RECENT EXPERIENCES WITH SQUEEZING ROCK IN ALPINE TUNNELS

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ABSTRACT

Tunnelling through brittle fault zones requires special considerations during all phases of a tunnel project from design to construction. The geotechnical problems during construction relate primarily to the substantial heterogeneity of fault zones in terms of geotechnical properties of the rock mass. Geotechnical aspects of fault characterisation and problems of investigating a tunnel in a fault zone are discussed. It is demonstrated that a technical and economical optimisation of construction can only be achieved by a flexibility requirement within the contractual set-up, which allows for adaptation of excavation and support to the conditions encountered on site. In this context it is emphasised that based on a continuous evaluation of geological and geotechnical monitoring data, the predicted geological model is adjusted to the actual conditions. Based on the updated model, excavation and support can be adjusted accordingly. The principal geotechnical difficulties and adequate measures during excavation related to tunnelling in brittle faults are discussed. The development of software to collect and interpret data, tools to aid the decisions on site, as well as up to date supports for fault zones are demonstrated by case studies of shallow and deep tunnels.

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 (1,2).

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 (3,4). Low-temperature solution transfer substantially contributes to the alteration of fault rocks, in particular of gouge, through transformation and neoformation of clay minerals (5,6,7).

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" (8). These mainly lenticular units exhibit a fractal distribution of dimensions, ranging from the microscale 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. 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 (9,10,11).

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

Groundwater conditions are also highly variable in brittle fault zones. Water pressures and flow direction may change dramatically across fault zones. Accordingly effective drainage and support measures for underground excavations become extremely difficult.

Brittle fault zones are a great challenge for geologists and engineers involved in the design and construction of tunnels (12). 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 driving a tunnel 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

CASE HISTORIES

The following case histories deal with tunnels in fault zones with moderate overburden [13]. Although conditions on the first glance are rather similar, rock mass behaviour showed to be quite different.

Fault Zone at the Inntaltunnel

The "Inntaltunnel", a double track tunnel 12.756 m long, penetrates quartzphyllites ("Innsbrucker Quarzphyllit") which form the Pre-Alpine crystalline basement of the Northern Calcareous Alps. The maximum overburden thickness is approximately 350m. Due to its position at the boundary between Northern Calcareous Alps and the gneiss massif of the Central Zone, the quartz phyllite series have been subjected to intensive tectonic deformation which has generated shear zones oriented generally in the direction of strike of the phyllites [14]. A major brittle fault zone intersecting the tunnel axis at an acute angle between stations 2700 m and 4800 m (North Lot), has caused difficulties during tunnel excavation. This fault zone consists of alternating layers of clayey gouge, cataclasite and phyllite, varying in thickness from decimetres to tens of metres. Dipping 20° to 45° to the NW, the fault zone cuts through the gently SW dipping phyllites.

Excavation and support methods were governed by the considerable displacement typical for the area (see Figure 1). Near station 2.680 m, deformations exceeded the deformability of conventional shotcrete – rockbolt systems support, resulting in shear failures of the shotcrete lining and broken bolt heads. To increase its deformability, the shotcrete lining of the top heading was divided into three segments with two deformation gaps, a method used frequently in similar projects [15,16,17]. The rock bolt density was increased and the round length was kept at 1,0 m. The invert arch was installed in a close distance to the bench excavation, the distance to the heading face being between 100 and 120 m.

From station 2.680 m to approximately station 3.170 m, crown settlements ranged from 3 to 15 cm after one day, with final settlements of 30 cm to 60 cm. As the deformation gaps in sections with higher displacements closed completely, occasionally leading to severe lining damage, the number of deformation gaps in the top heading lining was increased to four, plus free footings. Maximum roof settlements reached 120 cm around station 3.230.

The end of the fault zone was reached near station 4.800 m and crown settlements over the last 1.500 m of the fault zone continuously decreased, averaging around 40 cm.

