Capitolo 15

TBM tunnelling in complex rock formations

G. Anagnostou, R. Schuerch

ETH Zurich, Switzerland

M. Ramoni

Basler & Hofmann AG, Zurich, Switzerland

1 Introduction

As complexity can be understood in different ways depending on the context, some conceptual clarifications are provided as a starting point (Section 2). Subsequently, we address selected aspects of complex formations: the variability of squeezing, the adverse effects of water, tunnelling in fault zones and the applicability of closed shields in weak rocks (Sections 3 to 6). Finally, we present two case histories addressing two key problems of TBM tunnelling in complex rock formations: the loads that develop on the shield under variable squeezing conditions (Section 7) and the stability of the tunnel face in faults or weak water-bearing rocks (Section 8).

2 Complexity

2.1 Geological complexity

Complexity is an ambiguous term, even from a purely geological perspective. A formation is characterized as geologically complex either due to the complexity of its geological history, i.e. the processes that led to its present appearance (see Table 1 for an overview) or, more commonly, due to its present heterogeneity with respect to lithological and structural characteristics. Although heterogeneity may be the result of a complex geological history, the latter does not always lead to a heterogeneous formation (Dzulynski 1977): a complex series of genetic or epigenetic processes may actually result in a decrease of the degree of heterogeneity (e.g. a fine-grained fault gouge).

The notion of complexity is ambiguous even with the more specific term "structurally complex formations". The attribute "structural" may refer to the present appearance of the formation, regardless of the processes that have led to this appearance. On the other hand, the adverb "structurally" points to tectonic processes and suggests therefore that "structurally complex" denotes that subset of geologically complex formations whose heterogeneity is due solely to folding, faulting or fracturing (cf. Barbier 1977).

In the following, we understand geological complexity in the sense of heterogeneity with respect to structure and lithological composition (Fig. 1). As any material is intrinsically heterogeneous and discontinuous at microscopic level, the notion of heterogeneity per se is trivial unless applied with reference to a specific scale, which in turn will depend on the problem under consideration. The relevant scale is therefore different for a mineralogist, a structural geologist or a geotechnical engineer, for example.
### Table 1. Origins of lithological or structural heterogeneities (based upon Dzulynski 1977)

<table>
<thead>
<tr>
<th>Origin</th>
<th>Examples</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Genetic processes</strong></td>
<td></td>
</tr>
<tr>
<td>Sedimentation and</td>
<td>Rock formations consisting of alternating beds of various lithological types such as flysch, sedimentary melanges (“olistostromes”) and metamorphism; alluvium or other quaternary soil deposits consisting of alternating fine- and coarse-grained layers and lenses</td>
</tr>
<tr>
<td>Glacial movements</td>
<td>Moraines of central Europe or northwest America, characterized by variable fine- and coarse-grained soil fractions with variable quantities of bigger or smaller boulders</td>
</tr>
<tr>
<td>Mass movements</td>
<td>Landslide breccias</td>
</tr>
<tr>
<td>Igneous processes</td>
<td>Sedimentary formations penetrated by magmatic intrusion dikes</td>
</tr>
<tr>
<td><strong>Epigenetic processes</strong></td>
<td></td>
</tr>
<tr>
<td>Tectonic processes</td>
<td>Variably fractured rocks; rapidly changing orientation of schistosity and bedding planes due to intensive folding; tectonic melanges consisting of a sheared fine-grained matrix with rock fragments</td>
</tr>
<tr>
<td>Weathering</td>
<td>Rock mass with open discontinuities due to chemical weathering by meteoric water near the surface (Brekke and Howard 1972)</td>
</tr>
<tr>
<td>Alteration</td>
<td>Irregular occurrence of soil-like granite even at greater depths due to hydrothermal alteration of micas and feldspars into clay (Barbier 1977)</td>
</tr>
<tr>
<td>Solution</td>
<td>Carbonatic rocks with variable quantities of smaller or larger karst cavities, possibly filled by breccias following cavity collapses</td>
</tr>
<tr>
<td>Combinations</td>
<td>Hydrothermally altered rock in fault vicinity; fractured and/or weathered host rock in the vicinity of intrusion dikes</td>
</tr>
</tbody>
</table>

**Figure 1.** Gotthard Basetunnel in the region of the Chièra synform: (a) heterogeneity at the regional scale (geological profile after Rütti et al. 2008); (b) heterogeneity at the scale of a rock specimen from the Eastern TBM drive through the Chièra synform (photograph: courtesy H.-J. Ziegler).
2.2 Geotechnical complexity

Depending on the scale, geological complexity may (but will not necessarily) manifest itself in the form of difficulties in the analysis, design or construction process. For example, the geotechnical characterisation of a formation is difficult if its structural characteristics and composition are variable at a scale that is, on one hand, too small for considering distinct, practically uniform zones, but, on the other hand, too big for establishing representative rock mass properties in laboratory experiments.

In response to the fact that geological complexity does not always result in geotechnical complexity, Morgenstern and Cruden (1977) introduced a distinction between geological and geotechnical complexity, defining the latter in terms of the rapid variability and heterogeneity of geotechnical properties. AGI (1979) adopted the notion of geotechnical (rather than geological) complexity and characterized formations as complex when their heterogeneity posed major difficulties in terms of formulating suitable models and defining, obtaining and testing representative samples on a scale that was relevant for the given engineering problems and questions, as well as the construction processes and sequences.

Morgenstern and Cruden (1977) noted, furthermore, that geotechnical complexity may exist also in the case of geologically non-complex formations. As an example, they mentioned homogeneous deposits of highly sensitive marine clays, characterizing them as complex on account of the difficulties associated with the evaluation of their geotechnical behaviour. Lombardi (1997) shares this view of complexity in that he also regards as complex formations consisting of rocks that exhibit an anisotropic or a rheologically complex behaviour, regardless of their degree of heterogeneity.

Geotechnical complexity thus means something more than what is induced by the heterogeneity and variability of the geotechnical properties. It includes, for example:

- The so-called "collapsible porous red clays" of central Brazil. Their microstructure collapses upon loading, thus leading to a very atypical strain profile during tunnelling (compressive rather than extensive vertical strains between tunnel crown and surface) and larger surface settlements (cf. Farias and Assis 1996).
- Silty soils, on account of their particularly complex behaviour during tunnelling (Heuer and Virgens 1987). When saturated they resemble a flowing slurry; with a little moisture they exhibit a cohesion that may suffice for stability, but when dried they become a cohesionless powder; occasionally, one may observe all three forms simultaneously at different locations across the tunnel face (Anagnostou 2001).
- The anhydrite-containing claystones of the Gypsum Keuper formation, on account of our limited knowledge as to their swelling (stress-strain) behaviour and the underlying chemical reaction and transport processes (Anagnostou et al. 2010). (Gypsum Keuper is a complex formation also for another reason: swelling intensity is often very variable even along lithologically uniform tunnel sections. This variability is probably associated with the varying hydrogeological conditions on site, which in turn is due to the lithological and structural heterogeneity of the rock at the megascale; cf. Butcher et al. 2010).

