Günther M. Volkmann

Function, Design, and Specifications for Pipe Umbrella Support Systems

Doctoral Thesis

Department of Civil Engineering

Graz University of Technology

Reviewers:

Schubert, Wulf, O.Univ.-Prof. Dipl.-Ing. Dr.mont.

Institute for Rock Mechanics and Tunnelling
Graz University of Technology

Likar, Jakob, Prof. Dr.

Retired Professor from University of Ljubljana
Authorized representative of Geoportal d.o.o.

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Acknowledgement

Actually, I owe it to a coincidence that I have written a dissertation. If I had not become an assistant, it would never have occurred to me to write a dissertation. On the one hand, I owe this to the coincidence that a fellow student had to do his military service - otherwise I would not have applied for this job - and, of course, Wulf, who chose me from the candidates. That was a mystery to me, because I did not recommend myself during an excursion the year before. Alright, now I was an assistant and I wanted beneath teaching explore an interesting topic. After learning the ropes, a suitable topic was found and from that moment on days and weeks evaporated, because everything was fascinating, new results were analyzed and discussed and there was always something to do.

Wulf was a very important navigator at the time, guiding me through his profound knowledge and experience, and saving me from taking many one-way streets. This saved me a lot of time, which I lost later on by my own. The freedoms he gave me as an assistant enabled me to break new ground and get experience. Thank you very much, Wulf!

In addition to the institute work I spent a lot of time primarily on two construction sites - the Birgl Tunnel and the Trojane Tunnel - with new measurement techniques and its interpretations. A stroke of luck for me, because on both construction sites all sides supported me. Therefore, I would like to take this opportunity to thank everyone involved, the owners, the owner’s representatives and the construction companies for their support - I have never learned so much in such a short time and it was for me the most informative and interesting time of the entire dissertation.

At the Trojane Tunnel, Teddy very much supported me. In my eyes, he proves that genius and madness are close to each other. No one else spent so much time with me to discuss my scientific findings during the day and at night. At some point during this time we became good friends - Thanks Teddy for the many nice and fascinating hours!

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Since I wanted to make no compromises in my research, the contract period as an assistant was not enough to clarify all questions. Luckily, I was supported by ALWAG company from that point on. So, I was allowed to continue to use the infrastructure of the university and was funded financially from the economy. Thank you Wulf & Thank you Karl for making this impossible thing possible for me. Despite all these efforts, the dissertation unfortunately did not want to be completed at this time.

The years went by, experience got bigger, new developments came ... one woman, two sons, everything went great even without a dissertation. But then - almost surprisingly - came the information that the curriculum for my PhD expires & that meant: now or never? The decision was "now"! Especially, during the last time, I would like to thank my sweetheart! Karin, you had to endure many sentences like "I have to work" or "Unfortunately, I have no time" - Thank you so much, that you nevertheless supported me with full power and parallel you had our two little sweeties also "always" under control.

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Kurzfassung

Wirkungsweise, Planung und Spezifikationen von Rohrschirmsystemen

Rohrschirmsysteme gehören im Tunnelbau in die Gruppe der vorauseilenden Stützmittel. Sie werden vorwiegend im weichen Baugrund eingesetzt, um einen sicheren und stabilen Vortrieb zu gewährleisten. Rohre mit einem Außendurchmesser von 90 bis 170 mm werden an der Außenseite des Tunnels von der Ortsbrust nach vorne eingebohrt. Diese Rohre bilden dann während des Vortriebes einen Schirm über dem Arbeitsbereich, stabilisieren den umgebenden Baugrund und schützen die Arbeiter. Typische Einsatzgebiete sind: der oberflächennahe, innerstädtische Tunnelbau, Tunnel in fluvialem oder maritimem Baugrund oder im Hangschutt, Störungszonen, Portalbereiche und das Wiederauffahren von verbrochenen Tunnelabschnitten.

Beim Einsatz der Rohre als Rohrschirm ist das Biegeverhalten entscheidend, deshalb wurden im Labor Versuche durchgeführt um die ausschlaggebenden Stützmitteleigenschaften festzustellen. Die durchgeführten Biegeversuche zeigen den Einfluss von Füllungsgrad, Injektionslöchern oder Verbindungen auf die Tragwirkung der Rohre. Dabei stellte sich heraus, dass die Verbindungstype einen entscheidenden Einfluss auf die Tragwirkung des Stützsystems hat.

Die Wirkungsweise eines Rohrschirms wurde auch in-situ, baubegleitend gemessen. Dabei kam ein neuartiges Messsystem zum Einsatz - eine online Inklinometerkette. Durch die gesammelten Daten konnte das Systemverhalten des Rohrschirms beobachtet und definiert werden, Rückrechnungen der Kräfte im Rohrschirm gemacht werden und dafür notwendige Interaktionskräfte zwischen Baugrund und dem Stützmittel errechnet werden. Diese Ergebnisse präzisieren die Stützbereiche und Auflagerbereiche eines Rohrschirms und können zur Definition eines analytischen Berechnungsmodells für die Planung verwendet werden.

Aufbauend auf den bereits erwähnten Erkenntnissen wurden dreidimensionale Berechnungen durchgeführt und durch die in-situ Messdaten validiert. Dabei konnten alle Charakteristika der gemessenen Verschiebungen abgebildet werden und eine Übereinstimmung in Größe und Ort gezeigt werden. Somit liegt der Schluss nahe, dass alle relevanten Mechanismen in den dreidimensionalen Berechnungen abgebildet wurden. Variationen der Rohrschirmparameter im gleichen Berechnungsmodell zeigten, dass ein Rohrschirm nicht nur einen Einfluss auf die Stabilität hat, sondern auch auf die Größe der Gesamtsetzungen, die durch den Tunnelvortrieb verursacht werden.

Zusammenfassend werden notwendige Spezifikationen für den Rohrschirm als Stützmittel im Tunnelbau definiert. Das geotechnische Modell wird dargestellt und analytische beziehungsweise numerische Berechnungsmethoden für die Planung gezeigt.

Schlagwörter:
Tunnelbau, weicher Baugrund, Rohrschirm, vorauseilendes Stützmittel, Stahlrohre, Baugrundverbesserung, dreidimensionale numerische Berechnungen, analytische Berechnung

Abstract

Function, Design, and Specifications for Pipe Umbrella Support Systems

Pipe Umbrella systems are categorized in the group of pre-support measures in tunneling. They are predominantly used in soft ground to ensure a safe and stable construction process. Pipes with an outer diameter from 90 to 170 mm are drilled at the outer perimeter of the tunnel from the actual face to the front. These tubes then form an umbrella over the working area during excavation, stabilize the surrounding ground and protect the workers. Typical areas of application are: shallow, urban tunnels; tunnels in fluvial or marine deposits or talus, fault zones, tunnel portals, and the recovery of collapsed tunnel sections.

