Hydrogeological aspects for the design of deep seated tunnels

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Abstract

If deep tunnels are excavated in water saturated rocks, the hazard scenarios depend strongly on the permeability of the driven-through underground. Tunnel construction in high permeable rocks bear the classical risks of sudden heavy water inflow in combination with the destabilisation of the tunnel face. Excavation in low permeability underground lead to different risks like delayed deformations, which may affect the rear parts of a tunnel.

In the framework of this presentation, various risk situations for both cases are investigated by means of hydro-mechanical coupled calculations.

1. Introduction

1.1 Water pressure

If a deep seated tunnel is regarded in its longitudinal profile, the course of the groundwater table goes more or less along the earth's surface. Although the groundwater head is in many cases supposed to vary significantly during the year due to seasonal fluctuations, the effect of these variations on deep seated tunnels is of minor importance, but the general influence of the ground water on the tunnel remains very high. The approximate determination of the water table location can occur by

using small rivers, spring lines, or so called fixed potentials (natural or artificial lakes) as “starting points”.
The in-situ water pressure in pores (considering e.g. an ultracatacastic rock / kakirite) or joints around tunnels is normally determined by the vertical distance between groundwater head and gallery. The water pressure is gradually rising with increasing depth and reaches already in a depth of some 100 m an order of magnitude, which cannot be withstood anymore even by thick tunnel liners. As a consequence deep seated tunnels require normally permanent drains to avoid high water pressures and - in the case of a high permeable underground - injections in the surrounding massif to reduce the water inflow.

1.2 Permeability

Systematic back analyses in the central Swiss Alps showed that permeability decreases with increasing overburden. This tendency is less pronounced in granites than in laminated banded gneisses or schistose metamorphic rocks. Nevertheless it can be estimated, that the permeability in the rock matrix (term used to distinguish between fault zones and “normal” typical jointed rock) with an overburden of more than 500 m is low (< 1e9 m/s) for both types of rock. Thus for deep seated tunnels the risk of heavy water inflow can be assumed to be restricted to

- fault zones
- (hydrothermal) karst phenomena in carbonatic rock.

Flat orientated schistosity and increased horizontal stresses lead to a faster decrease of permeability with increasing depth. For the Brenner Basetunnel with mainly flatter orientated shists an even higher decrease of permeability may be expected in sections with adequate geology e.g. phyllites, shists.

The permeability contrast between normal rock and defined high risk zones is very high, so that especially in early planning phases an inflow prognosis, relying only on zones with high permeability, is acceptable.

As a remark it may be stated, that the permeability of fault zones is very difficult to judge. Whereas cataclasis increases the porosity and consequently also the permeability, hydrothermal growth of clay minerals in e.g. ultracataclastic rocks decreases the permeability dramatically, leaving the zone with highest relative displacements often as a hydraulic barrier. Under those circumstances the heavily jointed peripheral areas of a fault can be judged as the zone with probably the highest permeability.

1.3 Hazard scenarios

The development of hazard scenarios can be used as a systematic background for the design of tunnels of various sizes or purpose. The main idea can be summarized as follows: in collaboration between project engineers and geologists relevant hazard scenarios and degrees are elaborated or postulated and appropriate means for counteracting are planned.
In early planning stages categories of hazard scenarios might be used additionally to or aside of rock mass classifications for dimensioning purpose.
In detailed planning stages the procedures for different scenarios are described and summarized in concepts to provide the site office with the required basis for quick decisions for eventual countermeasures during the construction phase.

a.) Typical water-concerning hazard scenarios for deep seated tunnels in <u>high permeable zones</u> are:

- Sudden large water inflow
- Sudden large mud or sand inflow
- Hydraulic base failure - especially in the case of alternating zones with very different permeability each

The most probable zone for those scenarios is in general the tunnel face. In cases of high permeable zones which are parallel or obliquely orientated towards the tunnel also side walls can be affected.