Geotechnically complex formations are characterized by an inadequacy of familiar "conceptual schemes, calculation methods and experimental and construction techniques" (Croce 1977). They may present difficulties in modelling (characterization, analysis), in engineering (design, construction) or both.

2.3 Engineering vs. geological vs. modelling complexity

Variability in the composition, structure and geotechnical properties of a formation at the relevant scale, limited knowledge as to its constitutive behaviour or an insufficient understanding of the basic processes underlying the macroscopically observed phenomena certainly justify the attribute of "complex", at least from a modelling perspective. However, as we will see below, a formation that is complex from a geological or modelling perspective is not necessarily complex from an engineering perspective. Furthermore, engineering complexity may arise even if the formation does not exhibit geological or modelling complexity.
Figure 2. Formations exhibiting different types of complexity: (a) jointed sedimentary rock; (b) quaternary clayey soil deposit with coarse-grained layers or lenses and boulders; (c) rock overlain by soil; (d) to (f) alternating layers of variable thickness; (g) erratically distributed water-bearing layers within a low-permeability formation; (h) vicinity of an aquifer; (i) lithological variability due to faults; (j) fine-grained fault core bounded by competent rock.
Thus, for example, soft marine clays proved to be unproblematic for tunnelling in Singapore, despite exhibiting a complex and poorly understood mechanical behaviour (Osborne et al., 2008). The effect of their modelling complexity was eliminated by closed shield tunnelling with a face support pressure equal to about 100% of the \emph{in situ} stress, \emph{i.e.} by selecting an appropriate construction method.

Typical sedimentary rock formations with a sub-horizontal bedding and two orthogonal joint sets (Fig. 2a) are, strictly speaking, complex both from a geological and a modelling perspective: the rock mass is discontinuous at the scale of the opening; the persistence and the spacing of the bedding planes and of the joints generally exhibit a natural variation; the strength and deformability parameters of the rock mass cannot be determined by laboratory tests. However, these aspects do not result in engineering complexity. Tunnelling through such a formation is routine.

With the exception of massive, competent rocks, geomaterials generally exhibit a highly non-linear mechanical behaviour including, for example, hardening or softening, rate-dependency and anisotropy. Consideration of these aspects of mechanical behaviour may be indispensable in critical geotechnical situations close to the technical feasibility limit of a project. In most cases, however, the complexity of the actual mechanical behaviour does not result in engineering complexity, as simplified constitutive models are adequate from a practical point of view even for demanding engineering tasks (such as tunnelling through squeezing rocks, cf. Dong and Anagnostou 2013, 2014).

Quaternary soil deposits are often geologically complex in that they consist of alternating, chaotically arranged fine- and coarse-grained layers and lenses of varying thicknesses (Fig. 2b). Due to this variability and heterogeneity, their geotechnical characterization in terms of strength-, deformability- and permeability-parameters tends to be difficult, but this does not always result in engineering complexity, which will be high in the case of conventional tunnelling, but rather low in closed shield tunnelling.

Glacial tills often contain rock boulders. If their fraction is smaller than about 30 - 40%, the mechanical behaviour of the till will be governed by that of the surrounding matrix (cf. Irfan and Tang 1993, Lindquist 1994). If the fraction of the boulders is higher, they will contribute to the stiffness and frictional resistance of the ground. The distribution of the boulders is normally hard to predict with sufficient resolution. This introduces an additional modelling complexity, which is nevertheless of subordinate importance from a practical point of view. The occurrence of boulders may increase engineering complexity in TBM tunnelling, but for a completely different reason: depending on the cutterhead design and on the stiffness of the soil around the boulders, it may be impossible to get past them with the TBM, thus necessitating demanding manned interventions in the working chamber.

On the other hand, a formation may be complex from an engineering point of view without being geologically complex and without exhibiting modelling complexity. Consider, for example, a shallow tunnel crossing rock overlain by soft ground (Fig. 2c) – as encountered, for example, in Munich (quaternary gravels overlying tertiary marls; Kovári and Weber 1991), Zurich (fluvial gravel or glacial till overlying molassic rocks; Guertner and Bosshard 2004) or Singapore (residual soils and recent deposits overlying granite and sedimentary rocks, respectively; Shirlaw et al. 2003). Although the geotechnical behaviour both of the rock and of the overlying soil is in most cases well understood, and although such formations can hardly be characterized as geologically complex, they may be complex from an engineering point of view depending on the thickness of the rock cover above the tunnel crown and on the geometry of the rock-soil interface. For example, irregular weathering profiles or old erosion channels of unknown location and of variable depth introduce risks (face stability, surface settlement), the management of which may render construction highly demanding, particularly in conventional tunnelling (Kovári and Weber 1991). Complexity is less pronounced in closed shield tunnelling provided that the TBM can be operated in closed mode and that the contrast in the mechanical characteristics of rock and soil is not so big that it affects the boring process or the effectiveness of the face support (\emph{e.g.}, mixed face EPB tunnelling in granite and overlying soil; Della Valle 2001, Thewes 2004).

A rock formation consisting of alternating, lithologically different beds (\emph{e.g.} clay- and sandstones) can, depending on the thickness of the beds and on the size of the opening, be conceived as:
homogeneous and isotropic in sections of the alignment (Fig. 2d); heterogeneous (Fig. 2e); or homogeneous and transversally isotropic (Fig. 2f). Although such a formation cannot be characterized as complex from a geological or a modelling point of view, it may exhibit engineering complexity in the case of frequent lithological changes or mixed face conditions (Fig. 2d and 2e), depending on how different the layers are with respect to, for example, the boreability or support requirements. Anisotropy per se (Fig. 2f) does not increase complexity considerably (most rock formations exhibit a transversal isotropy), but can, when found in combination with intense folding, lead to highly variable behaviour during tunnelling, for example under squeezing conditions.

3 Squeezing

The variability of the ground behaviour is one of main causes for construction set-backs in tunnelling through squeezing rocks (Kovári 1998). Squeezing variability can be traced back to heterogeneities in the rock mass at different scales. On the one hand, even relatively small fluctuations in the mechanical or hydraulic properties of a macroscopically homogeneous rock mass may have a major effect on the development of deformations and pressures. On the other hand, a lithological and structural heterogeneity in the rock mass (even in the scale of only few meters) may lead to significant variations in the ground response along the tunnel.

