Górnictwo i Geoinżynieria • Rok 29 • Zeszyt 3/1 • 2005

Rudolf Pöttler, Franz Starjakob*, Daniel Spöndlin**

TUNNELLING THROUGH HIGHLY SQUEEZING GROUND — A CASE HISTORY

1. Strenger Tunnel

The Strenger Tunnel forms part of the S16 four-lane expressway between Landeck and Bludenz. The construction of this important East-West connection started in 1973 with the Arlberg Road Tunnel as its key constituent. The starting points were predetermind by the intersections at Pians in the East and at Flirsch in the West. As a result of the expected traffic load and the comparatively high gradient of 2.9%, both one-way, two-lane tubes were constructed at once. The north tube is 5.851 m long, the south tube 5.775 m, see Figure 1.

Fig. 1. Alignment of Strenger Tunnel [1]

* ILF Consulting Engineers, Feldkreuzstrasse 3, A-6063 Rum, Innsbruck

The tunnel cross-section was determined by 2 × 3.75 m wide, 4.70 m high roadways, two sidewalks and sufficient space for two fans above the clearance envelope (Fig. 2). When faced with stable rock conditions, a flat invert was sufficient, yet when rock pressure was expected, an invert arch was required. The excavation areas were 77 m^2 and 88 m^2, respectively.

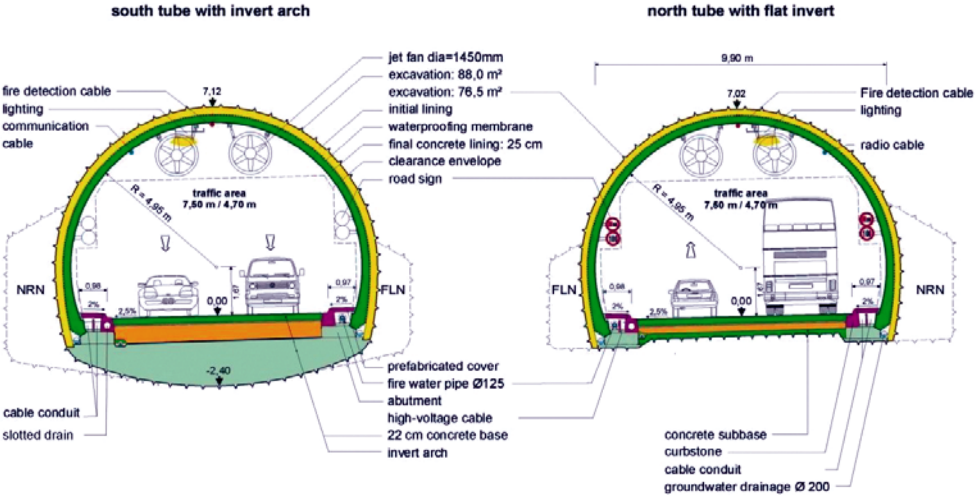


Fig. 2. Tunnel cross-sections [1]

To limit the excavation works and to save costs, the symmetrical axis of the tunnel was chosen to be perpendicular to the roadway and was thus adjusted to the crossfall [1, 2, 3, 4].

2. Geology

The Strenger Tunnel is located in the alpine region of Western Austria. The whole tunnel intersects a geological unit called „Landecker Quarzphyllit" which mainly consists of east-west striking and south dipping quartz phyllonites and quartzitic schists of low permeability. The maximum overburden amounts to 800 m. Due to the high overburden only one deep drill hole was carried out at the center of the tunnel; all other drillings were restricted to the portal areas. On the basis of these geological explorations, distinct fault zones were expected at acute angles. Different geotechnical models (especially in the fault area) have been developed based on the drilling and on geological mapping of the surface (Fig. 3).

Although the alignment was purposely chosen not to run parallel to the valley, fault zones extending over a stretch of approximately 2.500 m and striking almost parallel to the tunnel axis were ultimately encountered. The fault zones, several meters thick, consisted of mylonites and/or completely fractured material (kakirite) [3].