When the pipes are used as pipe umbrella, the bending behavior is crucial, and therefore tests have been carried out in the laboratory to determine the dominating support properties. The bending tests show the influence of filling grade, injection holes or connections on the load bearing capacity of the pipes. It turned out that the connection type has a crucial influence on the supporting effect of this support system.

The function of a pipe umbrella was also measured in-situ parallel to construction. A new measuring system was used for this investigation - an online inclinometer chain. With the collected data, the system behavior of pipe umbrellas could be observed and defined, back-calculations of internal pipe forces were made and consequential interaction forces between the ground and the support measure were calculated. These results specify the supporting sections and foundation areas of pipe umbrellas, and were used to define an analytical calculation model for design.

Based on the already mentioned findings, three-dimensional calculations were carried out and validated by the measured in-situ data. All characteristics of the measured displacements could be reproduced and agree in magnitude as well as in location. Thus, it can be concluded that all relevant mechanisms were reproduced in the three-dimensional calculations. Variations of the pipe umbrella in the same calculation model showed that a pipe umbrella has not only influence on the stability, but also on the deformation magnitude induced by the tunnel construction.

Consequently, necessary specifications for pipe umbrellas are defined as tunnel support measure. The geotechnical model is explained and analytical as well as numerical calculation methods for design presented.

Keywords:
tunneling, weak ground, pipe umbrella, canopy tube, umbrella arch, long-forepoling, pre-support, steel pipes, ground improvement

Contents

1 Introduction

Tunnel supports shall guarantee stability and safety, as well as allow long service life. For a long time, the support consisted of wood and stone, but a new era of support begun with the development of shotcrete and the ideas of the New Austrian Tunnel Method. Although the ground was still passively supported by shotcrete, the surrounding ground was improved in its load bearing capacity by bolts. In addition, in the case of difficult ground conditions, the cross-section was excavated in several stages.

In a next step, pre-support measures were introduced; Face bolts support the face, while lagging boards and spiles support the perimeter of the open span. This was not always sufficient, so specialized machines were developed, which could install heavier supporting measures at the perimeter of the open span. This allowed the first time to install a so-called pipe umbrella support. Because of further developments in drilling machinery, pipe umbrellas could soon be installed with standard drilling machines. To date, both technologies are still in use when it comes to the installation of pipe umbrellas as pre-support in tunneling.

But why is a pipe umbrella support needed at all? Tunnels are frequently built in challenging, weak ground. Under these conditions the probability of overbreaks increases even with partial face excavation. To allow for a reasonable construction process additional support is necessary. This is the reason why pipes are drilled at the outer perimeter of the tunnel, reaching ahead of the face. During the following excavation steps these pipes support the ground against local instability. As a result, the tunnel construction in challenging, weak ground can safely be executed under stable conditions and costs remain relatively low compared to more intensive construction measures such as horizontal jet-grouted columns or freezing technologies.

Because the pipe umbrella method is cost-efficient and can be installed in relatively short time, it has been used intensively as support in shallow, urban tunnels and tunnels through fault zones in mountainous tunnels. Furthermore, this system is also used for portals to increase safety due to uncertain stress conditions or to re-excavate collapsed tunnel sections.

The rapid increase in application could not be followed by the technical knowledge, and therefore generally accepted rules for design and construction were lacking for a long time.

1.1 Objective

This work focuses on the following main areas:

- Determination of mechanical properties for pipe umbrella pipes and its features
- A geotechnical model for the pipe umbrella method
- Comprehensible design rules

A pipe umbrella simply consists of pipes, which are drilled into the ground as a supporting measure, so everything should be clear from a static point of view. This statement may be correct, but these pipes have injection holes or connections and they are filled with cement suspension after drilling; so, the influence of these system-related characteristics must be clarified and assessed.

After installation of the pipes, the excavation is started again, and the support of the pipe umbrella is activated. The involved mechanisms, the resulting supporting effects or loads for a pipe umbrella must be described and defined. Investigations must clarify these characteristics and allow for developing a simple comprehensible design procedure.

On this basis, further investigations can be performed, which allow to convert a currently experience-based design into a technically sound design. If this is possible, this support measure can be further optimized, used more cost- and time-efficient in tunnel construction.

1.2 Method

The right way to gain conclusive approaches is the observation and measurement during tunnel construction. The experience and data collected on one site probably cannot be transferred to all projects, but they provide a basis for a certain scope of application, which provides clear solutions under similar conditions.

Therefore, in this work, the focus will be on observations and measurements during construction at the beginning. Afterwards, when all involved mechanism, connected to pipe umbrella supports are identified, further optimization measures can be investigated by, for example, numerical simulations.

The supporting pipe itself can only be specified on the basis of suitable laboratory tests. Therefore, proper tests must be defined as soon as the loads acting on a pipe are identified. Based on a sufficient number of tests, the decisive parameters can be determined.

2 Literature Review

The pipe umbrella method started in the 1980's. Advances in specialized drilling equipment allowed installing the pipes with bigger diameters into the ground horizontally. Its origin is in Italy; nevertheless, it also started in Japan nearly at the same time. The first considerable installation methods were the TREVITUB method (Italy), the RODINTUB method (Italy) and the AGF method (Japan). Both Italian methods use special patented machines for the installation of pipes while the Japanese method installs the pipes with conventional drill jumbos (Bruce et al. 1987, Muraki 1997).

The structural properties of the pipes changed over time. The increasing power of the machines have probably caused this effect. Bruce et al. (1987) describe pipes with an outer diameter of 56 mm or 76 mm. In 1997, Muraki's descriptions include pipes up to 100 mm as outer diameter, while current publications specify a range up to 200 mm as outer diameter for the Pipe Umbrella System.

Although this system is applied for approximately 40 years in tunneling, it is termed differently in publications. Bruce et al. (1987) used the term Horizontal Micropiling. Muraki (1997), who did the first classification of Umbrella Methods, used the term Injected Steel Pipe Umbrella Method for a series of 12 m to 15 m long, slightly upward-inclined pipes. These two early publications additionally mention the Italian term "Infilaggi" for the horizontal reinforcements. In newer publications, this forepoling method is termed as follows:

- Umbrella Arch (Method) (Carreri et al 1991, Kim et al. 2004, Hefny et al. 2004, Choi & Shin 2004, Kim et al. 2005, Eclaircy-Caudron et al. 2006, Ocak 2008)
- Steel Pipe Umbrella (Oreste & Peila 1998), Tube Umbrella (John & Mattle 2002), Forepole Umbrella (Hoek 2005),
- Umbrella method (Harazaki et al. 1998)
- Pipe Roof (Mager & Mocivnik 2000, Cirman et al. 2001), Pipe Roof Umbrella (Walchshofer, 2002), Pipe-Roofing (Bizjak & Petkovšek 2004), or injected steel pipe roof (Stojković et al. 2002), steel pipe roof (Fiumara et al. 2006)
- (Grout Injection) Long Steel Pipe Forepiling (Fujimoto et al. 2002, Yokoo et al. 2002; Kimura et al. 2005) or Long-Span Steel Pipe Fore-Piling (Miura 2003, Haruyama et al. 2005)
- Steel Pipe Canopy (Gibbs et al. 2002)
- Pipe screen protection (Maurhofer & Glättli 2004)

- All Ground Fasten (AGF) (Miwa & Ogasawara 2005)

As can be seen many different names are in use even though there is a very early, published classification system (Muraki, 1997).

