force amounts to $F_i = 150$ MN. The higher the ground permeabil-
ity $k$, the faster will be the consolidation and the shorter the critical standstill duration $t_{crit}$. In this numerical example, for a normal overcutting of $\Delta R = 5$ cm and a uniaxial compressive strength of the ground of $f_c = 1.5$ MPa, regular TBM operation as well as standstills of 1–1.5 day or more (depending on the ground permeability $k$) will be possible if the ground permeability is lower than $10^{-9}$ m/s. On the contrary, for a ground permeability higher than $10^{-8}$ m/s the shield would become jammed even during regular TBM operation (cf. Figure 10). During the standstill, the ground pressure acting upon the shield may increase further, thus making the job of freeing the TBM even more difficult. For ground permeabilities of $k = 10^{-8} - 10^{-9}$ m/s the critical standstill duration $t_{crit}$ decreases down to zero. At this permeability range, the required thrust force $F_t$ was close to the installed one $F_i$ even during the previous regular TBM operation that proceeded with a rate of $v = 10$ m/d (cf. Figure 10). From the practical point of view, this means that all possible precautions should be taken in order to make TBM operation as continuous as possible, as each further unexpected standstill will significantly increase the risk of shield jamming.

Figure 18. Required thrust force $F_t$ for different model assumptions (unsupported or fixed face, with and without backfilling of the segmental lining close to the shield) as a function of the standstill time $t$ (ground permeability $k = 10^{-9}$ m/s, advance rate of the preceding excavation $v = 10$ m/d, radial gap size $\Delta R = 5$ or 10 cm, shield skin friction coefficient $\mu = 0.45$, uniaxial compressive strength of the ground $f_c = 3.0$ MPa; other parameters according to Table 1).
As shown above (Sections 2 and 5.1), the ratio of advance rate \( v \) to ground permeability \( k \) governs the distribution of the pore pressure at the beginning of the standstill. Therefore, for a given ground permeability \( k \), the advance rate \( v \) of the preceding regular TBM operation will influence the conditions prevailing at the beginning of the standstill to a considerable extent: the higher the advance rate \( v \), the lower must have been the required thrust force \( F_r \) during excavation (cf., e.g., Figure 9) and, therefore, the more time will have to elapse during a standstill before the required thrust force reaches the installed one (Figure 19b). Figure 19b also suggests that a critical advance rate exists for which the critical standstill duration \( t_{crit} \) decreases up to zero (e.g., \( v = 2.6 \) m/d for \( \Delta R = 5 \) cm and \( f_c = 1.5 \) MPa, see point A in Figure 19b). This advance rate corresponds to the borderline case where the required thrust force \( F_r \) is already equal to the installed one \( F_i \) during regular TBM operation (cf. Figure 10). This confirms the practical experience that making the TBM advance as fast as possible may help to avoid shield jamming in squeezing ground.

Figure 19. (a) Critical standstill duration \( t_{crit} \) as a function of ground permeability \( k \) (advance rate of the preceding excavation \( v = 10 \) m/d); (b) critical standstill duration \( t_{crit} \) as a function of the advance rate \( v \) (ground permeability \( k = 10^{-9} \) m/s); radial gap size \( \Delta R = 5 \) or 10 cm, shield skin friction coefficient \( \mu = 0.45 \), uniaxial compressive strength of the ground \( f_c = 1.5 \) or 3.0 MPa, installed thrust force \( F_i = 150 \) MN; other parameters according to Table 1.
7 Closing remarks

Interruptions in the TBM operation may be unfavourable in squeezing ground. Tunnelling experience (cf. Part I) as well as theoretical considerations (cf. Sections 5 and 6) suggest that maintaining a high advance rate and reducing the standstill times may have a positive effect. This is, of course, a major goal for any TBM drive. Nevertheless, a fast TBM advance should not be seen as a panacea for coping with squeezing conditions. Firstly, it may be difficult to achieve it in the case of poor quality ground. Secondly, ground deformations may develop very rapidly and very close to the tunnel face (cf. Part I). And finally, the possibility of unpredicted long standstills cannot be excluded a priori with sufficient certainty.

