Burgenland's first tunnel takes its tribute – geotechnical retrospective of a tunnel collapse

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ABSTRACT: The tunnel Rudersdorf, which is part of the S7 Fürstenfelder motorway is a two-lane twin tube road tunnel with seven cross passages, and has a length of approx. 1.785 m. The overburden of Burgenland's first tunnel ranges between 2 and 72 m.
Especially the excavation works in the northern tube were accompanied by several geotechnical challenges, which required an adaption of the excavation concept and the support measures. In the subsequently excavated southern tube, situated in similar geological and geotechnical conditions, a collapse of the tunnel occurred after about 100 m of tunnel advance with a material ingress of approx. 1.600 m³. The paper discusses the rock mechanical processes which led to the collapse on the basis of the evaluation of monitoring data, the observed system behavior before the collapse, the observations during the re-excavation works of the collapsed tunnel section, and the encountered geological and geotechnical conditions.

Keywords: NATM, shallow tunnel, system behaviour, geotechnical monitoring, tunnel collapse.

1 INTRODUCTION

The S7 Fürstenfelder motorway will be constructed with a length of approx. 30 km between the interchange Riegersdorf and the Hungarian border at Heiligenkreuz. Near the city Fürstenfeld Burgenland's first tunnel is located. The tunnel Rudersdorf is a two-lane twin tube road tunnel, featuring seven cross passages (max. distance 250 m), and has a length of approx. 1.785 m. The cross section area is approx. 98 m². The overburden ranges between 2 and 72 m. The tunnel is located in the eastern section of the Styrian Neogene basin ("Fürstenfelder Becken"), which consists of Miocene deposits, built by marine base sediments. These rocks are heterogeneous sequences of silts, clays and fine to medium sand layers, with minor parts of gravel.

In the eastern part of the tunnel a terrain modelling (landscaping) with a length of approx. 360 m and a height up to 30 m was designed with excavated tunnel material as noise-mitigation measure. During the tunnel excavation and the simultaneous construction of the modelling, the unfavourable influence on the tunnel tubes became evident. Especially the excavation works starting from the eastern portal were accompanied by several geotechnical challenges.

In the northern tube excavation induced displacements up to decimetres were observed, which required an adaption of the excavation concept and the support measures. In the subsequently excavated southern tube, situated in similar geological and geotechnical conditions, a collapse of the tunnel occurred after about 100 m of tunnel advance with a material ingress of approx. 1600 m³. On the basis of the evaluation of monitoring data, the observed system behaviour before the collapse, the observations during the re-excavation works of the collapsed tunnel section, and the encountered geological and geotechnical conditions the paper discusses the rock mechanical processes and the causal relationships derivable therefrom, which led to the collapse.

2 OBSERVED SYSTEM BEHAVIOUR BEFORE THE TUNNEL COLLAPSE

The excavation works, following the principles of NATM, were performed with excavator with a subdivision of the cross section into top heading, bench and invert. The ring closure distance was set to 12 round lengths (round length between 1,0 and 1,2 m). The top heading, having massive elephant feet, was excavated in partial faces, with support core and face bolts. At chainage 45 the southern tube enters the area of the terrain modelling. From here, the terrain modelling increases steadily with a gradient of 20 % to 30 % up to a thickness of 23 m. Due to the low overburden of natural ground of approx. 6 m when entering the terrain modelling and thus an increase of dead load acting on the tunnel structure, an increase of tunnel displacements is to be expected.

Based on the chronology of the observed and measured system behaviour before the failure event one can deduce that the failure took place in two phases. In the first phase, lasting around two days, monitoring section MS76 showed a significant increase of displacement growth rates, although a decreasing displacement trend was already evident after the first follow-up measurement (Figure 1, Figure 2). Due to the progressive increase of vertical displacements, about 8 m behind the top heading face and total displacements of > 100 mm, a softening and loosening of the rock mass in the crown area and an increase of pressure acting on the shell is presumable. At this point it is likely, that shear failure already took place in the vicinity of the bench side wall, which could herald the beginning of a chimney type failure. In consequence of the overstressing of the rock mass, it attempts to transfer the loads in longitudinal direction.

