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TUNNELLING IN ALPINE FAULT ZONES EXCAVATION AND SUPPORT STRATEGIES

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ABSTRACT

Tunneling through faulted rock in general is associated with stability problems, high deformations and frequently changing stress conditions. Depending on the characteristics of the fault zone, different strategies for excavation and support of the tunnel must be applied. With the increased demand in Alpine base tunnel the issue of safe and economical tunneling is increasingly important. But also in tunnels with medium overburden squeezing conditions have been observed. Strategies applied in different regions are rather controversial. Some groups try to convey, that a certain system fits to any conditions. Unfortunately tunneling through fault zones is not that easy.
Extensive experience gained over the last decades from tunneling in the Austrian Alps as well as other alpine regions, combined with a continuous research has led to increased understanding of fault zones, as well as to a number of improvements in short term prediction and supports. Due to the heterogeneity of Alpine fault zones, the short term prediction of the rock mass behavior is an important issue. Several techniques have been developed to increase the reliability of short term predictions.
High displacements in general lead to unacceptable loads, respectively strains in conventional tunnel linings, leading to problems with stability and safety. In Austria a low-cost system has been developed, which allows one to control the stresses in the lining by integrating yielding elements into the lining. This system has been successfully applied on several projects. Improvements also have been made to grouted bolts, enhancing their performance.
Case histories are used to demonstrate the variability of different fault zones, and the development of the technology. It is emphasized, that only the combination of excellent technique, good workmanship, continuous engineering on site, up to date monitoring and interpretation techniques, as well as a suitable site organization lead to a successful mastering of the difficulties encountered, when tunneling through fault zones.

INTRODUCTION

Faults are elongated, complex zones of deformation, ranging from decimetres to kilometres in magnitude. For engineering projects the so-called brittle fault deserves our particular attention. This type of fault zone is generated in the upper 5 to 10 kilometres of the Earth's crust. A regular pattern of shear and tensile fractures has developed in brittle faults, reflecting the geometry of the strain field and, consequently, the orientation of the principal stresses (Mandl 1988, 1999). The brittle rock deformation, such as particle size reduction by crushing of grains and reorientation of grains by shearing, generates the characteristic fine-grained gouge (Scholz 1990, Twiss & Moores 1992). Low-temperature solution transfer substantially contributes to the alteration of fault rocks, in particular of gouge, through transformation and neoformation of clay minerals (Riedmueller 1978, Wu 1978, Klima et al. 1988). Brittle faults are geotechnically significant because of their substantial heterogeneity in rock mass properties. Brittle fault zones consist of randomly occurring units of more or less undeformed, unaltered rock, called “knockers" or “horses" (Goodman 1993). These mainly lenticular units exhibit a fractal distribution of dimensions, ranging from the micro scale to hundreds of meters in length and are typically surrounded by highly sheared fine-grained gouge and fractured, brecciated rock mass which appears to be flowing around the horses in an anastomosing pattern.

Fig. 1. Picture of a typical brittle fault, showing the extreme heterogeneity

The ratio of weak clayey gouge matrix to rock blocks of different sizes, shapes and strengths is extremely variable. Medley has introduced the term “bimrocks” to characterise tectonic block-in-matrix-rocks (Medley 1994, 1998, 1999).
Ground water flow directions and quantities may change dramatically across fault zones. Accordingly, effective drainage and support measures for underground excavations become extremely difficult.
Due to the heterogeneity of brittle fault zones the failure mode frequently changes. As an example of those changes the displacement vector plots in Figure 2 may serve. The two measuring sections are only 10 m apart, but show a different behaviour. The vectors in the upper plot show a nearly radial displacement. In the section of the lower plot obviously some shearing along the foliation and other discontinuities causes an uplift of the lower sidewall.