2.1 Classification of Umbrella Methods

The Umbrella Method in general describes ground reinforcement techniques shaped like an umbrella around the tunnel prior to the excavation. Muraki (1997) defined 3 categories of the Umbrella Method.

- sub-horizontal jet-grouting method
- injected steel pipe umbrella method
- pipe roof method

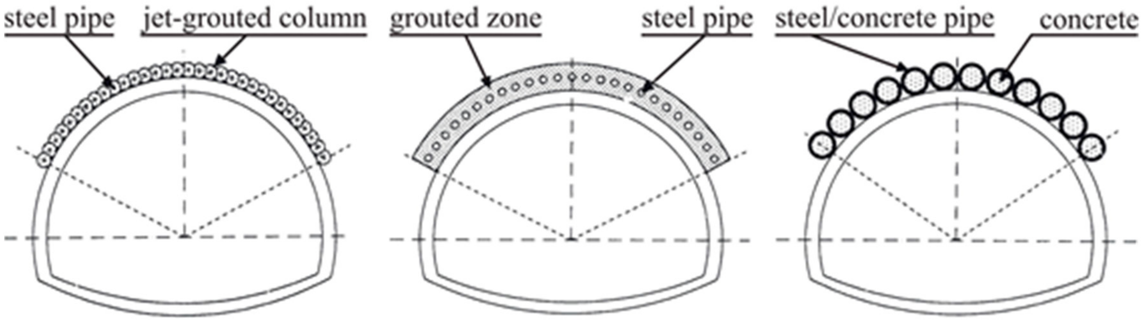

Figure 1. Umbrella types defined by Muraki (1997): from left to right; reinforced sub-horizontal jet-grouting method, injected steel pipe umbrella method and pipe roof method

20 years after this publication all three categories are in use and as mentioned by Muraki, the sub-horizontal jet-grouting method is used unreinforced or reinforced. Since then, ground freezing techniques evolved having the same principle shape so they could be added to Murakis classification as fourth group.

Due to this classification system the term "Pipe Umbrella Method" is used in this work.

This means that pipes are installed prior to the excavation at the outer perimeter of the tunnel ahead of the face to improve stability of a tunnel during excavation (Hefny et al 2004). Typical outer pipe diameters range from 60 mm to 200 mm (John & Mattle, 2002) with a pipe wall thickness from 4 mm to 12.5 mm (Oreste & Peila 1998, John & Mattle, 2002). A typical pipe length ranges from 12 m to 18 m. The flushing media can be air, any air-water mixture, water, or water with blows of pressurized air. The pipes are grouted (filled with cement suspension or chemicals) after drilling so the inside and the annular gap are filled. Additionally, permeation or fracture grouting treats the ground.

2.2 Supporting Mechanisms and Effects

Muraki (1997) states that Umbrella Methods create an arch-like shell ahead of the tunnel, which enables to excavate the tunnel under this support element. Umbrella Methods in general transfer the loads in both, the longitudinal and the transverse direction of the tunnel. This leads to the following effects during tunnel construction:

- Restricting ground surface settlements
- increase of face stability
- prevent slope failures and/or landslides
- seems to make it possible to decrease the quantity of the tunnel supports

Muraki mentions that the Pipe Umbrella Method, in general, transfers loads only in the longitudinal direction, but the injected grout, which treats the surrounding ground, may create an arch-like support in the transverse direction. The stiffness of this system consists of the following components:

- driven steel pipe
- grout filling inside the pipe and in the voids surrounding the pipes
- treated ground by permeation or fracture grouting

The ground conditions govern the importance and impact of the grouting (Muraki, 1997):

- cohesion less uniform soils: grouting is of vital importance to prevent caving of soil between adjacent tubes.
- morainic soils with a more or less closed matrix: the purpose of the grout is to fill open voids in the matrix.
- ground with real or apparent cohesion: the grout fills the tubes inside and the voids surrounding the tubes.

The injection of the grout is performed with either a single valve injection or a total valve injection. When using the first technique a double packer system charges each individual valve, while the grout is simultaneously injected through all valves of one individual tube when using the second technique (Muraki, 1997).

Regarding this point John & Mattle (2002) mention that the grout does not intrude into the ground. To their opinion, the pipe umbrella assists to achieve stability in the unsupported area and at the tunnel face by bridging loads ahead of the face in the longitudinal direction.

2.3 Case studies

Many articles are describing all different kinds of case studies without giving detailed technical information. As an outcome of these articles it can be summarized that pipe umbrella systems are mostly used at shallow tunnels but also in deep tunnels or portal situations. Additionally, this system is often used to recover collapsed tunnel sections. Typical ground conditions are soil, fault zones, (heavily) weathered rock mass, and morainic soils.

Most described cases were supported with the following pipe dimensions: 114.3x6.3 or 139.7x8.0. The primary lining was commonly equipped with an elephant foot that was additionally fixed with micropiles into the ground. The pipe lengths range from 9.0 m to 18m, in portal situations with only one pipe umbrella up to 30 m. The overlap in the longitudinal direction starts typically at 3 m and goes up to 8 m. The axial distance between the pipes is usually between 300 mm and 600 mm.

2.4 Empirical Results (Approaches)

Ishibashi et al (2002) explain with real data and numerical calculations the different support influences on the settlements. They analyzed the influence of stiffer support behind the face, in the face area and the periphery around the tunnel ahead of the working face. Their results show that a stiffer support system behind the face decreases the deformations as well as an additional support around the tunnel periphery, while an increase of ground stiffness ahead of the cutting face shows only small influence on settlements. The case study presented in this paper deals with a tunnel with pipe umbrella support. Inclinometers allowed measuring the induced roof settlement and the measurements indicate that increasing the pipe-overlap in driving direction clearly decreases the settlements.

Shin et al. (2008) published the results of a laboratory large-scale model test that investigated the influence of reinforcing patterns at the heading on stability, settlements, strains and stresses. They discovered that pre-reinforcement in general increases failure resistance and consequently increases heading stability. The tested pipes act as embedded beams transferring load in the longitudinal direction. Face reinforcement limits the settlement development, particularly when combined with crown reinforcement. Their investigations on the pipe length resulted in a failure plane with an inclination of 45°+φ/2 at the face. So, the pre-reinforcement at lower areas of a tunnel may be shorter than those at crown area.