Theoretical considerations, together with numerical investigations, improve the understanding of the time-dependency of ground behaviour. With respect to water-bearing squeezing ground, the governing factor is the ratio of advance rate $v$ to ground permeability $k$. During regular TBM operation (excavation including short standstills), favourable undrained conditions prevail near to the working face if the ratio $v/k$ is high enough (i.e., if the gross advance rate is high or the ground permeability is low). Otherwise, consolidation takes place simultaneously with tunnel excavation and already reaches the steady state in the machine area. As the ratio $v/k$ governs the conditions prevailing at the beginning of a standstill, for a given ground permeability $k$, a high advance rate $v$ not only reduces the risk of shield jamming when the TBM is advancing regularly but also increases the critical duration of a subsequent standstill.
CONCLUSIONS AND OUTLOOK
The main goal of each TBM drive is to achieve the highest possible gross advance rate. The latter represents the main outcome of the interaction between the ground, the tunnelling equipment (TBM and back-up) and the support and, at the same time, a factor influencing this interaction. As a series of factors resulting from the three main components of the system and "external" factors (such as the organization of the construction site) affect the TBM drive simultaneously and are usually coupled with each other, planning a mechanized excavation in squeezing ground is a complex task, where conflicting requirements and several feedback effects may be present. A careful assessment of such an operation therefore requires a thorough analysis which will take into account all the specifics of the geotechnical situation, the TBM and the back-up to be used as well as the tunnel support to be installed. In this respect, a systematic approach such as the one described in Part I is indispensable.

Due to the complexity of the problem, the assessment of a TBM drive in squeezing ground cannot be based solely on plausibility considerations or on experience from projects with comparable geological conditions. Besides engineering judgement, numerical investigations are helpful for evaluating potential hazards, as they provide indications of the magnitude of the key parameters. In this respect, the computational model presented and applied in this report – particularly the mixed boundary condition introduced in Part II and the extended "steady state method" of Part IV – undoubtedly represents a powerful tool for the simulation of a TBM drive in squeezing ground. On the one hand, it provides an accurate simulation of the interface between the ground and the shield as well as the ground and any kind of tunnel support. On the other hand, it incorporates the longitudinal distribution of the ground pressure acting upon the shield and the lining and takes due account of the stress history, thus avoiding the errors introduced by plane strain analyses (Cantieni and Anagnostou, 2009a). Furthermore, the efficiency of the numerical solution method applied makes it possible to complete a comprehensive parametric study with a justifiable amount of effort.

The design nomograms, together with the extensive collection of TBM technical data presented in Part III, represent a valuable contribution to decision-making in the design process. The nomograms show in a condensed form the statical conditions that have to be fulfilled in the design of a TBM (with respect to the thrust force requirements) in order to avoid jamming of the shield when crossing squeezing ground, while the collected data reviews the state of present-day TBMs. Their application enables quantitative statements to be made concerning the feasibility of a TBM drive as well as the effectiveness of potential design or operational measures.

In the case of time-dependent ground behaviour, the thrust force required to overcome shield skin friction in given ground conditions depends, among other factors, on the gross advance rate. A reduction in the advance rate leads, as a rule, to an increase in the ground pressure and thus to further deceleration or even standstill of the TBM. As shown in Part IV, in water-bearing ground the time-dependent development of ground deformations in the machine area depends on the ratio of gross advance rate $v$ to ground permeability $k$ as this governs the distribution of pore pressures during excavation and at the beginning of a standstill. If the ratio $v/k$ is sufficiently low (i.e., if the excavation is slow or the ground permeability high), unfavourable drained conditions will prevail near to the face and there will be practically no time-dependency in the ground behaviour. Otherwise, consolidation takes place simultaneously with tunnel excavation and reaches a steady state only in the back-up area. If the excavation is stopped, the excess pore pressures continue to dissipate and the zone with relevant consolidation deformations comes progressively closer to the face with the consequence that with increasing duration of the standstill the ground exerts an increasing
pressure upon the shield. Ground permeability plays an important role with respect to the critical standstill length, of course, as a low-permeability ground needs more time to reach the steady state. As stated previously, achieving and maintaining a high production rate is the main goal of any TBM drive. When tunnelling through squeezing ground, it may also play a role in preventing the machine from becoming trapped. Nevertheless, a high advance rate cannot be seen as a panacea for coping with squeezing, as there are a number of uncertainties with respect to both the ground and the TBM.

