Semmering Base Tunnel, Alternative Numeric Modelling of Long-term Rock Mass Behavior and Conceptual Support Design

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ABSTRACT: At the construction Lot SBT 3.1 of the Semmering Base Tunnel long-term displacement increases caused an overutilization of the support measures over a length of approximately 100m behind the excavation face. It was concluded that these increases were caused by a strain softening of the rock mass. In numerical calculations visco-elastic and visco-plastic material models are used very often to model long-term rock mass behavior. In the present case a linear elastic – ideal plastic material model with a Mohr-Coulomb failure criterion in combination with a creep function was used to model long-term behavior. The strength and stiffness values as well as the creep values of the rock mass were identified based on a curve-fitting of monitored displacements to limit excavation interruptions due to testing of the rock mass and parameter identification for numerical modelling. Support measures and construction sequences for further excavation were designed based on the calculation results.

Keywords: Long-Term Rock Mass Behavior, Semmering Base Tunnel, Numerical modelling, System Behavior.

1 INTRODUCTION

The Semmering Base Tunnel (SBT) is situated in Austria / Europe and is part of the Baltic-Adriatic Corridor, which is one of the most important cross-Alpine lines in Europe. The tunnel is constructed as a two-tube railroad tunnel with a length of about 27.3km and is excavated from the portal at Gloggnitz and three intermediate construction accesses. For a detailed description of the project refer to Gobiet et al. (2017).

The geological conditions in the project area are characterized by a tectonically extremely complex rock mass structure. The tunnel alignment passes through several tectonic nappe units, meets various crystalline rock complexes and Permo-Mesozoic cap rocks and is intersected by numerous tectonic fault zones.

2 SEMMERING BASE TUNNEL LOT SBT3.1

2.1 Observations

The project area of the Semmering Base Tunnel is characterized by very complex rock mass conditions consisting of an intensive fold, nappe and scale structure. The tunnel lies within an inverse rock sequence and contains karstified carbonate rocks and Semmering quartzite of the Permo-Mesozoic as well as overlying mica shists to quartz phyllites. The Tunnel section issued in this article is in the West part of the tunnel towards the portal at Mürzzuschlag and lies in the tectonically heavily disturbed transition area from carbonate breccias and quartzite to quartz phyllites with an overburden of around 225m. The rock mass conditions are characterized by small-scale changes of the degree of fragmentation and strength properties and therefore were very heterogenous. Additionally, cataclastic fault zones of different thickness (range from decimeter to meter) intersect the rock mass. Pronounced asymmetries in the elevation of the temporary top heading carriageway as well as distinct failure mechanisms were documented in the excavation face of the bench / invert (Figure 1).

Figure 1. Face images of the bench/invert excavation with distinct failure mechanisms.

Due to the observed system behavior, the support measures were adopted in several steps, with a distinction being made between stiff and ductile lining. The 3D displacement measurements showed displacements in the lower to middle decimeter range, from approx. 200mm to 600mm. The development of the displacements are characterized by relatively small initial displacements in the lower centimeter range, followed by a decrease of the displacement rates with further top heading advance due to reduction of the advance-induced stress redistribution up to approx. 5-7D (tunnel diameter) behind the tunnel face, ongoing displacement increases with constant displacement rates until ring closure and further slight displacement increases after ring closure. Although a ductile support including yielding elements was installed, an overutilization of the shotcrete shell occurred over several tens of meters.

2.2 Conclusions and Requirements

The observed system behavior was generally very unfavorable. The unexpectedly high overall displacements and the long-term increases in displacements were attributed to the following circumstances: overutilization of the shotcrete shell due to large-scale stress redistribution caused by the excavation works, stress redistributions from the second (neighboring) tunnel excavation, stress redistribution due to fracture formation and fracture expansion in the shotcrete shell, stress concentration in the competent rock areas between the fault zones, deep overstressing of the surrounding rock mass and an increased in situ stresses in horizontal direction. Furthermore, it was assumed that the rock mass observed a softening behavior ("strain softening"/"post-failure" behavior) being jointly responsible for the high overall displacements and long-term displacement increases representing a time dependent behavior.