Harazaki et al. (1998) describe the observed ground behavior induced by the excavation process of the Maiko Tunnel in Kobe (Japan) with evaluated on-site measurements. A detailed description of findings due to strain measurements in the pipes during construction follows. They conclude that the bending moments in the crown pipe correspond to those of a beam model. The remarks described at the conclusion are divided in two sections: effect in good ground (Diluvium) and effects in poor ground (Alluvium). The main difference is the position of highest bending moments in the crown pipe related to the face position. Because of this difference, they conclude that the main effect of a pipe umbrella might be the decrease of ground displacements ahead of the face to sustain the stability of the tunnel face in good ground. In poor ground conditions the deformation-decreasing effect is much smaller, because the loaded area might extend relatively far ahead of the face. On the contrary, the described surface measurements indicate an increase of settlements during installation of a pipe umbrella. This increase was primarily measured ahead of the face, where the drilling process happens. The authors propose an improvement of the ground ahead of the face by reinforcement and a stiffening of the umbrella arch to obtain a better settlement-decreasing effect by installed pipe umbrellas.

2.5 Analytical Approaches

Oreste & Peila (1998) describe a very sophisticated analytical approach and compare these results with results from 2-D FLAC analysis. By using this formulation, the design can be performed in a three-step analysis. First, the structural properties of the pipe umbrella pipes must be defined, and consequently mechanical parameters calculated. Secondly, one needs the stiffness properties of both foundations. So, the stiffness of the spring series representing ground, and/or steel sets is calculated. The series of ground springs depends on the deformation modulus of the ground, the distance between the springs, the pipe spacing in the transversal direction and the depth of the influencing zone. The single springs for the steel sets depend on the tunnel diameter, the structural properties of the steel sets as well as its foundation stiffness, the vertical load, the mean displacement in the crown and the transversal distance of the umbrella pipes. The third and last step of this approach allows calculating internal forces as well as deformation of umbrella pipes.

Möhrke (1999) classified three different types of collapses or over-breaks in pipe umbrella supported tunnels due to experience at railway tunnels between Platamon and Leptokaria (Greece). He describes that most observed failures started at the face. As a reason for this, he mentions a combination of fault zones near the face and the load transfer in the longitudinal as well as vertical direction to the face area by the pipes. He only mentions two cases, where local failures of a face could extend beyond the supporting pipes. Based on a loading model of Terzaghi (moveable ground above a ductile support) he defines the acceptable stress of the steel pipes. The calculation of the stresses is based on DIN V ENV 1994 Teil 1-1 EUROCODE 4 (EC4). Two real failures are back-calculated, one in the middle of a Pipe Umbrella field and one at the end. Due to the differences in the occurring moments, the elastically founded continuous beam did not fail, while the cantilever type failed. This was comparable to experience from this site.

Anagnostou (1999) describes another analytical approach. His examples are based on experience from the Metro Athens. The introduced formulation assumes that the pipes transfer the loads to both foundations, the ground ahead of the face as well as the primary lining. He assumes that the pipes are founded rigidly on the primary lining in the vertical direction and that this connection is pin-hinged to simplify the calculation. The foundation in the ground is supposed to be elastic. Obviously, this approach calls for stable face conditions. The verification of the stability is performed by calculating the stability of a wedge in the face region (including the stabilizing effect of face bolts). An increased number of face bolts increases the stability and decreases the calculated deformation. Additional details about this calculation are given in Anagnostou & Kovari (1996).

Mattle and John (2002) described a similar approach in their publication. They used the results of numerical calculations to define a static system as well as loads. A beam is chosen as static system with a rotational degree of freedom on one side and fully constrained support on the other side. The free span, used for calculation should be chosen as 1.5 times the excavation length. In their example, the loads acting on this static system are in good agreement with a formulation by Terzaghi.

2.6 Numerical Approaches

Muraki (1997) concludes that due to the complex ground behavior around the face as well as the way umbrella methods interact with the ground and other support systems, only 3-dimensional numerical calculations can appropriately assess results for understanding the behavior of the umbrella structures.

Hoek (2003 & 2005) notices, that shallow tunnels in weak ground usually involve the problems of face instabilities and failures in the ground surrounding the tunnel. Because of these facts, he recommends that a complete analysis requires a full 3-dimensional calculation. Even so, he discusses approximations in 2-dimensional numerical calculations (Phase2) because the 3-dimensional possibilities are currently not user-friendly for the average tunnel designer in his opinion. Another approximation showed a longitudinal section with support dependent forces in the excavated sections. The even more difficult analysis of pipe umbrella systems for a full solution requires the use of a program like FLAC3D. In the discussed 2-

dimensional approximation, the method of weighted averages is used to estimate the support effect of the forepoles and the calculated example was stable, while the unsupported one ended up in a caving of the surface.

Hefny et al. (2004) compared two ways of assessing the supporting influence of pipe umbrella systems in 2-dimensional calculations after a short discussion of former publications dealing with this topic. Both models assume the strengthening of a defined zone around the tunnel perimeter. The first method models pipes, grout, and ground as composite zone of enhanced properties calculated by weighted averages, while the second one considers the pipes individually in the composite zone, which consists of weighted averages of grout and ground. The comparison shows that both surface and tunnel crown settlements differ a lot so they recommend that 2-dimensional numerical simulations should be calibrated with case histories as well as 3-dimensional numerical simulations.

Major deformations and a face collapse after 25 meters of excavation at the HB tunnel caused another study. Due to these unexpected events, investigations using FLAC3D were started. The result showed good agreement between the measured and observed in-situ data and the 3D calculation. After this validation, the support system was adapted. Pipes with an outer diameter of 114 mm replaced the spiles, and the grout was considered by increasing the Young's modulus. The results lead to the author's conclusion that the modeled pipe umbrella pre-support was very effective in decreasing the deformations on the surface as well as increasing safety during construction (Choi & Shin, 2004).

Currently there are no commonly accepted design rules for dimensioning pipe umbrella support systems. Hoek (2005) 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 often found in literature utilize a homogenization technique to improve the ground strength ahead of the tunnel face. While this is a simple and 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 works. This aspect is important for the support optimization.

The publication of Eclaircy-Caudron et al (2006) focuses on the influence of pipe umbrellas on settlements and the possibility to find a representative 2D model in comparison to a 3D model. They conclude that the pipe umbrella does have a small influence on reducing deformations and that this result cannot be verified by experimental results. A very interesting point in this article describes that the first reference calculation resulted in a collapse, so for following comparisons, ground cohesion was increased. Their comparison of 2D and 3D calculations showed a good agreement regarding longitudinal extrusion of the face but smaller roof settlements in the 2D model.

2.7 Miscellaneous

At the New Mainzer Tunnel in average 3.5 days were necessary to install and grout one pipe umbrella (20.5 m long and 27 pipes) (Lauffer & Gaulhofer, 2000). Rodio GmbH installed the pipe umbrella with a rotary-percussion drilling system at this site. (representing a time requirement of 9.1 min per meter injected pipe umbrella)

Schikora et al. (2000) discusses problems that occurred because the installation of pipe umbrellas needed longer than the excavation sequences. In this case, the pipe umbrellas were installed with special machines in about 4 days while the excavation of the section only took 2.5 days.