For example, squeezing deformations depend strongly on the orientation of the schistosity, which in complex formations with intense folding may vary considerably within short distances along the tunnel. This structural heterogeneity leads to a wide variability in squeezing intensity. The solid line in Fig. 3a shows the convergences that occurred along a section of the Gotthard Basetunnel with uniform lithology and shearing degree but variable schistosity orientation. The latter can be expressed by the so-called schistosity influence factor $S$ (Mezger et al., 2013), which combines the dip and strike of the schistosity (Fig. 3b). The convergence correlates reasonably well with the schistosity influence factor $S$ (dashed line in Fig. 3a), thus supporting the hypothesis that the variability in this case is due solely to the variable schistosity orientation.

As mentioned above, other reasons for variability may include relatively small fluctuations in mechanical properties. This can be illustrated by means of a computational example. Figure 4a shows the convergence of an unsupported opening as a function of the tunnel depth. The thick solid line applies to the average material constants of the so-called “kakirites” encountered in the Sedrun section of the Gotthard Basetunnel (Vogelhuber 2007). The hatched zone shows the effect of a variation in the friction angle of ±15%, which is small considering the variation in the friction angles determined experimentally (Fig. 4b). The results indicate that sensitivity with respect to the friction angle increases with depth and that the effect of a variation in the friction angle can be far greater than the effect of the overburden.

Finally, let us consider the case of tunnelling through a heterogeneous rock mass consisting of alternating weak and competent rock layers striking perpendicularly to the tunnel axis (as, for example, in the first half of the northern Tavetsch massif in the Gotthard Basetunnel). The squeezing intensity along the tunnel depends in this case not only on the strength, deformability and thickness of the different zones, but also on their interplay: tunnel excavation mobilizes shear stresses at the layer interfaces because the competent rock deforms less than the weak rock. This so-called “wall-effect” (Kovári and Anagnostou 1995) reduces the deformations of the weak layers. Figure 5 shows, for the example of a 10 m diameter tunnel, the ratio of the maximum to the minimum convergences (in the middle sections of the weak zones and of the competent rock, respectively) as a function of the layer thickness $w$. According to the diagramm the convergences may vary by a factor 3 to 6 within distances of just 2 to 5 m ($w = 2$ to $5$ m), i.e. even between the monitoring sections (usually spaced 5 - 10 m apart).

TBMs are particularly vulnerable to squeezing due to the very limited space available for deformations, the limited flexibility with respect both to the type of support and the location of its installation and the associated difficulties with remedial measures such as the application of additional support a posteriori (Ramoni and Anagnostou 2010a). A case history will be presented in Section 7.
Figure 3. (a) Convergence and schistosity influence factor $S$ along the NE tube of the Gotthard Basetunnel in the northern part of the Tavetsch-Massif; (b) definition of the schistosity influence factor $S$ (after Mezger et al., 2013).

Figure 4. (a) Convergence $u/a$ of an unsupported opening as a function of the overburden; (b) distribution of friction angle (after Cantieni and Anagnostou 2007; parameters for l.h.s. diagram: Young’s modulus $E = 2000$ MPa, Poisson’s ratio $\nu = 0.30$, friction angle $\phi = 26.7\degree \pm 15\%$, cohesion $c = 0.6$ MPa, dilatancy angle $\psi = 5\degree$).

Figure 5. Variability of convergence as a function of the thickness $w$ of the layers (after Cantieni and Anagnostou 2007; parameters: initial stress $\sigma_0 = 10$ MPa, Young’s Modulus $E_{hard} = 10$ GPa, $E_{weak} = 1$ GPa, Poisson’s ratio $\nu = 0.3$, friction angle $\phi = 25\degree$, cohesion $c_{hard} = 5$ MPa, $c_{weak} = 0.5$ MPa, dilatancy angle $\psi = 5\degree$).
4 Water

Geological complexity is often associated with alteration or tectonic processes and thus with the presence of weak rocks. Their mechanical behaviour, which is governed by the principle effective stress, is strongly affected by pore water pressure. The response of water-bearing ground is generally time-dependent due to the transient process of excess pore pressure dissipation. The rate of this process is governed by the permeability of the ground, which may be extremely variable in complex formations.

Thus, for example, the presence of permeable layers within a low permeability formation (Fig. 2g) will increase its overall permeability, thereby accelerating excess pore pressure dissipation, the development of deformations or ground pressures and instability phenomena. The complexity induced by thin permeable layers manifests itself in two ways: as a difficulty in determining a representative value for the overall permeability of the ground at a specific chainage; and as a variability in the overall permeability along the tunnel (due to the variable spatial arrangement of the permeable layers).

The time-development of excess pore pressure dissipation and thus of deformation also depends on the length of the drainage paths. The paths can be shorter or longer depending on the distance of the tunnel from aquifers (Fig. 2h). In complex formations, characterized by sequences of aquitards and aquicludes, the length of the drainage paths may vary considerably along the alignment, thus leading to a variable ground response to tunnel excavation.

In order to illustrate the sensitivity of ground behaviour with respect to permeability variations, let us consider a circular tunnel supported either by a practically rigid lining or by a deformable support that yields at a rock pressure of 0.15 MPa (the so-called “resistance principle” and “yielding principle”, respectively; cf. Kovári 1998). Pore pressure dissipation leads to a gradually increasing load (in the case of resistance principle) or deformation (in the case of yielding principle). This happens very slowly if the permeability of the ground is low (Fig. 6a). For permeability that is higher by a factor of ten the rock pressure and deformation will develop ten times faster.

The sensitivity of ground behaviour with respect to permeability becomes evident by considering the pressure (resistance principle) or deformation (yielding principle) at a specific time after excavation. Figure 6b applies to \( t = 15 \) days, which corresponds to a tunnel cross-section at a distance of a few diameters behind the excavation face. In this period the rock pressure (in the case of resistance principle) will increase to a value of between 0.1 and 1.2 MPa depending on the permeability of the ground. The difference between these two values is significant from a structural point of view. A relatively thick shotcrete shell would be easily sufficient for a pressure of 0.1 MPa, but would certainly fail under 1.2 MPa. A permeability variation in the range \( 10^{-11} - 10^{-9} \) m/s would lead to extremely variable behaviour (with respect to the rock pressure close to the face) along the tunnel, even if the mechanical parameters of the rock mass were constant. Consequently, depending on the overall permeability of the ground (which may be affected by the presence of few thin permeable layers), the tunnel engineer may experience the ground as competent or disturbed.

It should be noted that a determination of permeability in the range \( 10^{-11} - 10^{-9} \) m/s is subject to large uncertainties, thus rendering predictions of the time-development of rock pressure unreliable. The problem could be alleviated by installing a heavy support capable of resisting 1.2 MPa, but this is difficult within a short distance from the tunnel face.