3 NUMERICAL CALCULATIONS

It was assumed, that the geological and geotechnical conditions would endure for at least another 500m. Therefore, a support and construction concept had to be established as soon as possible to avoid an unnecessary long excavation stop and to guarantee a safe and economic construction. Based on the observed system behavior and the geotechnical interpretation the aim of numerical modelling was to back calculate the rock mass parameters including the effect of time dependency (strain softening) and the potential of an anisotropic in situ stress state as a basis for further support design.

3.1 Available Models for long-term rock mass behavior

The consideration of time-dependent, irreversible and long-lasting deformations in calculations of underground structures is a complex issue. There are many different approaches for different material behaviors / material laws in literature, which can be classified in empirical and theoretical models. An overview over the common material models / laws is given in Gschwandtner (2013). The empirical models largely describe the transient and steady-state creep phase. As an example, the power-law model represents the initial (first) creep behavior, and the exponential models represent the steady-state (secondary) creep. Both models can be found in many publications and in simulation programs.

In comparable situations often material models based on a visco-elastic / visco-plastic approach are used to model rheological behavior. Therefore, relevant material parameters are determined in appropriate tests. This full process requires a lot of time, which is commonly no issue in the design phase. In cases where appropriate material parameters are not available, an analytical / numerical parameter study for the determination of the required input parameters has to be performed. Another key issue of these material models is the availability within the used software packages (Barla et al., 2012).

3.2 Material Models for Back calculations

3.2.1 Rock mass

Due to the strict timeline, it was concluded that time intensive testing to determine the relevant parameters for viscous material models must be avoided. In the current situation a big advantage was, that a sufficient time span of displacement monitoring in a particularly good quality was available. Therefore, it was possible to choose a common way of modelling. Regarding the general rock mass conditions, a linear elastic – ideal plastic material model with a Mohr- Coulomb yield criterion was chosen in combination with a basic creep function to consider time dependent behavior. Therefore, the calculations had to be executed in real time steps. The calculations were performed using the software ZSOIL. To take the influence of dominant geological discontinuities causing asymmetries in the observed system behavior in failure simulation into account, a constitutive model combining a multilaminate model with three possible lamina directions, to simulate the weakness planes, and a general three-parameters yield surface, to simulate the soil matrix, was used. On each plane separately, the Mohr-Coulomb plasticity condition and the tension cut-off condition must be fulfilled (Commend et. al 2004). The alternations of different rock mass qualities as documented was not modelled due to a frequently change and small thicknesses. Therefore, the rock mass was modelled as one continuum with smeared properties.

In the current context creep is a time dependent deformation under a constant stress and is considered proportional to the elastic primal strain.

$$\varepsilon^{cr} = \varepsilon^{e}_{inst} f(t) = \sigma C(t) \quad (1)$$

The numerical implementation is performed by a series connection of Kelvin Elements. In the current investigations the Power Law brought the best fit with the monitored displacements.

$$C(t, t_0) = A * (\mathrm{t} - t_0)^B \quad (2)$$

The creep yieldingness over time can be controlled by the parameters A and B. These parameters can be entered for the deviatoric and the volumetric part.

In the first step as a basis for further calculations the values of the elastic and strength parameters (E, c, φ) were back calculated with the approach of the Ground reaction curve based on Schubert et al. (2010). The values of the strength parameters of the discontinuities were determined by an empirical approach in combination with available data from the design phase. In further steps a variation of the calculation values was performed until a satisfying match with the observations in the first displacement phase was reached, when time dependent influences were considered to be very low. The values for the creep parameters A and B could be determined by considering displacement increases during construction stops due to refurbishment measures in rear construction areas. The parameters A and B were varied until a sufficient fit of the calculated displacements with the measured ones was found. Additionally, the in situ lateral stress coefficient was varied based on the orientations of the monitored displacements. A lateral stress coefficient transversally to the tunnel axis of 1.35 and of 1.0 in all other directions showed the best accordance with the monitored displacements.

3.2.2 Support

To achieve best consistency of the calculated with the monitored displacements it was necessary to consider the executed support measures within the numerical model. The shotcrete lining was also modelled with a Elasto-plastic material model with a Mohr- Coulomb failure criterion considering the time dependent development of stiffness and strength. Moreover, a creep function was taken into account with a calibration of creep parameters as described above considering available test results of the used shotcrete. This modelling also requires a real time calculation. Installed yielding elements were considered by implementing beam-elements with corresponding stress-strain behavior at the given location in the cross section. Rock bolts were not considered in the numerical model.

