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Tunnel stability and deformations in water-bearing ground

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ABSTRACT: Water can affect the stability and the deformations of a tunnel by reducing the effective stress and thus the resistance to shearing, and by generating seepage forces towards the excavation boundaries. The seepage-flow may lead to a draw-down of the water-level and to time-dependent subsidence due to consolidation. Furthermore, when tunnelling in soft ground, the seepage forces acting towards the opening may impair its stability. The movement of water in a low-permeability ground is one major cause of time-dependent effects in tunnelling. This paper discusses the effects of water by means of examples covering a wide range of tunnelling conditions. Emphasis is placed on practical questions of tunnel engineering, on the mechanisms governing the stability and deformation of underground openings in water-bearing ground and on the significance of poromechanical coupling.

1 INTRODUCTION

The effects of groundwater in tunnelling are manifold. During tunnel excavation in a water-bearing ground, seepage flow towards the opening takes place, because the pressure at the excavation boundary is, in general, atmospheric and the tunnel acts therefore as a groundwater drain. The seepage-flow may lead to a draw-down of the water-level, to a decrease in the discharge of wells, or to consolidation and subsidence. Besides these - in the broader sense - environmental impacts, large water inflows may impede excavation works or have a serious impact on the serviceability of the tunnel during its operation phase.

The present paper focuses on the mechanical action of water: Water can affect both the stability and the deformations of a tunnel by reducing the effective stress and thus resistance to shearing, by generating seepage forces towards the excavation boundary; and by washing out fine particles from the ground. Transient seepage flow in a low-permeability ground is one major cause of time-dependent effects in tunnelling. Furthermore, when tunnelling in weak ground, the seepage forces acting towards the opening may impair its stability. The interactions between seepage flow and equilibrium, porewater pressure and stress field around a tunnel constitute perhaps the most important coupled process in geotechnical engineering.

Figure 1. (a) Face instability; (b) Failure of an open shell; (c) Subsidence; (d) Overstressing of a grouting body in a fault zone; (e) Squeezing pressure or deformation; (f) Floor heave in swelling ground.
In the next pages the effects of water on the stability and deformations of underground openings will be discussed by means of examples covering a wide spectrum of tunnelling conditions. Rather than delving into the details of mathematical modelling, emphasis will be placed on practical questions of tunnelling through water-bearing ground, on the mechanisms governing the observed phenomena of stability and deformation and on the significance of poromechanical coupling. The discussion will start with the case of shallow tunnels through weak ground, and will continue with the crossing of fault zones consisting of so-called “swimming” ground, finishing with tunnelling through squeezing or swelling rock (Fig. 1). What unifies all of these cases are the underlying mechanical principles which can be traced back to the fundamental works of Terzaghi and Biot.

2 SHALLOW TUNNELS IN WEAK GROUND

2.1 Introduction

The most serious risks in tunnelling through weak ground are associated with a collapse of the tunnel face. In shallow tunnels the instability may propagate towards the surface creating thereby a chimney and a crater on the ground surface (Fig. 1a). The face failure results then in excessive subsidence and damage to overlying structures. Depending on the construction method, i.e. on the sequence of the excavation and temporary support works, various other collapse mechanisms need also to be considered in conventional tunnelling. For example, when tunnelling by the “top-heading and bench” method, a collapse up to the surface may occur also as a consequence of an insufficient bearing capacity of the ground beneath the footings of the temporary support arch (Fig. 1b). In general, the tunnel portion close to the heading is particularly demanding, as the application of support in this area interferes with the excavation works. Besides the stability of the opening, the control of surface settlement is essential in urban tunneling (Fig. 1c).

The stability and deformations of tunnels in water-bearing ground depend greatly on the permeability of the ground. Tunnel excavation in a low permeability ground does not alter the water content around the opening on the short-term. Instead, excess pore water pressures develop. These dissipate over the course of time leading thus to consolidation and additional deformations of the ground. From the standpoint of stability two stages can be distinguished: the short-term stage, corresponding to undrained shear, and the long-term stage, corresponding to drained shear. In a high permeability ground the water content adjusts itself immediately to the stresses prevailing after excavation.

