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A Contribution to the Design of Tunnels Supported by a Pipe Roof

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ABSTRACT: The increased use of pipe roof umbrella systems as a pre-support method necessitates the need for a standardized approach to determine the basic design parameters during design. The knowledge gained by in situ measurements using in place inclinometer chains were used to identify key influencing factors and guide 3-D numerical investigations. These simulations acknowledged and advanced the geotechnical model based on the in situ measurement data. Due to this it is possible to calculate the estimated deformations and to determine the design parameters of a pipe roof umbrella system with numerical simulations. To control the estimated ground – support interaction and adapt the support to the actual rock mass quality the developed measurement system can be additionally used in sections that are very sensitive to subsidence. Using this knowledge an appropriate modeling scheme allows the determination of the required support. With a continuous adaptation of the support system to the encountered ground behavior during construction a safe end economical construction process is assured.

1. INTRODUCTION

The modernization of urban, as well as regional infrastructure has resulted in increased tunneling activities in soil and weak rocks within developed areas. A safe and economical construction is always desired even though the conditions of the ground may not be optimal. This often results in critical sections, especially in urban areas, being supported with cost intensive and time consuming pre- support systems like freezing or jet grouting to protect surrounding infrastructure from damages.

Over the last decades technological developments have led to the increased use of different pre-support technologies to help prevent undesirable events. The pipe roof support method is one of the pre-support concepts that has gained in popularity in conventional tunneling and has even been included in TBM support systems. This method of supporting potentially unstable ground ahead of the face provides a high degree of flexibility and can be adaptable readily to the encountered conditions. However, in our opinion there is a significant lack of knowledge about the ground–support interaction associated with this method and thus objective design criteria are currently not available.

In order to obtain a better understanding of this support system, in situ measurements with inclinometer chains installed parallel to the pipe roof support 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 [1].

Laboratory investigations on the rock mass materials and the pipe roof pipes were performed to develop input parameters for numerical simulations providing a basis to examine the acquired knowledge in detail. With this study a geotechnical model and the way to transfer it into a numerical model is investigated.

2. DEFINITIONS / STATE OF THE ART

In their book "Tunneling with Steel Support", Proctor & White [2] discussed the use of wooden "spiles" as a forepoling method for traversing weak and raveling ground. Since this time several slightly different concepts have evolved all with the goal of providing additional support above and directly before the working tunnel face to suppress local or global instabilities. Concurrently to the technical adaptations the terminology has also evolved but some pre-support methods are not delimited from each other by clear definitions or different names are used for the same system. There are 5 primary concepts of pre-support technology installed from the tunnel that are utilized in modern tunneling:

- The simplest forepoling method is the installation of spiles from the last arch to the face before the excavation takes place. Normally the spiles have diameters lower than 50 mm (rock bolts) and are either pushed or drilled into the ground at the perimeter of the working face with a very small spacing (fugure 1). After the installation the annular gap is filled with grout. Shorter spiles (3-4 m) are used to suppress local failures in the just excavated span by their shear resistance. Longer ones (up to 8 m) can be used to minimize the interruption to the normal excavation process required for drilling and installation. This system is commonly called forepoling, while the term spiling is commonly used in Austria.
- Pipes with a diameter lower than 200 mm (not exactly defined) can be installed using either special machines (e.g. Cassegrande drilling rigs) or a normal drill jumbo. This system requires a widening of the cross section resulting in a sawtooth profile. Their lengths can vary but typically are 12 m or 15 m long. After grouting the inner annulus and the annular gab the excavation advances under the supporting pipes. After a pre-defined length of excavation the same procedure recurrently starts. For this system a few names are used worldwide: e.g. pipe roof umbrella; umbrella arch method; long forepoling method; canopy tube umbrella.
- Pipes with a diameter up to 1 or 2 m can be drilled with special equipment or installed with micro-TBM's, on the outer side of the designed excavation profile from a starting shaft. After filling them with grout the excavation can be done under a very stiff supporting umbrella. A commonly used term for this system is pipe jacking.
- Jet grouted columns installed from the tunnel can be used to create a canopy surrounding the excavation profile. These columns can be either overlapping, creating a closed often watertight canopy, or non-overlapping.
- Freezing of the ground is the most cost and time intensive pre-support method. Using this system the ground water is used to produce an ice-umbrella acting as support for the following excavation.

Additionally, unique methods which are typically variations of the above mentioned systems can be found in the literature; for example the pre-cutting method [3], the "Farchanter umbrella" [4] or the "Ischebeck umbrella".

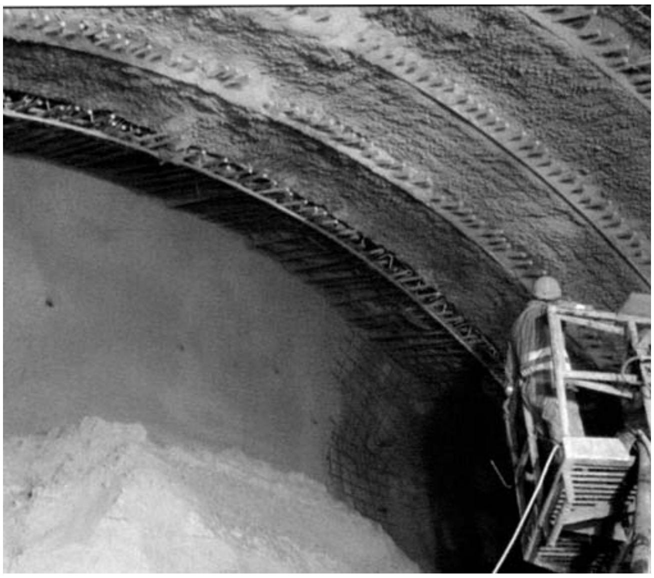

Figure 1. Spiles installed through the lattice girder to the face in every excavation step.