Day 1:
Identical longitudinal displacements at monitoring sections MS65 and MS76; stresses due to longitudinal strains dissipate through formation of a crack in circumferential direction at chainage 56.

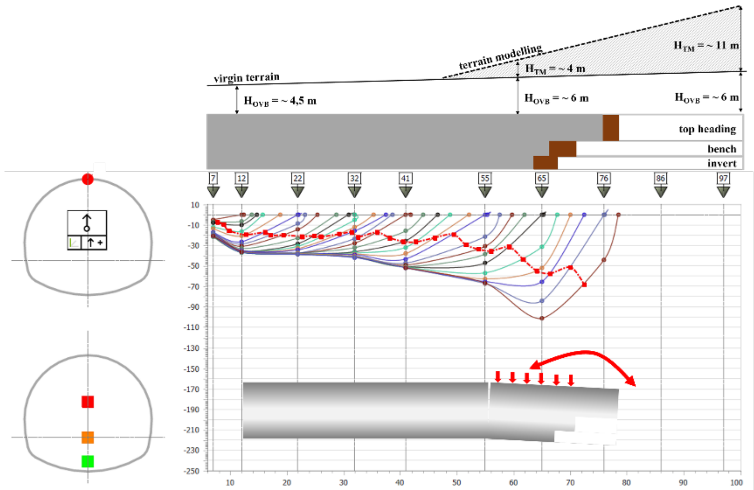

Figure 1. Situation at day 1; construction phases (top), deflection lines for vertical displacements for the crown point (middle), kinematics of the shotcrete shell (bottom).

At this time, the unfavourable situation in the area of the partial cross section top heading and bench without ring closure becomes evident. The shotcrete shell features two adverse characteristics in the open bench section: Firstly, the shotcrete of the bench rests with its standard thickness on the ground. The elephant feet of the top heading have only limited effect since the ground in the vicinity of the elephant feet abutment gets disturbed by the bench excavation.

On the other hand, the partial cross section top heading/bench has low lateral stiffness due to the almost vertically aligned shell in the bench area. Mobilization of support pressure is only possible to limited extent until the complete ring closure is installed.

On day three, the measurements of monitoring section MS65 showed almost no increase of vertical displacements (Figure 2). A stress transfer behind the face, in the area with already installed ring closure, is not observable. Rather, stresses relocate and concentrate in the section of the open bench. Thus, leading to further strain concentration and overstressing in the ground and hence formation of new shear failures and existing shear bands propagate outwards and upwards. Due to the low thickness of natural ground the development of a longitudinal arch is limited, and the ground is forced to transfer loads solely via transverse arches, resulting in a high degree of utilisation of the ground's shear strength. The high vertical displacements lead to tilting of the stiff shotcrete shell in the area of the already installed ring closure. This in turn imposes high longitudinal strains on the shell in the crown, which became evident by the formation of cracks in circumferential direction.

Day 3:
Significant displacement increases at monitoring section MS76 (open bench); load transfer to the rear part (increase of vertical displacements 10 m behind the ring closure); constantly identical longitudinal displacements at monitoring section MS65 and MS76; formation of a crack in circumferential direction at chainage 65.

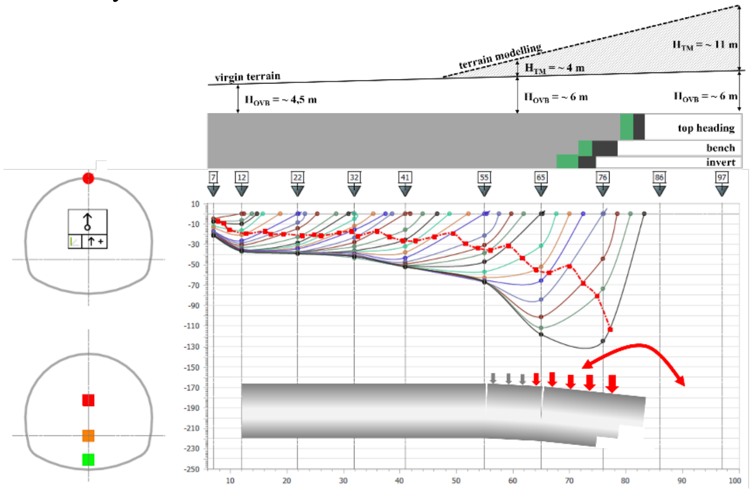

Figure 2. Situation at day 3; construction phases (top), deflection lines for vertical displacements for the crown point (middle), kinematics of the shotcrete shell (bottom).

