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A Contribution to the Effect and Behavior of Pipe Roof Supports

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ABSTRACT: The increasing use of pipe roof support systems in tunneling has not been adequately followed by an increased understanding of the interaction between the rock mass and this support system. A detailed investigation of the rock mass – pipe roof support system interaction was initiated using continuously recorded, horizontal chain inclinometer measurements to provide a basis for a better understanding of the mechanisms involved. The recorded data ahead and behind of the tunnel face indicate a typical settlement characteristic in a pipe roof field. The installation method can affect the settlements especially ahead of the face. The effectiveness of the support system decreases at the end of the pipe roof field showing in increased settlement amounts ahead of the face. The length on which pipe roofs significantly influence displacements depends on the rock mass quality and the height of the face.

1 INTRODUCTION

The technical advances of roof systems over the past few years have increased its use in weak ground. In many cases the design is merely empirical or based on non-validated numerical simulations.

In order to obtain a better understanding of this support system, in situ measurements with inclinometer chains installed parallel to the pipe roof were performed. The measurements of the inclinometer chain were linked to the geodetical displacement measurements taken inside the tunnel and on the surface. These measurements display the longitudinal distribution as well as the magnitudes of the settlements in the crown region of the excavation area (Volkmann, Button, Schubert, 2003). The results of the monitoring campaign allow a number of conclusions to be made with respect to the effectiveness of a pipe roof support.

2 MONITORING SYSTEM AND SITE APPLICATIONS

The chain inclinometer settlement measurements were acquired during the construction of two projects. Both of the instrumented tunnel sections are situated in extensive fault zones. For both tunnels sequential excavation was used, with an extensive monitoring program accompanying the construction process.

2.1 *Birgltunnel (Austria)*

The first measurement campaign was realized at the "Birgltunnel", a 950 m long double track railway tunnel, constructed as a part of the upgrade for the "Tauernachse" between Salzburg and Villach. The total excavated cross section of around 130 m^2 was done in up to 6 stages. The west portal and the first approximately 80 m long section of the tunnel are situated within the so called "Tauernnordrandstörung", which is a major Alpine fault zone. In this area a 44 m long section was instrumented with inclinometer chains. In the evaluated zone the overburden ranges from 30 m up to 50 m.

The rock mass in that section consists of clayey, cataclastic fault material with shear lenses composed of more competent blocks (3G & BGG, 2001). Laboratory tests on samples taken from the Birgltunnel show that the uniaxial compressive strength of fault gouges can be below 1 MPa and the Young's modulus below 100 MPa (Canali, 2004).

2.2 *Trojanetunnel (Slovenia)*

The second campaign was conducted at the Trojane tunnel, which is part of the Highway A10 between Ljubljana and Celje. The measurements were performed for more than 80 m in a critical section of the south tube, where the alignment passes a critical structure. The overburden thickness in this area is approximately 15 m.

The rock mass in the Trojane tunnel is dominated by faulted mudstone, claystone and sandstone. The rock mass contains clayey zones, transition zones and more component blocks. Laboratory test results performed with samples from the Trojane hilly area are presented in Table 1.

friction angle	**φ**	**18° - 20°**	Young's modulus	**E**	**70MPa - 120MPa**
cohesion	**c**	**0.001MPa - 0.060 MPa**	Poisson's ratio	**v**	**0.15 - 0.25 increasing with strain**

Table 1. Characteristic laboratory test results reported by [Zlender, 2003] for the Trojane tunnel

2.3 *Measurement Methods*

The geodetical survey in both projects was generally performed once a day, in critical situations this was increased to twice daily. Surface displacements were measured along the tunnel axis at approximately 5 m intervals, while measurement profiles normal to the tunnel axis were spaced at approximately 20 m intervals. The displacements of buildings close to the tunnel were also measured. The distances between the measurement sections in the tunnel were approximately 4 m in the Birgltunnel and 10 m in the Trojanetunnel. The beginning point of the inclinometer chains was also measured geodetically to allow the total settlements to be evaluated.