This paper will focus on the pipe roof umbrella system, which will be explained in more detail. There are currently two concepts used to install the pipes. The first installation procedure we define as the pre-drilling system. Normally several holes are drilled one after the other and afterwards the pipes are installed in the pre-drilled holes which are unsupported until this time. The flushing material is carried out in the annulus between the drilling rod and the rock mass with water or air. In weak or unstable ground conditions this may deteriorate the bore hole walls, increasing the hole diameter and potentially leading to increased displacements. However, use of air prevents the negative influence of water on the strength of many ground types. In the final step the pipes including the annular gap are grouted.

In the second installation procedure the pipe acts as a casing and is installed simultaneously with the drilling process into the rock mass ahead of the face. We call this the cased-drilling method. There appears to be two distinct advantages to this installation method over the pre-drilling system. The first is that 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, preventing degradation of the ground and enlargement of the borehole annulus. Secondly the borehole is supported the entire time by the simultaneous installation of the pipe; this prevents borehole stability problems that may lead to the inability to install the pipes correctly, or increased displacement due to larger open voids surrounding the bore hole.

Currently there are no commonly accepted design rules for dimensioning pipe roof support systems. Hoek [5] discusses how common practice has led to some basic “guidelines” and states that it is unpractical in most cases to try to perform 1 to 1 modeling of this support system. Instead, numerical studies most often found in the literature utilize as homogenation technique to improve the ground strength ahead of the tunnel face. While this is a simple and possibly an acceptable (in terms of general trends) method, it provides no information on the true support – ground interaction and thus limits the potential for truly understanding how this support system functions in terms of support optimization.

The homogenation technique corresponds better with a geotechnical model for an overlapping jet grouted column umbrella, where an area around the perimeter of the tunnel is strengthened by cementing. Mostly the pipes are not grouted with high pressures like during jet grouting. The pipes are filled with grout and through holes in the pipes the annular gap including the surrounding open joints are filled (Figure 2). For this reason the geotechnical model must differ to that one used in the publications up to now.

3. IN SITU MEASUREMENTS

The movement initiated by the construction of a conventional tunnel is normally quantified by a geodetic survey in the tunnel. Additionally, in shallow tunneling the subsidence of the surface is monitored. By the continuous observation of the displacements the deformation behavior can be observed. Changes from this behavior have to be interpreted and if necessary followed by an adaptation of the support system.

Pre-support systems like a pipe roof umbrella are primarily acting ahead of the face and in the non-supported area behind the face. Both sections cannot be controlled by the geodetic survey. For this reason an additional measurement system has to be used for collecting the data, which is necessary for a geotechnical control of this pre-support system.

An online inclinometer chain including a data acquisition system meets this demand. The used arrangement allows the evaluation and interpretation of settlements up to 20 m ahead of the face in the crown level. However, longer or shorter inclinometer chains may also be used.

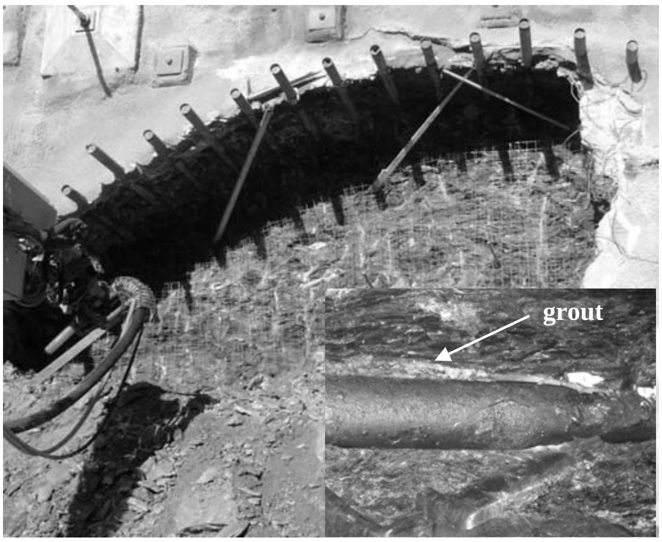

Figure 2. Portal pipe roof umbrella at Birgltunnel (Austria); the detail shows grout filling the annular gap.

3.1. Projects and Geological Conditions

The first measurement campaign was realized at the “Birgltunnel” (Austria), a 950 m long double track railway tunnel, constructed as a part of the upgrade for the “Tauernachse” between Salzburg and Villach starting in 2002. The total excavated cross section was approximately 130 m^2 was done in up to 6 partial excavation 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. In the evaluated zone the overburden ranges from 30 m up to 50 m.

The rock mass in this section consists of clayey, cataclastic fault zone material with shear lenses composed of more competent blocks [6]. Laboratory tests on samples taken from the Birgltunnel showed that the uniaxial compressive strength of the fault gouge can be below 1 MPa and the Young’s modulus below 100 MPa [7].

The second campaign was conducted at the Trojane tunnel (Slovenia) starting in 2003, which is part of the Highway A 10 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 beneath the Trojane village. The overburden 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 area are presented in Table 1.

Table 1. Laboratory results for the Trojane tunnel area [8].

	laboratory result	unit
modulus of elasticity	70 – 120	[MPa]
Poisson’s ration [ν]	0.15 – 0.25 increasing with strain	[-]
friction angle (φ)	18° - 20°	[°]
cohesion (c)	0.001 – 0.054	[MPa]

3.2. *Measurement System*

With the assumption that the ground responds symmetrically about the vertical axis the excavation induces only vertical deformations in the crown region. For this reason the inclinations measured by a horizontal inclinometer in a pipe above the tunnel crown can be used to calculate the movements at a certain moment in time without loosing information about the mechanisms involved. In a first campaign these measurements were done six times including the zero reading at the Unterwaldtunnel (Austria) during the excavation under a single measurement section [9]. A manually measured inclinometer appears to be too time consuming for more detailed measuring campaigns, especially because the construction process has to be interrupted during the measurements. The results acquired during this measurement program were geotechnical reasonable and showed promise for quantifying the deformations of the pipe roof.

