Muir Wood Lecture 2014

Some Critical Aspects Of Subaqueous Tunnelling

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Some Critical Aspects Of Subaqueous Tunnelling

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High potential for damage, relatively high pore pressures and limited pre-construction accessibility are all features of subaqueous tunnels. Potential hazards include high water inflows or even a complete flooding of the tunnel in the case of a connection opening up to the seabed. In subaqueous tunnels, very high pore pressures may occur at small depths of cover, i.e. often in combination with a low shear strength ground, resulting in particularly adverse effects in terms of stability and deformations of the opening. This lecture illustrates some of the geomechanical issues relating to subaqueous tunnels (face stability in fault zones, the limits of open mode TBM operation in weak sedimentary rocks and the effect of advance drainage in squeezing ground) with reference to five case studies – the Storebælt tunnel, the "Melen 7" Bosphorus tunnel, the Lake Mead Intake No 3 tunnel, the Zurich Cross Rail and the future Gibraltar Strait tunnel project.

1 >> Introduction

Limited pre-construction accessibility, high potential for damage and relatively high pore pressures are all features of subaqueous tunnels. In addition to the high cost of marine operations, the ship movements or sea currents that often prevail in straits (Fig. 1) may cause frequent interruptions to probing operations, thus making exploratory campaigns very demanding. Systematic and time-consuming advance probing during construction is often indispensable. The limited accessibility not only increases uncertainty in the planning phase, but also narrows the range of technical options for construction: offshore ground improvement works, for example, share the same difficulties as exploratory campaigns and are also extremely costly (Fig. 2).

Intermediate attacks (which might reduce the impact of geological uncertainty and, in the case of long tunnels, also construction time) are possible only in very exceptional cases and only for works of a limited nature (Fig. 3).

The high damage potential associated with subaqueous tunnelling arises from the possibility of high water inflows or even a complete flooding of the tunnel in the case of a hydraulic connection opening up to the seabed. Tragic events of this kind have occurred many times, as for example during the construction of Brunel's Thames tunnel, the first subaqueous tunnel in the world (Fig. 4).

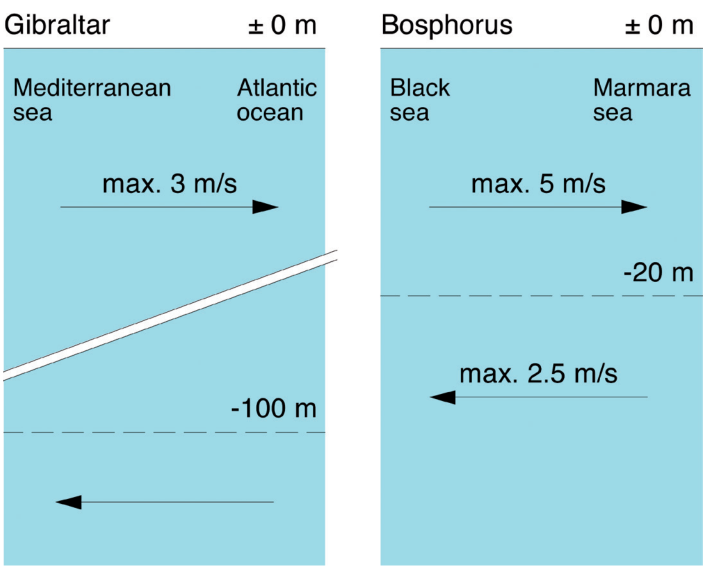


Figure 1 : Sea currents in the Gibraltar strait and in the Bosphorus strait

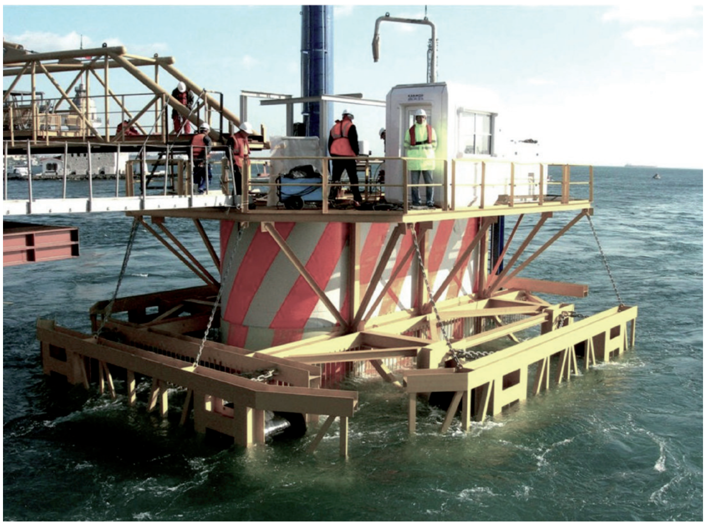


Figure 3 : Marmaray immersed tunnel project: An exceptional case of a shaft through water [2]

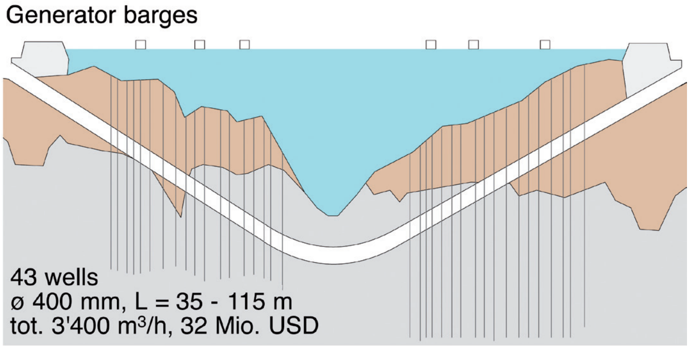


Figure 2 : Storebælt tunnel. Large scale pore water pressure relief below the seabed using pumping wells [1]

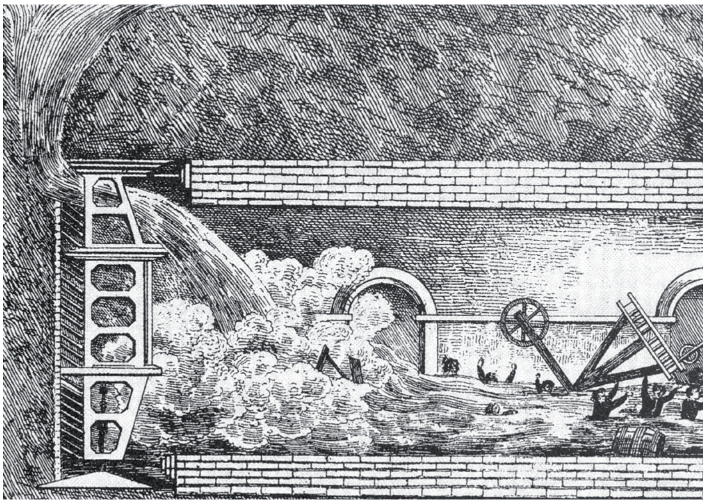


Figure 4 : Historic painting of the Thames tunnel flood of 1827

1 >> Introduction

The risk of flooding is of course not limited to subaqueous tunnels. It also exists in mountain tunnels where there are fault zones consisting of so-called "swimming ground" [3]. The characteristic of subaqueous tunnels, however, is that the potential volume of water recharge is practically unlimited.

High pore pressures are not specific to subaqueous tunnels either. In contrast to mountain tunnels, however, they may occur in combination with a small depth of cover. As the head difference between the water level and the tunnel has to be dissipated within a smaller distance, the pore pressure gradients and consequently the destabilizing seepage forces are higher (Fig. 5).

Furthermore, the small depth of cover means that the effective stress and the shear resistance of the ground may be low relative to the pore pressure. This effect is particularly pronounced in (but not limited to) underconsolidated marine deposits, i.e., soils that are still consolidating under the existing overlying sediment load and which contain an excess pore pressure that carries part of the overburden.