2.2 Stability

Stability issues are usually investigated by limit equilibrium analyses. As deformations are not taken into account in such analyses, the ground may be idealised as a plastic material obeying the Mohr-Coulomb failure condition either with the effective shear strength parameters \((c', \phi'\)) or with the undrained shear strength \(C_u\). As in other problems involving the unloading of the ground (i.e., a reduction in the first invariant of the total stress), undrained conditions are more favourable for the stability of underground openings. For common advance rates (up to 20 m/d), drained conditions are to be expected when the permeability is higher than \(10^{-7} - 10^{-6}\) m/s (Anagnostou 1995b).

When analysing the stability of the tunnel face, a simple collapse mechanism can be considered (Fig. 2a) consisting of a wedge and a prism which extends from the tunnel crown to the surface (Davis et al. 1980, Anagnostou & Kovári 1996). With the exception of closed-shield tunnelling, the piezometric head at the tunnel face is lower than that prevailing in the undisturbed ground. Consequently, water seeps towards the face, thereby generating seepage forces which have a destabilising effect and must be taken into account in a drained analysis. The seepage forces are equal to the gradient of the hydraulic head field. The computation of the seepage forces therefore calls for a three-dimensional steady-state seepage-flow analysis (Fig. 2b). Thus, a drained face stability analysis proceeds in three steps: (i) determination of the three-dimensional hydraulic head-field by means of a finite element computation; (ii) integration of the seepage forces acting upon the components of the specific collapse mechanism \((S, \text{Fig. 2c})\); (iii) solution of the limit equilibrium equations.

Fig. 2d shows the support pressure required in order to stabilise the face as a function of the safety factor (defined in terms of the shear strength constants). The upper line applies to the long-term conditions prevailing when tunneling under a constant water table (steady state piezometric head field as in Fig. 2c). The face support requirement decreases considerably if groundwater drainage is carried-out prior to tunneling (lower line). In low-permeability ground, the face would remain stable even without support (lowest line). The diagram shows the influence of groundwater conditions and of time on face stability.

As another example, consider a partial excavation with an invert closure in a distance \(L\) from the face (Fig. 3a). The overall stability of the top heading depends on the loads acting upon the temporary support shell and its bearing capacity, as
well as the bearing capacity of the footings. At the limit state, the load $V$ from the overburden and the forces $W$ at the shell footings can be calculated by means of the silo theory (Janssen, 1895) and by the common foundation bearing capacity equations, respectively. Fig. 3b shows the safety factor of the system as a function of $L$. The diagram illustrates a well-known fact from tunneling practice: in weak ground, rapid advance and closing the ring near to the face improve stability conditions considerably. According to Fig. 3c, which shows the critical length $L$ as a function of the shear strength $C_u$, not even short-term stability can be assured in adverse conditions and a closed support rind must be provided practically immediately. Note that in the range of low strength values ($C_u < 150$ kPa), relatively small variations of strength (of, e.g., ±25%), which may take place within short distances during tunnel excavation in a heterogeneous ground, affect the critical length $L$ considerably from the constructional point of view (ring closure within one versus four diameters).

Figure 2. (a) Collapse mechanism at the tunnel face (Horn 1961); (b) Numerical model for seepage flow analysis; (c) Hydraulic head field ahead of the tunnel face; (d) Results of a limit equilibrium stability analysis (geometry as in Fig. 2c) based upon the method of Anagnostou & Kovári (1996).

Figure 3. (a) Failure mechanism; (b) Safety factor as a function of top heading length $L$; (c) Maximum length $L$ as a function of undrained shear strength $C_u$. 
1.3 Deformations

Although limit equilibrium analyses are sufficient for the investigation of stability questions, coupled stress- and seepage-analyses provide useful insights into the mechanics of failure. With such analyses both short- and long-term behaviour can be studied consistently, based upon the effective strength parameters, i.e. without additional assumptions concerning the undrained strength.