The number of rockbolts with lengths of 6 and 8 m varied from 17 to a maximum of 29 per linear meter of top heading throughout the fault zone, while the shotcrete thickness were kept constant at 20 cm.

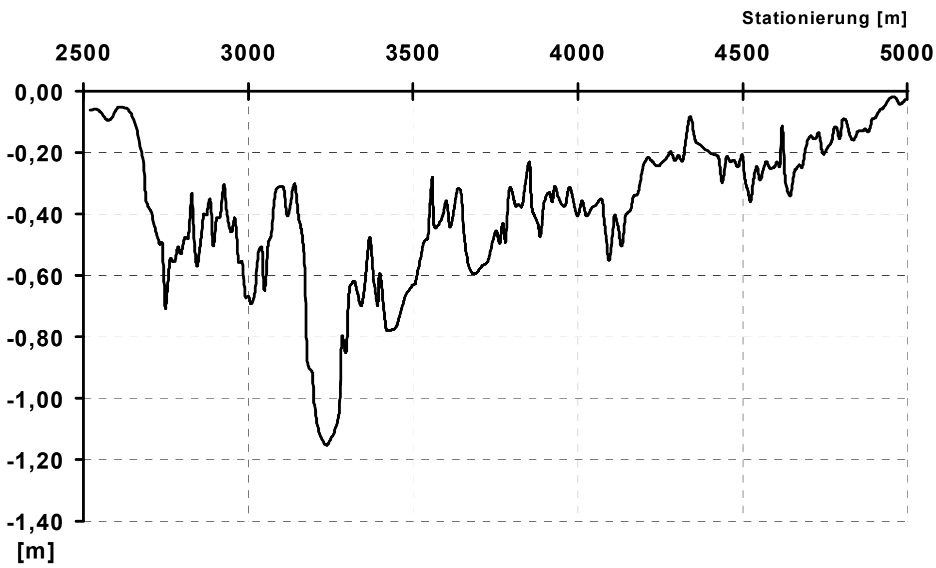

Figure 1**: Crown settlements along the fault zone at the Inntaltunnel**

The main influencing factors of displacement were shear planes striking more or less parallel to the tunnel axis and dipping to the NW. The combination of foliation and shear planes generated wedge type failures on the right sidewall, influencing the orientation of the

displacement vectors. The difference in longitudinal displacements between the left and right sidewall of several centimetres was remarkable.
In general displacement rates decreased smoothly indicating a “normal” stabilisation process. The varying stiffness of the rock mass within the fault zone made prediction of final displacements rather difficult. In a few areas, the amount of deformation was underestimated, requiring reshaping of the tunnel was required prior to installation of the inner lining.
Although the amount of displacement varied widely in short distances, the general behaviour of the rock mass was very similar throughout the fault zone and operations could continue routinely at an excavation rate of around 4 m per day.

Fault Zones at the Galgenbergtunnel

The NE -SW oriented ”Galgenbergtunnel” has a total length of 6.108 m. The maximum overburden thickness is 260 m. The tunnel cuts through extremely variable ground, consisting of gneisses, quartzites, phyllites, greenschists and marbles belonging to different tectonic units
The folded and faulted lithological series trending generally WSW - ENE intersect the tunnel axis at an acute angle. The two major fault zones, the ”Hinterberg” fault and the ”Haberl” fault, have proved to be excessively difficult to tunnel through [18, 19, 20).