After this first experience with inclinometers, the measurement system had to be improved for further applications during construction. To minimize the interruption to the excavation it was decided to utilize an inclinometer chain in combination with an automated data acquisition system. This arrangement allows storing the measured inclinations in pre-defined time intervals without interrupting the construction process. For the

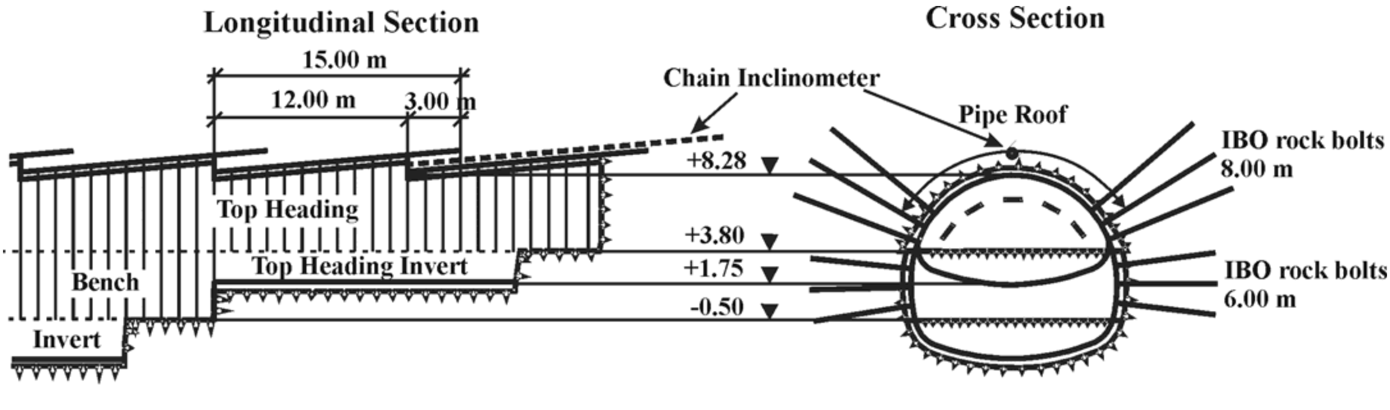

Figure 3. Position of the chain inclinometers [1]

investigations performed to date the inclinometer chain consisted of 10 links, which were connected to each other. Each of these links was 2 m long. The data acquisition system was either situated at the sidewall of the tunnel or in an office.

As can be seen in figure 3 the inclinometer chain was installed parallel to the pipes in the roof region. To protect the instrumentation from the construction work a 21 m long pipe was specially installed above the pipe roof pipes during the pipe roof support installation. In the time of the grouting process of the pipe roof pipes the inclinometer casing as well as the instrumentation was installed. Connecting the inclinometer chain to the data acquisition system was realized before the excavation under the newly installed pipe roof started. With this system it was possible to measure the total deformation path of the pipe roof pipes in the crown in real time.

Since the entire inclinometer chain will most likely move during the measurement period only a relative measurement is possible. To fix the inclinometer vertically in the absolute tunnel coordinates it is necessary to measure its exposed position geodetically. A geodetic target was fixed to the end of the inclinometer casing during installation and it is measured to provide a zero reading before the tunnel excavation resumes. The displacements of this target are then measured routinely during the normal geodetic survey to tie the inclinometer measurements with the tunnel displacements.

3.3. Data evaluation

Compared to the geodetic survey, which was performed once or twice a day, for the inclinometer we choose a data storage rate of once per minute during these projects. This allowed a very detailed assessment of the construction induced settlements along a traverse both ahead of and behind the tunnel face position.

In figure 4 the measured data at a point near the face is shown as a time-settlement line diagram for one excavation round. The practically continuous measurement of the settlement behavior allows the influence of each construction phase to be identified. The time-settlement line as shown only describes the settlements at one location along the inclinometer chain. To evaluate the influence in the longitudinal direction it is necessary to use a deflection curve diagram.

With the utilized data acquisition rate deflection curve diagrams [10] can be constructed for either single excavation steps or for the overall tunnel advance. A single deflection curve connects the displacements in the longitudinal direction at a specific point in time. Figure 5 shows the deflection curves for each construction phase of a single excavation step. The excavated length is highlighted in grey and the measurement locations are represented with the filled circles. Systematic evaluations of this plot type can be used to identify local changes in the ground response ahead of the excavation. This plot shows only the influence of one excavation step and has a high sensitivity to the local ground conditions. To evaluate the influence of multiple excavation steps on the settlement characteristics both ahead of and behind the tunnel face a deflection curve diagram is constructed from each excavation step as shown in Figure 9. This diagram can be used to assess global changes in the ground response. This figure discussed in detail in relation to the numerical simulations in Section 4.3.