Day 6:
At day 4 and 5 identical longitudinal displacements at monitoring section MS65 and MS76 (further crack opening at chainage 56 and 65). Since day 4 no further vertical displacements at monitoring section MS65; Stresses concentrate in the ahead-lying area of the open bench; Formation of further cracks in circumferential direction at chainage 71 and 78, reduction of distance between cracks and top heading face (> 20 m → 12 m).

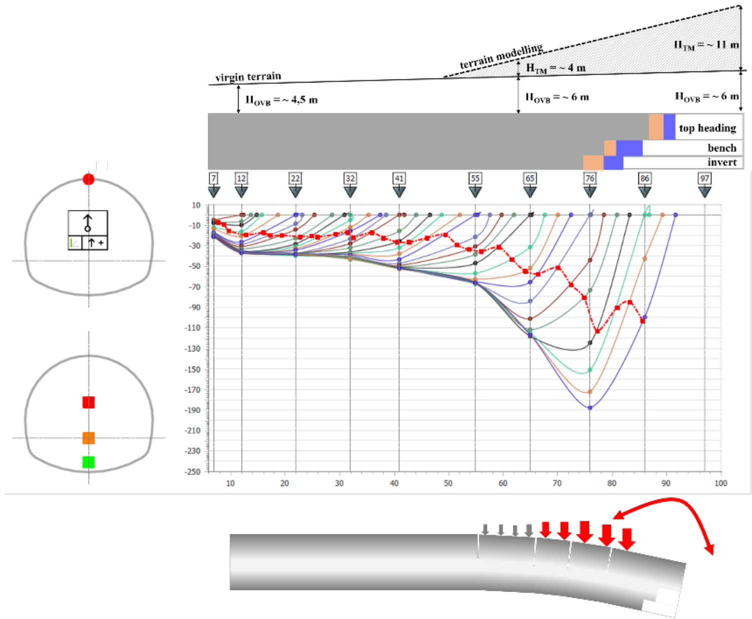

Figure 3. Situation at day 6; construction phases (top), deflection lines for vertical displacements for the crown point (middle), kinematics of the shotcrete shell (bottom).

By considering the advance-oriented 3D monitoring data evaluation at day five, it seems the situation becomes less critical (orange deflection line in Figure 3). The ascending tendencies of trend lines may be interpreted insofar that the observations of the last days might have been a local problem.

The developments from day six on initiate the second phase of the failure process: Monitoring section MS86, at that time situated in an open bench area, shows high initial displacements (Figure 3). Due to high displacements one can assume that overstressing of the ground with formation of shear bands developing towards surface now also took place in the area around MS86. The overstressing of the ground and the ever-decreasing capability to transfer loads via longitudinal and transversal arches now covers a tunnel area of at least 20 m (approx. chainage 65 to chainage 86). The loss of the ability of the ground to bear and transfer loads is also reflected by the increase of vertical displacements at monitoring section MS76, whereby ring closure was already 6 m ahead of this section. The shotcrete shell is subject to further additional loads in areas that were already exposed to higher loads the days before. The excessive loading of the shell was revealed by more and more frequent formation of shear cracks in the shotcrete shell, in combination with a reduction of the distance between place of crack origin and tunnel face. Due to the radial perforation, a shear force transfer in the shotcrete shell is significantly impaired. Starting from chainage 56 the shotcrete shell is separated in approx. 10 m long slices in longitudinal direction. The radial cracks at chainage 56, chainage 65 and chainage 71 already show crack widths to a degree where a load transfer by frictional coupling in the formed circumferential joint is highly limited.

The concentration of loads on an increasingly smaller area becomes evident by the observations on day seven, where shear cracks in the shell were already developing at the transition bench/invert (Figure 4). The immediate formation of a damage (cracks) when the impact changes (e.g. bench excavation) is an indication that the load-bearing capacity limit of the shotcrete shell had already been reached. The tunnel support is now no longer able to follow the complex kinematics and is increasingly evading the load. On day seven, the overstressing of the rock mass and the shotcrete shell have progressed so far that a prevention of tunnel failure was highly unlikely.