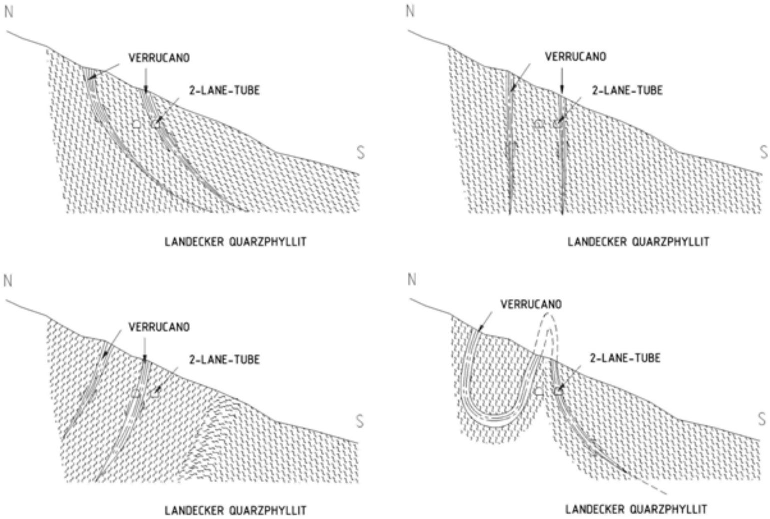


Fig. 3. Different geotechnical modells in the fault areas

In addition, the schistosity planes also parallel to the tunnel alignment, exhibited smooth and glossy surfaces with a high dislocation potential. Figure 4 clearly shows the difference between the predicted location of fault zones and the actual situation during the excavation process.

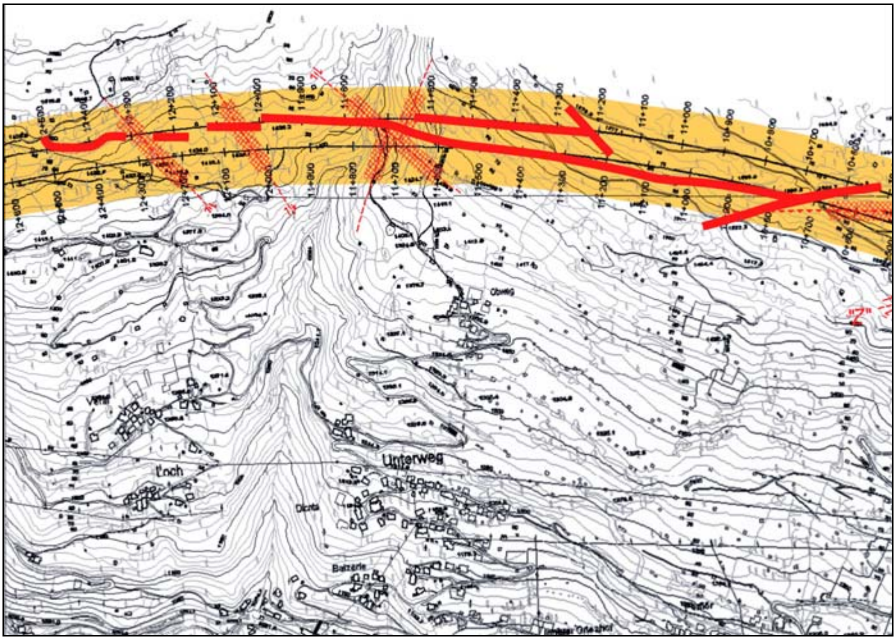


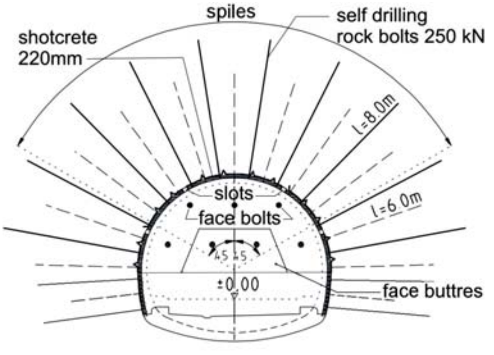
Fig. 4. Prognosis and orientation of fault zones

As the faults are more or less parallel to the tunnel alignment the poor geological situation in the fault zones had big impact to the tunnel construction. The primary lining was effected by high deformation and pressure caused by squeezing rock mass during a long distance.