Hinterberg fault

This fault zone, located between station 959 m and 1 342 m (heading Leoben), has developed between two units of massive marble. Source rocks are imbricated graphitic and carbonatic phyllites, greenschists and thinly bedded marbles of the Paleozoic, Upper Austroalpine Greywacke zone (”Veitscher Decke”). The orientation of foliation planes indicates a E -W trending syncline structure with a fold axis dipping gently to the west. Slickensided shear planes most commonly dip steeply to the SSE and occasionally to the SE and NE.
The main characteristic of this fault zone is its extreme heterogeneity concerning the ratio of soft, clayey gouges to variably fractured rock mass. Despite a chaotic fault structure, it has been confirmed by statistical evaluation that the amount of clayey gouges and the degree of rock mass fracturing increases significantly from NE to SW. It is pointed out that a disastrous collapse happened in July 1994 at the southeastern boundary of the fault zone.
The location of the Hinterberg fault zone was precisely predicted. Due to the heterogeneity of the rock mass, prediction of the tunnel performance was extremely difficult. Sections with low initial deformation exhibited long lasting displacements, while other sections stabilised rather quickly after high initial displacements. Determining the amount of overexcavation and support required was therefore extremely difficult. At the beginning of the fault zone, a rather stiff support approach was believed to limit deformations to amounts the lining could sustain. Due to severe damages, the method was soon changed to one similar used in the Inntaltunnel. The size of top heading, lining thickness, rock bolt density, and round length was comparable, but the invert followed the heading face in a closer distance than at the Inntaltunnel.
When the excavation approached the end of the fault zone, a sudden collapse at the face of the top heading occurred without warning. The ensuing investigation revealed that a combination of unfavourable circumstances caused the failure, including:

- Sudden alternations of relatively stiff and soft sheared rocks.
- Low friction angle of the fault gouge ($\phi = 12° - 14°$).
- Unusual primary stress situation.

From the trend of the displacement vector orientations, it could be deduced that an arch type initial stress situation exists within the fault zone. A lateral creeping of the fault material wedged in between the two massive marble units causes this unusual primary stress condition. This resulted in a higher maximum principal stress, and the development of potential shear failures in directions not usually experienced during tunnelling [21]. Long lasting displacements in certain sections of the fault zone required additional bolting and partial reshaping.

Haberl Fault

The ”Haberl” fault was encountered between stations 1.760 m and 1.990 m (heading Jassing Ost). The fault zone developed in graphitic phyllites and occasionally greenschists, meta-sandstones, calcareous phyllites and platy marbles of the greywacke zone. The undulating, most commonly sheared foliation planes generally dip gently to the south. Additional shearing, which generated clayey gouges and fault breccia, took place on moderately to steeply NE and SW dipping shear planes.

The ”Haberl” fault zone is characterised by pervasive homogeneous shearing which shows brittle and ductile deformation features. From the geotechnical standpoint, the most important feature of the fault zone is the alternating sequence of hard and soft rock layers. within the fault zone exists.

With the experience from the Hinterberg fault, it was agreed that the support and excavation approach should be modified for the Haberl fault [19, 22.] The modifications included:

- reduction of the top heading height to 4,5 m to increase face stability
- use of 8 to 12 m long regroutable self drilling bolts (IBI)
- integration of yielding steel elements in the shotcrete lining

The aim of the modifications was to reduce displacements in the excavation area and to guarantee rock bolt performance after big deformations, thus increasing stability and safety. Technically the solution was convincing. Final displacements did not exceed 15 cm, nearly a magnitude smaller than at the Hinterberg fault zone. At no time was there any sign of instability.

The reduction of displacements in the soft rock layers prevented high stress concentrations in the hard rock layers. Thus the risk of a brittle failure of the stiff layers as experienced in the Hinterberg fault zone was minimised.

Due to the time-consuming bolt installation and the reduction of the top heading height, the excavation rate could not be increased over an average of 1,70 m per day, increasing construction time and cost. On the other hand, no reshaping or repairs were required on the 350 m of tunnel where this approach was applied.

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. Using data from 3D optical displacement monitoring, Budil (23) 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 (24,25), who in detail analysed the development of stresses and displacements in heterogeneous ground during tunnel excavation. 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. Also assumptions of the primary stress field are possible by evaluating the spatial displacement vector orientation (26). The findings have been implemented in commercial geotechnical monitoring software.