Fig. 4a shows the crown settlement $u$ of a shallow tunnel as a function of the support resistance $p$ (normalized by the initial stress). The condition of constant water content applies for the short-term deformations, while the long-term deformations have been obtained by taking into account the steady state hydraulic head (Fig. 4c). In the short-term the opening remains stable even without support, while in the long-term a minimum support must be provided for stability. When the support pressure approaches a critical value $p_{cr}$, the ground fails up to the surface (Fig. 4c) and the settlement becomes asymptotically infinite (Fig. 4a). The seepage forces reach approximately 20 kN/m$^3$ at the tunnel floor (i.e., twice the submerged unit weight of the ground) indicating thereby the risk of piping in the case of an open invert.

Coupled analyses are indispensable for investigating questions of surface settlement. Fig. 5b and 5c show the settlement troughs as well as the steady state hydraulic head fields around a shallow tunnel for the case of a constant or a depressed water level, respectively. In agreement with field observations (O’Reilly et al. 1991), the settlement trough deepens and widens with time while the angular distortion remains approximately constant.

3 FAULT ZONES

3.1 Introduction

Tunnel sections in soil-like materials that are subject to high water pressures present a considerable challenge to tunneling operations. In the past such ground was described as "swimming", which aptly emphasizes the importance of the water. The width of such fault zones may vary from a few meters to decametres. In some cases they are accompanied laterally by a heavily jointed and fractured rock zone, in other cases the transition to competent rock is very distinct. When such a zone is suddenly encountered water and loose material flows into the opening. Often one speaks therefore of a "mud inrush", which in extreme cases can completely inundate long stretches of tunnel.

To overcome fault zones involving soil under high water pressures the ground is drained and strengthened ahead of the working face (Fig. 6). Experience show that in the case of small tunnel profiles in dense ground or in ground exhibiting some cohesion, drainage alone is often sufficient to enable excavation.
The ground can be strengthened and sealed either by grouting or by artificial freezing. Ground freezing however only offers a temporary solution. In deep tunnels, in general a permanent strengthening and sealing is required, which can only be obtained by grouting. By injecting a fluid into the ground, which then hardens, its strength, stiffness and imperviousness are increased. The aim is usually to obtain a cylindrical grouted body by carrying out the grouting works in a controlled way.

1.2 Effect of seepage flow

The effect of water on the stability of grouting bodies can be explained by considering the simple case of a circular tunnel (of radius $a$) in a homogeneous and isotropic initial stress field (total stress $\sigma_0$, Fig. 7a). The initial water pressure at the elevation of the tunnel is assumed to be uniform with the value $p_0$. After tunnel excavation the grouted zone takes the form of a thick-walled cylinder whose outer surface (at radius $r=b$) is loaded by the surrounding untreated ground and the water.

The ground (both the grouted and the untreated) is considered to be a porous medium according to the principle of effective stresses. Elastoplastic material behaviour with Coulomb's failure criterion is assumed. Seepage effects are taken into account based upon Darcy's law. The permeability of the grouted body is $k$, that of the untreated ground $k_0$. The excavation boundary $(r=a)$ represents a seepage face, i.e. the water pressure $p$ at that point takes on the atmospheric value. Under these assumptions the system fulfils the condition of rotational symmetry, and closed-form solutions can be derived (Anagnostou & Kovári 2003).

Due to the filling of the pores, the permeability of the grouted zone is very low compared to that of the untreated ground. In the borderline case of a very stiff and low-permeability grouting body, the tunnel excavation does not have any effects on the stresses and porewater pressures in the surrounding untreated ground. The conditions at the extrados of the grouting cylinder are given, therefore, by the initial values of effective stress $\sigma'_0$ and porewater pressure $p_0$. The hydraulic head difference between the untreated ground and the excavation boundary is dissipated entirely within the grouting body.
The porewater pressure distribution within the grouted body (Fig. 7a) is:

\[ p = p_o \frac{\ln(r/a)}{\ln(b/a)} \]  

(1)

The grouting cylinder is loaded by the ground pressure \( \sigma_o' \) acting at \( r = b \) and by the seepage forces associated with the porewater pressure gradient \( dp/dr \). Depending on the magnitude of these parameters and on the strength of the grouted zone, yielding may occur. The extent of the plastic zone is given by its radius \( \rho \) (Fig. 7a).