In order to provide more detailed information about the settlement characteristics at the crown level, inclinometer measurements were performed (Volkmann, 2003). The in-place chain inclinometer had a length of 20 m and consisted of ten 2 m inclinometer links. Each inclinometer chain was installed parallel to the pipe roof in the crown. This allowed displacements to be measured up to 20 m ahead of the face. The measured inclinations were recorded by an automatic data acquisition system every minute.

3 EVALUATION OF RESULTS

Compared to the geodetical survey continuous inclinometer measurements enable the observation of the settlement behavior in more detail. The influence of the excavation process on the settlements can be observed in real time. The measurements demonstrate that both the excavation and the installation of support, such as rock bolts, initiate settlements. Settlements initiated during the excavation increase rapidly after the beginning of each excavation phase and after the excavation is completed show a time dependent stabilization process. The drilling for rock bolts and micropiles increases the settlement amounts (Figure 1). Settlement trend observations can give

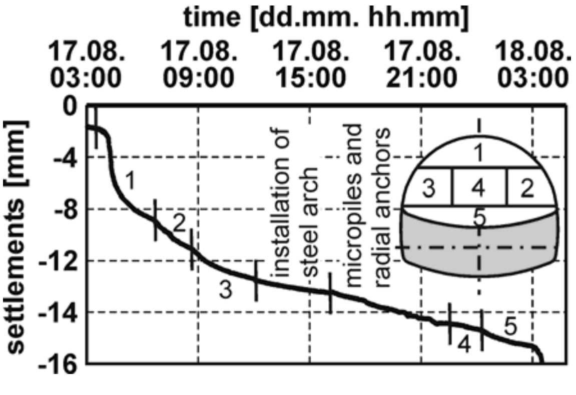

Figure 1. Measured time - settlement line for one excavation step in five phases with support installation

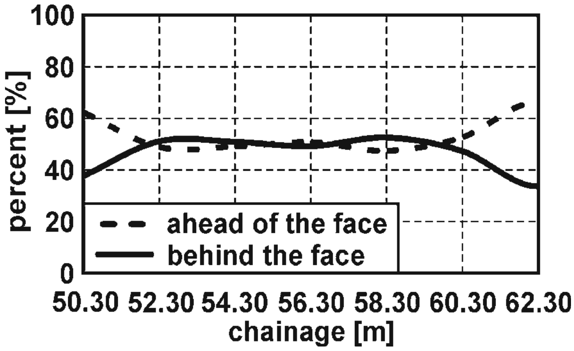

Figure 2. Settlement characteristic in a pipe roof umbrella field (Volkmann, 2003)

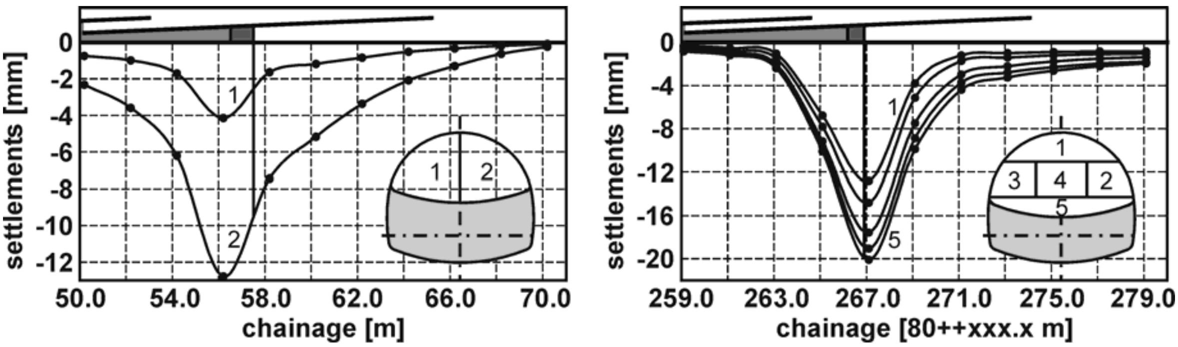

Figure 3. Deflection curves due to one excavation step; left side Birgltunnel, right side Trojanetunnel

information about the sphere of influence, if local face instabilities occur, as well as about the stability conditions ahead of the face in the time of an excavation break.