These hazard scenarios lead typically to the following dimensioning case:

- Design of an injection / grouting body ahead and around the tunnel for a zone of specified width

For the construction stage of e.g. the Gotthard Basetunnel, the procedures to reduce the above mentioned risks are typically described in:

- concepts for investigations
- concepts for special measurements
- concepts for injections

As a remark it may be stated, that in the shaft foot caverns of the central part of the GBT (section Sedrun) pumps with a maximum conveying volume of 1000 l/s are installed, on the bases of pessimistic inflow predictions.
Whereas the risks are clearly described for high permeable underground by means of the “classical” risk situations, for low permeable saturated rocks the risks concerning the pore water are more difficult to define due to the complexity of hydro-mechanical coupled processes.

b.) Possible risks encountered in <u>low permeable zones</u> are:

- Increased deformation potential due to pore pressure effects leading to low effective stresses
- Reduced deformation potential due to negative pore pressures near tunnel face, leading to
  - increasing risk of high deformations and failure of the tunnel face in the case of an excavation stop due to time dependent pore pressure redistributions
  - increased risk for a “too early” installed support respectively unexpected high “delayed” radial deformations in the rear part of the tunnel due to pore pressure redistributions in the massif

Whereas in high permeable rocks the highest risk is near the tunnel face, in low permeable rock the risks due to pore water concern to a larger extent the rear of the tunnel.
These hazard scenarios haven’t gained yet in the literature the status of classical dimensioning cases. To underline and illustrate the influence of the above mentioned scenarios, some results of fully coupled (hydro-mechanical) calculations and numerical approaches are presented below.

2. Design cases

2.1 High permeable rock

2.1.1 Theoretical considerations

To enable a tunnel construction, water-bearing high permeable zones have to be sealed by injections. These injections influence and change the mechanical parameters of the injected underground, so that the classical calculation methods of the interaction between underground and supporting structure cannot be applied any more. Analytical solutions for the design of a tunnel in an injected zone exist with reference to [1, Chang&Hässler] and [2 Anagnostou & Kovari]. The authors derive dimensioning concepts based on the theory of ground reaction curves (GRC).
As GRC are a priori only adequate for zones with large (infinite) length and as water bearing zones (faults) are generally of restricted width, the application of typical GRC approaches are restricted.

In [2] approximate closed form solutions (GRC) have been obtained taking siloeffects into account. So these solutions can be employed for a first dimensioning of a tunnel through a grouted underground.

Another problem concerning the application of GRC for a detailed analysis of a tunnel is the three-dimensional distribution of the radial support forces, namely high radial support forces especially at the rear of the tunnel and very low support forces at or near the tunnel face. This may lead to large face displacements (extrusions) when for example the tunnel enters a vertical, parallel to the tunnel face orientated, zone of weakness. Extrusions studied in various numerical models provoked strong stress relaxations ahead of the tunnel face. For short zones of weakness the radial deformations are consequently often of minor importance due to the activation of silo effects. Thus for detailed analyses axial-symmetric or 3D models are required even for the relative simple case of a full face excavation through short fault zones.

The following theoretical example summarizes the general procedure for the design of a tunnel in a grouted zone, however in the framework of the presented article the authors refrain from providing a too detailed description of the design procedure.

2.1.2 High permeable zone of weak rock sandwiched between competent rock layers: Typical dimensioning case for grouting.

A tunnel with a radius of 5 m encounters a 14 m wide vertical and oblique orientated fault zone with increased permeability, an overburden of 800 m, an expected total vertical pressure in-situ of about 21 MPa and an in-situ water pressure of 8 MPa. The fault is sandwiched between competent good rocks with a modelled length of 30 m each. A pronounced difference in terms of strength and deformability is assumed between the fault (friction angle / φ'= 28°, cohesion / c' = 0.1 MPa, Young's Modulus / E = 1.5 GPa) and the host rock (φ'=35°, c' = 1.7 MPa, E = 16 GPa,).

Using the „Steady-State-Approach", the untreated fault zone (permeability > 1e-5 m/s) would cause a water inflow quantity of > 100 l/s. To reduce the inflow significantly to something about 10 l/s, high quality grouting is required to decrease the permeability in the fault zone lower than k = 1e-6 m/s.