3.3 Results – Comparison with Observations

Figure 2 shows a comparison of the calculated displacements considering the final values of parameters for the rock mass and in situ stress state. For comparison two monitoring cross sections showing typical displacement developments were chosen. As is apparent, the monitored displacements show a deviation between the single cross sections due to local discontinuities. With respect to the described smeared rock mass properties the focus was laid on a fit of the calculated displacements with a mean value of the monitored ones. By using the multilaminate model considering the orientation of the main geological structures the observed failure mechanism in the bench/invert excavation face could be mapped in the calculation results (Figure 3). Moreover, the calculations showed the overutilization of the shotcrete lining as observed during construction. Finally, it must be stated, that the evaluated rock mass parameters just serve as a basis for the design of the required construction measures for further excavation and do not represent the actual ones.

3.4 Design for further excavation

As a first step a new cross section layout was created for the further excavation which's geometry is better suited for the expected load situation, six rows of yielding elements instead of four were implemented. The shotcrete lining was divided into two layers with a total thickness of 60cm to get a stiff support when closing the yielding elements. Different construction sequences consisting of the excavation of the top heading, shifted excavation of bench and invert and closing of the yielding elements with immediate installation of the second shotcrete lining were investigated. The criterion to find the best sequence was the lowest shotcrete utilization at the end of the calculation. This criterion was met with the construction sequence, where the bench + invert excavation is at minimum

20m and at maximum 40m behind the top heading face. Another 20m to 40m behind the yielding elements are closed and the second shotcrete lining is installed simultaneously. The calculation showed a very high utilization of the single support elements but no overutilization. Displacement increases were calculated till the end of the calculation which was set to 180d (app. 170m behind the tunnel face). The total displacements amount approximately 30cm.

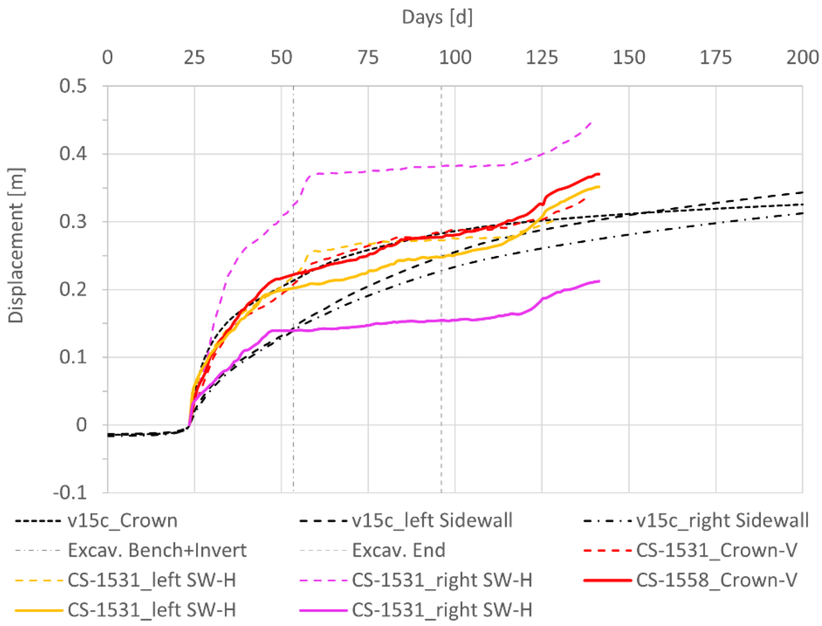

Figure 2. Comparison of calculated and monitored displacements of back calculations.