Ayaydin (2001) reported a collapse. This one happened at the Metro in Istanbul. They found that an old well caused this collapse. This well was about 12.0 m deep and about 1.5 m in diameter. The well bottom was about 1.5-2.0 m above the tunnel crown and mud and clay liquified. The liquefied material started to flow through the gap between the installed umbrella pipes and filled up a 35.0 m tunnel section causing severe damages on the surface and killing five people.

John & Mattle (2002) mentions a drill rate for pipe umbrella pipes of 2 m - 3 m per minute. Including grouting for up to 600 m of steel pipes (1 pipe umbrella) the installation for a pipe umbrella can be finished in about 24 hours by using the AT - Umbrella System. (time requirement of 2.4 min per meter injected pipe umbrella)

Stocović et al. (2002) report that in average 36 hours were needed to install 29 injected steel pipes with a length of 15.0 m. At this project, the AT-pipe umbrella system was used. (Using this support, 3.5 meters per day could be excavated in average under the unfavorable ground conditions at the St. Mark Tunnel in Croatia.) (5.0 min per meter injected pipe umbrella)

Fujimoto et al. (2002) and Kimura et al. (2005) report that the pipe umbrella installation at the Baikoh Tunnel in Japan needed about 2.5 days for 23 pipes with 12.5 m in length. (12.5 min per meter injected pipe umbrella)

Yokoo (2002) describes a measurement technique above the tunnel crown. They used settlement gauges in a pre-drilled hole at the Shin Kobe Tunnel to observe the development of settlements ahead of the construction. This campaign resulted in up to 50% of settlements ahead of the face.

Nemec (2002) reports a chimney type collapse that occurred half-side at the Brixlegg exploratory tunnel during excavation works. He does not explain the reason for the collapse but mentions that this event developed so slowly that no worker was hurt. After stabilizing the hole from the surface, the collapsed section was re-built with additional support measures, namely: pipe umbrella support (strengthened with steel bars inside) and 6-8m long IBO-spiles. After this event, the axial distance was reduced from 0.40 m to 0.35 m for the next pipe umbrella sections.

3 Bending Behavior of Pipe Umbrella Pipes

Before any technical definition or design of pipe umbrellas can be performed, it is necessary to know the pipe characteristics. In case of pipe umbrella tubes, the bending behavior is of interest, which is tested by an especially developed bending apparatus (Leitner, 2003) that is mounted in a standard load frame by MTS Systems Corporation. The four-point bending apparatus (Figure 1) allows to determine the bending behavior of tubes and changes of this behavior due to special features in the tube. An automatic data acquisition system records selected values during the test. The most important value is measured by two Linear Variable Differential Transformers (LVDTs) mounted on a measuring device. This is mounted on the sample so the LVDTs measures the sample's deflection only.

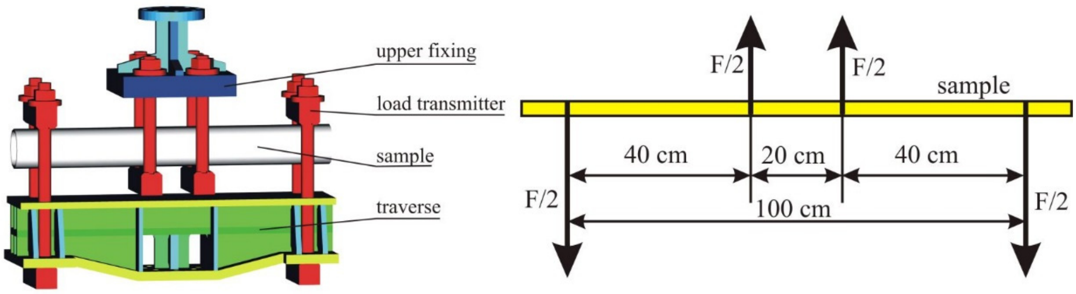

Figure 1. Layout of testing apparatus (left side); geometric conditions for load transmission (right side) (both Leitner, 2003).

Due to the simple test configuration it is possible to calculate the bending behavior with an analytical formulation. This result can be taken as basis for detailed interpretations.

The test program focuses on the following points:

- Influence of grout
- Influence of borehole debris in uncleaned and grouted tube
- Influence of injection holes
- Influence of different tube connection types

The test series is performed with pipes of steel quality S355 J2H (EN 10210-1, EN 10 025). Table 1 presents the significant properties of this steel type.

Table 1. Steel properties of S355 J2H.

	value	unit
Modulus of Elasticity	200.000	MN/m^2
Poisson's Ration	0.3	-
Shearing Modulus	76.923	MN/m^2
Yield Strength	360	MN/m^2
Tensile Strength	500	MN/m^2

3.1 Influence of Grout on the Bending Behavior

Pipe umbrella pipes are filled with grout after installation, so the grout can improve the quality of the surrounding ground by penetrating open joints or pores. Before the grouting process starts, the pipes are usually not cleaned, thus borehole debris may still lie at the bottom of the pipe. To evaluate this influence, a partly grouted sample with sand filling is tested as well.

As Figure 2 shows, the bending behavior of a perfectly filled pipe is adequate to the partly grouted sample with sand filling. Both show a linear behavior to a tension load of 0.1 MN. At this load level the outer fiber starts yielding. An empty pipe shows a differing behavior at loads higher than 0.7 MN. This is caused by an ovalization of the unfilled tube, resulting in a decrease of section modulus and resistance respectively.