The effect of the seepage forces can be demonstrated best by studying the equilibrium inside the plastic zone. The equilibrium condition is

\[ \frac{d\sigma_r'}{dr} = \frac{\sigma_t' - \sigma_r'}{r} - \frac{dp}{dr} \]  

(2)

where \( \sigma_r' \) and \( \sigma_t' \) denote the effective radial and tangential stress, respectively. The stress field within the plastic zone fulfils, furthermore, the yield condition

\[ \sigma_t' = \frac{1 + \sin \phi}{1 - \sin \phi} \sigma_r' + f_c \]  

(3)

where \( f_c \) and \( \phi \) denote the uniaxial compressive strength and the friction angle of the grouted body, respectively. From Equations (1) to (3) we obtain:

\[ r \frac{d\sigma_r'}{dr} = \frac{2\sin \phi}{1 - \sin \phi} \sigma_r' + f_c - \frac{p_o}{\ln(b/a)} \]  

(4)

From the above we note that the effect of pore water pressure is apparently equivalent to a reduction of the uniaxial compressive strength \( f_c \) by \( p_o/\ln(b/a) \) (cf. Egger et al. 1982). In the example of Fig. 7a, the seepage forces cause an apparent strength reduction from 3 to 0.9 MPa!

An extensive plastification, such as the one in Fig. 7a, is in general unacceptable as it may lead to a loosening of the grouted zone and carries the risk of an uncontrollable material and water inflow (inner erosion). The plastification of the grouted body can be limited by different measures, such as applying a tunnel lining of higher resistance or by producing a grouted body of larger diameter or higher strength. In view of the considerable effect of seepage discussed above, it is obvious that another very efficient measure is the systematic drainage of the grouting body.

Note that drainage does not reduce the total load acting upon the grouted body. Due to the seepage flow, which takes place within an extended region surrounding the grouted zone, the untreated ground consolidates towards the grouted zone and the effective radial stress on the external boundary of the grouted body increases by the same amount as that by which the water pressure decreases there. The
4 SQUEEZING ROCK

4.1 Introduction

When driving through zones of cohesive materials of low strength and high deformability the tunneling engineer is faced with problems of a completely different kind. It is almost as if the rock can be moulded, which is why it was formerly spoken of a "mass of dough". If suitable support measures are not implemented, large long-term rock deformations will occur, which can lead even to a complete closure of the tunnel cross section. The rock exerts a gradually increasing pressure on the temporary lining, which can lead to its destruction. In such cases one can speak of "genuine rock pressure" and the ground is characterized as "squeezing". The basic aspects of tunneling in zones of squeezing rock have been presented in a concise form by Kovári (1998). Typical examples of rocks prone to squeezing are phyllites, schists, serpentinites, claystones, certain types of Flysch and decomposed clay and micaceous rocks.

Experience shows that high pore water pressure promotes the development of squeezing. This is confirmed by the observation that considerably smaller deformations take place when the rock mass is drained in advance.

The effects of porewater pressure on the mechanical behaviour of squeezing rock have been investigated by a comprehensive laboratory testing program, which was carried out at ETH Zurich during the design and exploration phase of the 57 km long Gotthard highspeed railway tunnel. Heavily squeezing ground was expected particularly in a "mass of dough". If suitable support measures are not implemented, large long-term rock deformations will occur, which can lead even to a complete closure of the tunnel cross section. The rock exerts a gradually increasing pressure on the temporary lining, which can lead to its destruction. In such cases one can speak of "genuine rock pressure" and the ground is characterized as "squeezing". The basic aspects of tunneling in zones of squeezing rock have been presented in a concise form by Kovári (1998). Typical examples of rocks prone to squeezing are phyllites, schists, serpentinites, claystones, certain types of Flysch and decomposed clay and micaceous rocks.