A yielding support would offer more time to manage the rock pressure, because the deformation develops considerably more slowly than the pressure (Fig. 6a). As the convergence reaches maximum 0.25 m within the considered critical period of 15 d (Fig. 6b), one might envisage an overexcavation of 0.25 m in combination with a yielding support that can be strengthened later, at a greater distance behind the face. The consequences of overestimating the convergence margin would be (with up to 7 - 8 m² per tunnel metre of additional excavation and concrete for the permanent lining) far less costly than the consequences of underestimating the rock pressure in the case of the resistance principle (damage to the lining and the need for re-profiling).

In conclusion, a yielding support will alleviate the consequences of prediction uncertainties in the present example. In the case of a smaller depth of cover, the uncertainty would be smaller with the resistance principle, because the rock pressure cannot be higher than a fraction of the in situ stress, while the convergences may subject to a big variation depending on the deformability of the
ground. In a nutshell, geological complexity *per se* does not necessarily lead to engineering complexity. Its technical consequences may be greater or smaller depending on the construction method.

Permeability is also important for mechanized tunnelling through waterbearing weak rocks because it affects squeezing intensity close to the face and thus also the loading on the shield and the thrust force required to overcome friction and keep the TBM advancing. Figure 7 shows, for a computational example, the necessary thrust force as a function of the permeability of the ground. Depending on the permeability, a TBM with a fairly high installed thrust of 150 MN may become stuck or may advance without any difficulties. In conclusion, relatively small permeability variations may result in extremely variable TBM operational conditions along the alignment. These effects of the hydraulic properties of the ground are also relevant for the stability of the tunnel face. The stand-up time of the face in waterbearing ground depends on its permeability and on the length of the drainage paths, both of which are inevitably fraught with uncertainties in complex formations. A case history will be presented in Section 8.

![Figure 6](image-url)

**Figure 6.** (a) Time-development of rock pressure $\sigma$ (resistance principle) and of rock deformation $u$ (yielding principle) in a low-permeability ground ($k = 10^{-10}$ m/s); (b) rock pressure $\sigma$ (resistance principle) and rock deformation $u$ (yielding principle) 15 days after support installation as a function of the permeability $k$ of the ground (after Anagnostou and Kovári 2005; parameters: initial stress $\sigma_0 = 7.5$ MPa, initial pore water pressure $p_0 = 2.5$ MPa, Young’s Modulus $E_{\text{hard}} = 300$ MPa, $E_{\text{weak}} = 1$ GPa, Poisson’s ratio $\nu = 0.3$, friction angle $\varphi = 25^\circ$, cohesion $c = 50$ kPa, dilatancy angle $\psi = 5^\circ$).

![Figure 7](image-url)

**Figure 7.** Required thrust force $F$ during ongoing excavation as a function of the permeability $k$ of the ground (after Ramoni and Anagnostou 2011a; parameters: initial stress $\sigma_0 = 10$ MPa, initial pore water pressure $p_0 = 1$ MPa, Young’s Modulus $E_{\text{hard}} = 300$ MPa, $E_{\text{weak}} = 1$ GPa, Poisson’s ratio $\nu = 0.25$, friction angle $\varphi = 25^\circ$, uniaxial compressive strength $f_c = 3$ MPa, dilatancy angle $\psi = 5^\circ$).
5 Fault zones

Geological complexity is often associated with tectonic processes and more specifically with the substantial heterogeneity induced by faulting or shearing. Fault zones occur alone or in a group, with a single or a branching fault core and with more or less competent rock in between. The rock at the core of the fault may have been crushed to a fine-grained cohesionless material (fault gouge) containing a variable quantity of rock clasts (fault breccia). In some cases the fault core is accompanied laterally by a heavily jointed and fractured rock zone, while in other cases the transition to competent rock is sharp. The condition and the behaviour of the ground in the faults depend essentially on the dominant lithology of the competent host rock. In the case of limestones, for example, the fault zone will presumably be blocky and brecciated; while in predominately shaly rocks the fault material will be fine-grained (clayey or silty) and resemble soft ground.

Potential problems for TBMs in **blocky fault zones** include high water inflows, which may cause excessive wear as well as difficulties with mucking-out and with the installation of the segmental lining (washing-out of the annulus grout), as well as rock instabilities in front of the TBM which may block or damage the cutterhead, or rock instabilities in the machine area in the case of gripper TBMs. The risk of high water inflows and the need for impermeabilization grouting in advance of the excavation can be assessed on the basis of the water quantities observed in boreholes drilled ahead of the TBM during construction. Grouting, besides sealing the rock mass, also improves its strength, thereby helping to deal with the previously mentioned stability problems at the same time.

The possible hazards in **faults with fine-grained infillings** are associated with high pore water pressures combined with the low strength and high deformability of the fault material and they include instabilities of the ground in front of the TBM (which may block the cutter head), or rapidly developing convergences (which may lead to shield jamming). It should be noted that such problems cannot be identified in advance on the basis of the water quantities in the probe holes because the water inflows are very limited in fine-grained materials of low permeability. Very low stand-up times and face instabilities have been observed, for example, in the Vardoe and Ellingsoy subsea tunnels despite rather small water ingress in the probe holes (less than 10 and 30 l/min, respectively, Dahlö and Nilsen, 1994).

Problems such as those mentioned above may lead to full-day standstills and may decrease TBM utilization to just a few percent (Beckmann and Krause 1982). In extreme cases, time-consuming hand mining is necessary, with the TBM actually representing an obstacle to such operations. For example, two faults encountered during construction of the Western tube of the Gotthard Basetunnel in the Amsteg and Faido sections took 152 and 138 working days, respectively, to overcome (see Ehrbar et al. 2013 for a comprehensive report). It is characteristic of the variability of the ground that the Eastern TBM drives did not encounter any difficulties although the two tubes were spaced only 40 m apart.

The occasionally dramatic experiences from tunnelling through fault zones can lead us to overlook the fact that fault zones often do not cause any special difficulties and may even remain entirely unnoticed during a TBM drive. For example, fault gouge may be cemented by secondary mineralization or the faults may be very thin.

Thin faults, although not problematic *per se*, may cause, however, lithological changes within short distances along the alignment (Fig. 2i). If the rock units are different with respect to boreability, stability of the tunnel face, support demand in the machine area or squeezing intensity, the juxtaposition of different lithologies may reduce TBM utilization. Such a formation is experienced as complex from the tunnel engineering point of view, even if each tunnel section *per se* may be geologically and geotechnically simple.