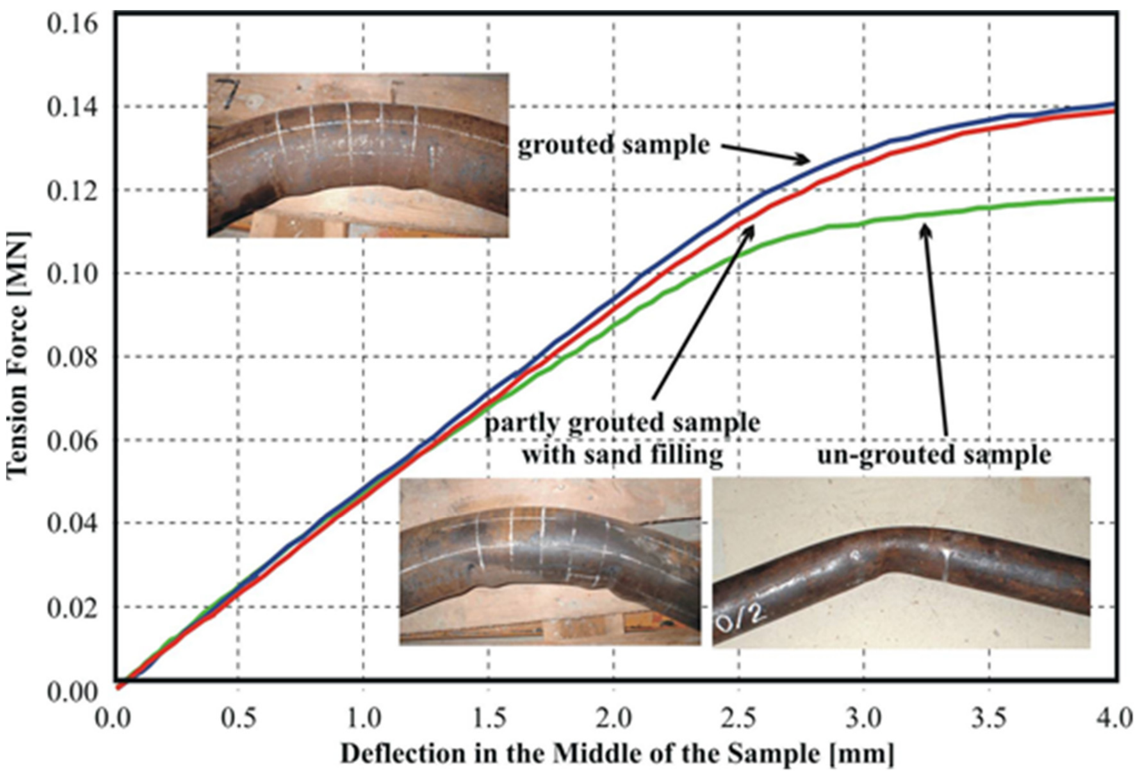

Figure 2. Comparison of samples that are un-grouted, grouted, or partly grouted with a sand filling in the remaining part (114.3 mm x 6.3 mm)

3.2 Influence of Injection Holes on the Bending Behavior

Pipe umbrella pipes are equipped with injection holes. This allows the grout to fill the annular gap as well as open pores or joints. The drilled holes have a diameter of less than 20mm and can be situated at the tension / compression zones as well as in the neutral axis.

During testing these features in none of the tested positions change the bending behavior before the outer fiber starts yielding. The stress concentration around the holes at higher curvatures lead to a failure of the pipe by cracking in the tension zone.

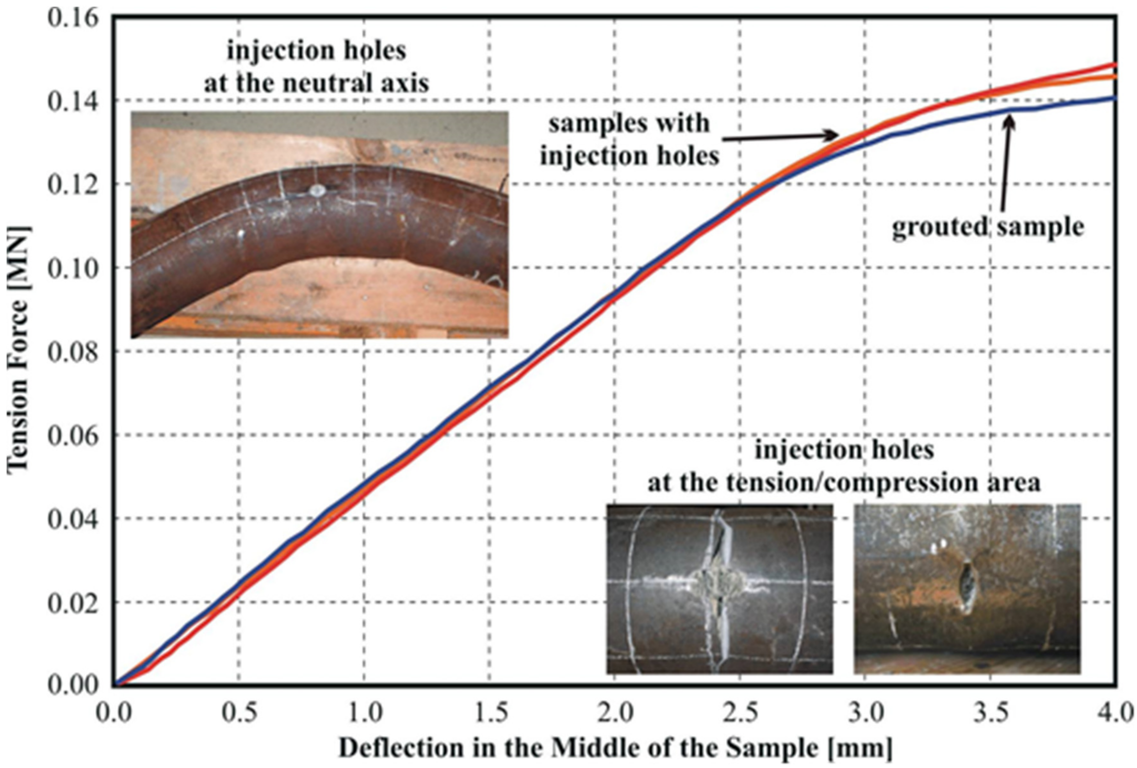

Figure 3. Test results of bending tests with injection holes at the tension / compression zone and at the neutral axis (114.3x6.3 grouted). Perfectly grouted regular pipe as reference (blue)

3.3 Influence of Connection Types on the Bending Behavior

Pipe umbrella pipes are installed piecewise, when using common drilling machinery for the drilling process. Initially, these connections were standard threads cut into the pipe ends. Due to observed failures of threads during construction, additional connection types were developed. Nowadays, two more types are available on the market. These are called “nipple connection” and “squeezed connection”. Characteristics of all three connection types are explained in Appendix A – Development of State-of-the-Art Connection Types for Pipe Umbrella Support Systems.

The results of the bending test show that the standard thread connection fails at the lowest load level. The ultimate bending moment of this connection type is at the level of the elastic design load for regular pipes under laboratory conditions. Both other connection types fail at much higher loading levels. The elastic loading range of the standard thread connection cannot be determined clearly but it must be lower than 40 kN. The bending behavior can be described as softer and weaker than the regular pipe. The elastic loading range of the squeezed connection ends approximately at a load level of 60 to 80 kN and can be described as softer than a regular pipe but stiffer than a standard thread connection. The sample equipped with a nipple connection shows comparable bending behavior as the regular pipe so it is as stiff and as strong as the regular pipe.

3.4 Result of bending tests

- The grout in the pipe prevents the pipe from ovalization effects, but this shows only at high load levels. Nevertheless, the grouting is of essential importance to fill the gap between ground and pipe for an immediate load transfer and to decrease the risk of additional subsidence.
- Cleaning the pipe before grouting does not increase the load bearing capacity.
- The connection type has a major influence on the bending behavior. Depending on the chosen type it may lead to a soft and weak link, plastic hinges, or no influence on the pipe umbrella support. Due to this influence it is recommended to consider the choice of connection in the design as well as define the connection in the tender documents.

4 In Situ Evaluations

To learn about support behavior of pipe umbrellas, observations and measurements on site are essential. A geotechnical model can be created with the knowledge about the behavior. This model is the basis for further investigations and developments.

Nowadays three-dimensional geodetic surveys of targets in the supported section are state-of-the-art for observing the system behavior during tunneling. Target movements are commonly surveyed once a day (sometimes twice a day) with a total station.