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The effects of porewater pressure on the mechanical behaviour of squeezing rock have been investigated by a comprehensive laboratory testing program, which was carried out at ETH Zurich during the design and exploration phase of the 57 km long Gotthard highspeed railway tunnel. Heavily squeezing ground was expected particularly in the so-called "northern Tavetsch massif" (TZM-N). During mountain formation this zone was subjected to intensive tectonic action resulting in alternating layers of intact and weak kakiritic gneisses, slates, and phyllites. "Kakirite" denotes an intensively sheared rock, which has lost a large part of its original strength. On account of a depth of cover of 800 m, of an initial pore water pressure of 80 bar and of the expected rock properties, the approximately 1100 m long TZM-N was a major challenge to the project. The laboratory tests revealed that control of porewater pressure during triaxial testing is indispensable. Specimen pre-saturation and maintenance of a sufficient back pressure are essential for obtaining reliable and reproducible parameters (Vogelhuber et al. 2004). Conventional triaxial tests are inadequate, as they may lead to a serious under- or over-estimation of the strength parameters. Consolidated drained (CD) and consolidated undrained (CU) tests provided, despite the complex structure of the kakiritic phyllite, remarkably uniform results. The distribution of the strength parameters obtained by such tests was very small in comparison with conventional triaxial testing data.

Squeezing normally develops slowly, although cases have also been known where rapid deformations occur very close to the working face. The development of rock pressure or deformation may take place over a period of days, weeks or months and can be traced back generally to three mechanisms:

(a) The three-dimensional redistribution of stress in the region around the working face. This mechanism cannot explain long-term rock deformations, because it occurs within two or three tunnel diameters from the working face.

(b) The rheological properties of the ground. So called "creep" is especially evident if the rock is highly stressed as the failure state approaches, which indicates, therefore, that it plays an important role in squeezing.

(c) The excavation of a tunnel in saturated rock triggers a transient seepage flow process, in the course of which both the pore water pressures and the effective stresses change over time. The latter leads to rock deformations. Thus we are faced here with a coupled process of seepage flow and rock deformation. The more impermeable the rock, the slower will process be, i.e. the more pronounced is the time-dependency of the rock deformation or pressure.

These three mechanisms are in general superimposed. Here attention will be paid exclusively to the processes associated with the development and dissipation of pore water pressures. The effects of porewater pressure will be discussed on the basis of computational results obtained with simple models.

A rotationally symmetric system in plane strain will be considered (see inset in Fig. 9). At the excavation boundary, an atmospheric porewater pressure \( p_a = 0 \) and a radial stress \( \sigma_r \) corresponding to the lining resistance apply. At the far field boundary, stress and porewater pressure are fixed to their initial values. The rock mass is modelled as a saturated porous medium according to the principle of effective stresses. Seepage flow is taken into account using Darcy's law. The mechanical behaviour
is described by an elastic, perfectly plastic material model obeying Coulomb's failure criterion.

1.2 Short- and long-term behaviour

We first consider the two limiting states of transient seepage flow: the state at \( t = 0^+ \) ("short-term behaviour") and the state at \( t = \infty \) ("long-term behaviour"). The first is characterised by the condition of a constant water content, while the second is governed by the steady state porewater pressure distribution. The short-term volumetric strains are zero, while the long-term deformations are associated with a volume increase caused by rock dilatancy. The short-term behaviour is therefore more favourable than the long-term behaviour.

This can be seen clearly by comparing the respective ground response curves (Fig. 9), which describe the interdependence of radial displacement \( u_a \) at the excavation boundary and lining resistance \( \sigma_a \).

The short-term radial displacement \( u_a \) stabilises at approximately 0.36 m even in the case of an unsupported opening (lining resistance \( \sigma_a = 0 \)). Long-term, however, the tunnel would close (upper curve).

If one assumes that about 30 - 40% of the short-term deformation occurs before the installation of the lining (ahead of the excavation or in the immediate vicinity of the working face, cf. Panet 1995) and that a temporary lining can withstand a radial displacement of 0.20 m without experiencing damage, then it has to be dimensioned for a rock pressure of approximately 0.30 MPa (Fig. 9, Point A). Long-term however the rock pressure would increase to the much higher value of 1.5 MPa (Point B).

1.3 Time-development of squeezing

To deal with the problems encountered in tunnel sections exhibiting squeezing rock conditions, basically two structural concepts exist (Kovári 1998): the "resistance principle" and the "yielding principle".