3. Support Measures

At the design phase, highly squeezing ground was predicted for distinct fault zones with radial deformations of up to 600 mm. In line with the New Austrian Tunneling Method (NATM), a flexible shotcrete lining (220 mm thick) with incorporated slots, flexible steel ribs and heavy rock dowels (6 m and 8 m long) was envisaged for the squeezing ground (Fig. 5). In addition, yielding steel elements were designed to be introduced in the shotcrete slots. The cross-section was divided into a 6 m high heading, a 2.5 m high bench, and a 2 m high invert arch.

a)



b)

Fig. 5. Support measures for squeezing rock according to design (a) and excavated heading (b) [1]

The fault zones, sometimes occurring in bunches striking mostly parallel to the alignment, resulted in stable face conditions and large radial deformations. Depending on the fault zone configuration, the deformations varied considerably (Fig. 6).

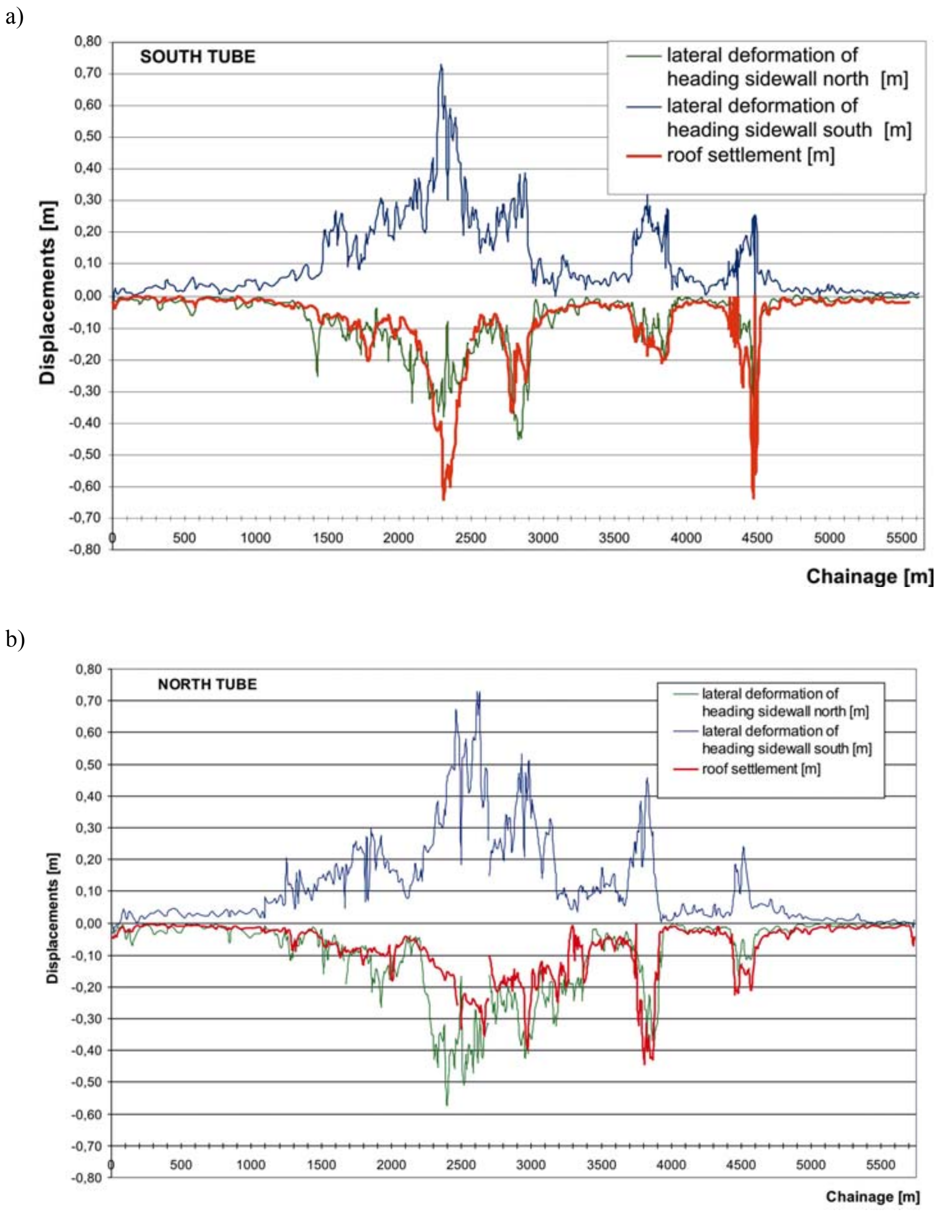
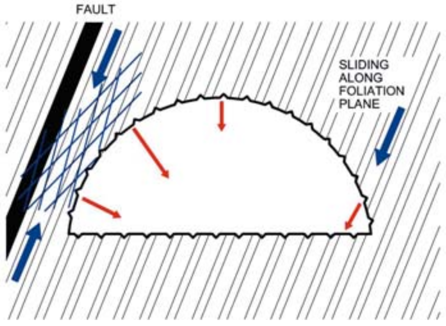