The present lecture addresses some of the geomechanical questions relating to subaqueous tunnels by means of selected projects, which, besides their obvious differences in terms of overburden and depth below the water table (Fig. 6), are also different with respect to the type and extent of the geotechnical challenges. We start with the issue of face support by earth pressure balance (EPB) shields under suboptimum ground conditions, referring to the example of the Storebælt railway tunnel (Section 2). This project provides an excellent example of the interplay between geotechnical, material-technological, mechanical- and process-engineering aspects in tunnelling. We next investigate the limits of open mode TBM operation by considering two hydraulic tunnels – the Bosphorus-"Melen 7" tunnel (Section 3) and the Lake Mead Intake No 3 tunnel (Section 4). In the first tunnel, which mostly crosses competent hard rocks, the potential hazard of face collapse is localized in distinct tectonic or volcanic structures (faults, dykes), while in the Lake Mead tunnel, where weak sedimentary rocks prevail over long portions of the alignment, this hazard is present across the entire construction process. Section 5 is about the conflicting criteria of settlement limitation and safety against blowout that had to be satisfied during the slurry shield construction of a part of the Zurich Cross Rail project due to the rare combination of subaqueous tunnelling and tunnelling underneath a building. The lecture closes with a future cutting-edge project, the Gibraltar Strait tunnel (Section 6). Heavily squeezing conditions (i.e. extremely large deformations of the ground) are expected over such a long portion of the alignment that they represent a key technical feasibility factor in this project.

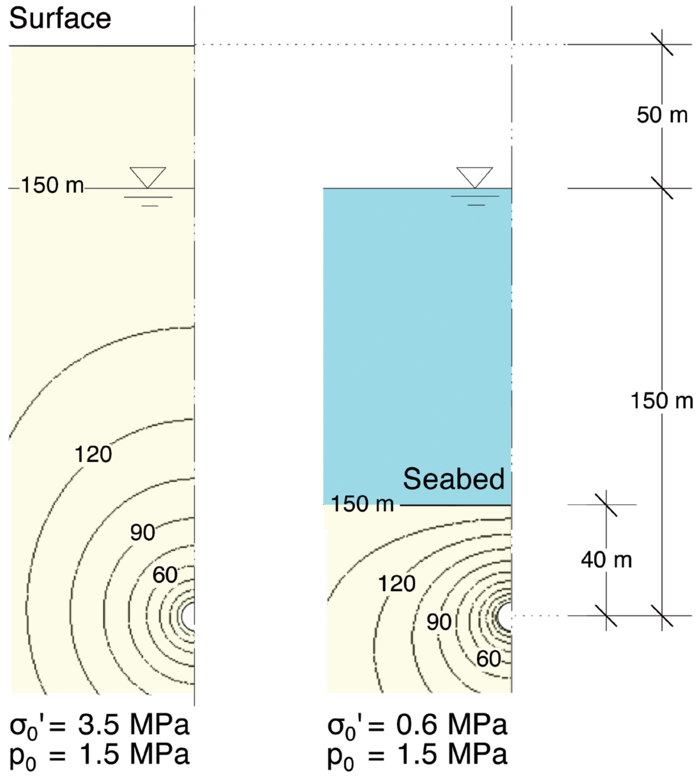


Figure 5 : Contour lines of the hydraulic head, initial effective stress σ_0' and initial pore water pressure p_0 for a deep tunnel (left hand side) and, for a shallow underwater tunnel (right hand side)

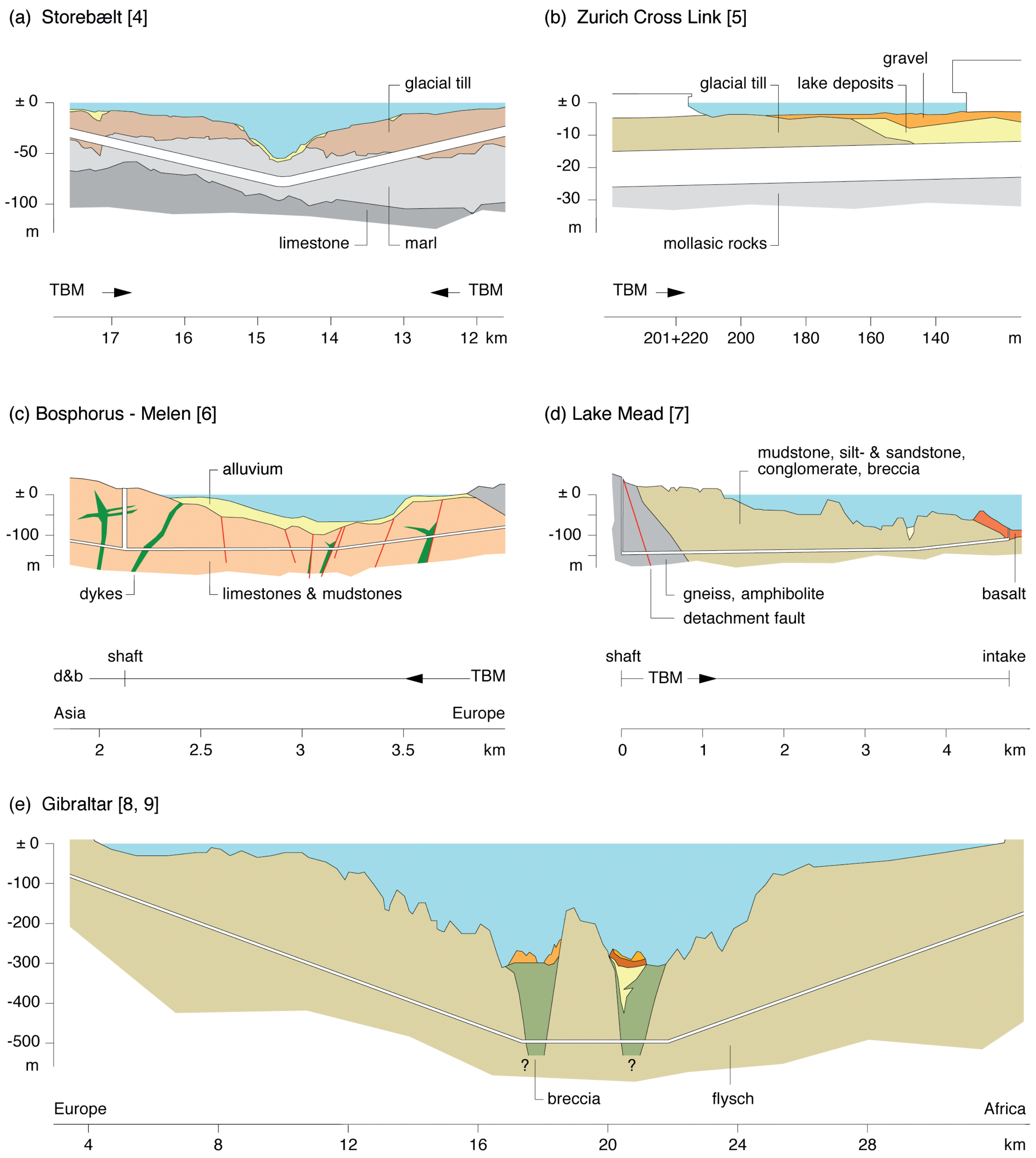


Figure 6 : Longitudinal profiles

2 >> Storebælt railway tunnel

2.1 PROJECT

The Storebælt fixed link project involves a twin bored railway tunnel 8 km long which connects the islands of Sprogö und Seeland in Denmark [4, 10, 11, 12]. The tunnel was constructed in 1990 - 1997 using four EPB shields (two starting from Sprogö and two starting from Seeland) of 8.75 m diameter. The ground in the project area consists of fissured marls and glacial tills, each accounting for about 50% of the alignment (Fig. 6a).

Tills are particularly unfavourable for tunnel construction: the till in question is an overconsolidated soil with up to 20% clay in the shallow part of the alignment and a very heterogeneous material in the deeper part (containing irregular sand lenses, gravels and glacial boulders and less than 10% clay). Typical parameter values for the till were a cohesion c' of 20 kPa, a friction angle φ' of 35° and a coefficient of permeability κ of 10^{-7} - 10^{-5} m/s [13].

Considering the low shear strength and the high permeability of the till as well as the high hydrostatic pressure (up to 5 bar), the main hazard scenario was a collapse of the tunnel heading with subsequent failure propagation up to the seabed. This became evident soon after the start of TBM excavation. During a weekend in October 1991, a face collapse occurred which created a hydraulic connection to the seabed. The quantity of water flowing in increased rapidly to 4 m^3/sec and led – through open bulkhead doors – to flooding of the tunnel (at that time 350 m long), the TBM launch pit in Sprogö and from there the parallel tunnel as well (Fig. 7).