The present PhD thesis attempts to contribute towards the understanding of the factors affecting the applicability of TBMs in squeezing ground. It represents the first time that all dimensions of the problem under consideration have been systematically summarised and structured. Furthermore, powerful design and decision aids have been developed and their applicability has already been verified with in the framework of consulting services. More specifically, the methodical approaches as well as the computational model of this research have been applied successfully in a series of real world projects involving TBM drives in squeezing ground: during construction of the Faido Section of the Gotthard Base Tunnel in Switzerland (Anagnostou and Ramoni, 2007) and the Uluabat Tunnel in Turkey (Kovári and Ramoni, 2008), in the planning phase of the Lake Mead No 3 Intake Tunnel (Anagnostou et al., 2010a, 2010b) and the Bosslertunnel in Germany (Anagnostou and Ramoni, 2009) as well as for the tender design of the El Teniente New Access Tunnels in Chile (Anagnostou et al., 2010c).

Of course, as is the case with all research work, some questions remain "open".

A series of practically relevant questions is currently under investigation within the framework of the research project "Design aids for the planning of TBM drives in squeezing ground" of the ETH Zurich. Starting from the considerations contained in Part I, Lavdas (2010) explored the applicability limits of segmental linings. With the goal of deepening the knowledge gained in assessing the TBM drives of the Faido Section of the Gotthard Base Tunnel (Switzerland), the investigations presented in Part II concerning the suitability of yielding supports installed behind a gripper TBM will be extended. A further open question concerns the evaluation of the pro and cons of the different TBM types and tunnel support designs with reference to the tunnel diameter (e.g., traffic tunnels vs. pilot tunnels). The first investigations into this topic are presented in Part IV.

A second group of "open" questions concerns the tunnel face. Firstly, the torque requirements in squeezing ground should be investigated more in detail. This issue is particularly challenging, since – as described qualitatively in Part I – several effects may affect both the friction to be overcome and the boring process. Additionally, questions arise concerning the stability of the face in weak ground. This is particularly important in water-bearing ground, where seepage forces develop towards the face (cf. Part IV). In this respect, pore pressure relief by advance drainage is a highly effective measure for reducing deformations (Anagnostou, 2009a, 2009b) and improving face stability (Anagnostou et al., 2010a, 2010b). Initial investigations have been carried out by Amberg (2009), Floria et al. (2008) and Lombardi et al. (2009). This topic is now under investigation at the ETH Zurich within the framework of a further research project with particular emphasis on the mechanized excavation of fault zones. Considering the possibility of having a superposition of squeezing and unstable ground, another interesting research project might be to extend the investigations of this PhD thesis to cover slurry or EPB shields, i.e., to analyse the effects of an active
face and tunnel boundary support on the development of ground deformations and pressure in the machine area.

Another future research topic concerns the time-dependency of ground behaviour. The present report discusses only the effect of consolidation. In a further research step, the effects of creep should be investigated in order to understand the mechanism governing the interaction between the ground, the tunnelling equipment (TBM and back-up) and the support in creeping ground. It should be noted here, that an efficient investigation of this topic will be possible only if the "steady state method" (Anagnostou, 1993, 1995, 2007a, 2007b; Nguyen Minh and Corbetta, 1991) is employed as was the case for various issues in the present report. This presupposes the implementation of this solution method in the finite element code HYDMEC of the ETH Zurich (Anagnostou, 1992) for creeping materials as well. A second step might then be to analyze the superimposed effect of creep and consolidation and to identify distinguishing features of these two mechanisms.
APPENDIX
1 Computation of the characteristic line of the yielding support YS25/C15

In order to carry out the numerical investigations of Section 6 of Part II, the characteristic line (i.e., the relationship between the ground pressure $p$ and the radial displacement $u_i$) of each considered tunnel support (cf. Table 3 and Figure 18 of Part II) has been implemented in the computational model. As an example, this Appendix will illustrate the detailed computation of the characteristic line of the tunnel support YS25/C15. This support system is composed of three basic elements (Figure A1): (1) a 25 cm thick shotcrete ring; (2) 15 cm high ductility concrete elements; (3) steel sets (TH36) at 1 m spacings having sliding connections.