A further result of grouting is, apart from sealing, the increase of the strength characteristics of the grouted rock mass. For calculation purpose the following properties are assumed after grouting the fault zone: E_i = 6 GPa, φ_i = 30°, c_i = 1.4 MPa. Geometry and extent of the grouted zone with increased parameters have been modelled by assuming a typical conical geometry with an average thickness of 10 m.

The calculations have been carried out by programming FLAC2D (axial-symmetric). To simulate the injection of the fault through the tunnelface, the rock mass parameters in the above mentioned area of the fault zone increased to the values E_i, φ_i and c_i, when the tunnel front is 10 m away from the fault zone.

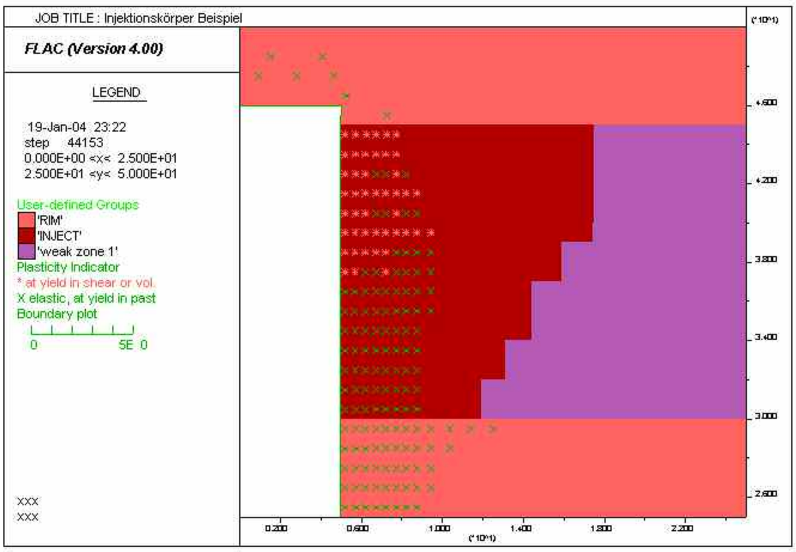
The in situ water pressure has been modelled as dropping from 8 MPa to 0 in the grouted zone due to drainage at the tunnel wall. The excavation is assumed as a full face excavation with a round length of 1 m. The radial support pressure (stiff steel arches+ shotcrete) increases to 2 MPa after 4 cm radial deformations. The systematic anchoring in the area of the tunnelface is simulated by an additional increase of the cohesion in the anchored massif (Δc = 0.2 MPa).

Results:

The radial pre-deformation at the tunnelface is calculated to 3 cm. Additional 4 cm of “measurable” deformations are encountered especially in the first part of the fault. The horizontal deformations at the tunnelface are higher than the radial ones when entering the fault zone. The strength of the grouted zone is partly exceeded (pict. 1). The steel arches are partly loaded up to their maximum load capacity.

The extent of the grouted zone is not sufficient in the first half of the fault. The smaller radius of the truncated cone in this part leads to a higher extent of the plastified zone and so consequently to a higher risk of an increase of permeability due to the development of cracks in this disturbed zone. Therefore measures like additional secondary injections should always be provided.

Very important for keeping the grouted body tight is the limitation of any deformations due to the subsequent tunnel excavation to a feasible minimum. To achieve this aim for deep tunnels driving through high permeable zones with marked deformation potential, a stiff support has to be installed as early as possible in combination with strong systematic anchoring of the tunnel front.