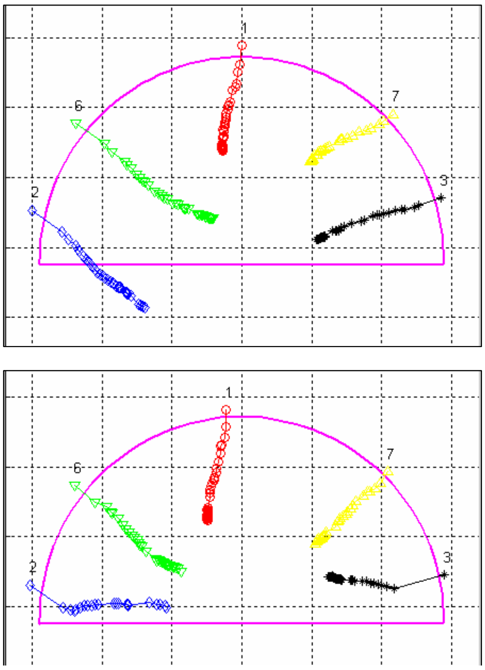

Fig. 2. Displacement vector plots of two measuring sections with a distance of 10 m, showing different behaviour

Brittle fault zones are a great challenge for geologists and engineers involved in the design and construction of tunnels (Schubert 1993). To successfully cope with severe geotechnical problems, which usually are encountered when tunnelling through a fault zone, a realistic three-dimensional geological model based on a geotechnically relevant investigation and characterisation of the fault zone has to be established. A sound geotechnical assessment of faults, followed by three dimensional modelling, has to continue through all design stages from route selection, preliminary, tender and final design to construction. This approach is essential to technically and economically optimise the design and construction of tunnels. Due to the frequently changing rock strength, deformability and groundwater conditions in brittle faults we have to provide construction methods with great adaptability to allow for uncertainties in the prediction of rock mass behaviour.

The principal geotechnical difficulties likely to be encountered when tunnelling through a fault zone are:

- instability of the face
- excessive overbreak
- excessive deformation by squeezing and/or swelling fault rocks
- instability of intermediate construction stages
- frequently changing stresses and displacements
- excessive water inflow frequently associated with flowing ground

Figure 3 shows an example of the heterogeneity of stresses as a result of tunnelling through a heterogeneous rock mass .

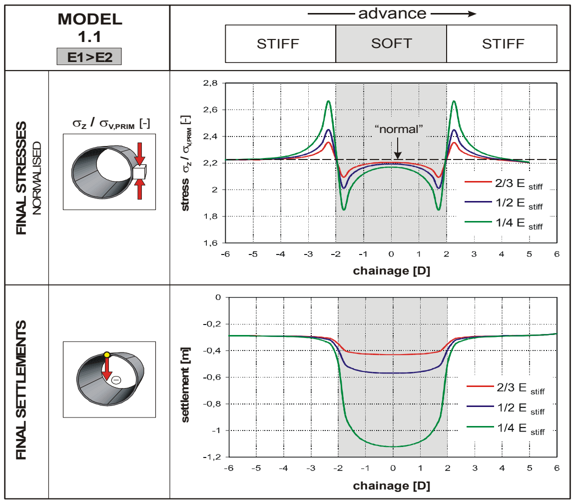

Fig. 3. Arching effect in a weak zone leading to stress concentrations in the stiffer material at the boundaries and the typical though shaped displacement distribution (numerical model).

The model consists of a sandwich-type rock mass with a weaker zone in the centre. Due to the different stiffness of the adjacent regions an arching effect develops in the weak zone, when a tunnel is excavated. The arching on the one hand leads to a relative stress increase in the stiffer material and a decrease in the weaker material in the area of the boundaries between weak and stiff material on the other hand (Grossauer 2001, Schubert et al. 2003).

CONSTRUCTION SEQUENCE

The standard excavation sequence when driving a tunnel in a fault zone in Austria is top heading, then bench, and invert. The excavation is mainly performed using a tunnel excavator with occasional blasting or cutting with a road header. Supports usually consist of rock bolts, shotcrete, wire mesh and steel ribs. The advance length in general does not exceed 1,5 m. Supplementary measures like forepoling, in extreme cases the installation of a pipe roof, or face support by bolting increase the stand-up time and stability of the face.
In other countries a preference for full-face excavation methods

can be observed. The stabilisation of the face in that case often requires extensive bolting or other auxiliary measures. Although this in many cases may be technically feasible, time requirement and costs are much higher than with a sequential excavation.