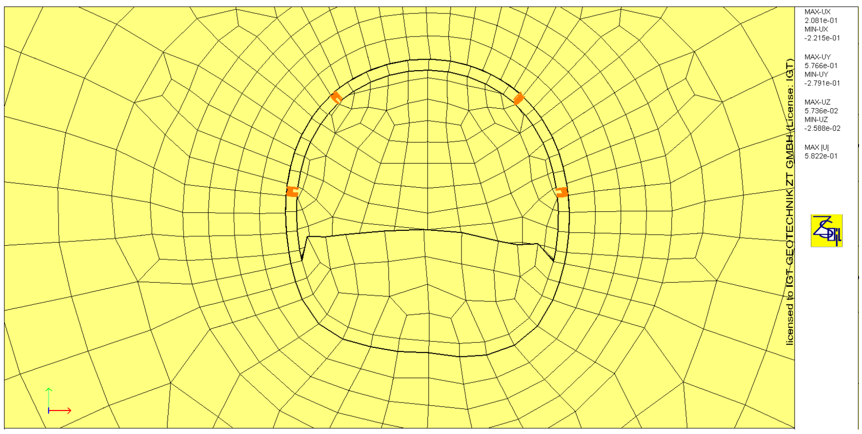

Figure 3. Deformed Mesh shows observed failure mechanism in the bench face.

4 FURTHER EXCAVATION – COMPARISON WITH CALCULATIONS

The monitored displacements were constantly compared with the calculated ones (Figure 4). The comparison showed that the measured displacements in many sections were higher than the calculated displacements. The actual support (number of yielding elements and rock bolts) was adapted based on the observed system behavior. In combination with the designed construction sequence a favorable system behavior with no further overutilization of the shotcrete lining was observed.

The long-term measurements showed vertical displacement increases of 1-2 mm per month over a period of approximately 1 year resulting in high vertical displacements which were monitored in the whole cross section. These increases were not shown in the results of the calculations, which point out a limit of the used modelling approach. It was concluded that the long-term displacement increases were caused by a loosening of the rock mass beneath the invert due to the observed failure mechanism described before in combination with a long-term consolidation process. Additionally, the design of the secondary lining had to be done on an empirical basis.

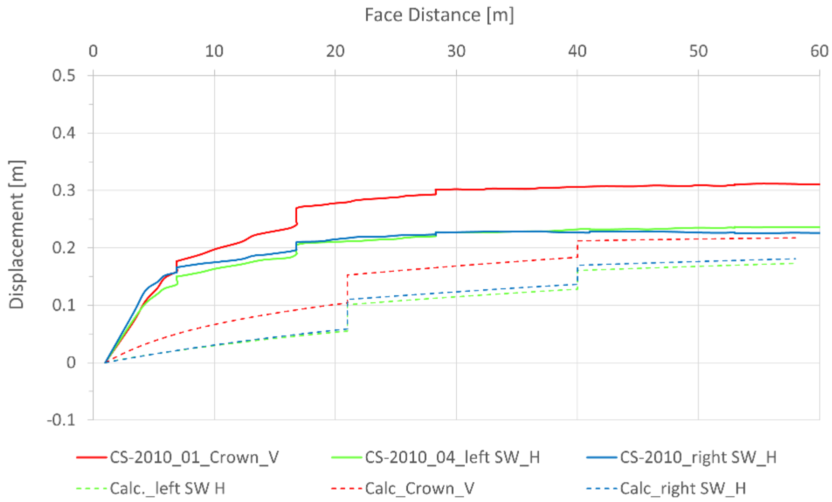

Figure 4. Comparison of the calculation with the measured displacements in a Path-Distance diagram.

5 CONCLUSIONS

Due to long-term rock mass behavior and resulting overutilization of support measures over a length of approximately 100m behind the excavation face a new construction concept had to be designed to enable a successful excavation. This had to be done as fast as possible to limit the delay of construction works. Therefore, it was not possible to perform sufficient tests of rock samples to identify values of relevant parameters required for appropriate material models used to simulate long-term rock mass behavior. In the current case a back calculation of monitored displacements was performed including a creep function to consider long-term behavior. An appropriate construction concept could be evaluated within a short time span for further excavation works. Further excavation showed a favorable system behavior with no further overutilization of the shotcrete lining. Nevertheless, slight displacement increases were observed up to one year after excavation which had to be considered in the design of the secondary lining. These displacement increases could not be identified in the calculations pointing out a limit of the used material model. Nevertheless, the chosen modelling approach served as an effective instrument to save time and costs during construction.

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