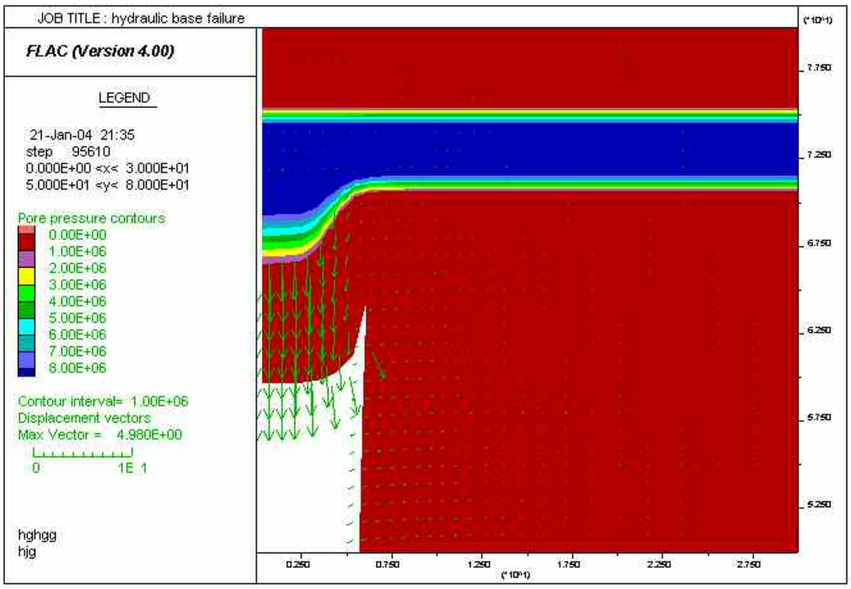


Plastified zone and groups with different properties

Picture 1

2.1.3 High permeable zone behind a cataclastic 20 m wide low permeable fault zone consisting of weak rock.

Without going into details, in this chapter only the fact is stressed, that the failure type "hydraulic base failure ahead of the tunnelface" can be observed in 3D or axial-symmetric numerical models, when a tunnel is excavated through a very low permeable fault zone bounded on its rear side by a high permeable zone. If - as a first judgement – the pore pressures in the low permeable fault are neglected and within the high permeable area are assumed as constant a sudden bulging out of the tunnel face can be observed in the models (pict. 2).



Picture 2

2.2 Low permeable rock

2.2.1 Theoretical considerations

In low permeable porous media, the excavation of galleries leads to both mechanical and hydraulic disturbances in their surroundings, namely a de-stressed damaged zone and a significant drop in pore water pressures. This phenomenon of instantaneous pore pressure decrease has already been observed in centrifuge tests (Mair,

1979) and around openings in low permeable formations (e.g. Boom Clay). It is brought about by hydro-mechanical coupling in the saturated media.

Furthermore, during and after the excavation stage, fluid flow occurs, causing time-dependent changes for both the structure and the medium: variations of the total pressure acting on the lining and redistributions of stresses and displacements in the medium (Gärber 2003; Gärber & Labiouse 2003; Labiouse & Gärber 2001).

To date, the design of underground structures is generally carried out by means of total stress analyses of monophasic media. Although their domain of use should be restricted to dry or compact rock masses, their application has frequently been extended to saturated media. Now, for several reasons, it may be stated that finite element calculations and analytical solutions available for monophasic media could become inappropriate to calculate the stability of galleries driven in low permeable saturated porous media. Indeed, these methods:

- consider the solid skeleton and the pore fluid as one phase and do not differentiate between the respective contributions of these two components.
- do not account for the in situ pore water pressure.
- do not yield information concerning the decrease in pore pressure generated during the excavation of tunnels as a result of hydro-mechanical coupling. When this is the case (Randolph & Wroth 1979, Mair & al 1992), the pore fluid is considered to be incompressible and the decrease in pore pressure is calculated based on the variation in mean total stress.
- do not provide any information about the fluid flow and the redistribution of pore pressures as a function of time and consequently on the long term equilibrium of the tunnel (support pressure and convergence).

Researches, undertaken at the Rock Mechanics Laboratory of the Swiss Federal Institute of Technology Lausanne, treated the up-to-now misappreciated role taken by pore water (pre-existing pressure, hydro-mechanical coupling, fluid flow and pore water redistribution, medium and lining permeability) for the short and long term stability of tunnels in saturated media. In the framework of a PhD-thesis [Gärber 2003] ground reaction curves for saturated and undrained conditions were developed, which take –amongst others- the three-dimensional effect of the tunnel excavation process into account. This method can be used for the design of deep tunnels in low permeable media and of course also for gaining insight into the behaviour of the underground during a tunnel excavation. The above mentioned work provides the user with the semi-analytical solutions to determine the underground-structure equilibria (support pressure, support convergence, pore water pressure, plastic zone extent) for both the short term and the long term state.