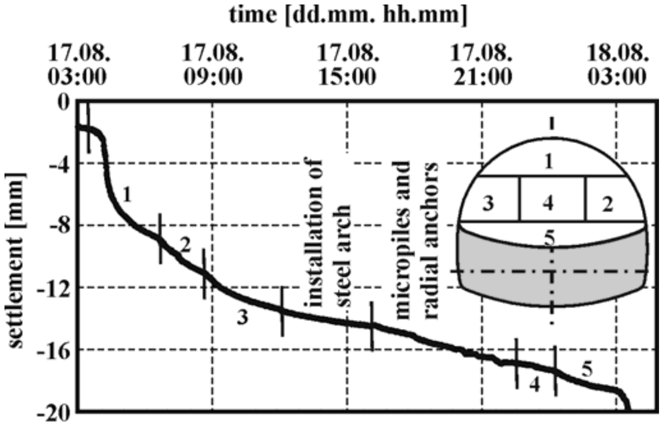

Figure 4. Time settlement line including the installation of radial rock bolts [11].

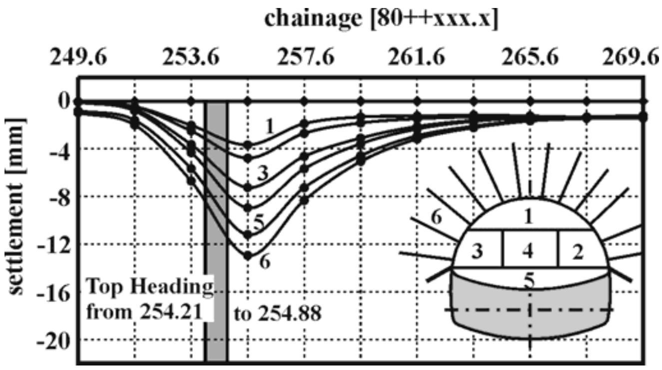

Figure 5. Deflection curve of one excavation step [12].

3.4. Results of the measurements

The settlement characteristics over time induced by one excavation step starting at 03:00 on the 17th of August is exemplarily displayed in figure 4.

The excavation of each phase can be identified by slowly increasing settlements when the excavator broke the shotcrete temporarily supporting the face. This is followed by a rapid increase in the settlement magnitude resulting from the excavation of the rock mass directly beneath the inclinometer. The stresses stored in this material have to be transferred to the remaining ground. This stress transfer induces the development of the displacements. When the unsupported span and the new face areas are supported with shotcrete the increase of the settlement values slows down. This observed behavior indicates a time dependent stabilization process around the heading. It should be noted that the measurement location influences this diagram, if a deflectometer is used at other locations (measure both horizontal and vertical displacement components) the importance of the different excavation phases will change.

After the excavation of the first three phases the installation of the support consisting of wire mesh, steel girders and shotcrete took place. In this time the stabilization process continues (figure 4) and the settlement velocity is decreasing.

With the high accuracy of the measurement system the settlements, induced by drilling radial rock bolts and micropiles in the top heading footing, could also be measured. In this case the settlement path indicates a change from stabilization to linearly increasing during the time of the drilling process, after which stabilization continues (figure 4).

The single deflection curve in figure 5 demonstrates the influence of the excavation step from chainage 254.21 to 254.88 on the settlements in the longitudinal direction of the Trojanetunnel. It can be seen that the majority of the settlements occur ahead of the face. Both the excavation steps and the installation of the radial bolts display a comparable distribution of settlements in the longitudinal direction. Due to the relatively stiff lining used on this site [11] the settlements induced by the excavation process behind the face are rather small. This behavior showed to be characteristic on this particular site. In contrast to this; face bolts, spiles or pipe roof installations cause settlements primarily ahead of the face. With geodetic monitoring alone, only a minor part of the total displacements can be recorded [1], possibly leading to wrong conclusions about the system behavior.

In the last decades methods have been developed to use the changes in the displacement vector orientation from geodetic monitoring data for the prediction of changes in the rock mass quality ahead of the face [13, 14]. In cases of a rather stiff lining however the value of this method of data evaluation is limited. The chain inclinometer on the other hand extends up to 20 m ahead of the face. This allows observing untypical deformation characteristics in this section directly. The excavation and support system can be adjusted to the ground conditions ahead of the face [12].

4. NUMERICAL SIMULATION

The results of on-site observations and the advanced evaluation of the measured data indicate that a tunnel supported with a pre-support system can only be simulated correctly in a full three-dimensional numerical simulation; as also discussed by Hoek [5]. Tunneling in weak rock masses usually generates local failures in the unsupported span as well as unstable face conditions. Only a three-dimensional model with an adequate mesh may reproduce these problems and allow investigations on the supporting systems in detail.

Recently several authors [15, 16 and 17] have used three-dimensional numerical studies to do investigations on the pipe roof umbrella support system. The support system was modeled as a homogenized area at the outer perimeter of the tunnel. The settlement magnitude reported from the numerical calculations was up to 10-times smaller than the amounts measured on site.

With the assumption that a homogeneous rock mass is adequate for the numerical model due to the fact that the measured deformations did not indicate a big influence of the pre-existing structures of the rock mass, a FLAC-3D model was created for the numerical investigations. For this discussion the measurements and numerical simulations are related to the Trojanetunnel discussed above. Two different models are discussed. One utilizes the pile elements in FLAC-3D to simulate the pipe roof as measured and the other is a comparison to the homogenation method where a shell of improved ground is used to instead of the pile elements.

4.1. Numerical Model

In order to decrease the boundary effects in the excavation direction the model was created with a length of 100 m. The area below the excavation is 15 m. From the sidewall of the tunnel to the outer edge of the model a distance of 35 m was used. The overburden is dependent on the investigated project, either 15 m or 40 m. Due to memory limitations it was necessary the use an axial-symmetric model.

Rock mass properties were determined for the pre-peak behavior with the values reported by Zlender [8] for the Trojanetunnel. The post-peak behavior was adopted with laboratory results from shear tests on fault zone material [7].

Table 2. basic rock mass properties used in FLAC-3D.