For the computation of the characteristic line, it has to be considered that the elements (1) and (2) are connected in series, while the element (3) is connected in parallel with the subsystem (1-2). The elements of a serial connection have the same hoop force (due to equilibrium condition) but experience different deformations. On the other hand, elements connected in parallel experience the same deformations (compatibility condition), while the hoop force is different. Therefore, the correct computation of the characteristic line of the support system requires firstly the formulation of the relationship between hoop force $N$ and radial displacement $u_i$ for each subsystem. On account of this, this Appendix will firstly compute this relationship for the subsystems (1-2) and (3). Later, the same will be done for the entire system (1-2/3), leading thus to the characteristic line of the support system under investigation.

(1-2) – Shotcrete ring with high ductility concrete elements

Taking into account the well-known kinematic relations, the radial displacement $u_{1,2}$ of the subsystem (1-2) reads as follows:

$$u_{1,2} = \varepsilon_{1,2} R,$$

where $\varepsilon_{1,2}$ is the hoop strain and $R$ the tunnel radius. The hoop strain $\varepsilon_{1,2}$ corresponds to the ratio between the reduction of the circumference $\Delta C_{1-2}$ of the ring – due to the tangential deformation of both elements (1) and (2) – and its initial circumference $C$:

$$\varepsilon_{1,2} = \frac{\Delta C_{1-2}}{C} = \frac{\Delta C_{sc} + \Delta C_{ce}}{2R\pi}.$$

The reduction of the arc length $\Delta C_{sc}$ of the shotcrete ring is:

$$\Delta C_{sc} = \varepsilon_{sc} C_{sc} = \frac{\sigma_{f,sc}}{E_{sc}} (2R\pi - n_{ce} d_2),$$

where $E_{sc}$ is the Young’s modulus of the shotcrete, while $n_{ce}$ and $d_2$ are the number and the height of the deformable concrete elements incorporated in the shotcrete ring, respectively (Figure A1). The hoop stress $\sigma_{f,sc}$ depends on the hoop force $N_{sc}$:

$$\sigma_{f,sc} = \frac{N_{sc}}{b'd_1} \leq f_{c,sc},$$
where $b'$ is the width of the shotcrete ring between two steel sets (i.e., the steel set clear distance),

$d_1$ is the thickness of the shotcrete layer and $f_{c,sc}$ is the uniaxial compressive strength of the shotcrete.

The reduction of the arc length $\Delta C_{ce}$ due to the deformation of the high ductility concrete elements reads as follow:

$$\Delta C_{ce} = \varepsilon_{t,ce} C_{ce} = \varepsilon_{t,ce} n_{sc} d_2 .$$  \hfill (A5)

The hoop strain $\varepsilon_{t,ce}$ depends on the hoop stress $\sigma_{t,ce}$:

$$\varepsilon_{t,ce} = f(\sigma_{t,ce}) .$$  \hfill (A6)

Equation A6 is the working line of the applied deformable concrete elements and is provided by the manufacturer. Figure A2 shows graphically the relationship used in this report. Analogously to Equation A4, the hoop stress $\sigma_{t,ce}$ can be formulated as a function of the hoop force $N_{ce}$:

$$\sigma_{t,ce} = \frac{N_{ce}}{b'd_1} \leq f_{c,ce} ,$$  \hfill (A7)

where $f_{c,ce}$ is the maximum compressive stress considered for the calculations (cf. Figure A2).

Taking into account the Equations A2 to A7 as well as that the elements (1) and (2) are connected in series, i.e., that

$$N_{1-2} = N_{sc} = N_{ce} ,$$  \hfill (A8)

the relationship between hoop force $N_{1-2}$ and radial displacement $u_{l,1-2}$ for the subsystem (1-2) of Equation A1 can be now computed (Figure A3a).
(3) – Steel sets having sliding connections

Figure A3b shows the assumed relationship between hoop force \( N_3 \) and radial displacement \( u_{i,3} \) for the subsystem (3). Neglecting stability problems (buckling), the idealized behaviour up to failure of a steel set having sliding connections can be divided in four phases (Figure A3b). In phase A, the steel set deforms elastically. When the hoop force \( N_3 \) reaches the yield load \( N_y \), the sliding connections start to close (phase B) until the deformation capacity \( \Delta u_{i,3,B} \) is exhausted. In phase C, the sliding connections are closed and the steel set deforms further elastically. When the hoop force \( N_3 \) reaches the yield load of the steel set \( N_{max} \), the steel set deforms further plastically since it reached its failure strain.