2.2.2 TBM excavation in low permeable rock, example Boom Clay
Development of pore pressures, deformations and stresses during excavation

The following example case is based on the underground research laboratory HADES in Mol, Belgium, which is built in a depth of 225 m in the Boom Clay. The main geotechnical characteristics of the clay and of the gallery are shown below (see details in Labiouse & Gärber 2001):

Boom Clay (elasto-plastic, Mohr-Coulomb, drained values):

E = 300 MPa, φ = 18°, c = 300 kPa, k = 2 10^{-12} m/s, water bulk modulus K = 2 GPa

Gallery:

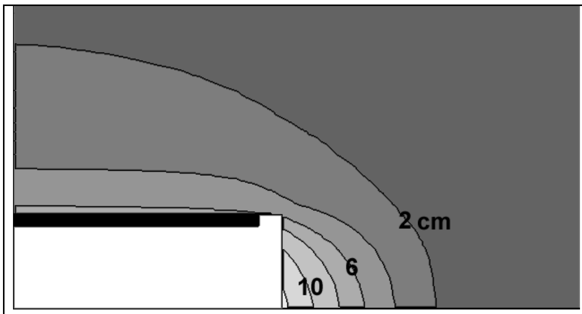
R = 1.91 m, in-situ total stress = 4.5 MPa, in-situ pore pressure = 2.25 MPa, excavation speed = 2 m/day

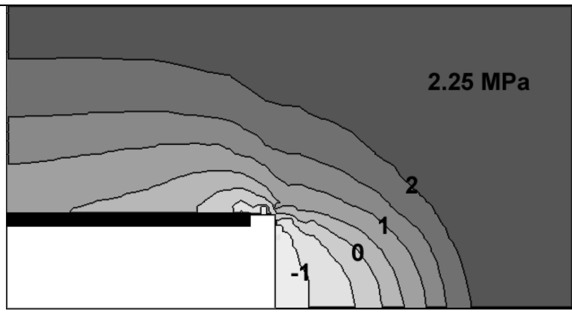
Lining

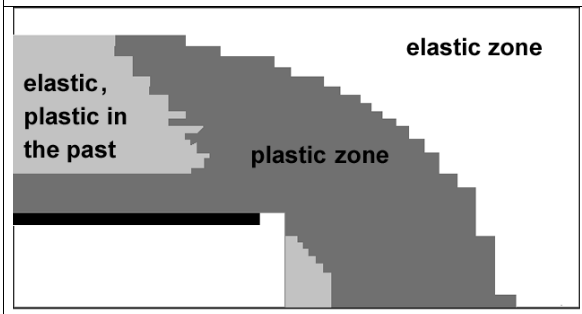
E = 75000 MPa, thickness = 0.1 m, placement distance to tunnelface = 0.5 m.

Due to the high tunnelling speed (2 m/day) in comparison to the low underground permeability (2 10^{-12} m/s), the numerical simulations were carried out *fully-coupled* (FEM-code Z_soil 5.0) in axial-symmetric configuration, taking account of the sequential tunnelling process, the stiffness and placement distance of the lining and the excavation speed.

Pict. 3 shows two contour plots of the displacement magnitude and the pore water pressures around the tunnel face during construction.







