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CONTRIBUTION TO THE DESIGN OF TUNNELS WITH PIPE ROOF SUPPORT

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The working area of tunnels is often supported with a pipe roof support system in weak ground. The experience gained from former projects led to the conclusion that this support system decreases the subsidence during tunneling. The lack of knowledge about the geotechnical system behavior of this support system disables the designer from determining the basic design parameters depending on analytical and/or empirical solutions. This fact often leads to conservative and uneconomical designs. In order to overcome this lack an extensive monitoring program was executed on site. Using these data sets and the results of laboratory investigations the geotechnical model for this support system was determined in numerical simulations. Starting with this back calculated model the variation of basic design parameters was investigated. In this publication the focus will be set on the influence of the dimension and number of pipes on the displacement magnitudes at the tunnel level.

Keywords: shallow tunnel; weak ground; pipe roof support system; design parameters.

1. Introduction

In the last decades shallow tunnels were increasingly constructed in weak ground. These tunnels are often situated in urban areas, where project requirements, such as limited settlement requirements, constitute the necessary support. The experience gained from former tunnel projects indicate that the pipe roof support system not only increases the stability of the working face but also decreases the subsidence induced by the excavation. Due to these experiences a number of tunnels were additionally supported by this system without clear design rules for the determination of the design parameters.

Before such necessary rules can be established the ground support interaction has to be monitored during construction and the system behavior identified. The result of these investigations was used for calibrating the geotechnical model in numerical simulations. The authenticity of the numerical investigation was increased by using laboratory results for the rock mass and support parameters. In contrast to the experience gained during construction the same tunnel section can be excavated again and again with different support parameters in numerical simulations.

The differences of these simulations demonstrate the influence of different design parameters on the ground support interaction. Based on these evaluations relevant parameters can be derived and design rules can be determined leading to a transparent design for pipe roof support systems.

2. Definition of Pipe Roof System

Some pre-support systems are not separated from each other by clear definitions. For this reason the pipe roof system should be shortly described to inhibit confusion with other systems.

In the literature the pipe roof umbrella support system is also mentioned with the terms “pipe forepole umbrella” (Hoek, 2003), “umbrella arch method” (Kim et al., 2004), “long-span steel pipe fore-piling method” (Miura, 2003) or “steel pipe canopy” (Gibbs et al., 2002). These terms all contain the words for describing this system. Normally steel pipes but also fiber glass pipes are installed from the actual face to the front (forepoling systems) arranged like an umbrella or canopy around the later excavated area. The diameter of the steel pipes is usually between 60 mm and 200 mm with a wall thickness of 4 mm to 8 mm. The length of one umbrella is commonly 12 m or 15 m. The excavated length underneath (pipe roof field length) ranges from 6 m to 12 m.

The pipes can be installed with both special machines and conventional drill jumbos. From the geotechnical point of view there are basically two different methods for the installation: the pre-drilling system and the cased-drilling system. The significant difference is, when using the cased-drilling system the pipe follows directly behind the drilling bit immediately supporting the installation hole. When using a pre-drilling system the hole for the installation is drilled first and in a 2nd step the pipe is installed in the unsupported hole. When the ground weakens a cased-drilling system is therefore less susceptible to settlements than a pre-drilling system (Volkmann, 2004).

3. Projects and their Geological Conditions

The in situ measurement program was performed at two projects. 130 m of excavation were overall observed in more detail for the investigations on the pipe roof support system.

3.1. *Birgl tunnel*

The 950 m long Birgl tunnel (AUSTRIA) is a double track railroad tunnel. The excavated area is approximately 130 m^2. The west portal and the following 80 m are situated in the so called “Tauernnordrandstörung”, a major Alpine fault zone. This section of the tunnel was constructed using the New Austrian Tunneling Method (NATM) with a pipe roof support as pre-support system. In the section, where the additional measurements took place, the overburden increased from 30 m to 50 m. The rock mass consisted of clayey, cataclastic fault zone material with shear lenses composed of more competent blocks (3G & BGG, 2001). The design rock mass parameters are shown in table 1.

3.2. *Trojane tunnel*

The Trojane tunnel (SLOVENIA) is a 2900 m long twin tunnel, located on the motorway section AC A10 connecting Lubljana and Celje. The diameter of the tunnel is about 11 m. The tunnel was driven using the principles of the NATM. In this section, where the measurement campaign was performed, the overburden is 15 m. The geological conditions encountered during the construction are dominated by a meta-sediment sequence including mudstone, claystone and sandstone. Alpine thrusting resulted in heavily sheared zones that varied in thickness from a few centimeters to more than 50 cm. The basic rock mass parameters are described by Zlender (2003) (table 2).

