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Pore water pressure and seepage flow effects in squeezing ground

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ABSTRACT

The 57 km long Gotthard high-speed railway tunnel with a maximal overburden of 2400 m will cross in some parts kakiritic rocks prone to squeezing. In order to establish their mechanical characteristics, a comprehensive testing program was carried out. Particular attention was paid to the control of pore water pressure during testing. Despite the complex structure of the kakiritic rocks, remarkably uniform results were obtained. The practical significance of the laboratory findings is demonstrated by discussing the ground response due to the tunnelling operation. In order to take into account the pore water pressure, the weak rock is modelled as a porous medium according to the principle of effective stress. With this approach both the short- and the long-term behaviour can be analyzed in a consistent manner with a single set of material parameters. The permeability of the rock should be taken additionally into account in the characterisation of squeezing formations, as it may govern to a large degree the rate of the rock pressure and deformation phenomena.

1. INTRODUCTION

Tunnel excavation in squeezing rock can lead to substantial long-term deformations of the ground. If these are impeded or prevented by a tunnel lining, instead rock pressure builds up, which can cause excessive damage or failure of the lining (Figure 1). This phenomenon occurs in rocks exhibiting low strength and high deformability and the problem is exacerbated by increasing rock overburden and a high initial pore water pressure (Kovári 1998).

The planning of the deep and long base tunnels through the French, Italian, Austrian and Swiss Alps highlights the problem of tunnelling in squeezing rock (Kovári 1995). In the 57 km long Gotthard high-speed railway tunnel, such conditions are expected particularly in the northern Tavetsch massif (Figure 2). During mountain formation this
zone was subjected to intensive tectonic action, whereby alternating layers of thickness in the range decimetres to decametres, consisting of intact to more or less strongly kakiritic gneisses, slates, and phyllites resulted. Kakirite describes a broken or intensively sheared rock, which has lost a large part of its original strength (Schneider 1997). Depending on the extent of disintegration, besides the structure also the texture of the rock has been considerably changed. Because of a depth of cover of 800 m, an initial pore water pressure of 80 bar and the expected rock properties the approximately 1100 m long tunnel section represents a key problem in the present project.

For this reason, the geological survey included the inclined, over 1700 m deep exploratory borehole SB 3.2 (Figure 3), which passed through the problematic series of rocks and extended to the level of the tunnel. Besides the geological exploration, it also served to retrieve large intact rock cores for various laboratory tests. To investigate the strength and deformation properties of the weakest zones within the investigated rock series, a detailed testing program was carried out at the Institute for Geotechnical Engineering of the Swiss Federal Institute of Technology (ETH) Zurich (Vogelhuber 1998, Vogelhuber 2000).

Figure 1: Squeezing phenomenon in the safety gallery of the Gotthard motorway tunnel.
In this paper some aspects of squeezing rock behaviour are discussed both experimentally and theoretically paying special attention to the pore water pressures and to the time-dependency of the squeezing phenomenon in tunnelling due to seepage flow.
2. TESTING PROGRAM

2.1 Laboratory equipment and testing procedure

The kakirite to be investigated is a porous medium, whose pores may be partially or completely filled with water. The pore water pressure influences the strength and deformability of the rock considerably. Thus according to Terzaghi all measurable effects of a change in pore pressure are solely the result of changes in effective stresses. The effective normal stresses are equal to the difference between the total normal stresses and the pore water pressure provided saturation is complete. For this reason the specimen must be brought into a saturated state beforehand and, in addition, the pore water pressure must be either controlled or measured during the whole period of testing.

Therefore, a new type of testing device was developed. Apart from a hydraulic press with a maximum load of 600 kN having an extremely stiff testing frame and a system for applying lateral pressure (max. 70 MPa), it also includes a system for applying back pressure (max. 10 MPa). Further, the triaxial cells have been dimensioned for samples having a diameter of 80 or 100 mm. A diamond-studded saw was employed to prevent the specimens from splitting along the edges when cutting them to length and to ensure plane ends normal to the axis. Lack of evenness along the circumference or smaller splits on the edges were touched up locally using plaster. Markers along the circumference of the sample help one to detect failure mechanisms.