4.1 Pre-Investigations

The chance to investigate the ground – support interaction by observing the excavation induced deformations is also used to get a first idea about the mechanisms involved, when using a pipe umbrella system. The Pipe Umbrella system is a pre-support system. These systems support the ground ahead of the primary lining. The geodetic survey in the tunnel may not give significant results. For this reason, observations of the surface are analyzed regarding the influence of the pipe umbrella support. The project Lainzer Tunnel, lot LT22 (Austria) is suitable for this investigation, because the overburden depth is small, the ground is weak, and a pipe umbrella support system is used.

The cover above tunnel crown in the selected section is approximately 10 m. Claystones, sandstones, and marls are typically covered by a 10 m thick layer of alluvial deposits in this territory. The ground is mostly tectonised and sometimes completely altered to soil in the eastern section of LT22.

The ground conditions with ongoing advance changed from bad to worse as can be seen in Figure 4 (Moritz et al. 2002). This figure shows the subsidence measured by geodetic survey in a deflection curve diagram including three trend lines. They indicate the influence of the construction at the face position and both 6 m ahead and 6 m behind the face. Face bolts and a pipe umbrella support system were used as pre-support systems. The length of the face bolts was 9 m while the pipe umbrella pipes were 12 m long. The pipe umbrella was newly installed every 9 m and the face bolts every 6 m. Even if the general trend indicates the ground conditions very well in this example, the trend lines fluctuate a lot around the general trend. If the fluctuations of the trend lines would be caused by changes of the ground quality, the increase and decrease of subsidence would be located at certain positions in all trend lines. But the fluctuations appear at the same time in all trendlines, indicating that the reason is excavation process dependent. The deformation characteristic appears every 6 m or 9 m in all trend lines, so it can be assumed that the fluctuations are caused by the pre-support measures.

The red annuli in Figure 4 mark sudden increases followed by decreases in the trend lines with distances of 9 m. This characteristic can generally be seen in all three trend lines, indicating this characteristic likely to be caused by the pipe umbrella support system. In this case, the increase appears before the installation of the next set of pipes takes place, indicating that the support efficiency decreases or the load transfer mechanism changes. The increase in subsidence is followed by a more or less pronounced decrease of subsidence for the following construction steps. So, the installation of a new pipe umbrella interrupts the deformation increase. At the point for installation an additional shotcrete arch is created that has time to harden during installation of the pipes. This fact increases support stiffness so the following excavation steps induce less subsidence than usual. Additionally, in between the discussed in- and decrease the same characteristic appears sometimes. These points have an appearance of multiple times 6 m so face bolts may cause the same effect when the bonding length decreases.

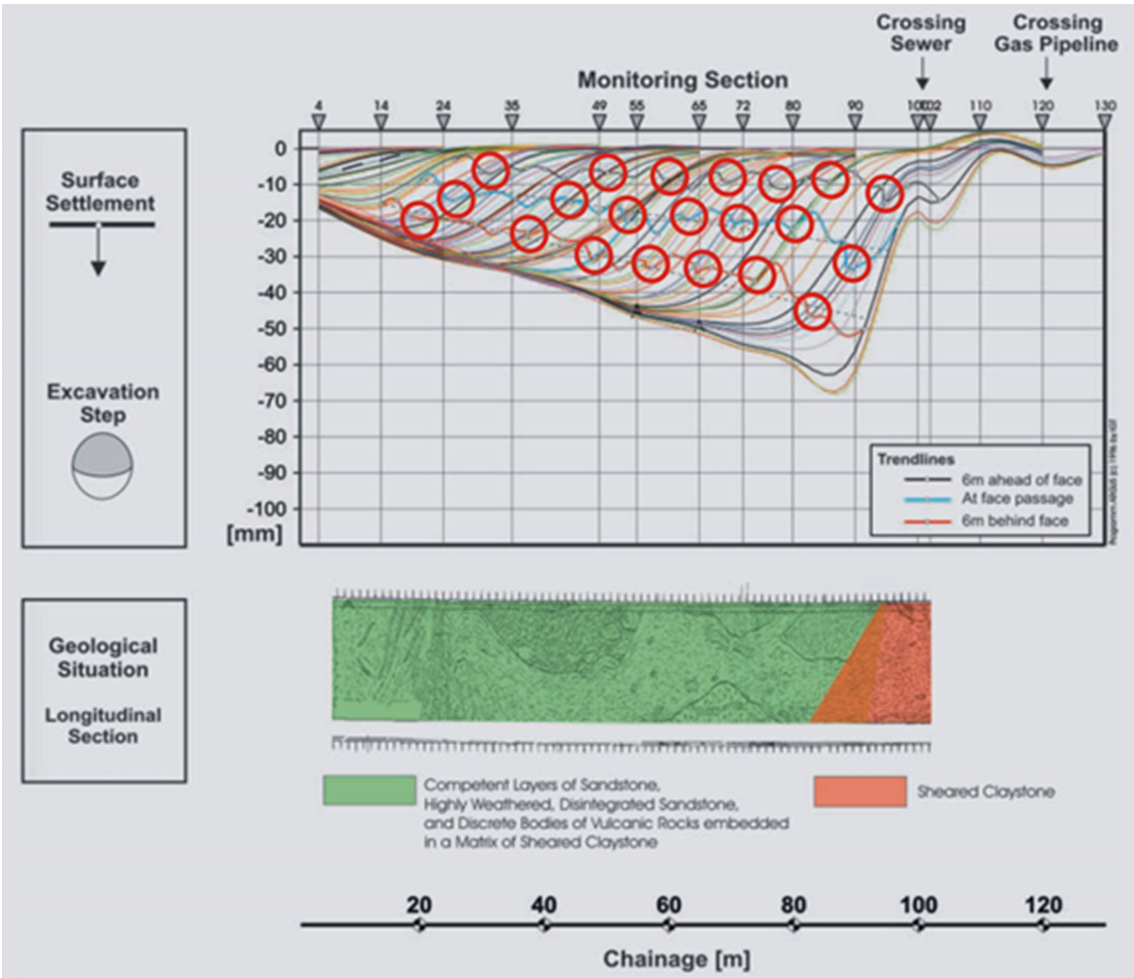

Figure 4. Deflection curve diagram for surface settlements along the tunnel axis of BL9 including the trend lines 6 m ahead of the face, at the face position, and 6 m behind the face (Figure 2 in Moritz et al., 2002)

Having these interpretations in mind, a geodetic survey is not sufficient for further investigations, because:

- The influence of all involved mechanisms cannot be identified clearly with daily measurements.
- Influences between the construction process and the surface points cannot be identified and eliminated.
- The geodetic survey in the tunnel starts in the supported section. There is no observation of the ground behavior ahead of the face and the pre-support.

Due to these facts, a new measurement technique is used for the further investigations on the pipe umbrella support system.