This is true also with respect to distinct fault zones containing a geometrically well-defined single fault core that consists of water-bearing highly sheared gouge and is bounded by competent rock (Fig. 2j). Apart from tectonic melanges containing a significant fraction of hard, unaltered rock fragments floating within the soft matrix, the mechanical behaviour of a fault core is governed by the lithology of the matrix. Although the geotechnical characterization of the fault may be quite straightforward, such a situation is extremely complex from the tunnel engineering point of view, because it necessitates measures such as drainage and grouting and possibly auxiliary adits to make these measures possible.
One remarkable feature of complex fault zones is the anomaly of pore pressure distribution, which is due to their extreme permeability heterogeneity (Ganarod et al. 2008, Faulkner et al. 2010). As pointed out by Evans et al. (1997), fault zones often exhibit permeability contrasts of several orders of magnitude and may simultaneously include both aquifers and aquicludes. The fault core (if fully developed and consisting of gouge) typically has a low permeability, while the adjacent rocks normally (depending on the connectivity of the joints) exhibit higher permeabilities than the competent host rock (Brekke and Howard 1973).

The spatial heterogeneity of the permeability affects the pore pressure distribution in the ground ahead of the tunnel face. If the fault core exhibits a lower permeability than the adjacent rock (e.g. silty fault gouge bounded by fractured rock), then the pore pressure gradient within the core will be high (particularly if the fault is relatively narrow), which is unfavourable with respect to face stability. If, on the other hand, the fault core consists of crushed rocks of higher permeability than the adjacent competent rocks, then the pore pressure gradients will be lower and less unfavourable than in the case of uniform permeability. Fault thickness and permeability are thus, in addition to the shear strength of the material, important for face stability. All of these parameters may be highly variable in faults.

The high pore pressure gradients that develop deep below the water table particularly in lower permeability faults may lead to failure of the excavation face even if the fault core exhibits cohesion (e.g. cemented fault gouge or rocks that are not completely sheared). In view of the dramatic effect of the pore pressure gradient, possible remedial measures include advance drainage or compensation of the hydrostatic pressure by closed shield tunnelling, possibly in combination with ground improvement by advance grouting (Section 8). Face stabilization by mechanical means (e.g. bolt reinforcement) is possible only at small depths below the water table or in narrow faults with a permeability not much lower than that of the adjacent competent rock. Consider for example a 10 m diameter tunnel crossing a fault with cohesive material 130 m below the water table. Figure 8 shows the required face support force as a function of the fault thickness and permeability. Even with a dense face reinforcement (which in the case of TBM tunnelling is not feasible), the support force would be not more than 10 - 15 MN. This force would be sufficient just for 2 - 3 m thick faults.

It should be noted that the heterogeneous permeability of the ground (which may be particularly pronounced in faults) is adverse not only with respect to the behaviour of the tunnel face but also concerning the effectiveness of advance drainage or grouting.

![Figure 8. Required support force S as a function of the fault thickness d_f and permeability k_f (after Zingg and Anagnostou 2012; parameters: friction angle $\varphi = 30^\circ$, cohesion $c = 100$ kPa, submerged unit weight $\gamma' = 12$ kN/m$^3$).](image-url)
6 Closed shields

Closed shields enable continuous control of the tunnel face and of the groundwater during excavation. An evaluation of the types of closed shield is sometimes difficult even for purely soil tunnels (e.g. glacial tills), and becomes even more demanding for tunnels that also contain sections through rock or with a mixed face. The difficulties increase if the lithological changes occur frequently along the alignment (thereby necessitating frequent changes of operation mode) or the tunnel also crosses weak rocks, which, although being outside the typical operational range of closed shields (they were developed for soils), may require closed mode operation (particularly under high hydrostatic pressures). Such conditions are not rare in geologically complex formations.

As mentioned in the last section, a high pore pressure gradient may cause failure of the tunnel face even if the rock exhibits a considerable shear strength. On the other hand, in the absence of a pore pressure gradient, a low cohesion in the order of 20 - 30 kPa is sufficient for face stability in typical traffic tunnels (Anagnostou and Kovári 1994). As weak rocks usually exhibit a higher cohesion, even if weathered and sheared, stability can be achieved in most cases just by compensating the hydrostatic pressure by the pressure in the working chamber.

With a slurry shield it is easy to establish hydraulic equilibrium between working chamber and the groundwater, because the slurry pressure can be regulated directly. Proven TBM technology now allows slurry shields to advance in closed mode under hydrostatic pressures of up to 15 bar (Section 8). A difficult situation may arise in the case of a mixed face with jointed hard rock and cohesionless soil. The stabilization of the latter necessitates a slurry pressure in excess of the hydrostatic pressure. Maintaining an excess pressure may, however, be impossible if the jointed rock is highly permeable.

The water pressure in the working chamber of an EPB shield depends on the characteristics of the excavated ground, the way it is mixed inside the working chamber and its interplay with added polymers or foams (Anagnostou and Kovári 1996). Achieving a sufficiently high water pressure in the working chamber may be impossible if the muck is extremely coarse-grained or contains a large fraction of rock chips. As EPB shields support the face with the excavated muck itself, they are more sensitive to deviations from the optimum geological conditions than slurry shields. In addition, EPB shields in rock are more prone to wear than slurry shields, thereby necessitating maintenance work in the working chamber more frequently.

Closed-shield operation under suboptimum geological conditions involves a higher probability of face failures. Hereinafter we focus on urban environments, which are particularly adverse in this respect, because of the possibility of damage to third parties. In many cases, both EPB- and slurry-shield tunnelling may be feasible only in combination with additional protective measures (such as injections, jet grouting or artificial ground freezing) over sections of the alignment. Lack of free space at the surface may limit the feasibility of such measures or increase their cost considerably. As a particular tunnel stretch may be favourable for one type of face support but unfavourable for another, the locations and the quantities of the additional measures and the time needed to implement them should be evaluated separately for each type and the associated costs should be added to the capital and operational costs of the TBMs.

Estimates of appropriate quantities tend to be highly uncertain in the case of complex geological formations. Procurement methods such as design-build with a lump sum shift a considerable portion of the geological risk onto the contractors, and experience from a number of projects shows this to be unsuitable: it may lead to high contingencies in the bids or, as is more often the case, to project delays or poor quality work, including unacceptable settlements or even collapses. The limited time available for bid preparation, in combination with quite understandable levels of optimism during the tender phase, do not encourage a cautious assessment of the additional measures required.

Although it is undoubtedly advantageous to leave machine type decisions to the market, the quantities of additional measures required should be estimated by the owner in the pre-tender design and included (separately for the different face support types) in the tender documents. Failing this, tender price comparability will be lost.
7 Uluabat tunnel

7.1 Introduction

The 11.8 km long Uluabat Tunnel has a cross-section of about 20 sqm and belongs to a hydropower scheme located about 100 km south of Istanbul. It was excavated with a TBM, except for a 2 km long section at the Southern portal, which was constructed conventionally (Fig. 9). The tunnel crosses Triassic, slightly metamorphic detritic rocks (meta-sandstones, meta-claystones and graphitic schists) that are overlain by Jurassic limestones over a relatively short stretch in the middle of the alignment (Fig. 9). Severe squeezing conditions were encountered in the Triassic formation.