In the former a practically rigid lining is adopted, which is dimensioned for the expected rock pressure. In the case of high rock pressures this solution is not feasible. The yielding principle is based upon the fact that rock pressure decreases with increasing deformation. By applying a flexible lining, the rock pressure is reduced to a manageable value. An adequate overprofile and suitable structural detailing of the temporary lining will allow for non-damaging rock deformations, thereby maintaining the desired clearance from the minimum line of excavation.

Interesting results concerning the efficiency of these two structural concepts can be obtained by investigating:

(a) the time-development of the rock pressure under a fixed radial displacement \( u_a \) (the "resistance principle" case);

(b) the time-development of the displacement \( u_a \) for a constant lining resistance \( \sigma_a \) (the "yielding principle" case).

Fig. 10a shows the results of two transient analyses carried out in respect of these two cases. The ordinates \( \sigma_a \) and \( u_a \) refer to the "resistance principle" (left axis) and to the "yielding principle" (right axis), respectively. For the sake of simplicity, it was assumed that the lining was installed after the occurrence of the short-term deformations. Thus the curves have their origin at \( \sigma_a = 0 \) and \( u_a = 0 \), respectively.

According to Fig. 10a, the radial displacement \( u_a \) of a flexible lining develops at a much slower rate than the rock pressure \( \sigma_a \) acting upon a rigid lining. The development of rock deformations in the first case needs more time, because it is associated with volumetric strains and, consequently, presupposes the seepage of a larger quantity of water. The rock pressure \( \sigma_a \) acting upon a stiff lining (the resistance principle) increases within a month to the considerable value of 0.5 MPa, while the corresponding deformation of the flexible lining at 0.03 m is still negligible. This result is interesting from the practical point of view, as it shows that with a yielding support one gains time.
1.4 Effect of permeability

The duration of consolidation is governed by the permeability of the ground. Increase permeability by a factor of ten and the rock pressure and the rock deformation will develop ten times faster. The great importance of permeability soon becomes clear when one plots the rock pressure and the rock deformation at a particular time $t$ in the function of permeability (Fig. 10b).

Permeability coefficients less than $10^{-9}$ m/sec are characteristic for practically impermeable rock. Since the determination of permeability in this range is subject to large uncertainties, reliable predictions of the time development of rock pressure or deformation are extremely difficult.

Additional prediction uncertainties exist in the case of heterogeneous rock formations. From soil mechanics it is known that the duration of consolidation is proportional to the square of the length of the drainage paths. Thin permeable layers embedded in a practically impermeable rock mass lead to a shortening of the drainage paths (Fig. 11) causing thus a substantial acceleration of the squeezing process.

For a permeability $k = 10^{-11}$ to $10^{-9}$ m/s, the rock pressure would increase to 0.1 - 1.2 MPa within 15 days (Fig. 10b). For a rock pressure of 0.1 MPa a light temporary lining is sufficient, while a heavy support is required in order to sustain a load of 1.2 MPa. The difference between the two cases is important from the tunneling standpoint. As a consequence of some randomly distributed permeable interlayers, the tunneling engineer may experience the rock mass in one case as "competent", and in the other as "disturbed".

Fig. 10b shows, nevertheless, that with a flexible lining the consequences of prediction uncertainties are alleviated. For a permeability $k = 10^{-11}$ to $10^{-9}$ m/s, the radial displacement $u_a$ of a flexible lining amounts to 0 - 0.25 m. The practical consequences of a poor estimate of convergence are modest compared to the consequences of a wrong estimate of the time development of rock pressure acting upon a rigid support according to the resistance principle (damage to the lining and the need for re-profiling).

Figure 10. (a) The time development of the rock pressure (resistance principle) and of rock deformation (yielding principle) compared in the same diagram. Permeability $k = 10^{-10}$ m/s, other parameters as in Fig. 9. (b) The rock pressure (resistance principle) and the rock deformation (yielding principle) after 15 days, as a function of permeability. Other parameters as in Fig. 9.

Figure 11. Shortening of the drainage path caused by permeable interlayers.
5 SWELLING ROCK

5.1 Introduction

Rocks containing certain clay minerals and in some cases anhydrite swell, i.e. increase in volume when they come into contact with water. The swelling is due to water adsorption by the flaky structure of the clay minerals (so-called osmotic swelling) and, in the case of sulphatic rocks, also due to the gypsumification of anhydrite.