Fig. 6. Total displacements of: a) South Tube; b) North Tube [1, 2]

Asymmetric deformations developed as a result of the poor strength of the schistosity planes, aggravated by the presence of fault zones outside the tunnel walls. Due to the high compressive stresses, failure along the schistosity planes formed „rhomboedric chips" which led to strong dilatancy and high radial deformation, whereas at the opposite side sliding along schistosity planes resulted in smaller radial deformation. In areas with fault zones above the tunnel crown, roof settlements dominated (Fig. 7).

a)



b)

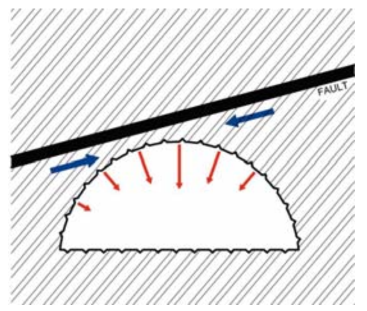


Fig. 7. Failure mechanism resulting from parallel striking schistocity (a) planes and fault zones (b)

During the heading excavation, a remarkable uplift of the heading invert of up to 1.5 m was observed. This heave, which occurred at some distance from the face and indicated continuous creep until excavation of the bench, could be explained by shear failure along the schistosity planes due to high horizontal stress components (Fig. 8).

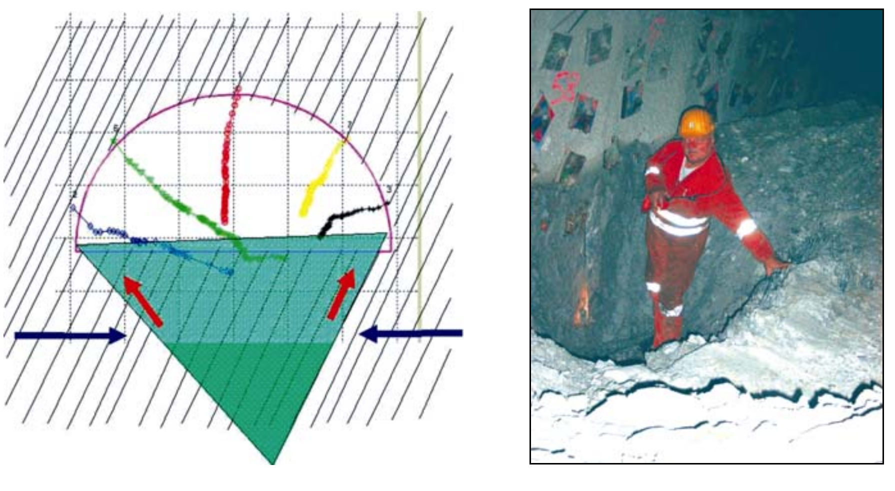


Fig. 8. Failure of heading invert

The excavation process caused progressive failure, which due to the parallelism of schistosity planes and fault zones, resulted in long-lasting deformations.

Deformations continued at varying creep rates of several tenths of millimeters per month after each excavation phase and only ceased after full closure of the initial lining. Due to variations in the deformation behavior, it was very difficult to predict the total amount of deformation to be expected.

In order to limit displacements along the schistosity planes, cement-grouted steel bars acting as dowels were the most effective support elements. For this reason the rock bolts were increased both in number and in length. In critical areas, 30 rock bolts ranging between 8 and 12 m in length had to be installed per meter of heading. The load-bearing capacity of the bolts was also increased to 350 kN. In order to reduce wearing of the rock bolt heads, special rock bolt plates were developed adding two steel pipes which are squeezed when large deformations occur.