Analogously to Equation A1, the radial deformation \( u_{i,3} \) of the subsystem (3) reads as follows:

\[
u_{i,3} = \varepsilon_{i,3} R . \tag{A9}\]

Therefore, the increase of the radial deformation \( \Delta u_{i,3} \) which occurs in each phase (cf. Figure A3b), is

\[
\Delta u_{i,3} = \Delta \varepsilon_{i,3} R , \tag{A10}\]

where \( i = A, B, C \) or D. Analogously to Equation A2, the increase of the hoop strain \( \Delta \varepsilon_{i,3} \) corresponds to the ratio between the reduction of the circumference \( \Delta C_{3,i} \) of the subsystem (3) – due to the tangential deformation of the steel set or of the sliding connections – and its initial circumference \( C \) before deformation:

\[
\Delta \varepsilon_{i,3,i} = \frac{\Delta C_{3,i}}{C}. \tag{A11}\]

In phase A, as mentioned above, only the steel set deforms. Therefore, \( \Delta C_{3,A} \) only depends on the increase of the hoop stress \( \Delta \sigma_{t,ss,A} \) in the steel set (occurred in phase A) and on the Young's

![Figure A2. Assumed load-deformation behaviour of the high ductility concrete elements after Solexpert (2007).](image-url)
Figure A3. Hoop force $N$ as a function of the radial displacement $u_r$: (a) shotcrete ring with high ductility concrete elements; (b) steel sets having sliding connections; (c) support system YS25/C15.

Modulus of the steel set $E_{ss}$:

$$
\Delta C_{3,A} = \Delta \epsilon_{1,ss,A} C_{ss} = \frac{\Delta \sigma_{1,ss,A}}{E_{ss}} \cdot 2R \pi ,
$$  \hspace{1cm} (A12)
where $\Delta \sigma_{s,s,A}$ is
\[
\Delta \sigma_{s,s,A} = \frac{N_f}{A_{ss}}
\]  
(A13)
and $A_{ss}$ is the cross-sectional area of the steel set.

The yield load $N_f$ depends on the number of friction loops $n_f$ of each sliding connection and on the friction loop resistance $N_f$:
\[
N_f = n_f N_f.
\]  
(A14)

In phase B, the sliding connections deform and close. The consequent reduction of the circumference $\Delta C_{3,B}$ depends on the number of slots and on the maximum slot deformation:
\[
\Delta C_{3,B} = n_{ce} \frac{d_2}{2}.
\]  
(A15)

In this case, as the steel sets are applied in combination with high ductility concrete elements, the number of slots is equal to the number of deformable concrete elements $n_{ce}$. The allowed slot deformation has to be chosen according to their deformability. As the deformable concrete elements become practically rigid at a strain of $\varepsilon_{s,ce} = 50\%$ (cf. Figure A2), the maximum slot deformation corresponds to the half of the height $d_2$ of the deformable concrete elements.

For the phase C, where only the steel set deforms (the sliding connections closed at the end of phase B), the reduction of the circumference $\Delta C_{3,C}$ reads as follows:
\[
\Delta C_{3,C} = \varepsilon_{l,ss,c} C_{ss} = \frac{\Delta \sigma_{r,ss,c}}{E_{ss}} 2R \pi,
\]  
(A16)

where
\[
\Delta \sigma_{r,ss,c} = \frac{N_{\max} - N_f}{A_{ss}}.
\]  
(A17)

The yield load of the steel set $N_{\max}$ can be calculated as the product of the yield stress $f_{y,ss}$ by the cross-sectional area $A_{ss}$ of the steel set:
\[
N_{\max} = f_{y,ss} A_{ss}.
\]  
(A18)

In phase D, the steel set deforms plastically until the failure strain $\varepsilon_{ss,max}$ is reached. Taking account of the elastic deformations which already occurred in phases A and C, and assuming that in phase B the steel set does not experience hoop strain changes, the reduction of the circumference $\Delta C_{3,D}$ is
\[
\Delta C_{3,D} = \Delta C_{ss,max} - (\Delta C_{3,A} + \Delta C_{3,C})
\]  
(A19)
where $\Delta C_{ss,max}$ is the maximum reduction of circumference, that the steel set can accommodate up to failure:
\[
\Delta C_{ss,max} = \varepsilon_{ss,max} 2R \pi.
\]  
(A20)
(1-2/3) – Support system YS25/C15

The relationship between hoop force $N$ and radial displacement $u_i$ for the entire system (1-2/3) can be now calculated taking into account the parallel connection of the subsystem (1-2) with the subsystem (3), i.e., adding their hoop forces and equating their deformation:

$$\begin{align*}
N &= N_{1,2} + N_3, \\
u_i &= u_{i,1,2} = u_{i,3}.
\end{align*} \tag{A21}$$

Equation A21 is represented graphically in Figure A3c.