Geological formations with swelling rocks (Jurassic claystones, tertiary marlstones or Gypsum-Keuper) are widespread in France, Switzerland and Southern Germany.

In tunnelling, the swelling manifests itself as a heave of the tunnel floor (Fig. 12a). When the heave is constrained by an invert arch, a pressure develops (Fig. 12b), which may damage the lining. If the depth of cover is small, also a heave of the entire tunnel tube may occur (Fig. 12c). Swelling develops usually over several decades thereby seriously impairing the long-term serviceability and stability of underground structures (Kovári et al. 1988, Amstad & Kovári 2001).

5.2 Significance of hydraulic-mechanical coupling

One interesting feature of the swelling phenomenon is that the deformations occur only in the tunnel floor. The walls and the crown remain stable over many years.

The first continuum-mechanical models were based upon the hypothesis that the swelling is caused by the stress redistribution resulting from the tunnel excavation. Accordingly, swelling was considered as a stress-analysis problem. Since significant differences between floor and crown do not exist in respect of the geometry or the initial and boundary conditions, stress-analyses predict swelling not only in the tunnel floor, but also in the crown, independently of the specific constitutive model. This contradicts the facts and leads to unsafe predictions concerning the resulting stresses in the lining (overestimated axial forces, underestimated bending moments).

Realistic predictions of the observed deformation pattern are possible only when taking into account the seepage flow and the hydraulic-mechanical coupling (Anagnostou 1995a). In a coupled analysis, the displacement field depends on the porewater pressures prevailing around the tunnel and, consequently, on the hydraulic boundary conditions. The latter are different for the tunnel crown than they are for the floor. In several tunnels, free water can be observed on the floor, while the crown and walls appear to be dry.

5.3 Sulphatic rocks

Besides clay minerals, some swelling rocks also often contain anhydrite (CaSO$_4$). In a closed system, i.e. a system without mass exchange with its surroundings, anhydrite dissolves in the water and gypsum begins to precipitate. Since the volume of gypsum crystals is about 61% bigger than that of anhydrite, the swelling of sulphatic rocks has often been attributed to the hydration of anhydrite.

In an open system, as in situ, a great variety of processes are possible. Depending on the water circulation, and on the reaction kinetics, either leaching of rock or hydration may occur. In the first case solid matter (dissolved anhydrite) is transported away by the water. In the second case the solid matter volume increases by about 61%. Provided that porosity remains constant, hydration causes a volumetric strain whose magnitude depends on the volume fraction of the original anhydrite. It is also possible, however, that the formation of gypsum causes a gradual stopping up of the pores, and thus a sealing of the rock. In this case, a smaller volumetric strain would occur.

So, the properties of the CaSO$_4$ - H$_2$O system do not allow for a definite statement concerning the contribution of anhydrite to swelling. They show, nevertheless, that the seepage flow conditions must be decisive. This hypothesis is supported by field observations: The intensity of swelling may vary considerably within small distances alongside a tunnel even in the case of a macroscopically homogeneous rock mass with constant mineralogical composition. This can be explained only as a consequence of varying water circulation conditions.

Empirical evidence concerning the swelling of anhydritic rocks, actually a mechanical-hydraulic-chemical coupled process, is very limited at the present. The heterogeneity of sulphatic rocks (in the specimen-scale) in combination with the extremely long duration of swelling tests (several years even for small specimens) makes experimental research very difficult. In addition, the value of common laboratory tests is questionable, because the water circulation conditions are in general different in situ from the ones prevailing in oedometer tests.

Figure 12. Phaenomena in swelling rock.
Field observations show that the rate of swelling can be reduced by applying a counterpressure on the tunnel floor. The relationship between steady-state heave and support pressure, which is important for the conceptual design of tunnels in swelling rock, is unknown.

6 CLOSING REMARKS

Water has a decisive influence on the stability and the deformations of underground openings for a wide spectrum of geotechnical conditions. Taking into account the seepage flow by appropriate modelling improves our understanding of the observed phenomena and of the inherent design uncertainties.

REFERENCES