The testing procedure is divided up basically into two phases. A uniform confining pressure follows a displacement-controlled increase of the axial stress $\sigma_a$ at constant radial stress $\sigma_r$. For shortening of the specimen in the axial direction one speaks of compression tests, whose classification is carried out on the basis of the hydraulic conditions during the two phases. The triaxial tests are either of type consolidated-drained (CD) or consolidated-undrained (CU). Due to the low permeability of the rock the consolidation procedure was slow. An observation time of up to 4 h was needed for that. The sample must be brought to failure very slowly, so that the pore water pressure $p$ corresponds to the value applied or measured external to the sample. Therefore, the rate of axial strain exhibits a value of either $0.5 \times 10^{-5} \, 1/s$ (CD) or $1.0 \times 10^{-5} \, 1/s$ (CU). This resulted in an observation time of up to 8 h. The triaxial tests were performed in a single step or in several steps, whereby the latter is characterized by the multiple repeating of individual test phases with different lateral stresses.

As a consequence of the rock core retrieval and the specimen preparation, the samples are unsaturated before testing. Thus they first have to be saturated. This is done in steps by removing the first part of the pore air by specifying a pore water pressure gradient (passing water through the specimen: period of observation = 100 h) and by dissolving the second part of the pore air in the pore water by applying a sufficient back pressure (saturation: period of observation = 30 h). For a value of approximately 2.0 MPa the required verification was achieved for all specimens by determining Skempton’s B-value. Independent of the hydraulic conditions a period of approximately one week results for the execution of the tests (Vogelhuber 2004).
2.2 Testing results

Figure 4 shows representative results of CD- and CU-tests. The diagrams illustrate the variation of the stress deviator $\sigma_a - \sigma_r$, the pore water pressure $p$ and the change in volume $\varepsilon_{\text{vol}}$ (a decrease is positive) in function of the change in length $\varepsilon_a$ (a decrease is positive).

![Figure 4: Deviatoric stress, pore water pressure and volumetric strain as a function of axial strain: Left sided diagrams CD-test. Right sided diagrams CU-test.](image-url)
In the CD-tests (open drainage valves) the pore water pressure is controlled permanently by the existing back pressure. With the unhindered expulsion or absorption of pore water there is a corresponding change in the volume of the rock. Under drained conditions the yield limit is reached when the stress deviator reaches a maximum value. Subsequently, the rock deformation is accompanied by a small decrease in strength. Even after a compression of more than 10% the specimens remain ductile and do not disintegrate at all (Figure 5). The observed failure mechanism exhibits, independently of the extent of strength loss, either a few pronounced or many less pronounced shear surfaces. In some cases the slope of the shear surface follows the texture of the rock, if present. As failure progresses the rock exhibits a dilatant behaviour. This expresses itself in a steady increase in the volume of the rock, whereby the material behaviour was observed at no time to approach a so-called critical state involving no further change in volume (Schofield and Wroth 1968).

![Figure 5: Documentation of the rock specimens: (a) Before testing. (b) After testing.](image)

If the drainage valves are closed (CU-tests) the pore water pressure is always different from the back pressure. This is because the expulsion or absorption of pore water is completely prevented and thus the volume of the rock remains almost unchanged. Before the development of failure the pore water pressure drops continually, which causes a steady increase in the effective normal stresses. Thus, after reaching the yield limit the stress deviator increases as the rock deforms during failure. The constrained dilatancy causes a behaviour similar to strain hardening.
The test results can be reproduced by the linear elastic, perfectly plastic constitutive equations obeying Mohr-Coulomb yield criterion formulated in effective stresses (solid lines in Figure 4). The undrained condition is characterized by the condition of a constant volume of rock (neglecting the compressibility of the solids and the water), and the drained condition by a constant pore water pressure. It is remarkable that the test results for both types of test, despite the relatively simple material model, can be reproduced satisfactorily using a single set of parameters. An additional evidence for this is provided by comparing the observed and calculated effective stress path in the principal stress diagram (Figure 6).

Figure 6: Testing results in the principal stress plane: (a) Area of 63 CD- and CU-tests (single or multistep). (b) Yield condition for $c = 0.7$ MPa & $\varphi = 31^\circ$. (c) Yield condition for $c = 0.5$ MPa & $\varphi = 21^\circ$. Conventional triaxial tests (multistep), i.e. without sample saturation and control or measurement of pore water pressure: (d) Dried specimen ($c = 1.9$ MPa & $\varphi = 35^\circ$). (e) Wetted specimen ($c = 0.3$ MPa & $\varphi = 11^\circ$).
The hatched area (a) in Figure 6 contains the effective stress states at failure for all the 63 CD- and CU-tests. Despite the complex and changeable structure of the kakirite, remarkably uniform test results were obtained. The observed scatter is due to the heterogeneity of the samples. The straight lines (b) and (c) may be considered as an upper and lower bound for the yield condition, and are used in the calculations of the next section.