4. In situ data

Observations and measurements already have a long tradition in tunneling (Rabcewicz, 1944). The measurement data are used to control the excavation induced movements (Rabcewicz 1963, Steindorfer & Schubert, 1997). These movements reflect influences of the surrounding ground as well as the construction method and the involved support methods. Altogether the measured data represent the so called “system behavior”.

4.1. *State-of-the-art measurements*

Nowadays geodetic three dimensional observations are state-of-the-art for collecting displacement data in tunneling. Using this system the positions of systematic located points in the tunnel and on

Table 1. Basic rock mass parameters for the Birgl tunnel.

	Parameter	Value
Matrix	rock mass strength	0.3 – 0.8 MPa
	friction angle	20°
	cohesion	up to 0.03 MPa
Blocks	rock mass strength	up to 100 MPa

Table 2. Basic rock mass parameters for the Trojane tunnel (Zlender, 2003).

Parameter	Value
Young's modulus	20 MPa
Poisson's ratio	0.25
friction angle	18°
Cohesion	0.016 MPa

the surface are determined in a local coordinate system during excavation. Such surveys are normally executed daily.

The measured movements, induced by the construction process, are usually displayed in time settlement diagrams, deflection curve diagrams and vector orientation plots. These display methods enable to observe the ground support interaction. Special evaluation techniques allow estimating parts of the pre-displacements by using the characteristics of the measured displacements (Sellner, 2000). This estimation increases the quality for the geotechnical evaluation of the survey data.

Surface settlements measured above shallow tunnels in weak ground are often larger than those measured in the tunnel. This characteristic indicates that a significant part of the displacements occur before the observation starts at the tunnel level (figure 1). To increase the information gained by the geodetic survey an additional measurement system was applied at both the Birgl and the Trojane tunnel during construction.

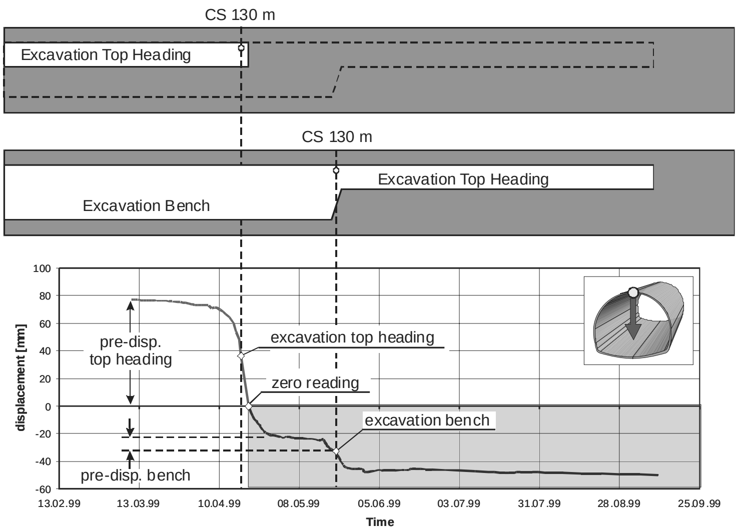

Figure 1. Time-settlement diagram: the grey shaded area in the diagram highlights the part of displacements that can be measured by a geodetic survey at the tunnel level (Sellner, 2000)

4.2. *Additional measurement system*

The purpose of the additional measurement system is to obtain more information about the pre-displacement characteristic at the tunnel level. Assuming homogeneous rock mass conditions only settlements have to be measured in the crown ahead of the face without losing important information.

The instrumentation is installed in an extra steel pipe in the time of the pipe roof installation. The 21 m long pipe is situated in the roof, parallel to the other pipes. The two executed measurement campaigns used 2 m long inclinometer links. In each measurement section 10 links were connected to each other to a horizontal, continuous, 20 m long, in-place inclinometer chain. The instrumentation was connected to a data acquisition system that stored the measured data every minute. This allows a very detailed observation of the settlements in the instrumented section without interrupting the construction process (Volkmann & Schubert, 2005).