According to Bilgin and Algan (2012), who have reported comprehensively on experience from the Uluabat Tunnel, tunnel construction started in 2002 with conventional excavation but was halted in 2003 due to the squeezing. Although the overburden was not particularly high (100 m), the displacements were mostly in the order of 0.5 m (10% convergence) and locally even higher (more than 1 m).

Mechanized excavation started in June 2006 und was completed in March 2010. The TBM was equipped with a 12 m long single shield, it had a boring diameter of 5.05 m and an installed thrust force of about 30 MN. The radial overcut was 3 cm at the front part of the shield, increasing to 4 cm at the shield tail. Squeezing caused jamming of the shield even in the first 3 km of the TBM drive, where the depth of cover was still quite moderate (120 m). The TBM was stuck 18 times, which caused a down-time of 192 days in total. Rescue galleries were needed on each occasion in order to free the TBM and restart excavation. The photograph in Figure 10 was taken during such an intervention and shows the extrados of the shield tail and of the segmental lining. The ground has visibly closed the gap around the shield, but has not yet established contact with the lining.

Although progress fell to 50 - 80 m/month in the critical stretches, the overall production rate (including all problems) was 210 m per calendar month, which is a remarkable performance.

Figure 9. Longitudinal section of Uluabat tunnel, locations of shield jamming and progress of the TBM drive.
The results available from monitoring are sparse, but they indicate a large variability in squeezing intensity, with maximum deformation rates of up to 60 mm/h. The observed variability can be explained in terms of the variability of the mechanical parameters of the weaker components of the formation (Section 7.2) as well as the overall structure of the ground along the alignment, which is characterized by weak zones of variable length alternating with stretches of more competent rock (Section 7.3).

### 7.2 Variability of the mechanical parameters

In the critical zones the ground consisted of a claystone matrix containing sandstone lenses 1 - 50 cm in size. The claystones appeared intensively sheared with several slickensides and they disintegrated quickly under the action of water. Laboratory testing revealed an angle of internal friction of about 20º, strongly variable cohesion values (50 - 400 kPa) and a Young’s modulus of 200 - 1000 MPa. As the fraction of the hard sandstone lenses was only about 20%, the mechanical properties of the claystone matrix were decisive for the behaviour of the ground.

In order to evaluate the effectiveness of possible TBM improvements (in relation to the risk of shield jamming) and the feasibility of an acceleration in tunnel completion by means of a second TBM from the Southern portal, sensitivity studies were carried out based on an axially symmetric model (Ramoni and Anagnostou 2011b). Due to the high convergence rates observed in situ, the time-dependency of the ground behaviour (which is favourable with respect to shield loading) was disregarded. The necessary thrust force was estimated considering the most critical phase for shielded TBMs in a weak, squeezing rock (restart of advance after support installation).

For given depth of cover and TBM parameters (shield length, installed thrust force etc.), 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 the TBM may become stuck. In the present case, the critical parameter range was determined in terms of the Young’s modulus $E$ and the cohesion $c$ of the rock mass (all other ground parameters being kept constant). The reason for this choice was the large uncertainty concerning the values of these two parameters in combination with the great sensitivity of the ground response with respect to their variation.

The solid curve in Figure 11 was computed using the nomograms of Ramoni and Anagnostou (2010b) and it shows the critical ground conditions for the actual TBM and an overburden of 120 m. All points of this curve (hereafter referred to as “critical curve”) fulfill the condition that the required thrust force is equal to the installed thrust force. Points above the critical 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, points below the critical curve indicate 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. The TBM operating conditions can be assessed quickly by considering the position of the critical curve relative to this box. As the solid...
critical curve applying to the parameters of the actual TBM crosses the grey box in its upper half, one may conclude that the TBM may (but will not necessarily) get stuck, which agrees with the variable conditions experienced during the TBM drive. The lubrication of the shield extrados by injections of bentonite, the installation of a higher thrust force $F$ or an excavation with a larger overcut $\Delta R$ would shift the critical curve towards the lower-left part of the box (dashed curves in Fig. 11), which indicates that these measures improve the situation considerably, but some prediction uncertainty still remains (the dashed curves intersect the grey box). The risk of shield jamming could be mitigated by a combination of all these measures (point A in Fig. 11). 

As mentioned above, the option of an additional TBM drive from the other portal of the tunnel was investigated. 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). According to Figure 11 (point A), 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 of the alignment ($H = 240$ m).

Figure 11. Critical ground conditions for shield jamming at a depth of 120 m (determined after Ramoni and Anagnostou 2010b for a single shielded TBM with $L/R = 5$; initial stress $\sigma_0 = 3$ MPa, friction angle $\varphi = 20^\circ$).

Figure 12. Required thrust force as a function of the length of the weak zone (after Ramoni and Anagnostou 2011b).
7.3 Variability of the length of the weak zones
During the TBM drive in Uluabat, the ground behaviour (as seen from the thrust force required to keep the TBM advancing) changed frequently within short intervals, thus indicating a succession of weak zones with stretches of more competent ground (Bilgin and Algan 2012). Shield jamming occurred mostly (in 13 out of 18 cases) in weak zones which were longer than 10 m. The average length of the critical zones was about 14 m.
The length of a weak zone is significant with respect to the risk of shield jamming because of the stabilizing effect of the adjacent competent rock ("wall-effect", Section 3). The shorter the weak zone, the more pronounced will be the wall-effect and the lower will be the risk of shield jamming for given TBM parameters (installed thrust force, overcut etc.). Figure 12 shows the results of a computational investigation into the effect of the length of a weak zone on the required thrust force. (The assumed values of Young’s modulus and cohesion are marked by a cross in Fig. 11.) The installed thrust of 30 MN would be sufficient for weak zones shorter than about 8 m. With lubrication of the shield extrados the critical length increases to 13 m. These results agree with the experience from Uluabat concerning the effect of lubrication and the critical length of the weak zones.