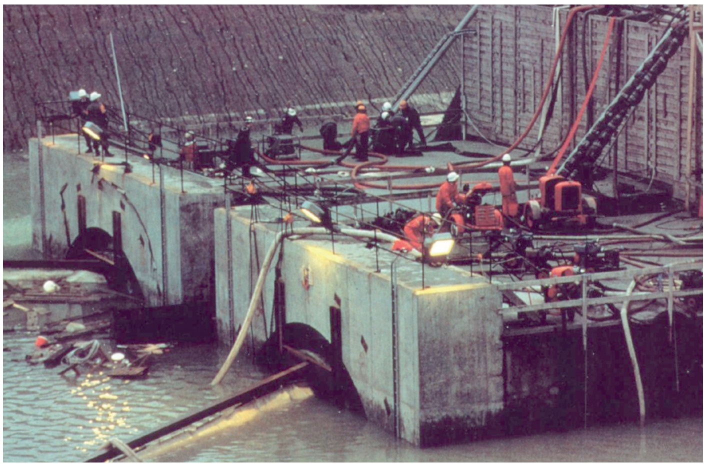


Figure 7 : Storebælt tunnel. Flooding of the Sprogö construction site [1]

The remedial work took about 8 months to complete. In addition to this spectacular incident there were several delays due to the high abrasiveness of the tills and the frequent occurrence of granitic boulders which necessitated time-consuming man entries into the working chamber. Furthermore, it was often impossible to stabilize the tunnel face by operating the EPB shield in closed mode. Towards the end of 1993, when the two machines coming from Seeland reached the deepest tunnel stretches in the till, these problems became so critical that the TBMs were no longer able to advance.

2.2 MECHANICS OF FACE STABILIZATION

The difficulties outlined above triggered thoroughgoing investigations into the mechanics of face stabilization by EPB shields under the specific non-ideal operational conditions that prevail in high permeability ground [14].

Under ideal operational conditions (characterized by a fine-grained, low-permeability soil under low hydrostatic pressure), the excavated muck can be seen – from a soil mechanics point of view – as a monophasic medium. The tunnel face is supported by the «*total pressure*» in the muck, *i.e.* the pressure that is monitored by the pressure trans-ducers in the cutterhead. A coarse-grained muck, however, is a bi-phasic material consisting of solid grains and pore water. In this case, a distinction must be made between the pore water pressure and the pressure in the solids (hereafter referred to as «*pore pressure*» and «*effective support pressure*», respectively). Although the sum of these two pressures is equal to the total pressure, they act in different ways: the solids exert a stabilizing stress *directly upon the face*, while the pore pressure is decisive in terms of the hydraulic head field in the ground *ahead* of the tunnel face. If the pore water pressure in the muck is equal to the *in situ* hydrostatic pressure (i.e. the hydraulic head h_F in the working chamber is equal to the sea level elevation h_g), then the ground is acted upon only by the force of gravity (represented schematically by the submerged unit weight y' in Fig. 8a).

In this case a low effective support pressure *s*' or a low cohesion *c*' are sufficient for face stability. For the typical shear strength parameters of glacial tills, limit equilibrium computations after [14] show that the necessary effective support pressure *s*' is equal to zero, *i.e.* compensation of the water pressure will suffice for stability.

If, on the other hand, the hydraulic head h_F in the chamber is lower than the sea level elevation, then water will flow towards the tunnel face, thus exerting seepage forces upon the ground. Figure 8 shows the contour-lines of the piezometric head under atmospheric pore water pressure in the working chamber. The seepage forces f_s are oriented perpendicular to the contour-lines. Their magnitude increases linearly with the head gradient and is, therefore, higher close to the tunnel face. The seepage forces are unfavourable for stability and necessitate a higher effective support pressure s' in order for the face to be stable. The lower the hydraulic head h_F in the working chamber, the higher the seepage forces and, consequently, the higher will be the necessary effective support pressure *s'*.

In conclusion, the ground response is controlled by the joint effect of pore pressure and effective support pressure. However, these two parameters cannot be controlled directly in practice. They depend on the characteristics of the excavated ground, the way the ground is mixed in the work chamber, the rotational speed of the screw conveyor and the excavation advance rate. So, the ground response to tunnelling by an EPB shield depends to a large degree on a complex interplay of geotechnical and operational factors.

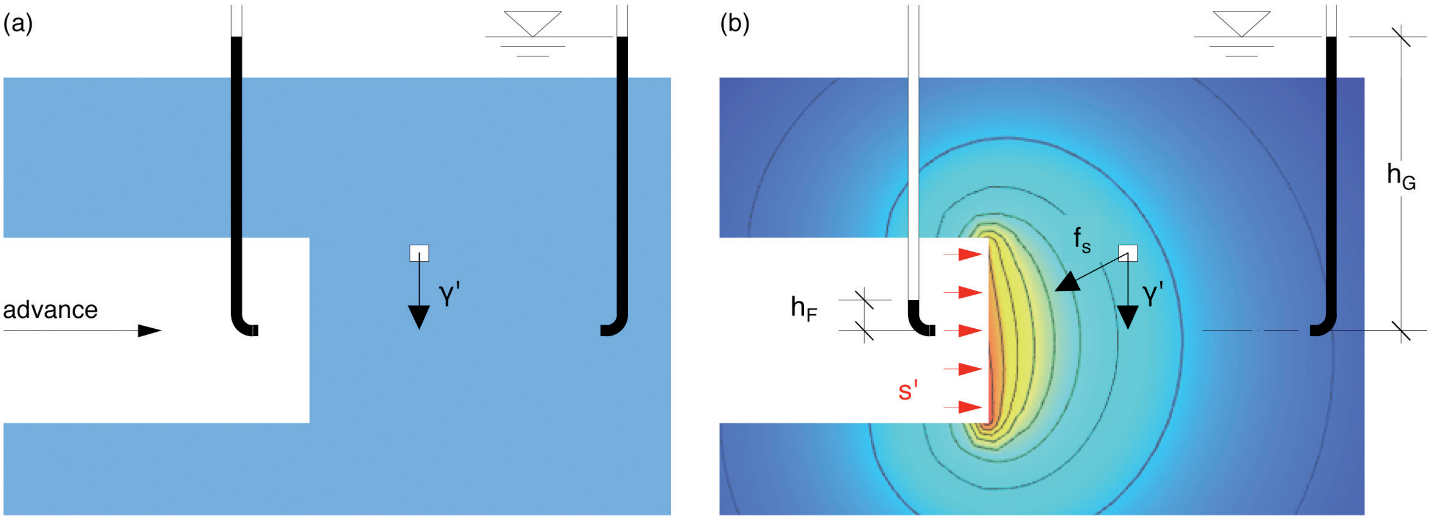


Figure 8 : Body forces (γ', f_s) acting upon the ground, necessary effective support pressure s' and contour lines of the hydraulic head h: (a) hydraulic head in the working chamber according to the sea level; (b) atmospheric pore water pressure in the working chamber

Let us consider now the conditions at chainage 12+700 of the Storebælt tunnel, where it proved impossible to maintain water pressure within the working chamber. The black line in Figure 9 shows the relationship between the necessary effective support pressure and the hydraulic head h_F in the excavated muck.

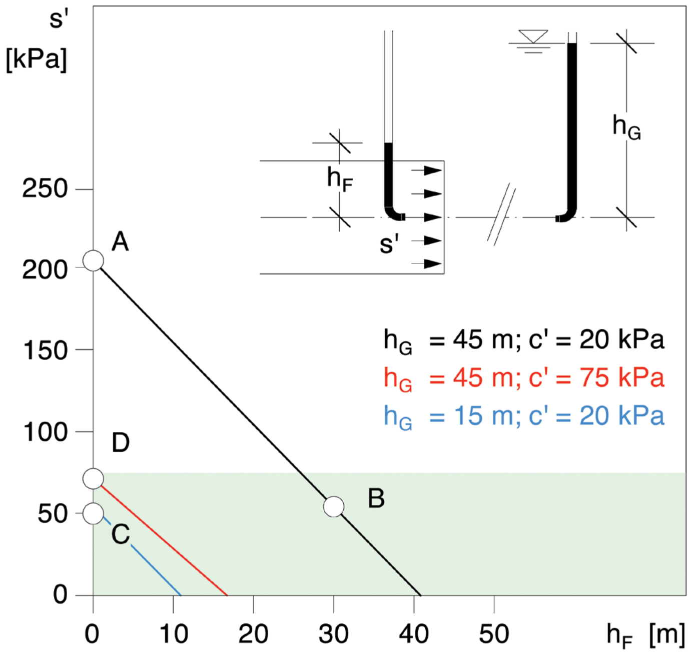


Figure 9 : Storebælt tunnel, ch. 12+700. Necessary effective support pressure s' as a function of the hydraulic head h_F in the working chamber (computation after [14])

If the pore water in the muck is atmospheric (point A), then the necessary effective support pressure s' amounts to about 200 kPa. Such a high effective stress increases the frictional shear resistance of the muck against the rotating cutterhead, thus leading to excessive torque demand, which explains why it was impossible to advance the TBMs. In addition, due to its high frictional resistance, the muck no longer behaves as a fluid, with the consequence that the distribution of the support pressure across the face may be highly irregular instead of hydrostatic. Furthermore, a high effective stress compacts the muck, thus increasing its stiffness, which, in combination with irregularities in the mass flows into and out of the working chamber, may cause pressure fluctuations over time. The large spatial or temporal variations in support pressure mean that the pressure may be locally or temporarily insufficient, which explains the difficulties observed in face stabilisation.