	Value	unit
K	80	[MPa]
G	28	[MPa]
$\varphi_{elastic}$	19	[°]
φ (ε_{pl}=1%)	35	[°]
φ (ε_{pl}=5%)	30	[°]
$\varphi_{residual}$	30	[°]
$c_{elastic}$	0.040	[MPa]
c (ε_{pl}=1%)	0.019	[MPa]
$c_{residual}$	0.019	[MPa]

The area of the excavation and the surrounding area are created with a maximum zone length of 0.5 m this allows high enough resolution to capture local failure and plastic deformations while maintaining a reasonable model size.

The shape of the tunnel is modeled as that of a pipe roof umbrella supported top heading excavation including the top heading invert. The implemented support systems are shotcrete, heavy steel beams or lattice girders, radial bolts, face bolts and pipes.

4.2. *Excavation Sequences*

The excavation and support sequence is explained and shown in figures 6 to 8. In figure 6 the top heading is displayed before the next excavation takes place. The stresses are shown only for the rock mass while the support is shown in a light grey tone. The stress transfer related to the last excavation step is finished and the support consisting of shotcrete with heavy steel beams is installed. The shotcrete strength is raised relative to the age of the single shotcrete slices using the formulations of Aldrian [18] and Müller [19]. The additional working space between the tunnel face and support, which is necessary during construction, is only supported by the pipes at the perimeter. Every excavation step includes a little widening for the necessary working space that is used for

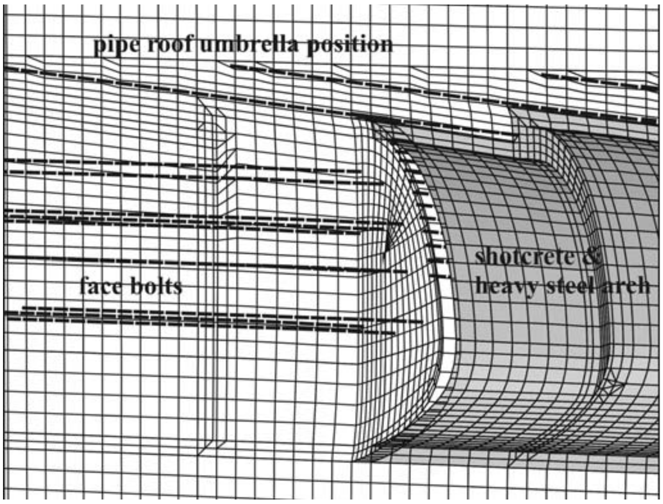

Figure 6. The numerical model before the excavation takes place

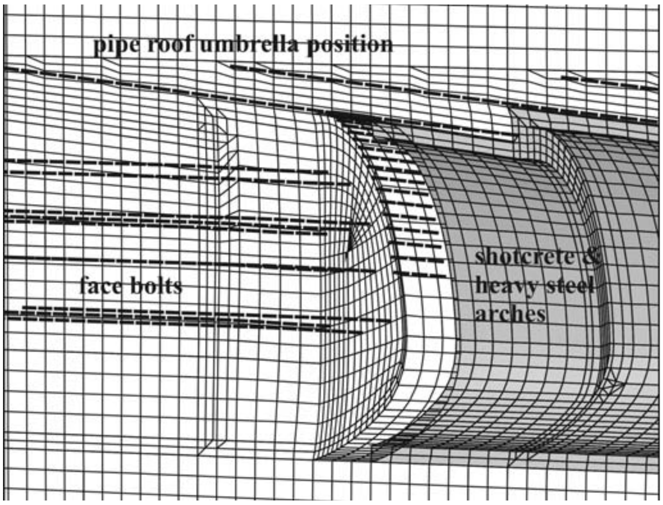

Figure 7. The modeled situation when the excavation has reached equilibrium is done.

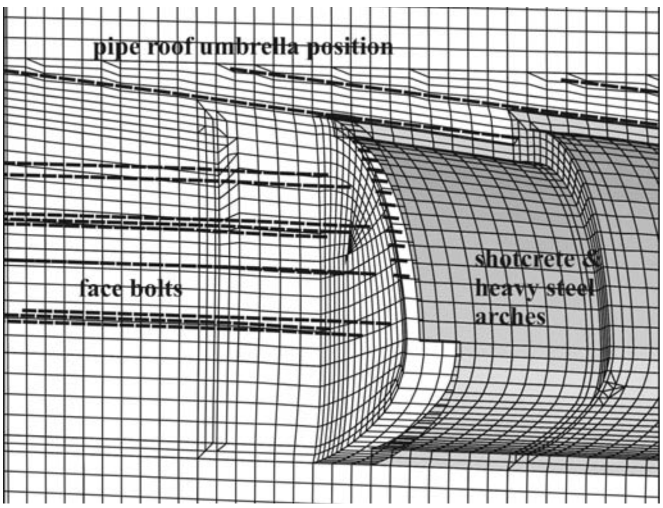

Figure 8. The first supporting phase used in the simulation.

installing the pipe roof pipes, related to the pipe installation angle. This leads to a recurrent geometry in the model. The magnitude of the widening corresponds to the length between 2 pipe roof pipe installations we refer to this hereafter as the pipe roof umbrella field length.

While the face of the top heading was excavated in up to 5 phases during construction we show only the results for excavating the entire face (1 m) in one excavation phase. Figure 7 displays the stress distribution after reaching equilibrium after one excavation step. The next simulation phase is to install the support and update the shotcrete properties for their age as shown in figure 8. This is followed by the installation of the top heading invert shotcrete, which completes the excavation cycle and leads to the starting position for a new excavation step as shown in figure 6.