4.3. *Benefits during construction*

At the tunnel crown level the information gained from the inclinometer measurements supplement the geodetical data ahead of that measurement cross section, which is nearest to the working face. Due to the position of the inclinometer instrumentation (up to 20 m ahead of the working face) the settlements are recorded directly above the working face and in the ground ahead of the face. The additional data obtained from the inclinometer chain are leading to the total, measured settlement path at the crown level. Behind the face the geodetical data additionally display the three dimensional ground support interaction.

The evaluation of the geodetic data catches the combined influence of all construction processes between two surveys on the ground support interaction. When constructing a shallow tunnel in weak ground, the survey of the tunnel primarily presents the stability conditions in the supported section behind the face. Around the heading the stress conditions change more rapidly due to the ongoing construction process. The frequent change of the stress situation in this section can be adequately observed by collecting the data in smaller time intervals, like the inclinometer measurement system does. The high resolution of the data in time enables one to observe not only the total settlement path but also the detailed development of settlements during single construction processes. This includes the development of settlements during the excavation of sequential parts of the cross section, the time dependent stabilization process afterwards and the settlements induced by the installation of every support system (figure 2). This evaluation identifies all settlement increasing construction phases (Volkmann & Schubert, 2005).

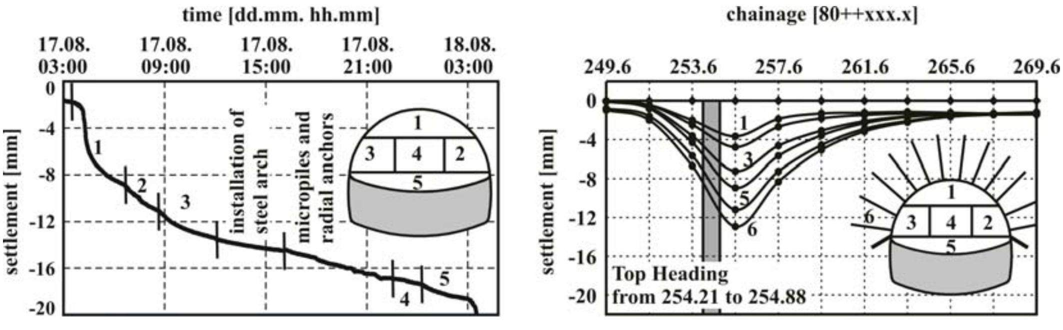

Figure 2. Time settlement diagram (left) and single deflection curve diagram (right) for one sequential excavation round

The short distance in between the measured points (2 m) enables the distribution of the settlements in the longitudinal direction to be displayed as well as the position of the maximum values. Changes in this characteristic behavior indicate changes in the ground support interaction: e.g. changes in the effectiveness of the pre-support system or changes in the ground quality ahead of the face.

The information gained from both geodetical and inclinometer data enable the understanding of the mechanisms involved during construction. Both data sets have to be evaluated and interpreted in a time span relevant for the tunnel advance (a few hours). Only in this way the ground support interaction can be continuously controlled and uncertainties in the ground properties can be followed by an adequate support adaptation leading to a safe and economical construction process.

5. Laboratory Tests

Numerical calculations require a lot of input parameters, which can significantly influence the results. For this reason multi stage triaxial tests and shear tests were performed on representative samples from the rock mass. The different pipe dimensions were also tested with and without grout to get input values for the simulations. With these results from the laboratory it is possible to delimit the unknown parameters to a minimum leading to a more reliable simulation.

6. Numerical Investigations

The numerical studies were done with the program “Fast Lagrangian Analysis of Continua in 3 Dimensions” (FLAC-3D, Version 2.1). In order to decrease the boundary influence the length of the model is 100 m. For the later discussed simulations the overburden is 15 m (Trojane tunnel). The distance between the sidewall and the outer boundary of the model is 35 m. In order to catch all mechanisms involved the geometry of the tunnel includes the saw-tooth shaped geometry in the upper part of the top heading, which is typical for a pipe roof supported tunnel. With a maximum finite element size of 0.5 m near the tunnel the memory limitation of the FLAC-3D Version 2.1 only enables to simulate one half of the tunnel.

The strain-hardening/softening model was chosen for representing the ground behavior due to the results of the laboratory tests. The shotcrete was simulated with a time dependent increase of stiffness and strength based on Aldrian (1991) and Müller (2001). The smallest time increment used for the aging process is 6 hours and the definition of age for the shotcrete is taken from the advance rate at the Trojane tunnel. The heavy steel beams in the shotcrete are simulated with beam elements. The face bolts and the pipe roof pipes are simulated with pile elements.