These operational and geotechnical problems can be avoided by keeping the effective pressure *s*' low. Figure 9 illustrates the basic options in this respect: pore pressure relief in the ground ahead of the face (blue line, Point C); ground improvement by grouting (red line, point D); high hydraulic head in the working chamber (point B).

As can be seen from the red line in Figure 9, which applies to the unfavourable combination of a high *in situ* hydraulic head (h_G = 45 m) with atmospheric pressure in the chamber (h_F = 0), even rather modest ground improvement (characterized by a cohesion increase from 20 to 75 kPa) leads to a considerable reduction in the necessary effective support pressure s' from 200 kPa to only about 70 kPa (point D).

Pore pressure relief by systematic drainage reduces the seepage forces and thus also the necessary effective support pressure considerably. The blue line in Figure 9 shows the relationship between the effective support pressure s' and the hydraulic head h_F in the chamber, assuming, for example, that drainage reduces the hydraulic head h_G in the ground from 45 m to 15 m. For the given shear strength of the till, the necessary support pressure would decrease to about 50 kPa (Fig. 9, point C). The effectiveness of drainage depends, however, on the permeability of the ground. As it was high in the present case, pore pressure relief was often insufficient.

For the given shear strength of the till and the given in situ hydrostatic head h_G (Fig. 9, black line), a low effective support pressure would be sufficient for stability only in combination with a sufficiently high hydraulic head h_F in the working chamber (e.g. moving from point A to point B of Fig. 9). Normally, the water pressure in the chamber can be kept high by creating a low-permeability muck that acts as a barrier to groundwater flow. The addition of bentonite slurry, polymers or foams to the muck can help here to some extent. In the case in point, conditioning attempts remained unsuccessful. In order to maintain a high water pressure in the chamber, it was necessary to apply back-pressure by installing a piston pump at the end of the screw conveyor. This made it possible for one machine to continue excavation in the glacial tills and to enter the underlying stable marls. The other machine was put out of operation; as the TBM coming from Sprogö made good progress in the meantime, the decision was taken to shift the joining point of the two drives to the location of the stuck TBM [12].

3.1 PROJECT

The “Melen 7” tunnel is located about 15 km north of central Istanbul. It is the first bored tunnel underneath Bosphorus and also the first bored tunnel in the world connecting two continents. The tunnel is a key element of Istanbul’s drinking water supply [18]. It has a bored diameter of 6.11 m and is 5550 m long. A 3400 m long part, which includes the section under the sea, was constructed using a shielded TBM (Fig. 6c). The TBM was designed to sustain a maximum hydrostatic pressure of 13.5 bar during standstills and to operate in EPB mode at pressures of up to 4 bar.

At the deepest section of the alignment, the TBM operated 135 m below the sea level and 70 m below the seabed. The bedrock in the project area consisted of alternating calcareous shales, clayey or sandy limestones and sandstones. Due to thick granular alluvial deposits, however, the minimum rock cover was as little as 35 m in the marine section of the tunnel. This, in combination with the existence of several faults as well as volcanic dykes in the project area, implied a higher probability of encountering very high water quantities or pressures in fault zones or in dykes communicating with the seabed. The thickness of the dykes ranges from a few meters up to 50 m and they consist of hard rocks with compressive strengths of up to 140 MPa, which may nevertheless be fractured and weathered. In the contact zones, the host sedimentary rock is often completely crushed or disintegrated into a fine-grained material. Due to this geological setting, together with previous experience of tunnelling in the region, where groundwater problems have been associated mostly with fault zones and/or dykes [6], hazards such as a face collapse with subsequent tunnel flooding were a major concern during the preparations for tunnelling.

The condition and behaviour of the ground in the faults depended mainly on the dominant lithology of the competent host rock. Fault zones in limestones with minor shale fractions appear blocky and brecciated (Fig. 10a), while in predominately shaly rocks the fault material is fine-grained (clayey or silty) and resembles soft ground (Fig. 10b). Potential problems in *fractured dykes* and *blocky fault zones* include high water inflows, which may cause difficulties in mucking-out and in the installation of the segmental lining and the annulus grouting, as well as rock instabilities in front of the TBM, which may block or damage the cutter head. The risk of high water inflows and the need for impermeabilization grouting can be assessed on the basis of the water quantities observed in exploratory boreholes systematically drilled ahead of the TBM during construction. Besides sealing the rock mass, grouting also improves its strength, thereby helping to deal with the stability problems mentioned above at the same time.

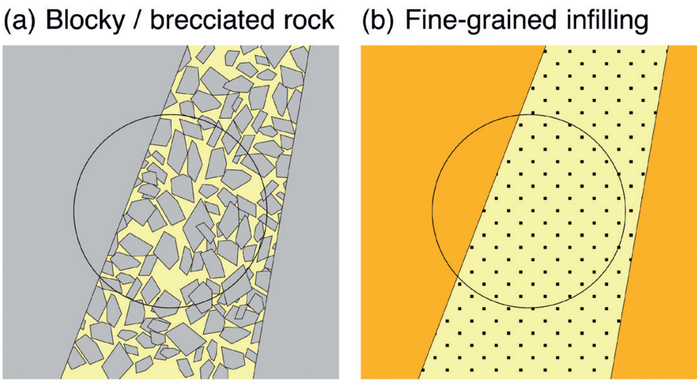


Figure 10 : Melen tunnel. Fault types

Faults with fine-grained infillings were likely to be encountered in the more shale-rich formations. The possible hazards arose from the combination of high pore pressures with the low strength and high deformability of the fault material and they included instabilities of the ground in front of the TBM, which might have blocked the cutter head, as well as excessive deformations of the ground, which can lead to shield jamming in the case of extended faults. It should be noted that such problems cannot be identified in advance on the basis of the water quantities in probe holes because the water inflows are very limited in fine-grained materials of low permeability and may give a false sense of security. Low stand-up times and face instabilities have been observed, for example, in the Vardoe and Ellingsoy subsea tunnels in spite of less than 10 and 30 l/min, respectively, water ingress in the probe holes [15].

3.2 STABILITY ASSESSMENT OF FAULTS WITH FINE-GRAINED INFILLINGS

In the following, we discuss the parameters that govern stability in faults with silty-clayey infillings on the simplified example of a vertical fault striking perpendicular to the tunnel axis (Fig. 11a).

Only undrained conditions will be considered, i.e. the conditions prevailing during continuous excavation or during short standstills. During longer standstills, stability conditions become increasingly less favourable. The assumption of short-term conditions is nevertheless reasonable given the limited extent of the fault zones. Should the geological pre-exploration show the presence of a fault, organizational measures (such as rescheduling machine maintenance) can be undertaken in order to allow continuous rapid excavation through the fault without interruptions.

The parameters governing short-term stability are: the undrained shear strength S_u of the ground, the depth of cover, the sea depth and the thickness d of the fault. Their effect can be estimated by means of limit equilibrium computations. The diagram in Figure 11 shows the necessary face support pressure s as a function of the fault thickness d in respect of three cross sections in the subsea section of the tunnel.

The following conclu-sions can be drawn from this diagram: For a given depth below the sea level, the neces-sary face support pressure depends sensitively on the depth of cover; the highest values correspond to the cross section A in the central portion of the alignment. Tunnelling ex-perience indicates indeed that the tunnel stretches with the highest water pressure and the smallest depth of cover are the most critical [15, 17]. Furthermore, the narrower the fault, the lower will be the necessary face support pressure. This is due to the stabilizing effect of the shear stress at the interface between the fault and the competent shale.

As can be seen from Figure 11b, face support is needed only if the fault thickness *d* is greater than the critical thickness d_{cr}. Figure 11c shows the critical fault thickness dcr as a function of the undrained *shear strength* S_u *of the ground for the three cross sections.* Faults narrower than 2 - 3 m do not present a stability problem even at the deepest portion of the alignment and under extremely low shear strength values. Depending on the strength of the fault material, thicker faults may necessitate face support (operation in closed mode), ground improvement by grouting or pore pressure relief in the ground ahead of the TBM by advance drainage. For shear strengths S_u higher than 50 - 60 kPa, the face would remain stable without any measures. In view of the difficulties of thorough advance grouting (due to the low permeability of fine-grained soils and the constraints imposed by the tunnelling equipment on the drillhole pattern), it is interesting to note that even a relatively small ground improvement suffices to stabilize the face.