The chosen instrumentation allows all evaluations every 2 meters in the longitudinal direction over a length of 20 m. The distribution of the settlements in the longitudinal direction indirectly describes the stiffness properties of the support as well as the rock mass. Due to this relationship the evaluation results demonstrate that settlements decrease faster behind the face with a stiffer support system (Figure 3). On the other side, stiffer blocks and weaker sections can be detected in front of the excavated area. The support can be adapted and the resulting changes in the interaction can be evaluated. The settlement recordings expectedly displayed that the highest partial excavation step in the top heading causes the highest amounts of settlements. The lower partial excavation phases in the top heading initiate lower total values of settlements but spread further in the longitudinal direction than those from the upper excavation phases.

3.1 *Characteristic Settlement Behavior*

The inclinometer measurements from these two projects show a recurrent settlement behavior in a pipe roof umbrella field (Figure 2). In the tunnel section shown the total settlements due to the excavation of the top heading were nearly constant, but the distribution of the settlements ahead of and behind the face changes with position in the pipe roof field.

In contrast to the normal support stiffness the stiff arch which results from the saw tooth shaped support at the onset of the new pipe roof field result in a very stiff support element. This element takes a lot of the loads associated with the stress transfer process induced by the excavation in the first excavation steps. Due to that fact the settlements occurring in the first excavation steps are smaller.

This effect decreases with distance to the starting point of the pipe roof field and a nearly constant settlement distribution starts. In this example the settlements ahead of and behind the face are nearly equal (Figure 2). The supporting pipes are founded on the tunnel lining and in the rock mass. The stiffness contrast of these foundations primarily defines the distribution of the settlements. By using a stiffer lining the settlement amounts behind the face are decreased which is increasing the proportion of the settlements ahead of the face by decreasing the total settlement amounts. The stiffer foundation also affects the settlements ahead of the face by the balancing effect of the pipe roof support in the longitudinal direction.

This balanced stress transfer changes at the end of the pipe roof field. The settlements increase ahead of the face due to the decrease of the support effect ahead of the face. The system behavior resembles the system behavior of a non pipe roof supported tunnel. This effect is also the reason for the higher pre-displacements at the beginning of the pipe roof field.

3.2 *Influence of Installation Method*

Nowadays two variants are applied for the installation of pipes with diameters between 80 mm and 200 mm.

The first installation procedure starts normally with the drilling of a few bore holes which are made one after the other without changing the drilling equipment. The flushing material is carried out in the annulus between the drilling rod and the rock mass with water or air as used in Trojane. The use of air prevents the negative influence of water on the rock mass strength. But it is possible that the mixture of spoils and air expands the boring by eroding away additional material. After finishing the preparation of the holes the pipes are installed in the borings which are unsupported until this time and then grouted (pre-drilling system).

In the other installation procedure the pipe acts as a casing and is installed simultaneously with the drilling process into the rock mass ahead of the face. In this case, the drilling rod and the backflow of the flushing material are inside the pipe. Therefore the water only encounters the rock mass in the area around the drill bit. This minimizes the ability for the penetration of water into the rock mass. An advantage of this installation method is that the borehole is supported by the simultaneous installation of the pipe (cased-drilling system).

The measured data acquired until now allow a limited comparison of these two installation systems and their influence on the settlement magnitude during installation. The settlements for the cased-drilling system which was used at Birgltunnel is shown in Figure 4 on the left side and the data for Trojanetunnel, where the pre-drilling system was used, is shown in Figure 4 on the right side. Both diagrams present the face position during the installation with chainage zero and the excavation direction is from the left to the right side. This permits a comparison of the settlement amounts ahead of and behind the face. As mentioned before the influence of stiffer and accordingly weaker support systems can be seen by the decrease of the settlements behind the face.