Besides the tests described above, several triaxial tests have been carried-out by using the equipment commonly employed in rock mechanics for conventional triaxial testing, i.e. without sample saturation, and control or measurement of pore water pressure. In these tests, a much larger scatter of the results was observed. A detailed investigation revealed that the test results depend essentially on the moisture content before the start of the test. The strength parameters may be either overestimated (when the moisture content is low due to previous drying, see curve (d) in Figure 6) or underestimated (when the moisture content is high due to previous wetting, see curve (e) in Figure 6). So the large scatter observed in these tests is obviously the result of a completely unsuitable testing procedure. This underlines the importance of control or monitoring pore water pressure during triaxial testing. It is interesting to note that in the CD- and CU-tests the strength parameters obtained were not affected by the moisture content before testing. This was shown by additional tests in which the moisture content was intentionally subjected to a large change before testing by drying or wetting.

3. APPLICATIONS TO TUNNELLING IN SQUEEZING ROCK

3.1 Time-dependent rock behaviour

In the following we shall look more closely at various aspects of the time-dependent development of rock pressure and rock deformation. Rock deformations normally develop slowly, although cases are also known of intense and rapid deformations close to the tunnel heading. The development of rock pressure or rock deformation may take place over a period of days, weeks or months. It is encouraged by the presence of high initial pore water pressure. The gradual increase in rock deformation or rock pressure can be traced back generally to three mechanisms.

(a) The first is connected with the three-dimensional redistribution of stress in the region around the working face. This takes place in every tunnel and is of great importance for the dimensioning of the lining (Lombardi 1971). However, it cannot explain long-term rock deformations, because it occurs in proximity to the working face within a region, whose length corresponds approximately to twice the diameter of the tunnel.

(b) The second mechanism is connected with the rheological properties of the rock. So called "creep" is especially evident if the rock is highly stressed as the state of failure is approached, and is therefore of great significance for the processes taking place under squeezing rock conditions (Fritz 1981). Creep does not necessarily imply the presence of pore water, as it is also observed in "dry" formations.

(c) The excavation of a tunnel in water-bearing rock triggers a transient seepage
process around the opening, in the course of which both the pore water pressures and the effective stresses change with 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 this process takes place and the more long-term are the rock deformations - or by their prevention - is the development of rock pressures on the lining.

These three mechanisms are in general superimposed. In this contribution we are concerned exclusively with a discussion of the processes linked to the development and dissipation of pore pressures. The analysis is based on the classical consolidation theory of Terzaghi (see e.g. Terzaghi and Jelinek 1954). The ground is assumed to consist of a saturated porous medium for which Darcy's law holds. A linearly elastic, perfectly plastic material model obeying Coulomb’s failure criterion describes the deformational behaviour.

In Section 3.2 we focus on the two limiting states of the transient phenomenon associated with the development and dissipation of pore water pressures (Anagnostou and Kovári 1999): the state at \( t = 0 \) ("short-term behaviour") and the state at \( t = \infty \) ("long-term behaviour"). The time-development of rock pressure and deformation as well as the effect of permeability are discussed in Sections 3.3 and 3.4, respectively. We deal here also with the influence of the length of the squeezing zone between two faces of competent rock.

### 3.2 Short- and long-term behaviour

The short-term response is determined by the condition of a constant volume of rock, the long-term response by the steady state distribution of the pore water pressures in the rock surrounding the tunnel.

Assuming an elasto-plastic material behaviour, a plastic zone develops around the opening. In it the rock is stressed to the limit of its bearing capacity and accommodates overstressing - as in the triaxial tests - by deforming plastically. In the short-term only deformations at constant volume occur, whereas in the long-term they are associated with an increase in volume. The latter is due to the dilatant behaviour of the rock at failure. With regard to the rock deformations, therefore, the short-term behaviour must be considered more favourable than the long-term one.

This is confirmed by the theoretical analyses carried out. Figure 7 shows the relationship between the support pressure \( \sigma_A \) and the radial displacement \( u_A \) at the tunnel boundary in the special case of rotational symmetry (so-called ground response curves) (Kovári 1986).