Upper left: displacements

Upper right: pore water pressures

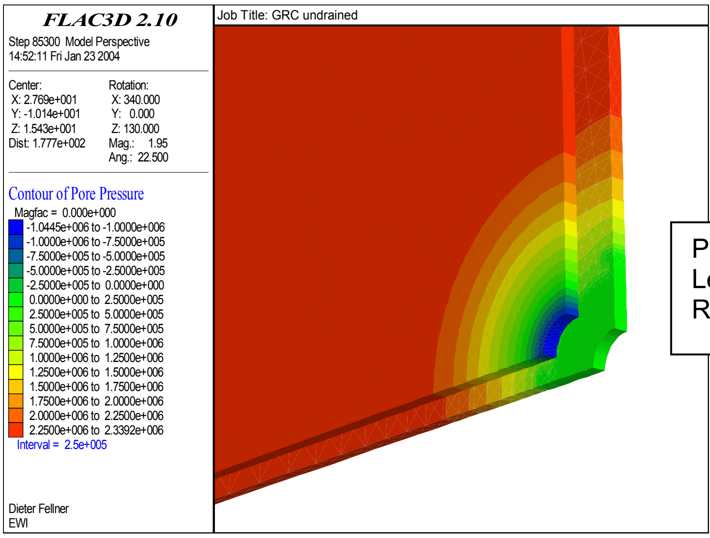
Left: plastic zone extent

Picture 3

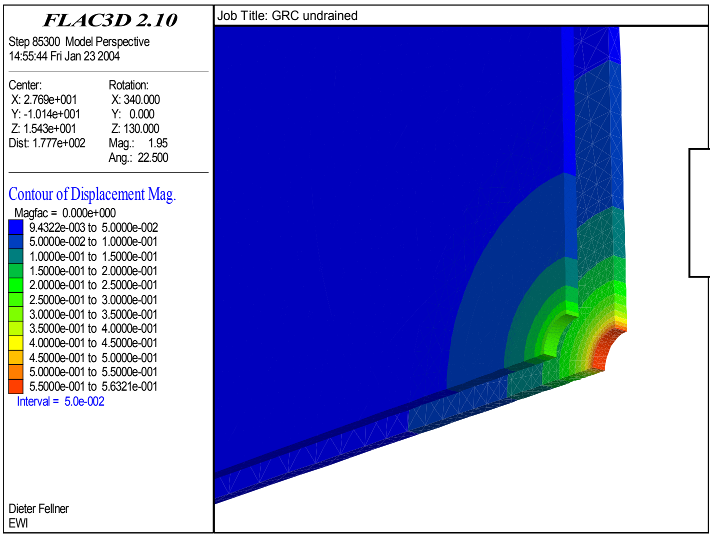
The plot of the displacement magnitudes shows, that a significant part of the convergence occurs already before the lining installation and ahead of the excavation face. Behind the laying distance, further radial deformation of the tunnel sidewalls takes place, inducing load in the support system. Equilibrium is finally reached when the soil pressure is exactly balanced by the lining pressure.

The filled contour plot of the pore pressures represents the drop of pore water pressure brought about by the mechanical deformations undergone by the clay mass during tunnelling. One notices that, for this axial-symmetric problem, the hydraulic disturbance only occurs in the damaged zone around the gallery. It may be stated, that this drop of pore water pressure has a mechanically seen positive effect on the stability of the tunnel and the tunnelface, as well as it reduces the arising deformations. On the other hand, this pore pressure distribution is logically time dependent and changes after a certain time (e.g. in the cae of an excavation stop).

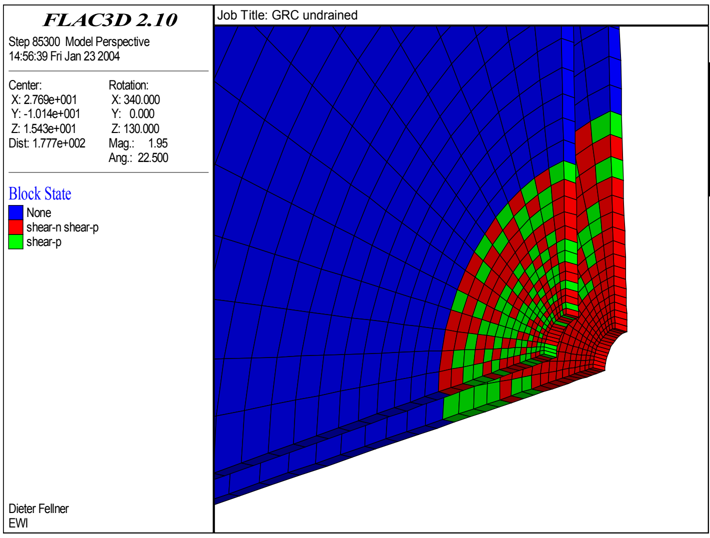
At the tunnelface, the pore pressure drops due to the high plastic deformations of the clay even to a negative value of around -1.3 MPa. This calculated pore water tension has to be regarded critically, because Horseman et al (1996) assume for the Boom Clay a limit (“air entry”) of –1.0 MPa. Additional simplified calculations (FLAC) to investigate the importance of this value show the interesting result, that for example the restriction of the water tension limit to “0” (= no negative pore pressures allowed) has a significant influence on the pore pressure distribution and leads to higher deformations and a higher plastic zone extent (Pict. 4).