The TBM drive started March 2008 and finished without particular problems in April 2009, about one month earlier than planned [18]. The evaluation outlined above assisted in defining the criteria for decision-making during construction with respect to advance probing, ground improvement and TBM operational mode. The measures foreseen were: rotary percussive drilling for routine advance probing and core drilling only for fault zones thicker than 2 m consisting of ground of uncertain characteristics (fractured rock or soil that was predominantly clayey or silty); stabilization grouting and/or closed mode TBM operation in the case of fault zones thicker than 2 m consisting of low to medium permeability soil (e.g. silty fault gouge, but not a practically impervious clay).

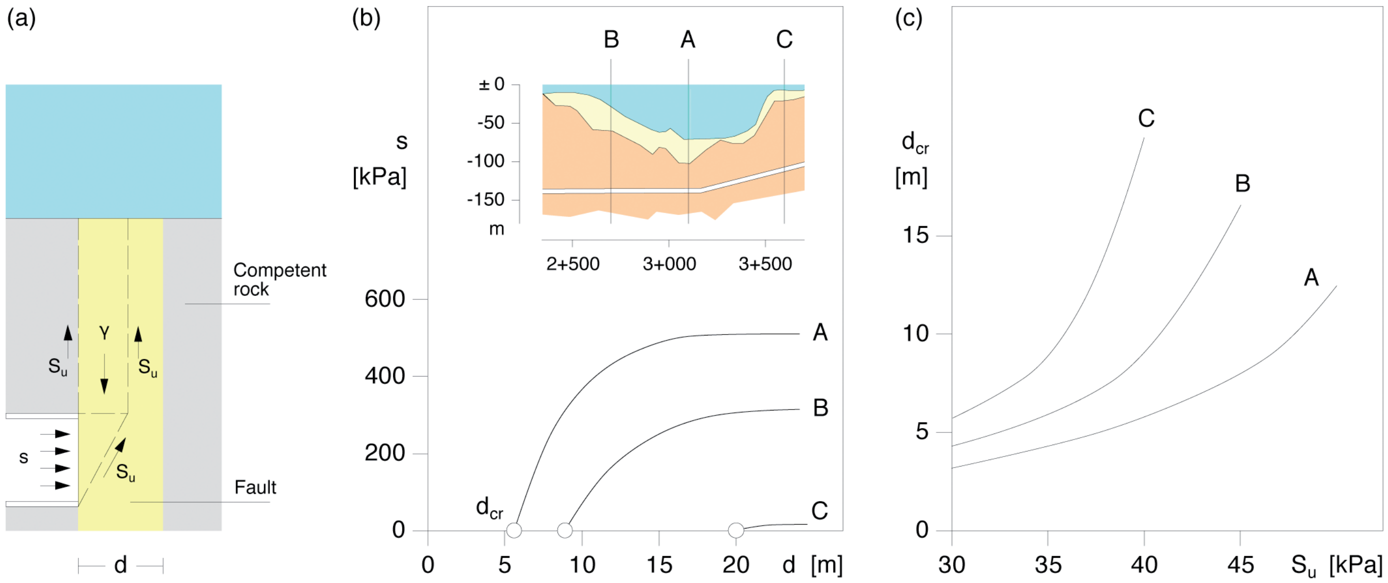


Figure 11 : Melen tunnel. (a) Problem layout; (b) Required support pressure s as a function of the thickness d of the fault (undrained shear strength S_u = 40 kPa); (c) Critical fault thickness d_{cr} for an unsupported tunnel face as a function of undrained shear strength S_u (computation after [16])

4 >> Lake Mead Intake No 3 Tunnel

4.1 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 [7]. 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 tunnel crosses metamorphic rocks 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. The rock cover decreases in the last portion of the alignment, amounting locally to just 20 - 30 m (Fig. 6d).

Due to the existence of several faults in the project area, the ground at the elevation of the tunnel may be recharged directly from Lake Mead, which implies the possibility of considerable water ingress during construction. Given the high hydrostatic pressures and the poor quality of the prevailing sedimentary rocks over long portions of the alignment, 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 [7, 19]. 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. 12). 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 operation and the lack of experience with hyperbaric interventions at 14 bar in tunnelling made it necessary to develop fallback strategies involving open mode operation, potentially in combination with advance grouting and/or drainage.

4.2 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 initially stable 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 which 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).

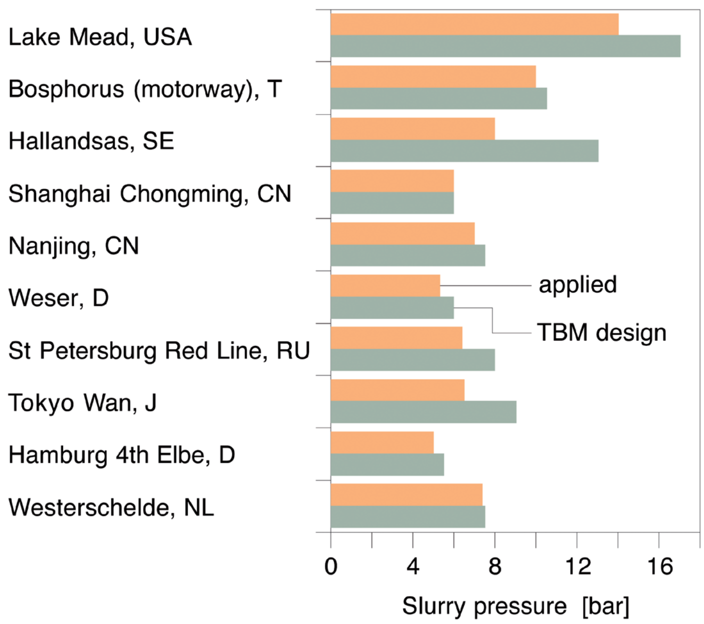


Figure 12 : Maximum slurry pressures in selected tunnel projects with high hydrostatic pressure (after [24], revised)

In the present case, it was expected that an unsupported face would be stable under undrained conditions (i.e. in the short term), but unstable under drained conditions (i.e. in the long term) for long stretches through tertiary rocks [20]. The central questions were thus: how long would the face remain stable? how certain was it that favourable undrained conditions would prevail during ongoing 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.

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 [21, 22, 23]. Figure 13 shows the results of such an analysis. 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. For the expected range of permeability, the stand-up time can be anything between a few hours and several days. 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 the construction point of view.

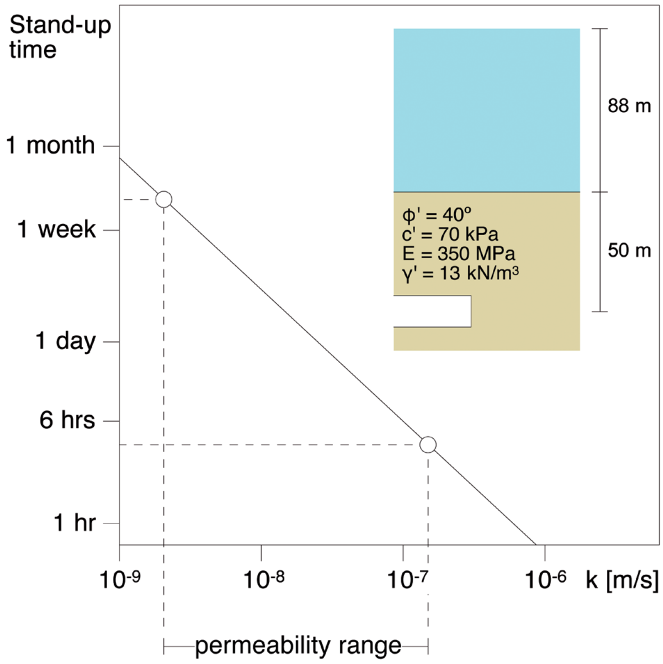


Figure 13 : Lake Mead Intake No 3. Stand-up time of the tunnel face as a function of permeability (computation after [22])

4.3 CONTROL OF THE FACE

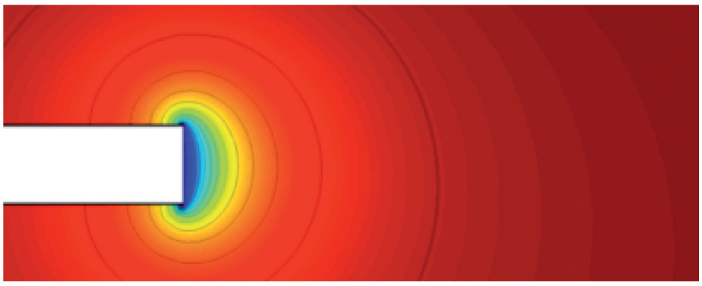
The computation of Figure 13 was carried out for a specific tunnel section (in the so-called Red Sandstone formation), but the results are also typical for the other tertiary rocks that prevail in the major part of the Lake Mead tunnel. They indicate that long portions of the alignment are in the geotechnically demanding transition zone between drained and undrained conditions. In this zone it cannot be said with certainty that favourable short-term conditions apply. This introduces an element of uncertainty concerning tunnel face stand-up time, with direct consequences for the operating mode. Drained conditions in combination with low shear strength of the ground would necessitate either closed mode operation or auxiliary measures such as advance drainage or grouting of the ground ahead of the tunnel face.