4.3. Results of the Numerical Simulation

Pile Element -Model

The model was developed to investigate the influence of pre-support on the ground behaviour and vise versa. Therefore the model considers all the pre-support elements utilized during construction. The standard support was shotcrete, heavy steel sets and 35 face bolts (IBO 250 kN). The top heading footing was disregarded in the modeling. The pipe roof pipes were modeled as pile elements, which are implemented in FLAC-3D. The observations during construction displayed that the drilling of the pipe roof holes ahead of the face in this weak rock also produces movements [20]. This led to the assumption that the grouting in the annular gap cannot be guaranteed. For this reason the grout was neglected and the rock mass parameters were used for the determination of the parameters, which control the pipe – ground interaction. The strength parameters of the rock vary with plastic strain therefore the properties controlling the interaction between the support elements and the ground were continuously adapted to the actual rock mass parameters.

The settlements calculated in the numerical simulation are measured and evaluated using the same geometry as during construction, with an additional, longer pipe in the roof. We show the modeled settlements as a deflection curve diagram to show the longitudinal settlement characteristics for the excavation steps. The upper most diagram in figure 9 shows a deflection curve measured at the Trojanetunnel. This, as well as the two lower diagrams, does not include the settlements occurring before the installation of the pipe roof. Each line represents the settlement values related to 2 excavation steps. The trend line separates the pre-settlements ahead of the face from the settlements occurring behind the face. The settlements behind the face are nearly constant with a magnitude of 35 mm. The measured pre-settlements continuously increase during construction. At the end of the pipe roof field the measured pre-settlement value is around 85 mm.

Using the described excavation sequence and the support installed at the Trojanetunnel the results of the numerical calculation display a similar result to that measured during construction. The calculated displacements behind the face are approximately 30 mm. After a comparable advance length the settlements occurring ahead of the face are 72 mm.

The characteristic settlement behavior measured at the Trojanetunnel is displayed in the upper diagram of figure 9. The settlements slowly increase from the end of the measurement section to about 6 m ahead of the face. At approximately 6 m ahead of the face the settlements magnitude begins to increase indicating the zone directly influence by the excavation. At approximately 2m ahead of the face the settlements increase linearly to the face position. In the first 3 m behind the face the settlement increase slows down and approximately 3 m behind the face nearly no additional settlements can be measured. A comparable characteristic settlement behavior was simulated using the pile elements as pipe roof pipes (figure 9 middle diagram).

Homogenized Model

The lower diagram of figure 9 displays the results of a calculation simulating the pipe roof pipes as a homogenized area with a thickness of about 40 cm at the outer perimeter of the tunnel. Only the Compressive Modulus, Bulk Modulus and the Cohesion were adopted using the surface percentage of the ground mass, pipes and grout and their properties. Using this simplification the settlement values behind the face decrease to 10 mm, those ahead of the face to 25 mm. With this model type the magnitude of the calculated settlements differs considerably and settlement behavior no longer reproduces the field measurements, which is much

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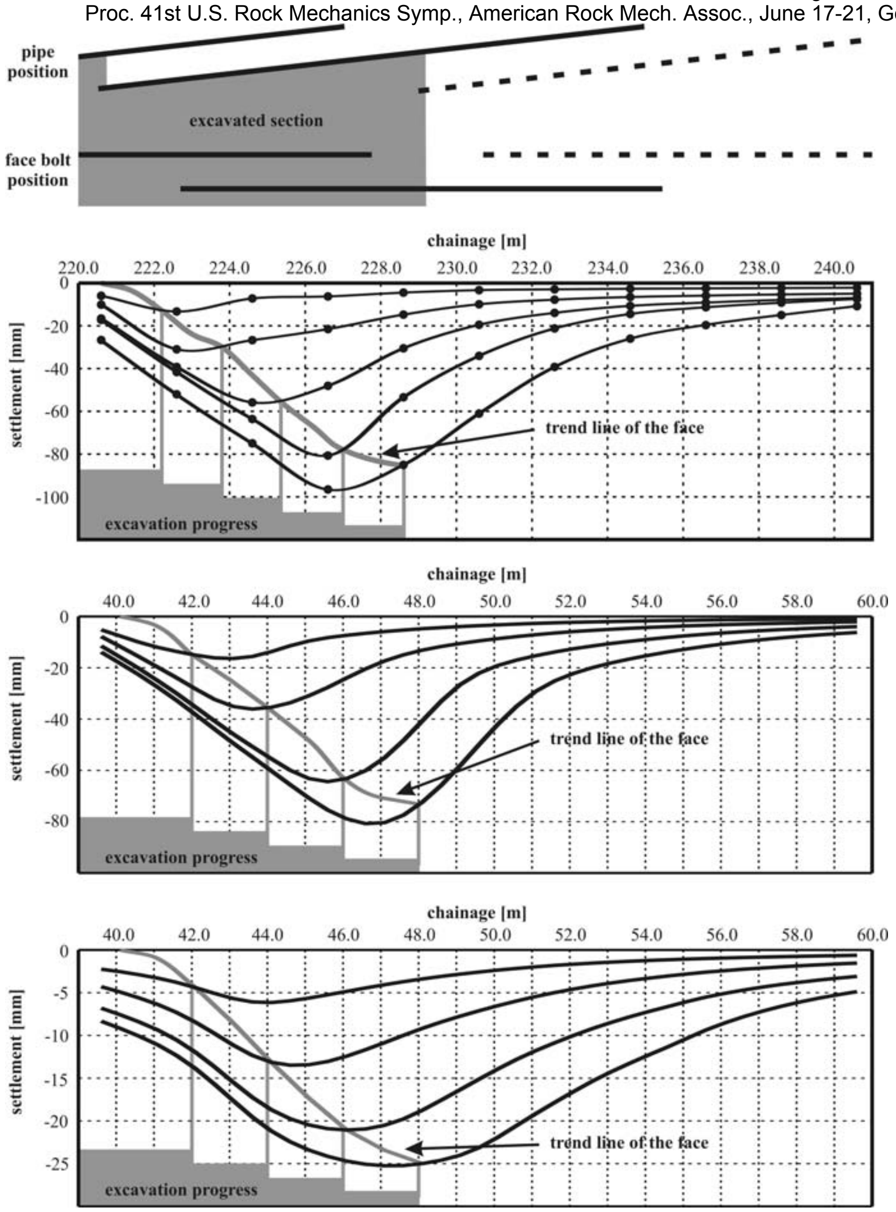