Pore pressure distribution:
Left: no water tension limit
Right: no negative pore pressures allowed



Displacement magnitude:
Left: no water tension limit
Right: no negative pore pressures allowed



Plastic zone:
Left: no water tension limit
Right: no negative pore pressures allowed

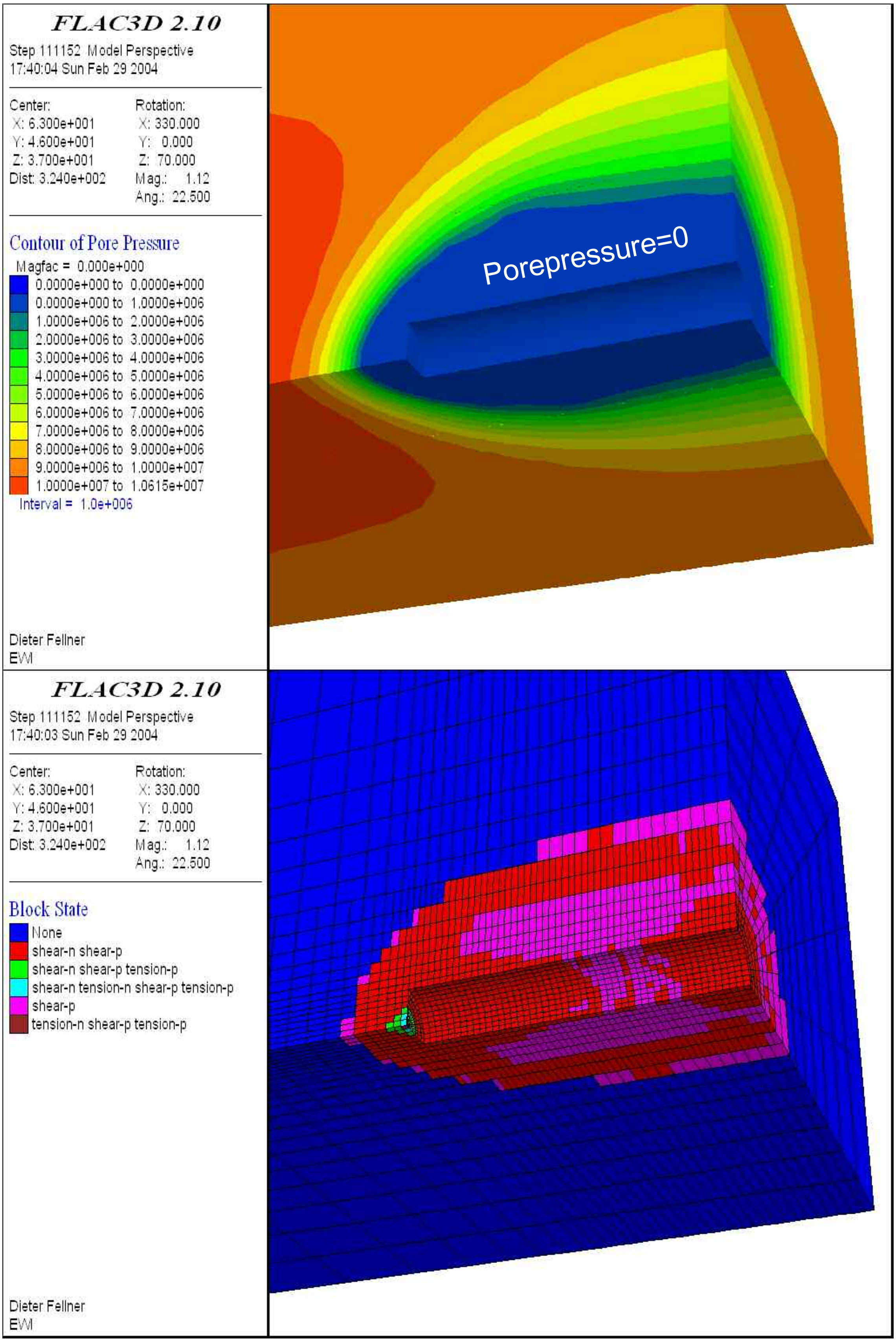
Picture 4

2.2.3 Low permeable homogeneous rock, 3D-calculations

Case: a tunnel with a radius of 6 m encounters 150 m a wide vertical and oblique orientated zone of weak rock with low permeability (k=1e-10 m/s) and with a porosity of 15%; the overburden is 1000 m; the expected total vertical pressure in situ is 32 MPa and the in situ water pressure 10 MPa. For the weak zone low strength parameters (φ = 30°, c = 0.2 Mpa) and a moderate Young's Modulus (E = 7.5 GPa) is assumed.

A fully coupled calculation for 50 m excavation with an assumed excavation velocity of 1 m per day has been carried out by means of FLAC 3D. The support is modelled as yielding with different levels of support pressures increasing up to 1.4 MPa for the resisting phase. The change from yielding to resisting phase occurs after 50 cm radial displacement undergone by the support. The excavation is accompanied with intensive radial and tunnelface bolting modelled as explicit cables. The fluid bulk modulus is 2 GPa and the water tension limit is set to -1 MPa (see chapter 2.2.2)

Picture 5 shows the hydromechanical disturbance after 50 cuts into the 150 m wide homogeneous zone. The pore pressures drop to 0 in the plastified zone. The results are predetermined by the fact that "realistic" water tension limits are exceeded in the plastified zone and have been set to zero (cavitation) by the calculation code. In comparison with chapter 2.2.2 the rock is too soft, the overburden is too high and the deformations are too large to keep negative pore pressures within realistic" water tension limits of e.g. -1 MPa.


FLAC3D 2.10
Step 111152 Model Perspective
17:40:04 Sun Feb 29 2004