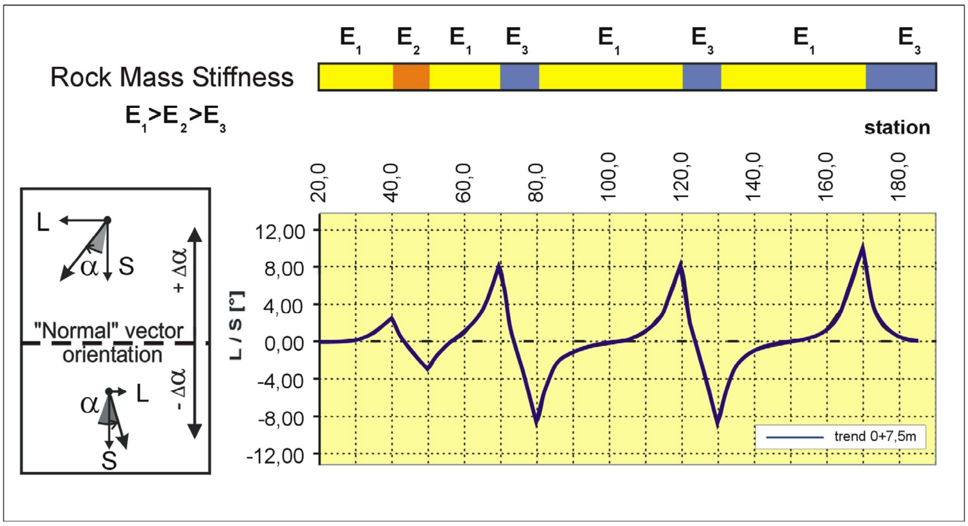

Figure 2. Variation of displacement vector orientation when tunnelling in a heterogeneous rock mass, result of numerical simulation

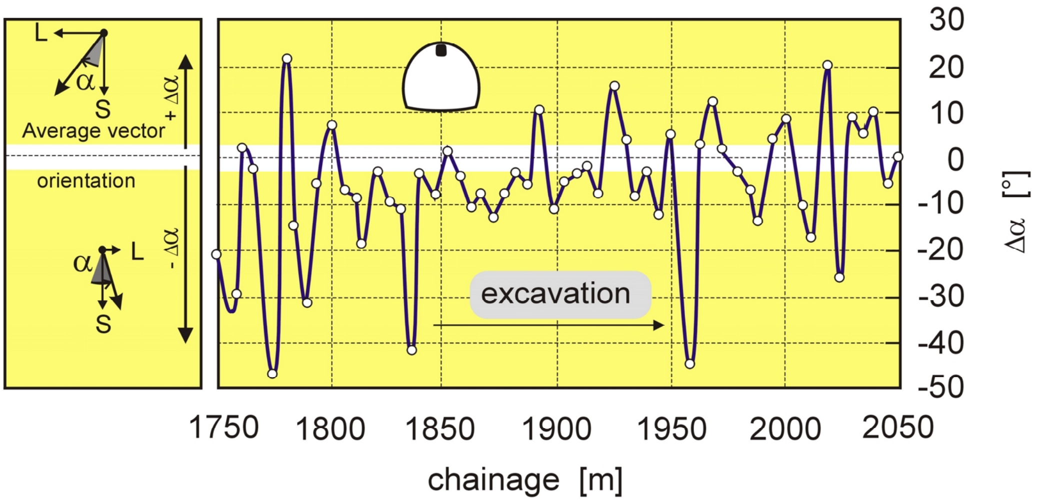

Figure 3. Monitored displacement vector orientation in extremely heterogeneous ground at the Haberl fault, Galgenbergtunnel

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 to adjust excavation and support in time an at the same time minimises the requirement of reshaping additional support after excavation.

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 to safe side, does increase the amount of required support and displacements.
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 developed a procedure (27), which is based on analytical functions developed by Guenot, Panet and Sulem (28) and the modified functions proposed by Barlow (29). 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 (30), 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 rock mass structure ahead of the face (25).