Day 7:
No increase of longitudinal displacements at monitoring section MS65 and MS76; significant increase of longitudinal displacements at monitoring section MS86; loads concentrate on an ever-decreasing area (chainage 78 to top heading face); formation of a crack in circumferential direction at chainage 82 at the transition from bench to invert; load-bearing capacity of the shotcrete shell reached.

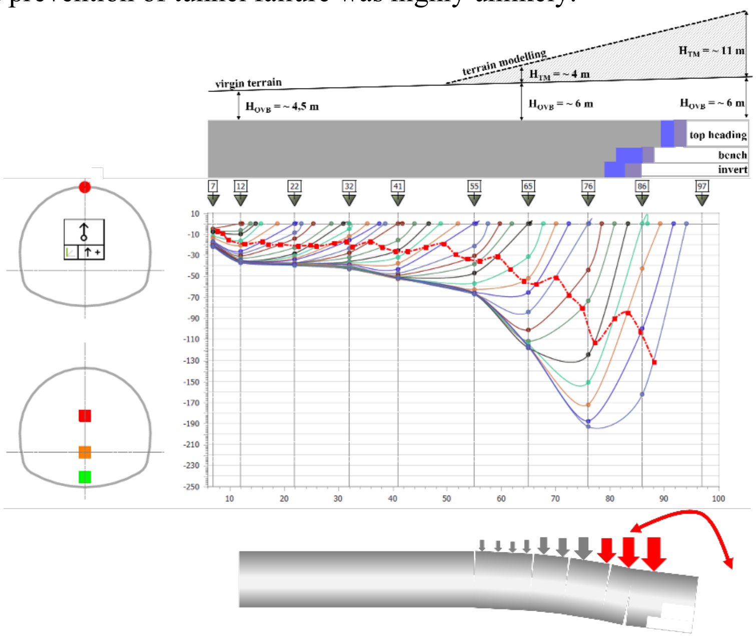

Figure 4. Situation at day 7; construction phases (top), deflection lines for vertical displacements for the crown point (middle), kinematics of the shotcrete shell (bottom).

Due to the high loads on the shotcrete shell because of the overlying material in the crown area on the one hand, and the shear failure of the ground in the bench area, which yields to pushing ground in horizontal direction on the other hand, longitudinal cracks form in the shotcrete shell (Figure 6, left). A chain reaction is triggered: The shotcrete shell below the longitudinal cracks provides almost no support pressure against the laterally pushing ground. This results in progress of the shear failure in the ground and finally in the chimney type failure which happened at day 8 (Figure 5).

Day 8:
Load-bearing capacity of the shotcrete shell is reached between chainage 80 and chainage 90. Circumferential crack at chainage 82 entirely formed; new cracks at chainage 85 and 88,1; progressive increase of longitudinal and vertical displacements at monitoring section MS86.

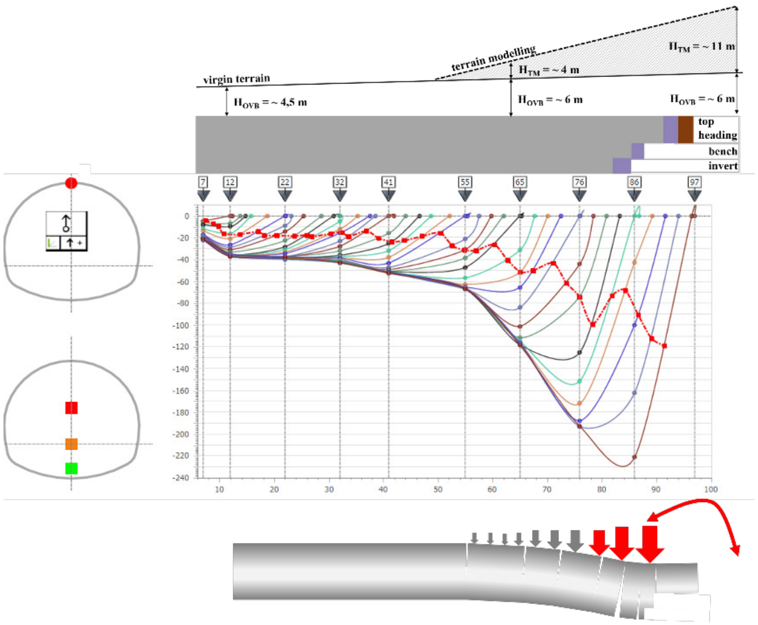

Figure 5. Situation at day 8; construction phases (top), deflection lines for vertical displacements for the crown point (middle), kinematics of the shotcrete shell (bottom).