The three measured installations from Birgltunnel display nearly the same settlement amounts. The values are all lower than 10 mm with maxima about 1 m to 3 m ahead of the face. From this position the settlements decrease slowly in both directions. In the installation which induced the highest settlement amounts, the annulus between the rock mass and the pipe closed due to the displacements of the borehole walls. This was noted because the volume of the injected grout was the same as the internal volume of the pipes. The supporting effect of the pipes effectively prevented the holes from closing, minimizing potential settlements.

At the Trojanetunnel the different settlement characteristics observed during the installation display a clear correlation to the system behavior measured in the excavation process. In the

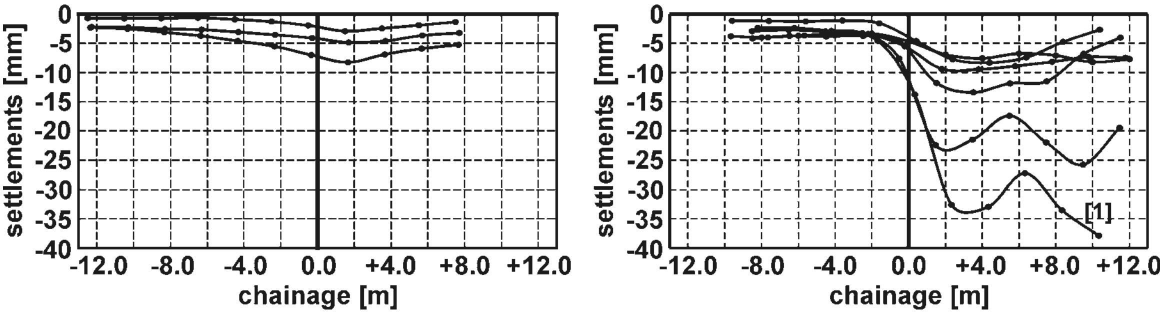

Figure 4. Settlements during the installations of different pipe roofs; left Birgltunnel, right Trojanetunnel

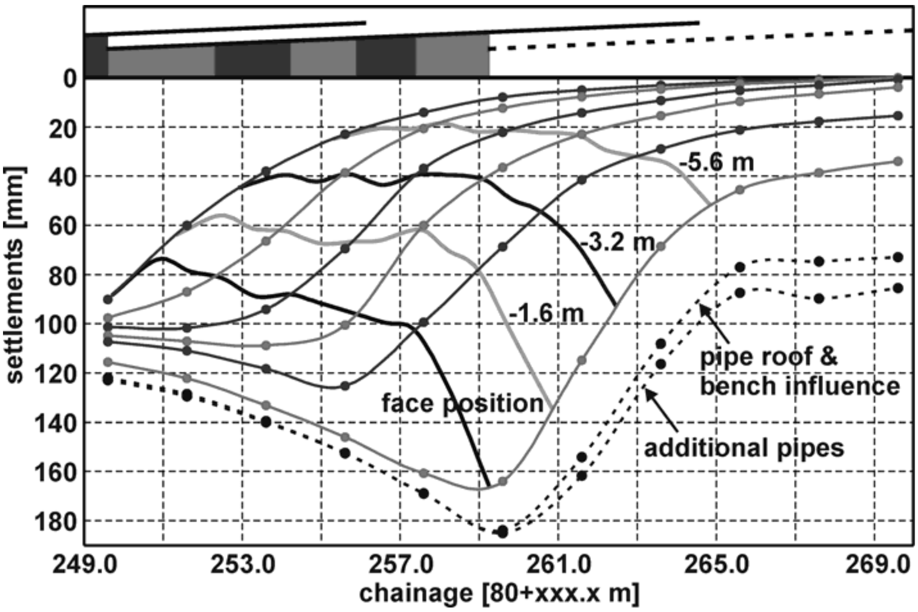

Figure 5. Deflection curves diagram showing every second excavation step including the installation of the following pipe roof umbrella with additional pipes

sections where the installations indicated settlement amounts smaller then 10 mm the system behavior did not vary. Wherever the measured settlements during the installation increased, weaker rock masses had to be excavated, which also created larger settlements during the excavation. For example, Figure 5 displays the trend lines for the tunnel section at which the installation of the pipe roof umbrella created the highest measured settlement amounts (Figure 4 right [1]). The trend lines indicate that the displacements ahead of the face rise significantly. Evaluations of the measurements indicated that the settlements increase simultaneously with the drilling of the unsupported boreholes due to the stress transfer and the dynamic loading. The pipes could not be installed correctly in single cases due to the closure of the pre-drilled holes. In the example Trojanetunnel the closure of the pre-drilled holes caused settlement values up to nearly 4 cm. A large proportion of these values could also be measured on the surface.