Advance drainage decreases the pore pressures and thus also the destabilizing seepage forces acting within the ground towards the face. Drainage-induced pore pressure relief is significant even under the practical limitations imposed by the construction equipment with respect to the spacing and number of boreholes [25, 26] (Fig. 14).

As can be seen from Figure 15, 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. 15).

In the case of particularly poor quality ground, advance drainage – either alone (Fig. 15, BC) or in combination with grouting (Fig. 15, ABC) – allows reductions to be made in operational pressure.

without advance drainage



advance drainage by three boreholes

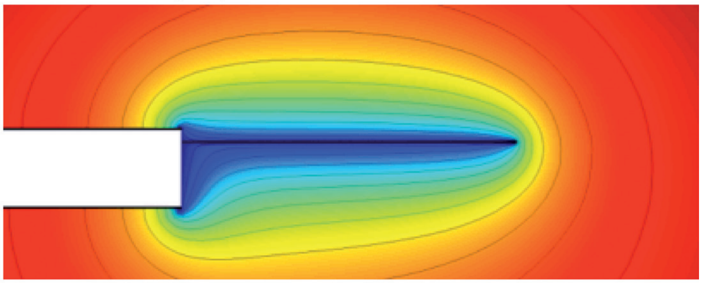


Figure 14 : Lake Mead Intake No 3. Contour lines of the hydraulic head (dark blue: almost atmospheric pore pressure)

4.4 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 1800 m (in a period of 10 months). The sedimentary rocks proved to be sufficiently stable at least in short term, making it possible to operate the machine practically always in open mode, always in combination with three 45 m long drillholes (overlapping by 10 m) through the cutter-head, which were used for advance probing and drainage. During longer standstills (more than two days), 2 to 3 additional 10 m long drillholes were installed.

Considerable difficulties were encountered, however, in the first part of the alignment through metamorphic rocks. These were caused by the unfavourable combination of high water pressure, extremely high rock permeability and the presence of an unexpected fault zone. The fault, consisting of almost cohesionless material, was oriented subparallel to the tunnel and therefore affected the works for a considerable portion of the alignment. These conditions made it 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.

However, the biggest problem 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 by using the TBM as a large-scale constant-head permeameter, i.e. by reducing the slurry pressure stepwise and monitoring the water inflow using a pre-defined procedure [27]. Attempts to lower the slurry pressure from the *in situ* hydrostatic pressure (14 bar) to atmospheric pressure were often interrupted because the water inflows reached hundreds of cubic metres per hour even at relatively high slurry pressures (Fig. 16).

Conditions such as these leave two possible options for carrying out 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). A series of pre-excavation grouting campaigns succeeded in reducing water inflow to an extent which allowed maintenance work to be carried-out at least on the slurry lines, which was indispensable for continuing excavation. Work could be performed at the cutterhead only later, after reaching competent rock [27].

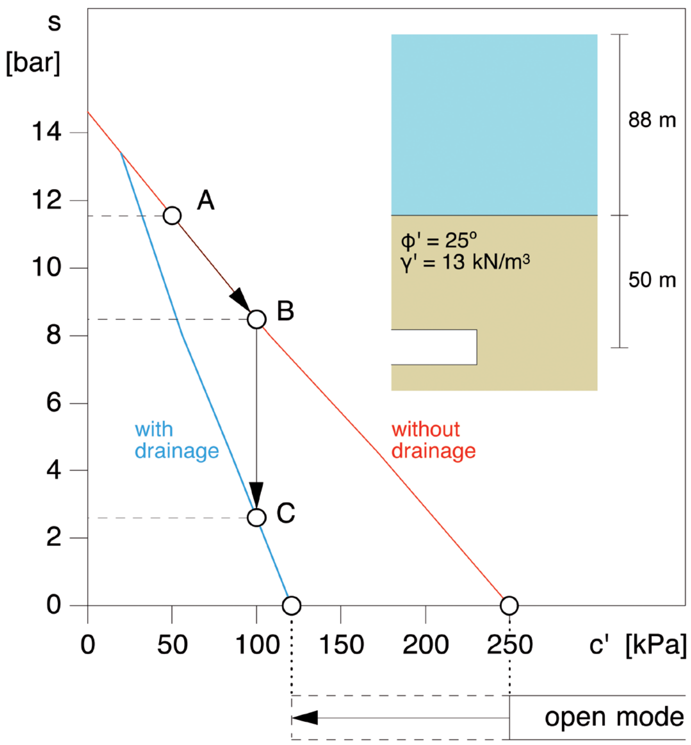


Figure 15 : Lake Mead Intake No 3. Slurry or compressed air pressure s required for long-term stability as a function of the effective cohesion c' with and without advance drainage (computation after [25])

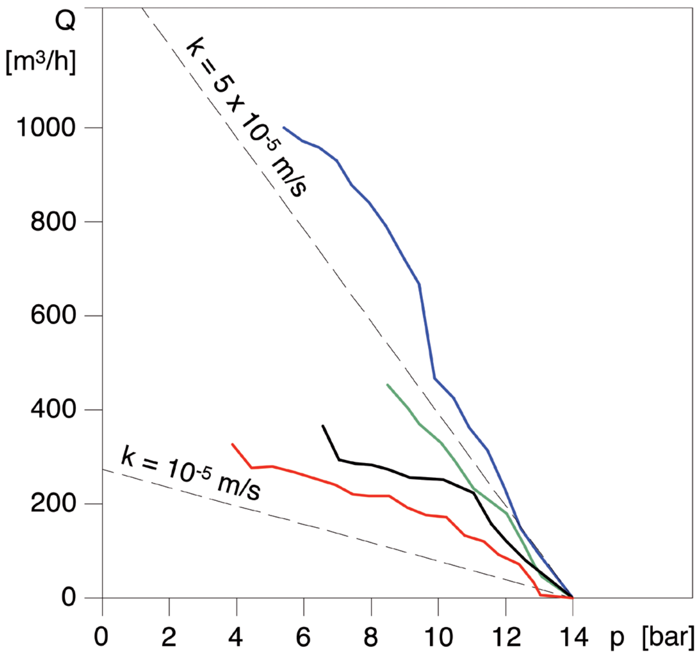


Figure 16 : Lake Mead Intake No 3. Measured quantity of water inflow as a function of the slurry pressure at different locations (solid lines) and theoretical prediction for two values of the permeability coefficient k (dashed lines)

5 >> Zurich Cross Rail

5.1 PROJECT

The Zurich Cross Rail project includes a 4500 m bored double-track tunnel of 11.3 m diameter [5]. The last 280 m of the tunnel crosses a glacial till overlain by lake deposits and fluvial gravels. An 86 m long part of the soft ground section runs at a minimum depth of about 9 m below bed of the Limmat river and 13 m below river level (Fig. 6b).

The tunnel was constructed using a shielded TBM, which was operated in the soft ground section in closed mode as a slurry shield. In order to reduce the impact of tunnelling on surface structures, the last 140 m were excavated under the protection of a large diameter pipe arch (Fig. 17a).

In the following we will focus on an interesting peculiarity of this tunnel that arose from the rare combination of subaqueous tunnelling and tunnelling underneath a building on the west bank of the river (Fig. 17c).

5.2 SUBAQUEOUS TUNNELLING UNDERNEATH A BUILDING

In the soft ground section, face stability was ensured through a pressurized bentonite slurry that compensated the hydrostatic pressure and exerted an excess pressure on the face (Fig. 17b).