Center:
X: 6.300e+001
Y: 4.600e+001
Z: 3.700e+001
Dist: 3.240e+002
Rotation:
X: 330.000
Y: 0.000
Z: 70.000
Mag.: 1.12
Ang.: 22.500
Contour of Pore Pressure
Magfac = 0.000e+000
0.0000e+000 to 0.0000e+000
0.0000e+000 to 1.0000e+006
1.0000e+006 to 2.0000e+006
2.0000e+006 to 3.0000e+006
3.0000e+006 to 4.0000e+006
4.0000e+006 to 5.0000e+006
5.0000e+006 to 6.0000e+006
6.0000e+006 to 7.0000e+006
7.0000e+006 to 8.0000e+006
8.0000e+006 to 9.0000e+006
9.0000e+006 to 1.0000e+007
1.0000e+007 to 1.0615e+007
Interval = 1.0e+006
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Porepressure=0
FLAC3D 2.10
Step 111152 Model Perspective
17:40:03 Sun Feb 29 2004
Center:
X: 6.300e+001
Y: 4.600e+001
Z: 3.700e+001
Dist: 3.240e+002
Rotation:
X: 330.000
Y: 0.000
Z: 70.000
Mag.: 1.12
Ang.: 22.500
Block State
None
shear-n shear-p
shear-n shear-p tension-p
shear-n tension-n shear-p tension-p
shear-p
tension-n shear-p tension-p
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Picture 5

Final comment concerning the negative pore pressures:

As has been demonstrated above, there can be a great influence of negative pore pressure on the deformations and stability of a tunnel. Concerning the limitation of negative pore pressures an estimation of "realistic" values is required taking also geological aspects like the grain size distribution into account. The development of the pore pressures is influenced by several parameters like the in-situ stress state and in-situ pore pressure, the strength parameters of the rock mass and the mechanical characteristics of the pore water. In reality it would be very difficult to distinguish between time dependent pore pressure effects leading to delayed deformations and creep effects with the same effect. The influence of the ventilation on pore pressures is not treated within this paper. The coupled processes will or should be the subject of further research for the following years.

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