The curves have been obtained by elasto-plastic plane strain analyses. The initial effective stress field was taken homogeneous and hydrostatic. According to the existing depth of cover and ground water level, an initial effective stress of 12 MPa results. The pore water pressure at the tunnel boundary was assumed to be atmospheric.

The results of Figure 7 apply to the symmetry plane of a short squeezing zone (see inset in Figure 7). When crossing a short squeezing zone, the adjacent competent rock experiences considerably smaller deformations than the weak ground within the critical
zone. Shear stresses are, therefore, mobilized at the interface between competent rock and squeezing ground. Due to this "wall-effect", ground movements as well as rock pressures in short fault zones are considerably smaller than the ones predicted by a plane-strain analysis. This effect can be taken into account by applying a correction factor to the ground response curve obtained for plane strain conditions. This correction factor was established by means of three-dimensional elasto-plastic finite-element analyses. Computational details are given elsewhere (Anagnostou and Kovári 1998, Kovári and Anagnostou 1995).

![Diagram](image)

Figure 7: Ground response curves: (b) and (c): Coupled stress-seepage analysis based upon the parameters obtained by triaxial testing with consideration of pore water pressure (Figure 6). (d) and (e): Stress analysis based upon the parameters obtained by conventional triaxial testing without consideration of pore water pressure (Figure 6).

For other material constants see Figure 4.
The hatched areas in Figure 7 (bounded by the curves (b) and (c)) show the range of short- and long-term ground response curves based upon the strength parameter envelopes established by all the 63 triaxial tests carried-out with consideration of pore water pressure. The other material constants were taken from the CD- and CU-tests of Figure 4. It should be noted here that the value of the Young's modulus is uncertain as the test results indicate a dependency on the first stress invariant. This cannot be taken into account by the assumed linear elastic, perfectly plastic constitutive equations.

Figure 7 shows that the short-term radial displacement is very small even without a support pressure. Long-term however the underground opening would close, as shown clearly by the upper hatched area. These results are of practical interest in the case of a low permeability rock, since only then a pronounced short-term behaviour does exist.

Furthermore, it can be clearly seen from Figure 7 that the possible range of variation of rock behaviour in the long-term is greater than in the short-term. This is because the long-term plastification is greater and therefore the scatter in the strength parameters has more influence on the calculated results.

For comparison, Figure 7 includes also two additional ground response curves based upon total stress analyses with the strength parameters obtained from conventional triaxial tests carried-out without consideration of pore water pressure (curves (d) and (e)). In this case, a distinction between short- and long-term behaviour is of course impossible. The diagram shows, furthermore, that such tests are useless as the large scatter of the parameters leads to an extreme variation of possible ground response curves.

3.3 Time development of rock pressure and rock deformation

To deal with the problems encountered in tunnel sections exhibiting squeezing rock conditions there are basically two design concepts: the "resistance principle" and the "yielding principle" (Kovari 1998). In the former a practically rigid lining with a high load bearing capacity is adopted, which is dimensioned for the expected rock pressure. In the case of high rock pressures this solution is not feasible. By contrast, in the latter, by allowing large convergence of the lining, the rock pressure is reduced to a value that is manageable during construction. An adequate overprofile and suitable structural detailing of the temporary lining permit an occurrence of rock deformations without damage to the lining, thereby maintaining the desired clearance from the minimum line of excavation.

A question of practical interest for the "resistance principle" concerns the time-development of rock pressure under conditions of full prevention of the radial displacement $u_A$. On the other hand, with regards to the "yielding principle", the time-development of rock deformation for a constant lining resistance $\sigma_A$ is interesting.

Figure 8 presents the results of numerical transient coupled stress-seepage flow analyses for the "resistance principle" (left) and the "yielding principle" (right). The two curves as well as the respective ordinates $\sigma_A$ and $u_A$ refer to these two cases. For the sake of simplicity, for both 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. For the calculation referring to the yielding principle, the radial
stress at the excavation boundary (lining resistance) was taken to 0.15 MPa. For the resistance principle, the excavation boundary deformations have been fixed to their short-term value (i.e. the displacement $u_A$ at $t = 0$ for $\sigma_A = 0$). The pore water pressure $p_a$ at the tunnel wall is taken atmospheric in both cases. The far field conditions are taken according to a depth of cover and a water level of 300 m. The material constants are given in Figure 8. Rotational symmetry and plane strain condition (without "wall-effect") have been assumed. The plane strain condition implies that the transient processes associated with pore water pressure dissipation do not interfere with the spatial stress re-distribution in the vicinity of the face (i.e. that the dissipation of pore water pressures as well as the respective time-dependent deformations take place slowly relatively to the advance of the tunnel heading). Here obviously the following factors are important: The continuity and the speed of advance, and the permeability of the rock. The numerical calculations have been carried-out by the finite element method using the computer program HYDMEC developed at the Swiss Federal Institute of Technology, Zurich (Anagnostou 1991).