Due to the small depth below the riverbed, the slurry pressure acting upon the crown involved the risk of a blowout. During TBM advance under the river, the slurry pressure required at the crown was 20% lower than the critical blowout pressure and thus not problematic. More critical in this respect were the hyperbaric interventions needed for inspection and maintenance of the cutter head, because of the inherently higher excess air pressure at the crown. The risk of a blowout proved to be manageable, however, by a partial lowering of the slurry level (up to the tunnel axis). The conditions under the river were demanding but nevertheless rather typical and well known for this type of work.

An interesting situation arose under the west bank of the river at a specific phase in the construction where the cutterhead was located underneath a commercial building, while a part of the shield was still underneath the riverbed and with reduced cover due to existing bridge abutments. Here, the uplift resistance of the overlying ground to the slurry pressure was slightly lower than before due to the smaller depth of cover above the rear part of the shield. At the same time, in order to limit the settlement of the building, the TBM had to be operated at a higher slurry pressure than before (Fig. 17c). In addition, there was a possibility that the TBM might encounter obstacles in the form of old sheet pile walls. In the worst case this would mean work at the cutterhead having to be carried out under slurry support by divers due to the conflicting criteria of settlement limitation and safety against blowout. Tunnelling under such conditions resembles a tightrope walk.

The pressure required for settlement control is higher than that required for stability alone, but, due to the presence of the pipe arch, probably considerably lower than the theoretical maximum pressure, i.e. the pressure corresponding to the condition at rest. The experience from the TBM drive confirmed this expectation, thus underlining the importance of the pipe arch. Non-stop operation, precise slurry pressure control, continuous real-time settlement monitoring and the excellent performance of the TBM during the advance through the sheet piles were all essential to the successful completion of the work.

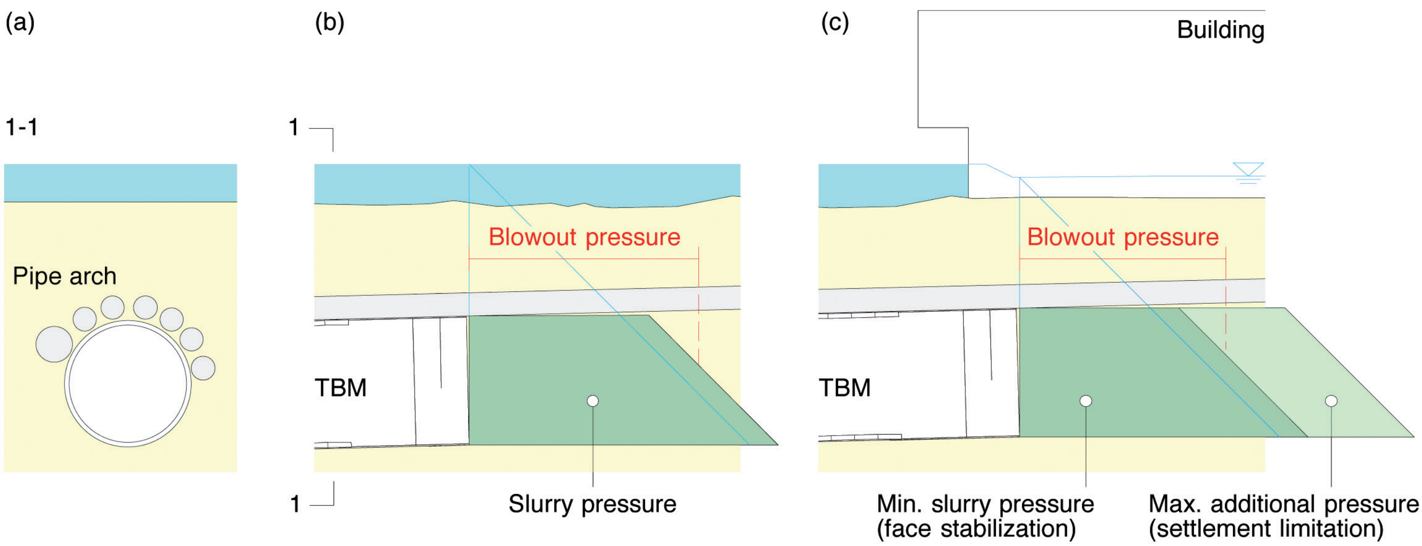


Figure 17 : Zurich Cross Rail. Tunnel cross section (a), slurry pressure and critical blowout pressure underneath the river (b) and at the river bank (c)

6 >> Gibraltar strait tunnel

6.1 PROJECT

The project under study for the Gibraltar Strait crossing foresees a bored tunnel approximately 38 km long, of which about 28 km will be under the sea [8, 9]. The largest part of the tunnel is located in Flysch. At the central section of the alignment, where the sea is 300 m deep and the overburden amounts to approximately 200 m, the tunnel will cross two so-called paleochannels filled with clayey breccias of extremely poor quality (Figure 6e). The very low strength and high deformability of the breccias in combination with the 50 bar pore pressures prevailing at tunnel elevation will cause squeezing conditions, i.e. large deformations of the opening and development of high loads upon the tunnel lining. This, in combination with the probable length of the critical stretch, its great depth and the distance from the European and the African coast (all of which limit the construction options) makes crossing the breccias a key technical challenge of the project [28, 29].

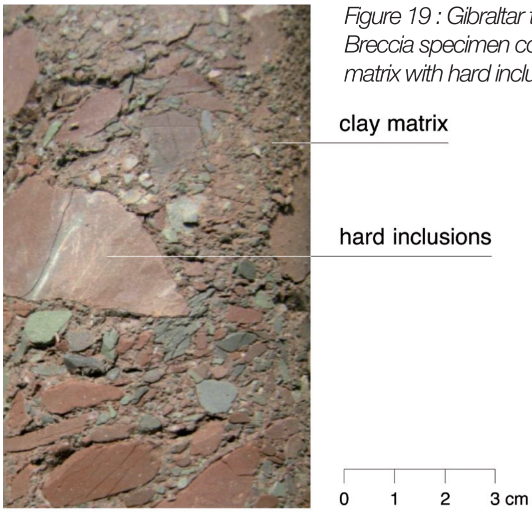


Figure 19 : Gibraltar tunnel. Breccia specimen consisting of a clay matrix with hard inclusions

6.2 THE BRECCIA SECTION

Due to the great sea depth, the periodically strong sea currents and the heavy ship traffic through the strait, even the geological exploration of the breccias is difficult. Sea currents and ship traffic lead to frequent interruptions in the probe drilling. Special devices were developed to allow re-entries of equipment and restarts of probing (Fig. 18a).

In addition, the direction of the sea currents is different in the deeper areas of the strait (Fig. 1), which, in combination with the great sea depth, may cause twisting of the drill rods. In order to reduce hydrodynamic resistance and avoid overstressing of the rods, aerofoil type fairings were developed (Fig. 18b).

The geotechnical testing of the breccias is also particularly demanding. Due to the prominent role of pore water pressure, the mechanical behaviour of the breccias is investigated by means of consolidated drained (CD) and consolidated undrained triaxial (CU) tests [32]. The difficulties of testing are associated with the structure and permeability of the material and the high in situ pore pressures and effective stresses. As a consequence of the low permeability of the clayey matrix of the breccias (measured $k = 10^{-13}$ - 10^{-11} m/s), triaxial testing takes long time (4 - 8 weeks, sometimes more) even

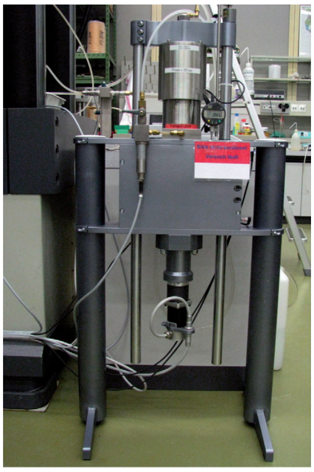


Figure 20 : Back-pressure device with step-motor for triaxial testing of extremely low-permeability materials