SHORT TERM PREDICTION AND DISPLACEMENT CONTROL

A main problem when tunnelling in squeezing ground is the heterogeneity of fault zones. Prediction of rock mass behaviour and thus proper choice of support, construction sequence, as well as determination of required overexcavation in heterogeneous rock mass is extremely difficult.
During the excavation of the Inntaltunnel Schubert (1993) observed, that the ratio between longitudinal and radial displacements considerably changes when the face approaches a zone of different stiffness. Using data from 3D optical displacement monitoring from different sites, Budil (1996) studied the phenomenon of longitudinal displacements, and found an interesting relation between displacement vector orientation and rock mass heterogeneity. This work was continued by Steindorfer (Steindorfer 1997, Steindorfer & Schubert 1997), who in detail analysed the development of stresses and displacements in heterogeneous ground during tunnel excavation (Figure 4). Numerous numerical simulations have been performed to verify the observations on site, and to improve the predictive capacity. Using the spatial displacement vector orientation, the prediction of changes in rock mass stiffness ahead of the face is possible. It also allows assessing the primary stress field (Golser & Steindorfer 2000). The findings have since been used successfully on various sites with heterogeneous rock mass conditions, helping in adjusting construction sequence, support and overexcavation.

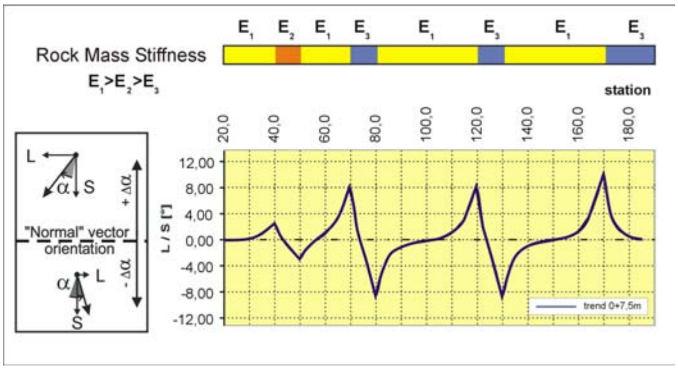

Fig. 4. Variation of displacement vector orientation when tunnelling in a heterogeneous rock mass, result of numerical simulation

Figure 5 illustrates the variation in the displacement vector orientation in a tectonic melange, where a stiff block was embedded in a fault zone.
Experience gained during the last ten years on sites with poor rock mass conditions shows that significant changes in the rock mass stiffness or faults can be detected several diameters ahead of the tunnel face. This allows adjusting excavation and support in time and at the same time minimises the requirement of reshaping, as well as additional support after excavation.

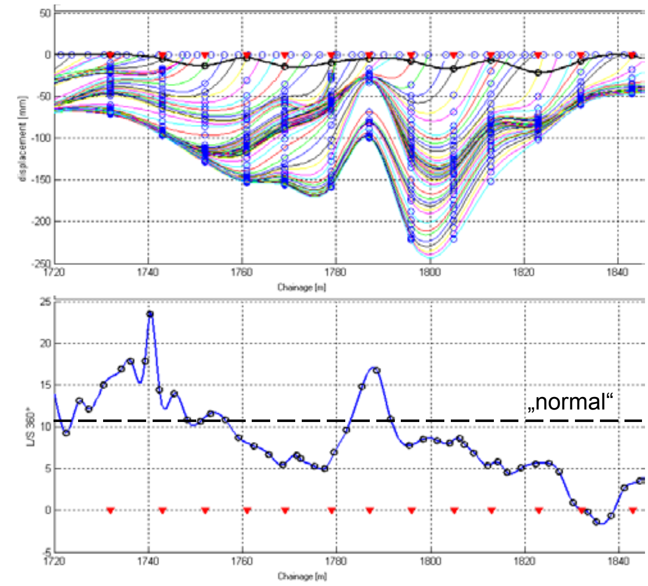

Fig. 5. Settlements and displacement vector orientation trend for a side wall point at the tunnel Spital, Austria.