Finally, the characteristic line of the support system YS25/C15 used for the investigations of Section 6 of Part II (cf. Figure 18) can be computed substituting the hoop force $N$ in Equation A21 with the ground pressure $p$, applying following expression:

$$p = \frac{N}{Rb}, \tag{A22}$$

where $b$ is the steel set spacing.

2 Notation

\begin{align*}
A & \quad \text{field function (co-ordinate system fixed to the advancing tunnel face)} \\
A^* & \quad \text{field function (co-ordinate system spatially fixed)} \\
A_{ss} & \quad \text{cross-sectional area of the steel set} \\
b' & \quad \text{steel set clear distance} \\
b & \quad \text{steel set spacing} \\
C & \quad \text{circumference} \\
c & \quad \text{cohesion of the ground} \\
C_{ce} & \quad \text{arc length of the deformable concrete elements} \\
c_g & \quad \text{compressibility of the rock or soil grains} \\
C_{sc} & \quad \text{arc length of the shotcrete ring} \\
C_{ss} & \quad \text{circumference of the steel set} \\
c_w & \quad \text{compressibility of the water} \\
D & \quad \text{boring diameter} \\
d_1 & \quad \text{thickness of the shotcrete layer} \\
d_2 & \quad \text{height of the deformable elements (yielding support)} \\
D_l & \quad \text{outside diameter of the segmental lining} \\
d_l & \quad \text{thickness of the lining} \\
D_s & \quad \text{diameter of the shield} \\
d_s & \quad \text{thickness of the shield}
\end{align*}
\( e \) extrusion rate of the core
\( E \) Young's modulus of the ground
\( E_k \) entity of \( N^2 \) chart (the subscript \( k \) refers to the entity numbering)
\( E_l \) Young's modulus of the lining
\( E_s \) Young's modulus of the shield
\( E_{sc} \) Young's modulus of the shotcrete
\( E_{ss} \) Young's modulus of the steel
\( F \) thrust force
\( F_a \) thrust force (auxiliary system)
\( F_b \) boring thrust force
\( F_c \) maximum cutter force
\( f_c \) uniaxial compressive strength of the ground
\( f_{ce} \) maximum compressive stress of the deformable concrete elements
\( f_{cl} \) uniaxial compressive strength of the lining
\( f_{sc} \) uniaxial compressive strength of the shotcrete
\( F_f \) thrust force needed for overcoming friction
\( F_f^* \) normalized required thrust force (for overcoming friction)
\( F_g \) thrust force that can be reacted by the grippers
\( F_{g,i} \) installed gripper force
\( F_i \) installed thrust force
\( f_i \) boundary condition for the simulation of the tunnel support
\( F_{m} \) thrust force (main system)
\( F_r \) required thrust force
\( f_s \) boundary condition for the simulation of the shield
\( f_{y,ss} \) yield stress of the steel
\( G \) ground
\( H \) depth of cover
\( h \) hydraulic head
\( h_0 \) initial hydraulic head
\( h_0^* \) reduced initial hydraulic head
\( k \) permeability of the ground
\( K_l \) stiffness of the lining
\( K_s \) stiffness of the shield
\( l \) length of a critical geological zone
\( L \) length of the shield
\( L_f \) length of the front shield (double shielded TBM)
\( L_r \) length of the rear shield (double shielded TBM)
\( N \) hoop force
$N$ number of physical and functional entities of a $N^2$ chart

$n$ surface normal

$n_c$ number of cutters

$N_{ce}$ hoop force in the deformable concrete elements

$n_{ce}$ number of deformable concrete elements

$N_f$ friction loop resistance

$n_f$ number of friction loops

$n_g$ ground porosity

$N_{max}$ yield load of the steel set

$N_{sc}$ hoop force in the shotcrete ring

$N_p$ yield load of the sliding connections

$p$ ground pressure

$P$ penetration

$P_{A}, P_{B}$ problems in the back-up area (zone A and zone B, respectively)