One meter of the top heading area is excavated at once in the simulation even though the excavation of the top heading was done in 5 sequences at the Trojane tunnel with a design excavation length of 0.8 m. Another 0.5 m is added to the excavated length as working area. After the excavation the model is calculated until stability is reached. With stable conditions the support consisting of shotcrete and heavy steel beams is installed behind the face and updated to its current age values.

6.1. *Comparison to the in situ data*

The first exercise was to find the correct geotechnical model for the simulation. In contrast to the publications of Bae et al. (2005), Hefny et al. (2004) and Kim et al. (2004) the grouted pipes were not simulated as a homogenized area. Each pipe was simulated as a pile element and the grout was neglected because the measuring of the grout volume indicated that only the pipes were filled with grout. Other important points for modeling the construction process were the longer unsupported span (0.5 m working area) and the calculation of stability before installing the support behind the face. The adaptation of the bonding properties of the pile elements to the actual rock mass properties also displayed an effect on the results.

In figure 3 the upper deflection curve diagram shows the in situ settlement values. The measured values after every second excavation round are displayed as black lines. Additionally the trend line at the face is drawn in grey. The lower diagram shows the results of the numerical calculation. The comparison of the diagrams shows a good correlation of the settlement values with the little knowledge about the ground. The maximum settlement values as well as the settlement distribution between pre- and total settlement amount can be simulated correctly.

Behind the face the increase of settlements is stopped in both cases after a few meters due to the stiff support in this area. The geotechnical model for this part is therefore correct. The settlement increase in the area of the unsupported span can also be observed in both diagrams. The section ahead of the face also shows the same characteristic areas of faster and slower increasing settlement values with respect to the uncertainties and continuous changes in real ground.

The implemented geotechnical model seems to be correct because the results of the measurement campaign agree very well with the results of the numerical simulation, which is the basis for further variations to evaluate the influence of the pipe number and dimension.

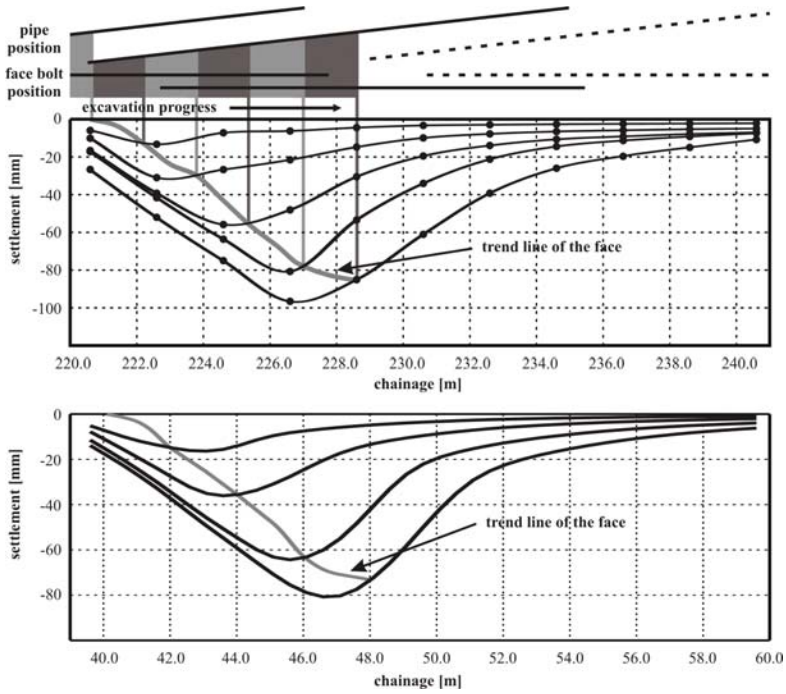

Figure 3. In situ deflection curve (upper) and simulated deflection curve diagram (lower) without pre-displacements.

6.2. *Influence of design parameters*

The influence of number and dimension of the installed pipes on the settlements is shown in simulations from another Trojane tunnel section, where the pre-settlements increased up to 16.0 cm at the crown level.