PREDICTION OF DISPLACEMENTS

Tunnel convergence in highly stressed rock will always be an important issue. It determines the amount of required overexcavation, influences machine and equipment selection, as well as the support concept. When underestimating the convergence, extremely costly reshaping is required. When overestimating the amount of displacements, in general the "unused" overexcavation has to be filled with concrete, which is comparatively cheap. On the other hand, choosing the overexcavation on the too safe side does increase displacements and the amount of required support.
Because of the importance of the prediction of tunnel closure, it is surprising, that not much research has been done in this direction so far. Using analytical or numerical methods only does not appear to lead to sufficiently accurate results because of the huge number of parameters involved.
Sellner (2000) developed software (GeoFit), which is based on analytical functions developed by Guenot, Panet and Sulem (1987) and the modified functions proposed by Barlow (1986). To determine function parameters an expert system in combination with Artificial Neural Networks (ANN) can be used. The expert system consists of numerous site date, stored in the data base system DEST (Liu et al. 1997), numerical simulations, and monitoring data. Numerical simulations focus on the influence of support and excavation sequence on displacements, while monitoring data are used to predict the rock mass structure ahead of the face.
The code GeoFit has options of simulating sequential excavation, and allows to modify supports, and also to simulate the installation of additional support. Thus various options of excavation sequence and support capacity can be studied, and the

optimal solution chosen. One feature is the prediction of the development of the displacements. Continuously comparing the measured displacements to the predicted ones allows an assessment of the “normality” of the stress rearrangement process after excavation. Common practice on most sites is a simple visual inspection of the displacement history plots. The assessment of the “normality” of the displacement history can to a certain degree be done this way in the case of a continuous advance, but such a visual inspection becomes very difficult and unreliable in case of a discontinuous excavation.
Figure 6 shows the prediction of the displacements for a section of an alpine tunnel three days after excavation. Figure 7 shows the comparison between predicted and measured values.

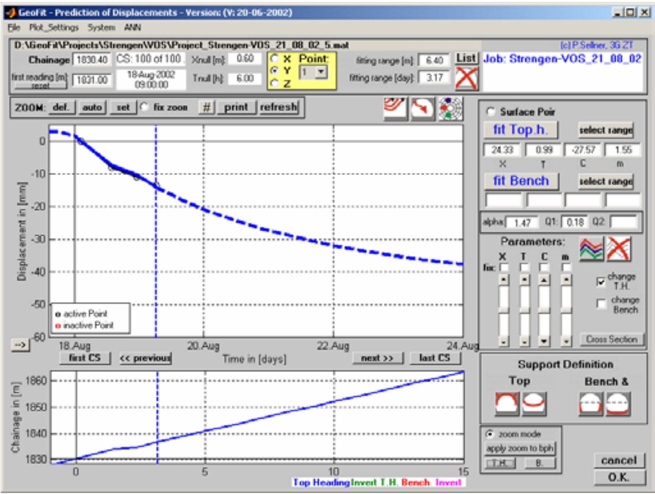

Fig. 6. Prediction of displacements at the Strenger tunnel; the dashed line indicates the predicted development of the displacements

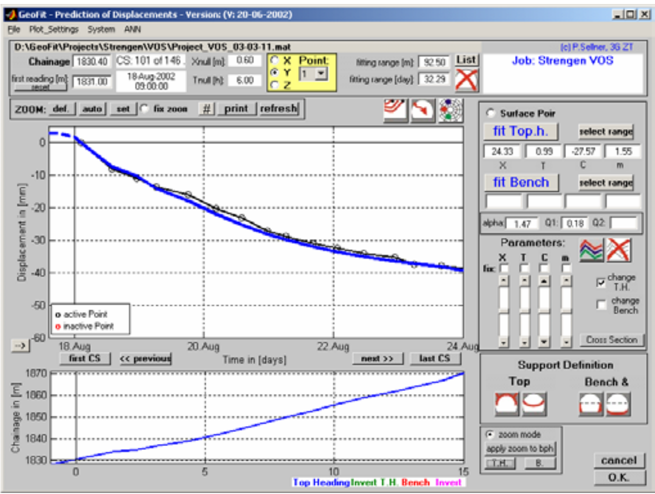

Fig. 7. Comparison between predicted and measured values at the Strenger tunnel.