8 Lake Mead Intake No 3 tunnel
8.1 Project
Lake Mead, behind the Hoover Dam, supplies about 90% of Las Vegas valley’s water. Over recent years, drought has caused the lake level to drop by more than 30 meters. In order to maintain water supplies, a third intake is under construction that is deep enough to function at the lowest lake levels (Feroz et al. 2007). The main structures of the project are a 170 m deep access shaft, a tunnel approximately 4'700 m long with a bored diameter of 7.22 m and an intake structure in the middle of the lake.
The Lake Mead Intake No 3 tunnel is located in an area of past intense tectonic activity resulting in a complex geological structure over the alignment. The tunnel crosses an alternation of metamorphic and tertiary sedimentary rocks (conglomerates, breccias, sandstones, siltstones and mudstones of very variable quality), at a maximum depth of about 139 m beneath the current lake level (Fig. 13). The rock cover decreases in the last portion of the alignment, amounting locally to just 20 - 30 m. Due to the existence of several faults in the project area, the ground at the elevation of the tunnel can be recharged directly from Lake Mead, which implies the possibility of considerable water ingress during construction.
Due to the variability of the ground conditions, the poor quality of the sedimentary rocks over long portions of the alignment and the high hydrostatic pressures, attention was paid right from the start to the potential hazards of a cave-in of the rock at the working face or a flooding of the tunnel, and a decision was taken to construct the tunnel using a convertible hybrid TBM (Feroz et al. 2007, McDonald and Burger 2009).
The TBM is capable of boring in open- or closed-mode. In open mode, a screw conveyor extracts the excavated rock from the working chamber. In closed mode, the screw conveyor is retracted from the cutterhead and the TBM operates as a closed shield by applying a pressurized bentonite slurry which counterbalances the hydrostatic pressure and stabilizes the tunnel face. The machine is designed to cope with water pressures of more than 14 bar – the highest pressures ever seen in closed shield tunnelling anywhere in the world (Fig. 14a). Although the TBM can bore in closed mode at this depth below the water level, however, the high hydrostatic pressures make inspection and maintenance in the working chamber extremely demanding.
The inherent technological risk of such high-pressure closed-mode TBM operations and the lack of experience with hyperbaric interventions at 14 bar in tunnelling made it necessary to develop fallback strategies involving open mode or low pressure closed mode operation, partially in combination with advance grouting and/or drainage.
Figure 13. Geological profile of Lake Mead Intake No 3, construction progress and distribution of permeability along the alignment.

Figure 14. (a) Maximum slurry pressures and, (b), maximum air pressures for interventions at the cutterhead in selected tunnel projects with high hydrostatic pressure (after Holzhäuser et al. 2006, revised).
8.2 Assessment of ground behaviour

The main difficulty with assessing the behaviour of the prevailing weak, water-bearing, low-permeability tertiary rocks is that their response to tunnel excavation is time-dependent. This means that the tunnel face might be stable initially but fail after a period of time: the short-term behaviour of the ground (i.e. the behaviour under so-called "undrained conditions") is characterized by a constant water content and the development of negative excess pore pressures that have a stabilizing effect. This effect is only temporary, however, because the excess pore pressures dissipate over the course of time. The long-term behaviour (so-called "drained conditions") is characterized by a fully developed seepage flow towards the tunnel, which is unfavourable for face stability and may necessitate stabilisation measures (depending on the shear strength of the ground).

The central questions in the tertiary sedimentary rocks were thus: How long would the face remain stable? How certain was it that favourable undrained conditions would prevail during excavation or short standstills?

The decisive parameter in this respect was the permeability $k$ of the ground, which governs how rapidly the excess pore pressures dissipate and how rapidly the transition from undrained to drained conditions occurs. In general, an assumption of favourable short-term conditions will be more reasonable the less permeable the ground, the more rapid the excavation and the shorter the standstills.

![Figure 15. Stand-up time of the tunnel face as a function of permeability (computation after Schuerch and Anagnostou 2012).](image)

The stand-up time of the tunnel face can be estimated by coupled hydraulic-mechanical numerical calculations that take account of the time-dependent processes in the ground ahead of the tunnel face (Höfle et al. 2008, Schuerch and Anagnostou 2012, 2013). Figure 15 shows the results of such an analysis for a tunnel section located in the so-called Red Sandstone formation. For permeability values less than $10^{-8}$ m/s, the stand-up time amounts to several days, which means that the conditions during TBM advance (including short standstills of about 0.5 - 1 day) are practically undrained. For higher permeabilities or longer standstills, however, unfavourable drained conditions must be taken into account. The computation of Figure 15 was carried out for a specific tunnel section, but the results are also typical for the other weak wocks of the tertiary formation prevailing along the major part of the Lake Mead tunnel. The results of Figure 15 in combination with the distribution of the permeability along the alignment (Fig. 13) indicate that conditions will be transient (i.e. between drained and undrained) over long portions of the tunnel: the stand-up time can be anything between a few hours and several days. This introduces an element of uncertainty concerning tunnel face stand-up time, with direct consequences for the operating mode. A stand-up time of several days would allow open mode TBM operation and maintenance under atmospheric
conditions. A stand-up time of a few hours might allow TBM advance in open mode or at low slurry pressure, but would probably necessitate hyperbaric interventions for maintenance. The difference between a few hours and several days is thus very significant from a construction point of view.

In order to alleviate the effects of the uncertainty concerning stand-up time in the sedimentary rocks, the TBM was operated either in low-pressure closed mode (a partial compensation of the hydrostatic pressure was in most cases sufficient) or in open mode in combination with advance drainage.

It should be noted that the drainage-induced pore pressure relief is significant (Fig. 16) even under the practical limitations that are imposed by the equipment with respect to the spacing and number of boreholes (Zingg and Anagnostou 2012, Anagnostou and Zingg 2013). As can be seen from Figure 17, pore pressure relief has direct consequences in terms of the slurry or compressed air pressure that is needed in order to ensure face stability. Advance drainage increases the feasibility range of open mode operation (bottom of Fig. 17). In the case of particularly poor quality ground, advance drainage – either alone (Fig. 17, BC) or in combination with grouting (Fig. 17, ABC) – allows reductions to be made in operational pressure, thus reducing the technical risks associated with operation of the TBM at very high support pressures over long portions of the tunnel.

Figure 16. Contour lines of the hydraulic head (dark blue: almost atmospheric pore pressure).

Figure 17. Slurry or compressed air pressure required for long-term stability as a function of the cohesion with or without advance drainage (computation after Zingg and Anagnostou 2012).
8.3 Experience in the field

The TBM started in December 2011, and after 18 months and a 950 m drive it reached the sedimentary rocks, in which it has so far tunnelled 3100 m (in a period of 15 months). The sedimentary rocks proved sufficiently stable at least in the short term, making it possible to operate the machine mostly in open mode, but always in combination with three 45 m long drillholes (overlapping by 10 m) through the cutterhead, which were used for advance probing and drainage. During longer standstills (greater than two days), 2 to 3 additional 10 m long drillholes were installed. Closed mode operation in the sedimentary rocks was necessary only in some sections with higher quantities of water inflow, because the excavated material resembled a flowing mud and could not be extracted by the belt conveyor.