The borehole stability can be considered as a limit for using the pre-drilling system in areas which are sensitive for settlements. This limit does not exist for the cased-drilling system due to the immediate support of the borehole wall. The measurements do not indicate significant settlements in the time of the grouting process for both systems.

3.3 *Bedding in the Rock Mass ahead of the Face*

The installation of the pipe roof support system is a time consuming procedure which decreases the advance rate. Therefore a short but effective overlapping length in the longitudinal direction is desired economically to decrease the number of installations. An effective face support strengthening the rock mass ahead of the face is a basic requirement for this goal. As mentioned the height of the top heading influences the sphere of influence for the stress transfer ahead of the face. On this account a higher top heading needs a longer overlapping length.

Depending on parameters like rock mass strength, overburden, dimensions of the cross section and face support the displacements induced during each excavation result in additional loads on the pipe roof pipes. These loads are transferred to both foundations - the rock mass ahead of and the support behind the face. Ahead of the face the stiffness of the support transfers the stresses from the highly stressed areas near the face to the rock mass ahead of the excavation. Additionally, the pipes decrease the relaxation in the longitudinal direction, increasing the strength of the rock mass ahead of the face. This stress transfer process which is influenced by the pipe roof umbrella causes the uniform transfer conditions in the middle part of a pipe roof field.

At the end of the pipe roof field the bedding length of the pipe roof pipes decreases. Parallel to this the loads which can be transferred to the pipes ahead of the face are also reduced. The efficiency of the pipe roof support gradually declines by the reduction of the induced loads. This results in the displacements increasing ahead of the face and in the area immediately behind the face. As a consequence the supporting effect decreases and the rock mass behavior ahead of the face resembles with progressing tunneling more and more the behavior without pipe roof support. By the reduction of the embedment length ahead of the face both the changes of the longitudinal stresses and the transfer of the stresses in the longitudinal direction to sections with more capacity disappear. This reduces the supporting effect of the system and increases the settlement amounts primarily ahead of the face.

4 CONCLUSION

The increased use of pipe roof support in tunneling calls for design rules which are based on the support characteristic of the pipe roof support system. For the evaluation of these characteristics a measurement system was applied that measures the settlements ahead of the face in the crown region of the excavation. The results display that the pipe roof support system follows a recurrent settlement behavior beginning with the installation of the pipe roof umbrella.

Due to the fact that the pipe roof support system is used in areas where the total and/or differential settlements are limited, parameters that influence the settlements were evaluated. The recorded settlement data demonstrate that as the rock mass weakens pre-drilled installations become more problematic. A cased-drilling system compared to a pre-drilling system is less susceptible to settlements in the time of the installation.

The excavation under the support of the umbrella results in smaller settlements at the beginning of each pipe roof umbrella field due to the load transfer in the stiffer shotcrete arch. The next excavation steps produce nearly constant values and distributions of settlements in the longitudinal direction. Dependent on the rock mass and on the height of the top heading the decreasing effect of the pipe roof support system can be defined by increasing settlement amounts ahead of the face. A combination of declining reinforcement of the rock mass and the reduced bedding length ahead of the face results in this phenomenon which is increasing with progressing tunneling. The stiff arch at the beginning of the next pipe roof field counters that effect and the recurrent settlement behavior starts again.

Analyses of the measurement data show that the pipe roof support system is a truly three-dimensional problem. To fully understand the influence of different installation methods and support geometries during design it is necessary to use detailed 3-D numerical calculations, 2-D numerical simplifications cannot capture the observed behavior. During construction the excavation and support methods can be optimized for the encountered rock mass and boundary conditions by using correct monitoring systems and evaluation methods.

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