Figure 9. deflection curves diagram for the in situ measurements (above), the structural element model (middle) and the homogenized case (below).

smoother than in the real case. Both of these results indicate that the pipe roof umbrella system cannot be correctly simulated using this simplification.

Settlement Reduction

In the publications using a homogenized model it is stated that the pipe roof umbrella is reducing the settlement values. As discussed before, this numerical model is not correctly simulating the pipe roof support. For this reason the same simulation as shown before was also calculated without the installation of the pipes.

In figure 10 the isolines for the calculated settlements are displayed for the case with pre-support. In the crown the settlement values are lower than 100 mm. This value is lower than the measured one in the pipe because of the pipe embedment. In the simulation the pipes move less than the surrounding material. In figure 11 the isolines are drawn for the case without pre-support. When the excavation advances under the old pipe roof umbrella the settlements did not significantly increase in the calculation. After this the settlement values started increasing and only after 2 more excavation steps the settlement values increased to more than 140 mm in the same position where the supported calculation showed 100 mm of subsidence.

The same settlement characteristics could be observed when the residual cohesion was set to 0.017 MPa but the settlement values logically increased. As shown in figure 12 the settlements at the level of the pipe roof are 150 mm. Similar amounts were also measured during the excavation of the Trojanetunnel. In Figure 13 the same boundaries without the pipe roof support lead to a maximum settlement value of 260 mm at the same position. The percentage of settlement reduction increased with the higher settlement amounts. A reason for this effect is that the bending of the pipes mobilizes the supporting effect of this system.

5. GEOTECHNICAL MODEL

For tunneling in weak ground the pipe roof umbrella system is supporting the excavation in the radial direction at the area of the unsupported span as well as approximately 2 m ahead of the face. The loads taken in this area are transferred both to the support behind the face and the ground further ahead of the supported area (figure 9). Due to the fact that a pipe is also taking bending moments, this system is activated through movements (passive support system). Therefore its effectiveness increases with subsidence in the supported area.

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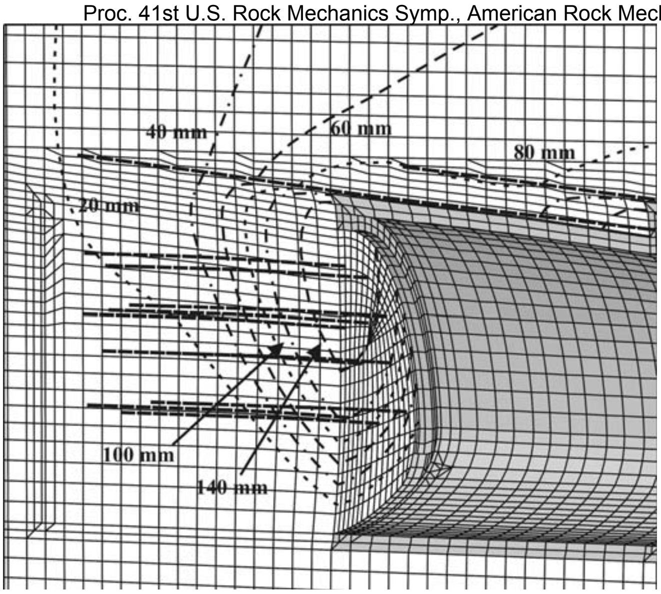

Figure 10. Settlement-isolines for the pre-supported case

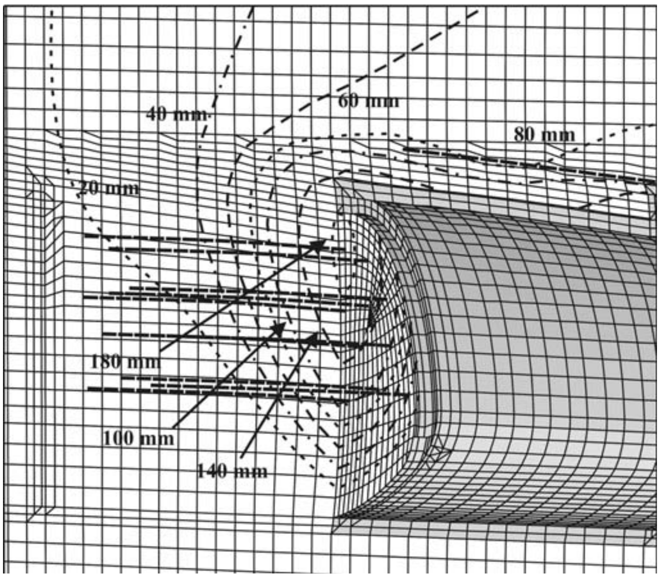

Figure 11. Settlement-isolines for the case without pre-support

Each pipe is independently transferring the loads to its foundation without creating an arch normal to the tunnel axis ahead of the face. This effect can only appear, when a closed pre-support system like grouted columns is used. On this account a homogenization of the ground in the area of the pipes does not correctly reproduce the system behavior in a numerical simulation. Each pipe has to be modeled separately to catch the correct support system behavior.