In case of large deformations, the shotcrete lining cannot be applied monolithically but has to be subdivided by longitudinal slots adopting the NATM principles. Slots have been used for more than 30 years since they were introduced at the Tauern and the Arlberg Road Tunnels. In recent years it was found [5] that the introduction of yielding steel elements helps to increase the support resistance of the shotcrete, thus reducing deformations. These elements consist of longitudinal steel plates with a length of 1 m according to the advance rate with two or three steel cylinders in between.

The deformation behavior of these steel elements shall follow the behavior of the shotcrete lining. At the beginning, the elements shall be highly deformable since otherwise the young shotcrete would shear off underneath these elements. To find the proper combination of stiff steel plates and soft cylinders required, several on-site tests varying the thickness of steel plates, the number of cylinders and the weakening of cylinders by holes and/or wedge-shaped slots were undertaken (Fig. 9). The best fit was finally found to consist of 15 mm thick steel plates and three 4 mm thick steel cylinders, 127 mm in diameter with four 70 mm wide and 100 mm high slots.

a) b)

Fig. 9. Yielding steel elements at beginning of deformations (a) and within closed slots (b)

FE analyses using the Phase 2 program [6] were carried out to optimize the rock bolt pattern and to predict deformations during bench and invert excavation. In a first step, rock parameters were determined using the method developed by Hoek and Brown [7]. By back-analyses comparing calculated to measured deformations, the horizontal to vertical stress ratio (k0) was adjusted. The rock mass was modeled by joint elements and small continuous elements representing fault zones. Rock bolts were modeled as discrete beam elements. The yielding steel elements were modeled by limiting axial forces and bending moments in beam elements. The shotcrete was modeled employing a reduced modulus of elasticity [8, 9]. Variations (length of rock bolts, density of the rock bolts) allowed the support elements to be optimized.

4. Conclusion

Fault zones with squeezing ground which were expected to be encountered in short sections, unfortunately turned out to run parallel to the alignment in the central tunnel section for a rather long stretch. The rock mass behavior was characterized by stable faces and varying fracture processes resulting in high deformations of up to 700 mm in total. Adapting the support measures by introducing deformable rock bolt plates, installing yielding steel elements within shotcrete slots and increasing the rock bolt pattern, allowed shotcrete damages and rock bolt failures to be reduced, ensuring a safe excavation. Over-excavation, as a result of long-lasting fracture processes, was nevertheless underestimated, requiring re-profiling in certain areas. By introducing the inventive measures described, the NATM once more proved to be capable of mastering even highly squeezing ground.

REFERENCES

[1] *John M., Spöndlin D., Ayaydin N., Huber G., Westermayr H., Mattle B.*: Means and Methods for Tunnelling through highly squeezing ground: A case history of the Strenger Tunnel, Austria. RTC 2005, to be printed

[2] *John M., Spöndlin D., Mattle B.*: Lösung schwieriger Planungsaufgaben für den Strenger Tunnel. Felsbau, 1/2004, 18–24

[3] *Pöttler R.*: S16 Arlberg Schnellstraße Pians – Flirsch: Außergewöhnliche Defomrationen beim beim Vortrieb des Strenger Tunnels auf der S16 aus Sicht der Planung. Luzern, Schweiz, Vortrag DACH Treffen 2004

[4] *Westermayr H.*: S16 Arlberg Schnellstraße Pians – Flirsch: Außergewöhnliche Defomrationen beim beim Vortrieb des Strenger Tunnels auf der S16 aus Sicht der Bauausführung. Luzern, Schweiz, Vortrag DACH Treffen 2004

[5] *Moritz A.B.*: Ductile Support System for Tunnels in Squeezing Rock. Geotechnik Gruppe Graz, Heft 5, 1999

[6] Phase 2: 2D finite element program for calculating stresses and estimating support around underground excavations. Users's Guide, Toronto, Rocscience Inc., 1998–2001

[7] *Hoek E., Carranza-Torres C.T., Corkum B.*: Hoek-Brown failure criterion – 2002 edition. Proc. North American Rock Mechanics Society meeting in Toronto in July 2002

[8] *John M., Mattle B.*: Shotcrete Lining Design: Factors of Influence. RETC 2003, Proceedings 726–734

[9] *Pöttler R.*: Time-dependent Rock – Shotcrete Interaction. A numerical Shortcut. Computers and Geotechnics (9), 1990, 149–169