For this comparison only the simulated pre-support is changed for one 8 m long pipe roof field. Up to this position a 15 m long pipe roof support consisting of 20 pieces 114.3 mm x 6.3 mm pipes was used. The pre-settlement values at the crown level are evaluated and compared. The reference calculation for this comparison is done without installing a pre-support system. As can be seen in figure 4 and table 3 the pre-settlements at the face are 20.90 cm.

The maximum settlement value in the working area is 25.00 cm for the case without pre-support.

Another three calculations were performed with a pipe roof support consisting of 10, 20 or 30 pieces of 114.3 mm x 6.3 mm pipes. The installed pre-support decreases the pre-settlement values depending on the number of pipes. The decrease of settlements increases with the quantity of pipes (table 3).

The calculation with 20 pieces for the pipe roof support was also performed with the pipe dimension 139.7 mm x 8.0 mm. The steel area per cross section is in this simulation comparable to the simulation with the 30 pipes from the earlier mentioned case. During the construction on site this case would be a little more time consuming than the case with the 20 smaller pipes but the increase in stiffness decreases the settlement values again. Compared to the case with the 30 smaller pipes the reduction of settlements is nearly equal but one third more pipes usually need more time for the installation.

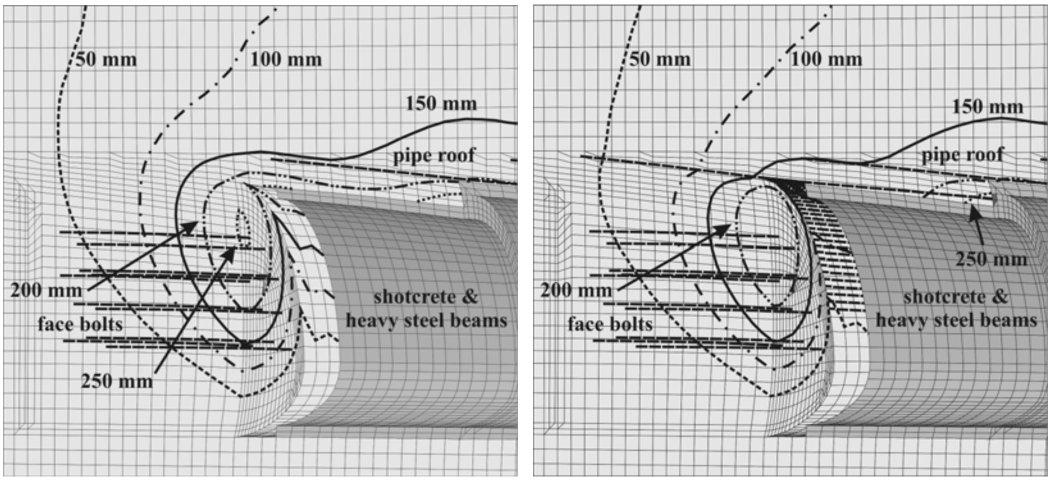

Figure 4. Calculated settlement values without (left side) and with pre-support (30 pieces of 114.3 x 6.3) (right side)

Table 3. Calculated settlement values for the different cases

	without pre-support		10 pieces 114.3 x 6.3		20 pieces 114.3 x 6.3		30 pieces 114.3 x 6.3		20 pieces 139.7 x 8.0	
at the face	20.90 cm	100.0 %	17.10 cm	81.8 %	16.15 cm	77.3 %	15.75 cm	75.4%	15.60 cm	74.6 %
maximum pre-settlement value	25.00 cm	100.0 %	19.70 cm	78.8 %	18.10 cm	72.4 %	17.50 cm	70.0 %	17.30 cm	69.2 %

7. Conclusion

Pipe roof support system design is usually based on experience although their application has increased in shallow, weak ground tunnel projects in the last decades. By performing additional horizontal inclinometer measurements to supplement the state-of-the-art geodetic survey the first step for understanding the system behavior was done. On site this additional data can be used to optimize the construction process of pipe roof supported tunnels as well as to determine changes in the ground quality ahead of the face. For this study the detailed data in combination with laboratory data was used as input and control parameters for numerical investigations. This back calculation clarified at first the geotechnical model for the pipe roof support system. Afterwards the number and dimension of pipes in one pipe roof field was investigated.

Even though the grout was neglected due to the ground conditions of the investigated projects the calculations clearly showed a decrease of the pre-settlement amounts up to 30 % at the tunnel level when using a pipe roof pre-support system. The different cases displayed that the decrease of pre-settlements increases with increasing number of pipes as well as with bigger dimensions.

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