Deviations from the “normal” behaviour can be detected early, thus enabling contingency measures to be implemented in time. This capacity of the program considerably enhances the safety and economy of a project. Such an example of a deviation of the measured values from the predicted ones is shown in figure 8. The tunnel is excavated in phyllites, the steeply dipping foliation striking approximately parallel to the tunnel axis. The overburden is approximately 600 m. In an intensely faulted section a wedge was squeezed up along the foliation and slickensides from the invert, leading to a relaxation. In figure 8 it can be clearly seen that the development of the displacements considerably deviates from the predicted ones, starting around the beginning of April.

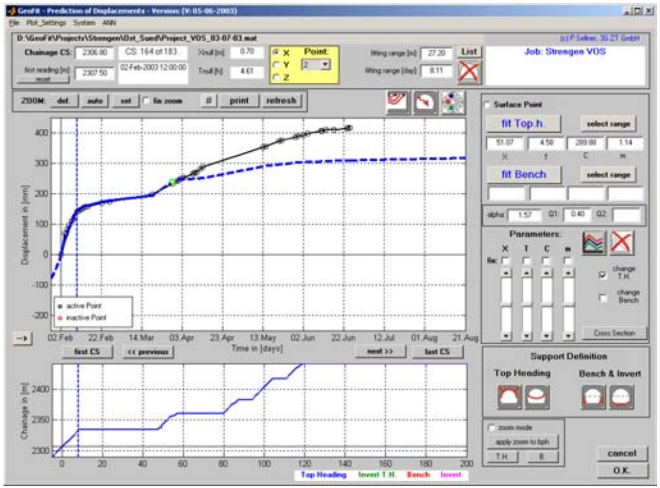

Fig. 8 Deviation of the measured horizontal displacements of left sidewall point from the predicted ones due to failure of the invert (Strenger tunnel, Austria).

SUPPORTS FOR TUNNELS IN SQUEEZING GROUND

Although rock bolts and shotcrete in many cases proved to meet the requirements, there is still some room for improvement of supports for squeezing ground. Blümel (1996) investigated the bond between rock bolts and grout under squeezing conditions in numerous laboratory tests and FE simulations.

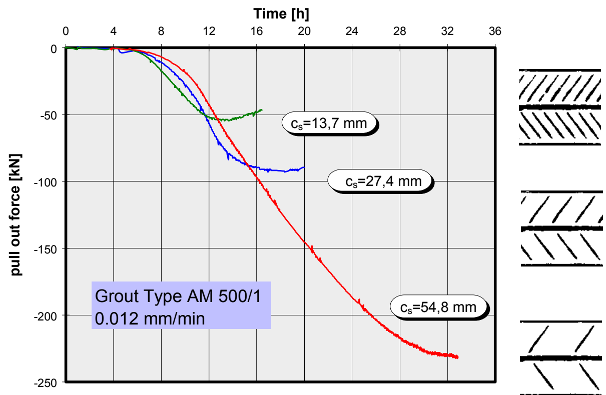

Fig. 9. Performance characteristic of 3 bolts with different rib distance, using the same grout type and displacement rate

This work yielded a recommendation for a modified rib geometry, which considerably improves rock bolt performance (figure 9). Support types and quantities have to be adjusted according to expected deformations and potential failure mechanisms of the rock mass. For instance, bolt length and