$\rho_g$ gripper pressure

$\rho_s$ average ground pressure acting upon the shield

$\rho_w$ pore water pressure

$\rho_{w,0}$ initial pore water pressure

$Q$ water flow

$q_n$ boundary flux

$r$ cutter head rotational speed

$R$ tunnel radius

$R_{f,T}$ reaction force

$R_m$ outer radius of the computational domain

$r_{\text{max}}$ maximum possible cutter head rotational speed (TBM design)

$r_{\text{max}*}$ maximum possible cutter head rotational speed (operational conditions)

$s$ step length (step-by-step calculations)

$S_1, S_2$ tunnel support (type, quantity, parameters, distance behind face) in the machine area and in the back-up area, respectively

$t$ time

$T$ torque

$t_1$ operational standstill time

$t_2$ standstill time due to jamming of the TBM

$t_3$ standstill time due to other problems

$t_{\text{crit}}$ critical duration of a standstill

$T_f$ required torque for overcoming friction

$T_g$ torque that can be reacted by the grippers

$T_i$ installed torque
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\( T_r \)  
rolling resistance of the cutter head

\( u \)  
radial displacement of the ground (at the tunnel boundary)

\( u_e \)  
extrusion of the core

\( u_l \)  
radial displacement of the lining

\( v \)  
advance rate

\( v_g \)  
gross advance rate

\( v_n \)  
net advance rate

\( x \)  
radial co-ordinate (distance from the tunnel axis)

\( y \)  
axial co-ordinate fixed to the advancing tunnel face (distance behind the tunnel face)

\( y' \)  
position of the tunnel face

\( y^* \)  
axial co-ordinate spatially fixed

\( z \)  
elevation

\( # \)  
number

\( \{x-y\} \)  
interactions between the entities \( E_x \) and \( E_y \) of a \( N^2 \) chart

\( \Delta D \)  
overboring (facility, amount of the increase of the boring diameter \( D \))

\( \Delta D_l \)  
annular gap (in diameter)

\( \Delta D_s \)  
overcut (in diameter)

\( \Delta R \)  
size of the radial gap between shield and bored profile

\( \Delta R_f \)  
radial gap size of the front shield (double shielded TBM)

\( \Delta R_l \)  
size of the radial gap between lining and bored profile

\( \Delta R_r \)  
radial gap size of the rear shield (double shielded TBM)

\( \alpha \)  
utilization degree of the TBM

\( \delta \)  
ground deformation

\( \varepsilon_{ss,max} \)  
failure strain of the steel

\( \varepsilon_t \)  
hoop strain

\( \varepsilon_{ce} \)  
hoop strain of the deformable concrete elements

\( \varepsilon_{sc} \)  
hoop strain of the shotcrete

\( \varepsilon_{ss} \)  
hoop strain of the steel set

\( \gamma \)  
unit weight of the ground

\( \gamma_w \)  
unit weight of the water

\( \varphi \)  
angle of internal friction of the ground

\( \mu \)  
shield skin friction coefficient

\( v \)  
Poisson’s ratio of the ground

\( \sigma \)  
stress

\( \sigma_0 \)  
initial stress

\( \sigma_1 \)  
maximum principal stress

\( \sigma_3 \)  
minimum principal stress

\( \sigma_0 \)  
ground pressure acting upon the tunnel support
\( \sigma_{\text{max}} \) maximum possible ground pressure acting upon the tunnel support (bearing capacity)

\( \sigma_l \) hoop stress

\( \sigma_{ce} \) hoop stress in the deformable concrete elements

\( \sigma_{sc} \) hoop stress in the shotcrete

\( \sigma_{ss} \) hoop stress in the steel set

\( \sigma_{\text{TBM}} \) ground pressure acting upon the TBM (cutter head or shield)

\( \sigma_t \) tangential stress

\( \sigma_{xx} \) radial stress

\( \sigma_{xy} \) shear stress

\( \sigma_{yy} \) axial stress

\( \tau_{\text{TBM}} \) shear stress acting upon the TBM (cutter head or shield)

\( \psi \) dilatancy angle of the ground
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