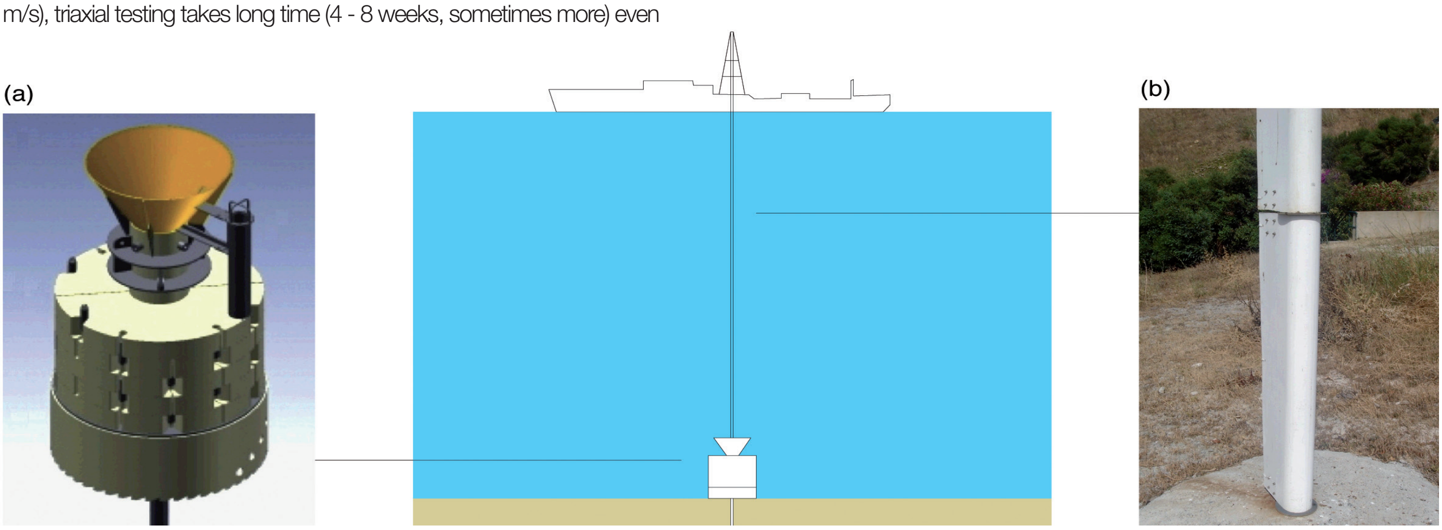


Figure 18 : Gibraltar tunnel. (a) Template for re-entry probe borehole; (b) hydrodynamically favourable fairing [30, 31]

when using small specimens. The presence of hard, gravel-size inclusions within a very soft clayey matrix (Fig. 19) in combination with low permeability (which makes the testing of large samples impossible) also renders the selection of representative specimens difficult.

Due to the hard inclusions, specimen preparation is very demanding. Satisfactory results have been achieved by cutting the samples with air-flushing through the use of a special electronically controlled diamond band saw.

Due to the high *in situ* pore pressures and effective stresses, testing equipment with high nominal loads must be used. (It is almost a matter of soil-mechanics testing under conditions which are more typically associated with rock mechanics problems). On the other hand, in spite of the high nominal loads, the strain rate has to be finely tuned within an extremely low range due to the low permeability of the material. An electromechanical high precision step-motor was developed specifically for controlling the pore pres-sure (Fig. 20).

According to the results of the CD and CU triaxial tests, the breccias exhibit cohesion values of only a few 100s of kPa, friction angles in the order of 15 - 20° and Young's moduli of a few 100s of MPa [32]. Figure 21 gives an impression of the deformability of the material.

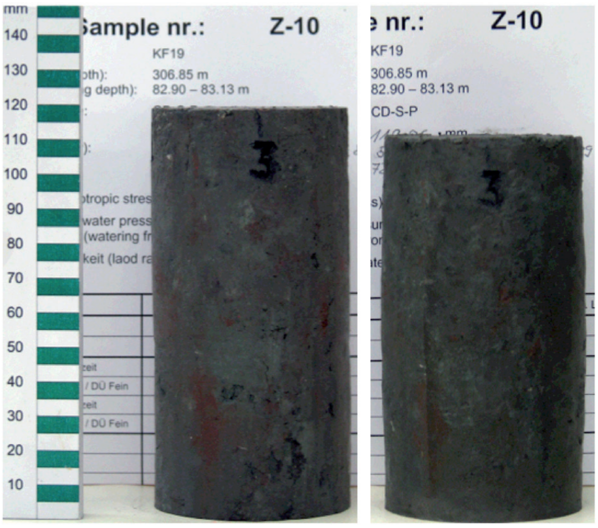


Figure 21 : Gibraltar tunnel. Breccia specimen before and after CD-testing [33]

Figure 22 : Shield jamming due to squeezing rock [36]

In the following, the practical significance of such a poor quality ground for tunnelling will be illustrated by means of computational results concerning the short-term behaviour of the ground, i.e. the behaviour under constant water content. The short-term response of the ground to excavation, i.e. its behaviour close to the tunnel face, is important from the constructional point of view. Depending on the intensity of squeezing, it may be necessary to apply large amounts of support close to the working face in order to control the ground. This slows down tunnel advance considerably, as support installation interferes with the excavation work. In mechanized tunnelling, which is an important construction option in the present case due to the great tunnel length, squeezing may result in the complete immobilization of the tunnel boring machine [34, 35] (Fig. 22).

The black solid line in Figure 23 shows the relationship between the radial displacement of the excavation boundary and the pressure supporting the ground under the simplifying assumption of rotational symmetry ("*ground response curve*"). For the given parameter set, which applies to the conditions prevailing in the breccias in the deepest section of the alignment, the maximum convergence of an unsupported opening is equal to 50% of the tunnel radius. Even with a heavy support pressure of 1 - 2 MPa, the estimated convergence reaches 20 - 30% of the tunnel radius. It should be noted that the ground response curve was obtained by means of a large strain analysis; neglecting the geometric non-linearity arising in the case of such large deformations would lead to a convergence of more than 100% of the tunnel radius (Fig. 23, dashed line), which is physically meaningless. Tunnelling experience shows that drainage not only improves the stability of the tunnel heading (Section 3), but is also favourable with respect to squeezing, because the consolidation of the ground increases its resistance to shearing [37, 40], thus leading to a dramatic reduction in the deformation of the opening (Fig. 23, blue line). Advance drainage is in fact an important auxiliary measure for the breccia section of the Gibraltar tunnel [41, 42]. In view of the very low permeability of the breccias, complete pore pressure relief by advance drainage may take a prohibitively long time. The effect of drainage is, nevertheless, remarkable even with partial pore pressure relief (Fig. 23, red line).

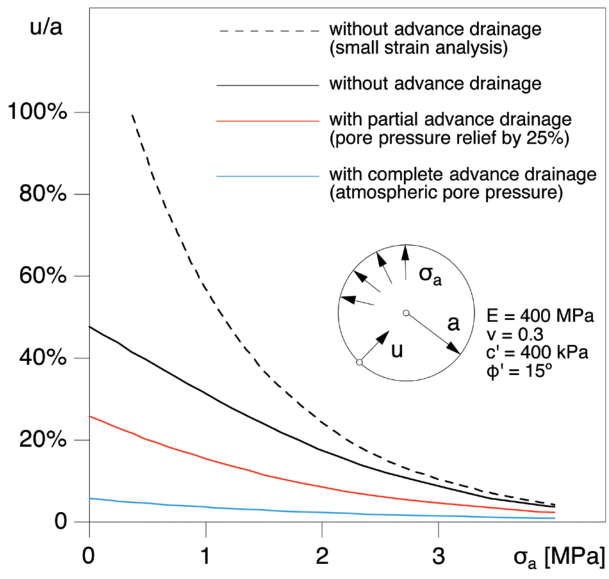


Figure 23 : Gibraltar tunnel. Short-term ground response curves with and without advance-drainage (computation after [37, 38, 39])

7 >> Closing Remarks / 8 >> References

Subaqueous tunnels are demanding projects and, as with every demanding project, a motor for technological progress. They are also a motor for advancing the science of tunnelling, which benefits the entire industry. Thus, for example, the Storebælt project was instrumental in improving our understanding of the mechanics of face stabilization by EPB shields [14]. Or, to take another example, the Seikan and Gibraltar tunnels triggered important research on grouting and drainage [43, 44] with results that have been very beneficial in the planning phase of the base tunnels through the Alps.

Demanding projects also increase the attractiveness of the civil engineering profession and provide a powerful motivation for our students. Recent large-scale infrastructure projects that gained public attention have been influential in increasing civil engineering student numbers in Switzerland. Despite the wide range of project conditions and construction methods in the case studies discussed, there is a unity in the underlying physical principles. The success of a project often depends on our respect for these principles and our understanding of what lies behind phenomena such as instability or large deformations. It is important to pass this message on to students and to enable them to push back the boundaries of knowledge even into areas where no experience is yet available to draw on – as they respond to the challenge of increasingly difficult tasks and the rapid development of technology.

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