bolting pattern are mainly determined by the rock mass structure, intermediate construction stages and by the geometry of possible shear failures.
In squeezing conditions stiff supports in many cases cannot sustain the loads, respectively the strains developing. Destroyed linings are the consequence, which require a considerable effort in repair and maintenance. In addition, continuous lining failures are a safety hazard for the crew, even if the overall stability is not an issue, and part of the capacity is lost due to the failure, again increasing displacements.
This has been a problem in the past, as most supports, such as steel sets or shotcrete do not provide enough ductility over the range of displacements encountered in squeezing rock. To increase flexibility of the lining previously gaps in the shotcrete lining have been left (Rabcewicz & Hackl 1975, Schubert 1993). To prevent loosening of the rock mass and restrict the displacements, a lining with considerable resistance has to be applied right at the face. On the other hand the lining should have enough ductility to avoid shearing.
To combine ductility with resistance, a support system was developed and first applied at the Galgenbergtunnel" in Austria (Schubert et al. 1996). The basic idea of this support system is to integrate ductile elements into relatively stiff standard supports. The system consists of sets of concentrically arranged cylinders, yielding a nearly bilinear load line (Figure 9). By varying number and dimensions of the so called Lining Stress Controllers (LSC), the system can be designed to the capacity of the linings used and displacements expected. Field applications and numerical simulations have shown the effectiveness of the system (Schubert & Moritz 1998, Moritz 1999). A big advantage of the system is that it is not sensitive to even abrupt changes in rock mass quality, as the capacity of the yielding elements in all phases is lower than the capacity of the lining. Figure 10 shows a yielding element after testing in the laboratory.

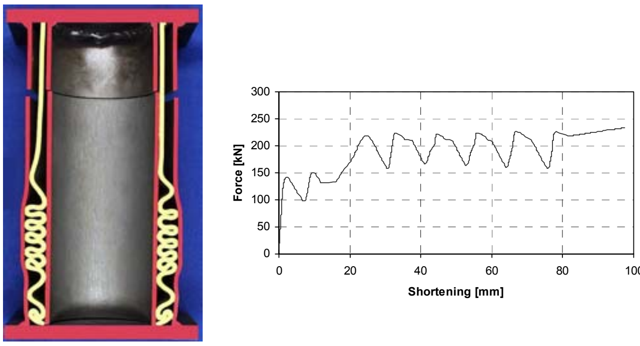

Fig. 10. Yielding element after laboratory testing (left), and load line (right)

At the Semmering base tunnel in a weak phyllitic rock mass section with an overburden of approximately 600 m the lining was severely damaged, requiring considerable repairs. It was decided to use yielding elements for the following section. Three rows of yielding elements have been used (figure 12). With a radial displacement of 200 to 300 mm the lining remained undamaged, showing the effectiveness of the system.

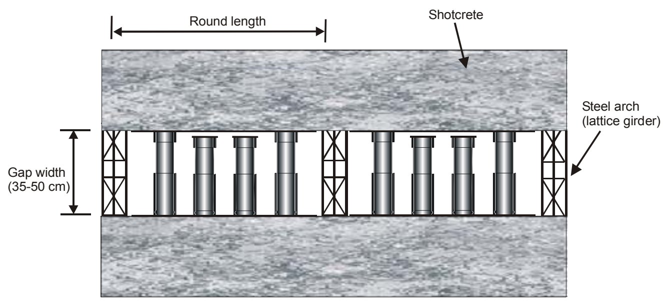

Fig 11. Typical assembly of LSC groups between steel arches

Fig 12. Three rows of yielding elements installed in the shotcrete lining at the Semmering base tunnel

In a big scale yielding elements have been recently used also at the Strenger tunnel project in the western Alps of Austria.
For the design of the yielding elements the time dependent development of the shotcrete strength and the deformation modulus, as well as creep and shrinkage are considered. For the calculations a spreadsheet program (LSC_DIM) was developed. Time dependent displacements are imposed on the combined LSC-shotcrete lining, and the resulting load developing in the lining compared to the shotcrete strength. The model used for the calculations assumes circular tunnel geometry and radial symmetric displacements. The formulations are based on the “rate of flow method” (England & Illston 1965), with some modifications (Schubert 1988 and Aldrian 1991). The routine uses an iterative process to guarantee compatibility of the loads in the LSCs and the lining.
The results allow the development of the strains and stresses of the system to be evaluated and compared to the materials capacity. Varying the capacity, arrangement and length of the LSCs and the number of sets used in the cross section, as well as lining thickness allows the support to be optimized in terms of the stress intensity.
Figure 13 shows an example of the development of the stress intensity factor in the shotcrete lining in the case of a closed lining (dashed line) and with the integration of two sets of LSC groups (solid line) for a tunnel of approximately 13 m diameter and a final radial displacement of around 200 mm. The advance rate was assumed at 1 m per day. It can be clearly seen, that the closed lining would fail within the first few days, while stresses

of the ductile lining are at all times lower than 40% of its capacity.