Figure 6. Crack formation/disintegrated shotcrete shell at chainage 85, right sidewall (left); left sidewall, bench excavation to chainage 94,3 – intact elephant foot and radial rock bolts of the top heading (right).

3 FINDINGS AND MEASURES FOR THE FURTHER ADVANCE

The tunnel collapsed on a length of about 30 m and led to a material ingress of approx. 1600 m³. During the curse of the re-excavation up to chainage 78,5 an almost intact shotcrete shell at the side walls was observed. The separation edge of the collapsed shotcrete shell is situated close above the upper end of the elephant feet (area of lowest bending stiffness). The horizontally displaced elephant feet confirm the primary cause of failure of formation of shear failure at the bench: Up to chainage 80,9 the original shotcrete shell of the bench was intact, furthermore the invert was undamaged up to chainage 85,7. In this regard it became evident that the area of the shotcrete shell where longitudinal cracks formed, was completely damaged. In the portal section the bench and invert withstood the loading. A forward reaching cavity in the central lower face area of the top heading was determined. The lowered crown cap was intact. The parts of the shotcrete shell at chainage 87,7 showed a shearing of the shell above the elephant feet, analogously the area behind. During the re-excavation of the bench the sheared off elephant feet were found in their entirety (Figure 6, right).

The material pushes from outward in the area of the open bench, the elephant feet get sheared off and are shifted horizontally into the cavity. At the same time the shotcrete shell of the top heading, which is already severely damaged, fails. The cavity is thus filled almost simultaneously from the sides in the area of the bench and from above through the failed top heading shell. In the front area (projecting top heading shell) the shear fractures propagate at the upper end of the elephant feet. However, the shotcrete shell in the crown area still has sufficient residual load-bearing capacity so that it remains unbroken. This part of the shell is pushed downwards by the overlying material and pushes the elephant feet into the adjacent bench material.

To get further information about the ground behaviour, following measures for the upcoming excavation were defined:

- Measures underground/subsurface: extensive monitoring program (intensified monitoring interval for the 3D-displacement monitoring; installation of a face magnetostrictive extensometer, strain gauges in the shotcrete shell and in the inner lining, fibre optic sensors in the shotcrete shell and in the inner lining)
- Measures above ground: removal of the material for the terrain modelling to reduce dead load above the tunnel; installation of piezometer for measuring pore water pressures and their variation due to the excavation works, vertically aligned magnetostrictive extensometer to get information about the induced settlements due to the reconstruction of the terrain modelling and the influenced height above the tunnel.

The evaluation of the performed measurements has shown that the longitudinal displacement field of the tunnel face covers a length of 7 to 9 m ahead of the face. Based on this finding the length of the face bolts was increased and the residual bond length was increased to 10 m. Due to the intensified geotechnical monitoring program measures for the excavation works were continuously adapted.

4 CONCLUSION

The documented geological conditions show that in the southern tube a weak silt-clay layer was located in the face area with greater thickness and length in comparison to the northern tube. This results in the unfavourable combination of a higher load situation due to terrain modelling with weak rock underlying and the already higher stress situation (secondary stress state) due to the preceding tunnelling of the northern tube. A numerical analysis showed that a long-lasting consolidation process was initiated by the terrain modelling, hence the tunnelling works took place under constantly changing stress conditions in the ground. A single cause for the collapse event cannot be identified from the observations made; rather, a chain of the unfavourable conditions cited ultimately led to the collapse.

Although the excavation of Burgenland's first tunnel was extremely demanding, the application of a qualified geotechnical risk management, based on sound information sources as a basis for decision making and the willingness of all project participants to act in a solution-oriented manner allowed a successful completion of the excavation through this highly challenging area.

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