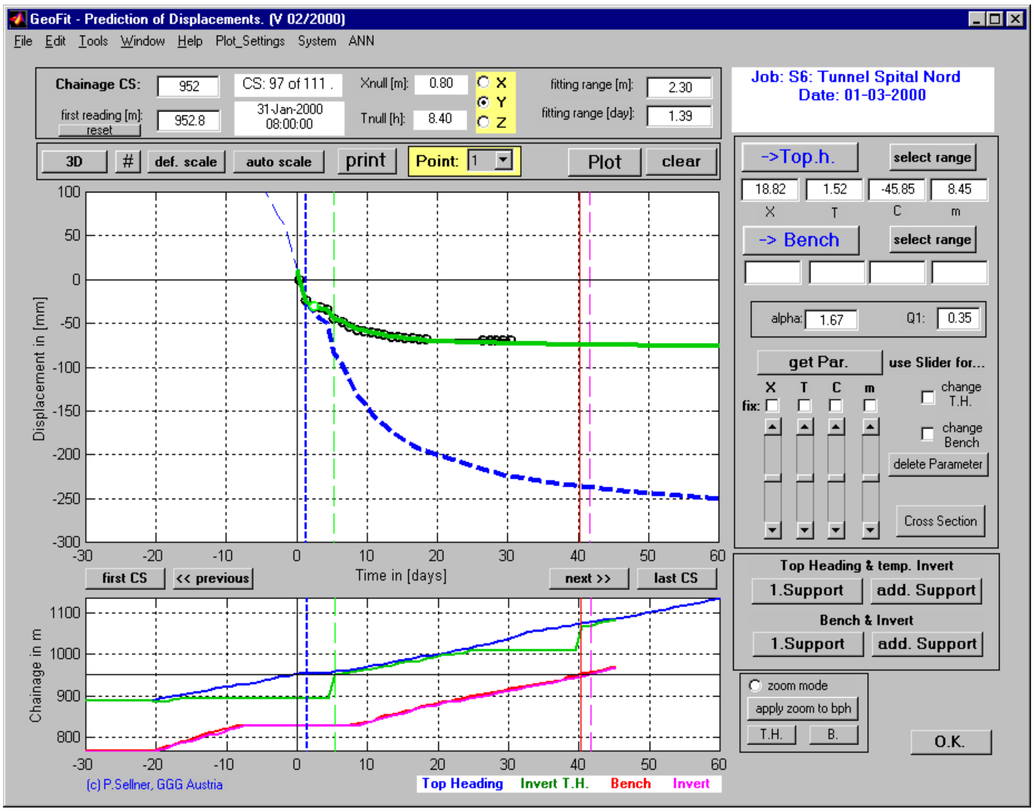

Figure 4 Prediction of displacements at the tunnel Spital. Dashed line: predictied displacements without temporary invert, solid line: displacements with temporary invert, circles: monitored displacement

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 investigated the bond between rockbolts and grout under squeezing conditions in numerous laboratory tests and FE simulations (31). This work yielded a recommendation for a modified rib geometry, which considerably improves rock bolt performance.