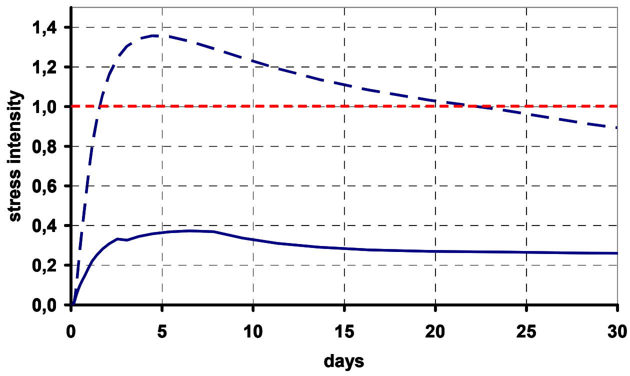

Fig. 13. Development of stress intensity factor in the shotcrete lining without (dashed line) and with yielding elements (solid line)

The rate of the face advance has a dominant influence on the development of the displacements over time, while the strength development of the lining is mainly time dependent. Different excavation rates or a discontinuous excavation have to be evaluated to ensure loads in the lining also remain below its capacity under such conditions. Figure 14 shows the development of the stress intensity in the lining for the same rock mass conditions as used for the example in figure 3, but with an excavation advance rate of 3 m per day. It can be seen, that the utilization of the linings strength within the first few days is considerably higher compared to the example shown in figure 12 due to the higher initial displacements.

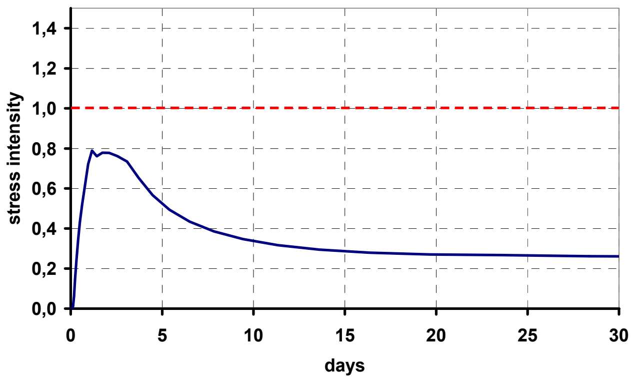

Fig. 14. Development of the stress intensity factor for the same rock mass conditions, but with higher advance rate

During construction monitored data can be used to calculate the actual stress intensity. With the recently developed code GeoFit© (Sellner 2000, 3-G Homepage) the development of the displacements of the tunnel can be predicted, which also allows the development of the lining stresses to be predicted.
The system of the yielding elements meanwhile has been further developed, and currently is used also in a test section with the final lining in a railway tunnel in Spain to cope with swelling of the invert.
In many countries, heavy steel sets and thick liners are preferred, obviously due to difficulties in rock bolt installation. The disadvantage of those systems is the relatively brittle behaviour of the support, which under unfavourable circumstances may lead to sudden collapses, while a densely bolted rock mass behaves more ductile. Geological conditions, the local stress field, as well as the size of the tunnel should govern the choice of supports. Different combinations of support elements may lead to comparable results in terms of displacements, and costs. For reasons of stability, and reserves against unforeseen conditions, a combination of steel arches, shotcrete with integrated yielding elements, and systematic rock bolting is recommended.

CONCLUSION

Tunnelling in squeezing rock definitely is one of the most challenging tasks in tunnel engineering. Strong regional preferences for methods to tackle stability and deformation problems can be observed. Prediction of rock mass behaviour is a crucial issue for successful tunnelling.
Experience with tunnels in squeezing ground in the Alps during the last decades has led to improvements in short term prediction, supports, and monitoring techniques. The increasing share of TBM excavations in poor ground calls for further development in order to prevent disastrous applications.
The heterogeneous nature of fault zones requires a continuous updating of the ground model, as well as adaptation of excavation and support to allow safe and economical tunnelling. Even with increasing mechanisation, there will remain a good share of geotechnical engineering also during construction in squeezing ground conditions.

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