Considerable difficulties were encountered, however, in the first part of the alignment through metamorphic rocks. Shortly after the beginning of the excavation of the starter tunnel (May 2010) an unexpected fault was encountered, which progressively entered the tunnel cross-section. With the advance of mining activities the rock at the face appeared more and more sheared and crushed and finally resembled a clay-like matrix with rock fragments. At the early phase (when the fault occupied only a small part of the tunnel cross-section) moderate water inflows and slow ravelling at the face were observed. The quantity of water inflow increased with the advance of the starter tunnel (Fig. 18). Finally, a major instability occurred at the end of June 2010 causing flooding of the tunnel and of the shaft.

In order to re-access the cavern at the bottom of the shaft, 2470 m$^3$ of concrete were pumped from the ground surface to fill the voids created by the instability. Moreover, an extensive investigation campaign (via sub-horizontal and vertical boreholes) was carried out to investigate the extent and geometry of the fault. The geological investigation showed that an unexpected normal fault consisting of almost cohesionless material was running sub-parallel to the tunnel alignment (Fig. 19).

The combination of poor ground conditions and extremely high water pressure required a realignment of the tunnel by rotating its axis eastwards by 23° (Fig. 19). A bigger rotation of the axis was impossible due to the constraints imposed by the TBM assembly. The change of the alignment made it possible to complete the excavation of the starter tunnel without further problems. The TBM drive started in closed mode with a low slurry pressure, which was needed only for mucking-out until a belt conveyor system was installed. The ground conditions, which were good during the learning phase, started to worsen after few hundred meters, as the TBM encountered branches of the above-mentioned normal fault intersected by the so-called “Detachment Fault” – another major sub-horizontal fault dipping in the direction of advance and having a core consisting of gravel. In order to stabilize the face it was necessary to operate the slurry shield in closed mode at 14 bar for several hundred metres. This is a remarkable achievement; it has never been done before anywhere in the world.

![Figure 18. Total quantity of water inflow measured in the TBM assembly cavern and starter tunnel (Brierley Associates, 2010).](image-url)
The biggest problem in the metamorphic rocks was the virtual impossibility of accessing the excavation chamber for maintenance under atmospheric pressure. In order to assess the feasibility of men entering the working chamber, the quantity of water inflow was estimated in advance (without the risk of an instability) by using the TBM as a large-scale constant-head permeameter (Nickerson et al. 2014). Starting from its initial value, the slurry pressure was lowered in steps of 0.5 bar. After each step the increase of water inflow was measured by observing the change in water outflow in the slurry line, while keeping the slurry level in the bubble chamber constant. The final value for water inflow was recorded after reaching stationary seepage flow conditions. After several steps (generally more than 10), the relationship between quantity of water inflow and slurry pressure could be determined and used for estimating (by extrapolation) the quantity of water-inflow under atmospheric conditions as well as the permeability (Fig. 20). During the water tests, observations were made of the force acting on the cutter head, the torque (by rotating the cutter head without advancing the TBM) and the colour of the drained water, in order to identify the possible onset of local instabilities and thus interrupt the test by increasing the slurry pressure immediately to its initial value.

Figure 19. Original and re-aligned starter tunnel (Brierley Associates, 2011).

Figure 20. Measured quantity of water inflow as a function of the slurry pressure at different locations (solid lines) and theoretical prediction for two values of the permeability coefficient $k$ (dashed lines).
Attempts to lower the slurry pressure from the *in situ* hydrostatic pressure (14 bar) to atmospheric pressure were often interrupted as the water inflows reached hundreds of cubic metres per hour even at relatively high slurry pressures. Conditions such as these leave two possible options for maintenance work: hyperbaric intervention or ground improvement by grouting. Although the necessary equipment and logistics for saturation diving were available on site, the decision was made to proceed with the second solution because of the higher chances of success (hyperbaric intervention has never been attempted in tunnelling at 14 bar, Fig. 14b). Three series of grouting campaigns were carried out successively. The pre-excavation grouting was executed in two stages (cement followed by micro-cement) and it succeeded in reducing water inflow to an extent which allowed maintenance work to be carried out at least on the slurry lines, which was a prerequisite for continuing excavation. However, due to the heterogeneity of ground and the limitations imposed by the cutterhead on the drilling layout, it was impossible to create a closed grouting body that would allow safe access in front of the cutterhead. Repair work at the cutterhead could therefore be performed only later, after reaching competent rock (Nickerson *et al.* 2014).

### 9 Closing remarks

The critical conditions for TBM tunnelling include an unstable face (*e.g.* in faults), squeezing ground, rocks with short stand-up time (*in the case of gripper TBMs*), high water pressures, karst cavities, blocky ground or a mixed face. TBMs generally achieve higher production rates than conventional tunnelling but respond more sensitively to deviations from ideal operational conditions (Fig. 21). Such deviations are more likely to occur in complex formations as they often involve highly variable ground including weak rocks. Geological complexity reduces the reliability of predictions as to lithological or structural characteristics, and the parameters or behaviour of the ground along the alignment. Depending on the construction method, this may (but will not necessarily) result in related uncertainties with respect to the construction process. Geological complexity is nature-made, but engineering complexity may also be partially man-made. In order to avoid subjectivity in the conception of complexity, complex formations might be defined pragmatically as: formations that are regarded even by experienced engineers as complex since they demand new technical solutions or design methods; formations that involve complex construction and decision making procedures; or formations that are fraught with construction time and cost uncertainties that persist even after all of the available technical options have been exhausted.

![Figure 21. Schematic construction progress for mechanized and conventional tunnelling (Kovári 1987).](image)
The uncertainty in complex formations sometimes concerns the rock mass parameters. In the literature, empirical methods have been proposed that allegedly allow sufficiently reliable parameter estimations to be made on the basis of index values. The empirical basis of the postulated relationships has never, however, been published in a way that would allow a critical examination to be carried out (Anagnostou and Pimentel 2012). Such methods are potentially dangerous as they provide a false sense of security in moments when an appreciation of the inherent uncertainties and necessary precautions in the design and in the contract are essential to the success of a project. In this respect it is also important to understand the nature of uncertainty in tunnelling. As the uncertainty arises from a lack of knowledge (rather than from stochastic causes) and the quantity of objective information is very small in most cases, statistical or probabilistic analyses can account for uncertainty only superficially and may distract attention from the actual problem. Qualitative systems with simple classification matrices are adequate for the evaluation of risks associated with the nature and the behaviour of the ground (cf., e.g., Ehrbar and Kellenberger 2003). Today, a wide range of instruments is available, along with methods of decision-making and risk management that make it possible to deal rationally with risks and to consider them adequately in construction contracts (cf. Anagnostou and Ehrbar 2013).

10 References
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