Thus the pipe roof system is supporting the entire heading it is not strictly a face support system. Additional support e.g. face bolts primarily have to guarantee the stability of the face because the pipes are acting outside the tunnel perimeter and not in the face.

6. DESIGN PARAMETERS

The use of pre-support systems primarily has two reasons. These systems are either used to guarantee the stability of the excavation or to reduce the subsidence. The project dependent limitations therefore influence the design parameters of the pipe roof umbrella system.

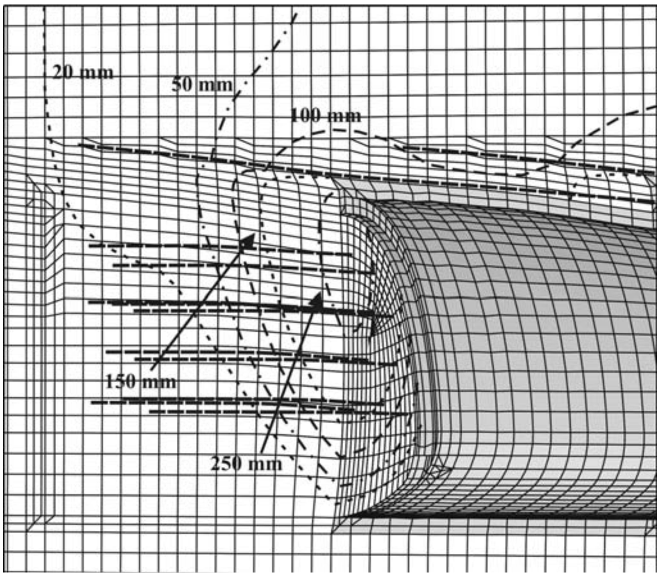

Figure 12. Settlement-isolines for the pre-supported case with lower residual cohesion

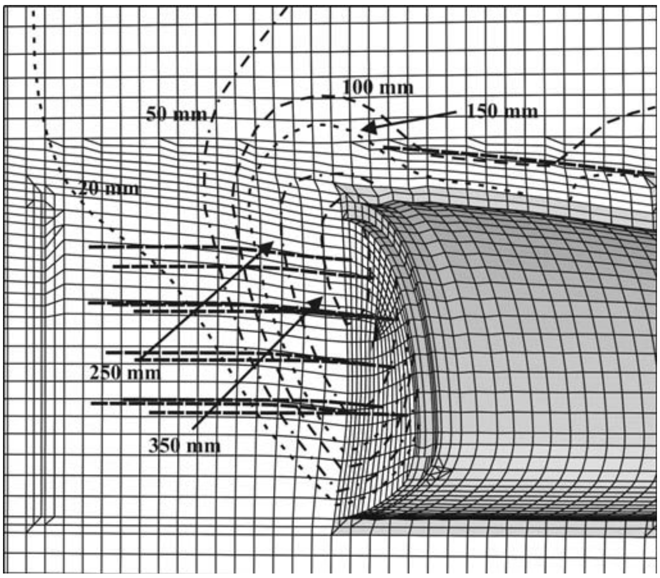

Figure 13. Settlement-isolines for the case without pre-support with lower residual cohesion

This system is a passive support system thus movements are required to mobilize the supporting effects. The developed support pressure is controlled by the pipe diameter and its thickness. The reaction forces of the pipes are related to the settlement magnitude, i.e. the smaller the subsidence should be the stronger should the pipe be. The diameter of the pipes seems to control the rate at which the support effects mobilize, with larger diameters developing the supporting effect faster due to increase bending.

The overlapping length of the pipe roof pipes in the longitudinal direction is dependent on the distance and length of the pipe foundation ahead of the face. Both are a factor of the tunnel shape and the ground quality. If one of the project limitations is subsidence the overlapping length should be the supporting length ahead of the face plus the total length of the foundation. This length can be identified in the data shown in figure 9 with the starting point of increasing settlement values ahead of the face. Every further excavation step is decreasing the foundation length and therefore decreasing the effectiveness of the pre-support system.

The spacing of the pipes in the tangential direction at the tunnel perimeter can either depend on the necessary support pressure or on the rock mass. A minimum value should allow the rock mass to create a local arch between successive pipe roof pipes. This guarantees the designed shape of the tunnel without local failure.

As long as the pipes can be installed correctly the drilling system used to install the pipe roof umbrella is not very import for a stability controlled design. For constructions, which are sensitive to subsidence, the stability and the movements due to unsupported holes for the pipes should be investigated [20]. Once the stability cannot be guaranteed or the unsupported holes significantly increase the settlement values a cased-drilling system should be used. This installation system is less susceptible for creating settlements than a pre-drilling system.

7. CONCLUSION

In the last decades the pipe roof umbrella system has increasingly been used to support tunnels in weak rock masses with low overburden. Because the knowledge about the geotechnical effectiveness is based on experience a measurement program was developed and executed to record the ground – support interaction during construction.

Using this data and laboratory tests for the rock mass and the pipes, numerical investigations were performed to acknowledge and advance the geotechnical model based on the in situ measurement data.

With the knowledge gained in these investigations and the correct parameters for the rock mass behavior under elastic as well as plastic conditions the estimated deformations and the basic design parameters for the pipe roof system can be determined with numerical investigations.

By using an appropriate measurement system including adequate evaluation and interpretation techniques the construction induced settlements can be controlled in critical sections, which are sensitive to subsidence during construction. Additionally, the support can be adopted to the actual rock mass quality depending on the project requirements.

Both the information gained from numerical analyses and the possibility to control the effectiveness during construction through appropriate monitoring result in a safe and economical construction advance in tunnels supported with a pipe roof umbrella system.

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