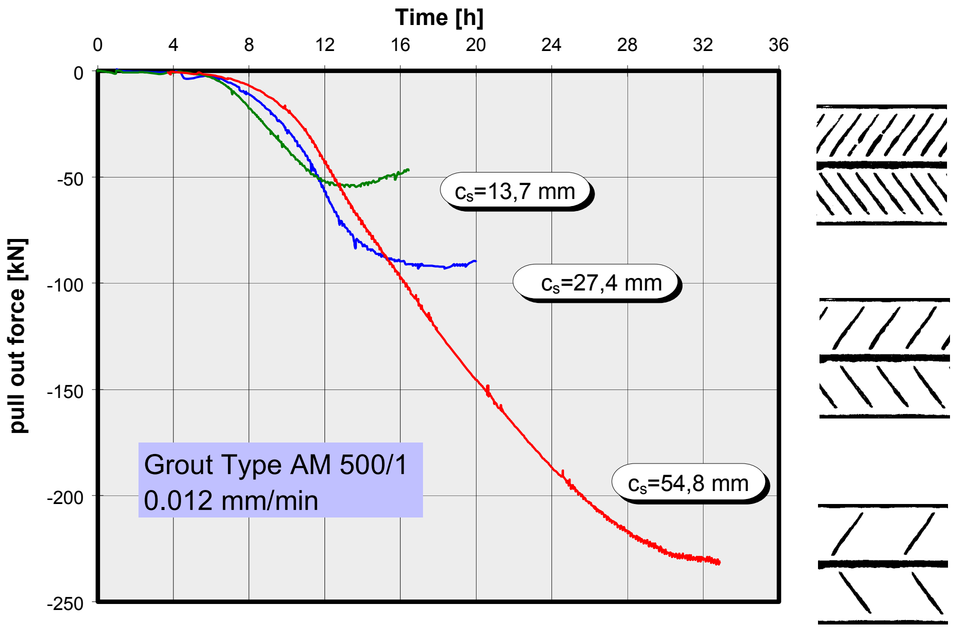

Figure 5. Performance characteristic of 3 bolts with different rib distance, using the same grout type; displacement rate 0,12 mm/hr.

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, continuos 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, 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 (32),33).
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 (34). 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 4). 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 (35, 36). 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 6 shows a yielding element after testing in the laboratory.

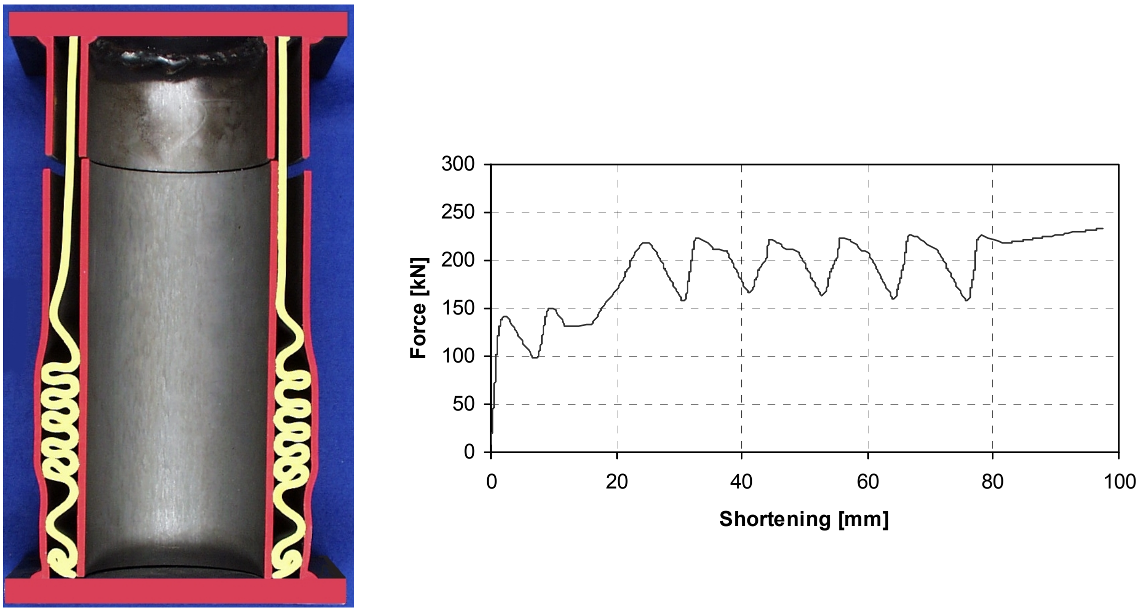

Figure 6. Yielding element after laboratory testing (left), and load line (right)

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 7). With a radial displacement of 200 to 300 mm the lining remained undamaged, showing the effectiveness of the system.

Figure 7. Three rows of yielding elements installed in the shotcrete lining at the Semmering base tunnel

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 rockbolted 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

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. 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.

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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