Figure 8 shows that the radial displacement $u_A$ occurring in the case of a deformable lining develops considerably slower than the rock pressure $\sigma_A$ in the case of a rigid lining. The development of radial displacement in the first case is slower, because it needs larger quantities of seeping water, as it is associated with larger volumetric strains and a larger increase of pore water content.
In the example in Figure 8 the rock pressure $\sigma_A$ acting upon a rigid lining (resistance principle) increases within a month to the considerable value of 0.5 MPa. When applying the yielding principle, however, only a negligible deformation develops in the same time period. This result is interesting from the practical point of view as it shows that with the yielding principle one gains time.

3.4 Influence of rock permeability

The rate of the deformations associated with the pore water pressure dissipation is proportional to the permeability of the rock. For permeability a factor ten higher the rock pressure and the rock deformation develop ten times faster. The great importance of permeability soon becomes clear when one plots the rock pressure (resistance principle) or the rock deformation (yielding principle) at a particular time $t$ in function of permeability $k$ (Figure 9).

Figure 9: The rock pressure (resistance principle) and the rock deformation (yielding principle) after 15 days in function of the rock permeability.

Note that permeability coefficients lower than $10^{-9}$ m/sec are typical for practically impermeable rock. Since the determination of permeability in this range is often subject to large uncertainties, reliable predictions of the time development of rock pressure or rock deformation are difficult. Further prediction uncertainties exist in heterogeneous rock formations. From the consolidation theory it is known that the duration of the tran-
sient flow is proportional to the square of the length of the drainage path. Thin permeable layers embedded in a practically impermeable rock mass lead to a shortening of the drainage path and cause therefore a substantial acceleration of the development of pressure and deformation (Figure 10a). Also, rock pressure phenomena can be more intense when approaching a permeable water-bearing formation (e.g. a fault zone) as the drainage paths become shorter (Figure 10b).

![Figure 10: Shortening of the drainage path due to (a) the existence of permeable layers, (b) the vicinity of a permeable formation.](image)

In the example of Figure 9, depending on the coefficient of permeability $k$ (between $10^{-11}$ m/sec and $10^{-9}$ m/sec), a rock pressure between 0.1 and 1.2 MPa develops within 15 days. To resist a rock pressure of 0.1 MPa a light temporary support suffices. In the case of a rock pressure of 1.2 MPa however a strong lining would be necessary. The difference between 0.1 and 1.2 MPa is very important from the technical point of view. So the existence of some permeable layers or the vicinity of a permeable formation may cause that the tunnelling engineer experiences the rock in one case as "competent", and in the other as "disturbed".

In the case of the "yielding principle", nevertheless, the consequences of prediction uncertainties are smaller. For $k = 10^{-11} - 10^{-9}$ m/sec the radial displacement $u_{a}$ of a flexible support amounts to 0 - 0.25 m (Figure 9). The results of a wrong estimate of convergence (in the actual example 7 - 8 m$^3$ per tunnel metre additional excavation and possibly more concrete for the permanent lining) are more modest than the consequences of a wrong estimate of the time development of rock pressure according to the "resistance principle".
4. CONCLUSION

The control of pore water pressure during triaxial testing of weak rocks prone to squeezing is indispensable. Conventional triaxial tests are inadequate in this respect, as they may lead to a serious under- or overestimation of the strength parameters. Consolidated drained (CD) and consolidated undrained (CU) tests provide, despite the complex structure of the kakiritic phyllite, remarkably uniform and useful results. The strength parameters show a considerably small scatter in comparison to conventional triaxial testing data. Specimen pre-saturation and maintenance of a sufficient back pressure are essential for obtaining reliable and reproducible parameters.

Taking into account the pore water pressure by appropriate modelling of the rock mass as a porous medium provides an explanation of the time-dependency of squeezing ground behaviour and highlights the importance of rock mass permeability as well as the respective prediction uncertainties in tunnelling through such water-bearing squeezing rock.

5. REFERENCES