4.2 Applied Inclinometer measurement system

The applied inclinometer measurement system is described in Appendix B and C. It consists of ten 2 m long inclinometer pieces that are connected to each other and installed in a roof pipe of a pipe umbrella. Advantages of this system are:

- Density of observed points (2 m) is higher than with a geodetic survey
- Measurements are up to 20 m ahead of the face and parallel to the pipe umbrella
- Data are recorded every minute and show the influence of all construction steps in detail

An in-time evaluation and interpretation of the data supplements the possibilities of a geodetic survey due to the mentioned advantages. Changes in ground conditions can be determined and adaptations of the support system can be initiated. Additionally, and most important for this work, the pipe umbrella behavior can exactly be observed.

The pipe umbrella system utilized at the Birgl tunnel (Austria) was 12 m long and supported nine 1 m long excavation steps. The evaluation of the measured data presented a similar deformation characteristic as already described in chapter 4.1; an increase of deformations ahead of the face during last excavation steps in a pipe umbrella field and decreasing values for the following excavation steps.

In contrast to this result, the measured data at the Trojane tunnel (Slovenia) (figure 5 Appendix D) displays no increase in deformations at the end of the pipe umbrella field. The overlapping length in the longitudinal direction was increased during the optimization process. This adaptation resulted in a positive change of the deformation characteristic.

The pipe installation was performed with special machines by using the so-called pre-drilling method at the Trojane tunnel, while the pipes were installed by using a cased drilling system with lost casing at the Birgl tunnel. The deformations measured during the installation point out that the installation method has a major influence on subsidence in weak ground conditions (Appendix C)

4.3 Geotechnical Model for pipe umbrella systems

With the assistance of on-site observations and measurements, it is possible to define a geotechnical model as described in Appendix D resulting in three supporting effects associated with pipe umbrella systems:

- Radial supporting effect: each pipe umbrella pipe supports the ground in the overloaded section around the heading and transfers these loads to less loaded sections in both longitudinal directions.
- Longitudinal supporting effect: compared to the ground stiffness the stiffness of a steel pipe is much higher; so, each relaxation process forces the ground to shear along the pipe umbrella, positively influencing the stress redistribution ahead of the face.
- Pipe spacing in the unsupported span: this distance is either defined by the overall necessary number of pipes or by the ground quality to allow for arching between the single pipes, thus preventing overbreak beyond the pipes.

This geotechnical model can be used for three-dimensional numerical calculations as well as analytical solutions. Furthermore, the result of the inclinometer measurement campaign can be used for back-calculations. The pipes are steel beams with a measured deflection in the longitudinal direction so internal forces or interacting forces between ground and pipe can be calculated as presented in Appendix E and Appendix F. A few of these results are highlighted in the following:

- The supporting area can be defined as free span plus an area up to 2.0 m ahead of the face in the back-calculated cases.
- The back calculated load is never higher than the load presented in Appendix F.
- The maxima of bending moment can be calculated, and their positions vary depending on the differences of ground stiffness and support stiffness.

With the load transfer model of Appendix F the increase of deformations due to inefficient load transfer to the ground – compare chapter 4.1 – can also be explained. In cases of short overlapping length, the static system changes from the presented one to that one of a spile (short forepoling) as shown in Figure 5. Thus, the already overloaded face region is additionally loaded, resulting in the measured and discussed increase of deformations.

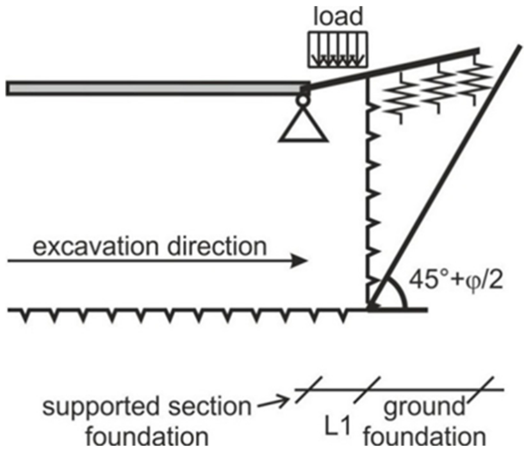

Figure 5. Static model for a spile (short forepoling).

4.4 Summary of In-Situ Evaluations

- The observation of the pre-support system behavior is limited when using only geodetical measurements. In case of shallow tunnels, regular measurements of closely spaced targets can provide a good basis for interpretations. In all other cases, special systems like the presented inclinometer measurement system can supplement the data to allow for adequate support control.
- The pipes stabilize the open span by decreasing exposed ground, small arches can develop between the pipes so the overbreak volume remains small. => the axial distance is a design criterion and it is defined by the creation of local arches.
- Pipe umbrella systems transfer loads from overloaded sections around the open span to less loaded sections in the longitudinal direction to its foundations, namely the ground ahead and the already supported areas. Result: the loading bends the pipes => elastic bending moment is a design criterion.
- A pipe umbrella supports ground outside of the pipe umbrella. To stabilize the face itself, face bolts or similar systems are the preferred measure.
- In weak ground conditions the perimeter should preferably be supported with a pipe umbrella system instead of spiles, because the face region is already overloaded. While spiles increase the risk for local face instabilities, a pipe umbrella system decreases this risk. If a pipe umbrella is designed correctly a face can even fail without resulting in a tunnel collapse, because the material on the outside is supported by the pipes.

- The weaker the ground is, the more important is the choice and/or definition of the installation method. Self-drilling methods with lost casing must be preferred because these are less susceptible to create subsidence.

5 Numerical Simulations

The major advantage of numerical calculations is the possibility to evaluate the influence of different support concepts on the stability conditions and on the induced deformations. A precondition for obtaining realistic results is the correct reproduction of all involved mechanisms. In the case under consideration, the data, which were collected during construction, enables to validate the results of the numerical simulation. In a second step, it is possible to compare different support and advance methods after calculating the case observed in situ.

As at the design stage, no detailed information on the ground behavior is available, previous experience can be used for the analysis, and the model refined during construction.

5.1 Numerical Model and Advance

The goal for these calculations is the evaluation of the stability conditions around the heading as well as the deformations induced by the construction process. The calculations are performed with a half model.

The program code Fast Lagrangian Analysis of Continua (FLAC-3D) was used for the calculations. Figure 6 shows an exemplary mesh type. This mesh geometry suits for a pipe umbrella support system with a pipe umbrella field length of 12.0 m. The length of the model is 72.0 m. Due to the influence of the boundary, the calculated results for the two pipe umbrella fields in the middle of the model can be evaluated without any boundary influence in the longitudinal direction.

The width of this mesh is 50.0 m. Due to width of the excavated section of about 7.0 m to 7.8 m, the remaining ground volume for the stress transfer is about 43.0 m in the horizontal direction. The bad ground conditions lead to wide-ranging stress redistribution; however, the chosen width eliminates the influence of the boundary in the horizontal direction perpendicular to the tunnel axis.

The distance between the excavation and the lowest point of the mesh is about 14.0 m, allowing to evaluate the stress relaxation below the excavated area and the induced deformations respectively. The ground cover above crown was defined by the topography of the Birgl Tunnel project to 22.0 m. This complies with the maximum cover in the monitored section.

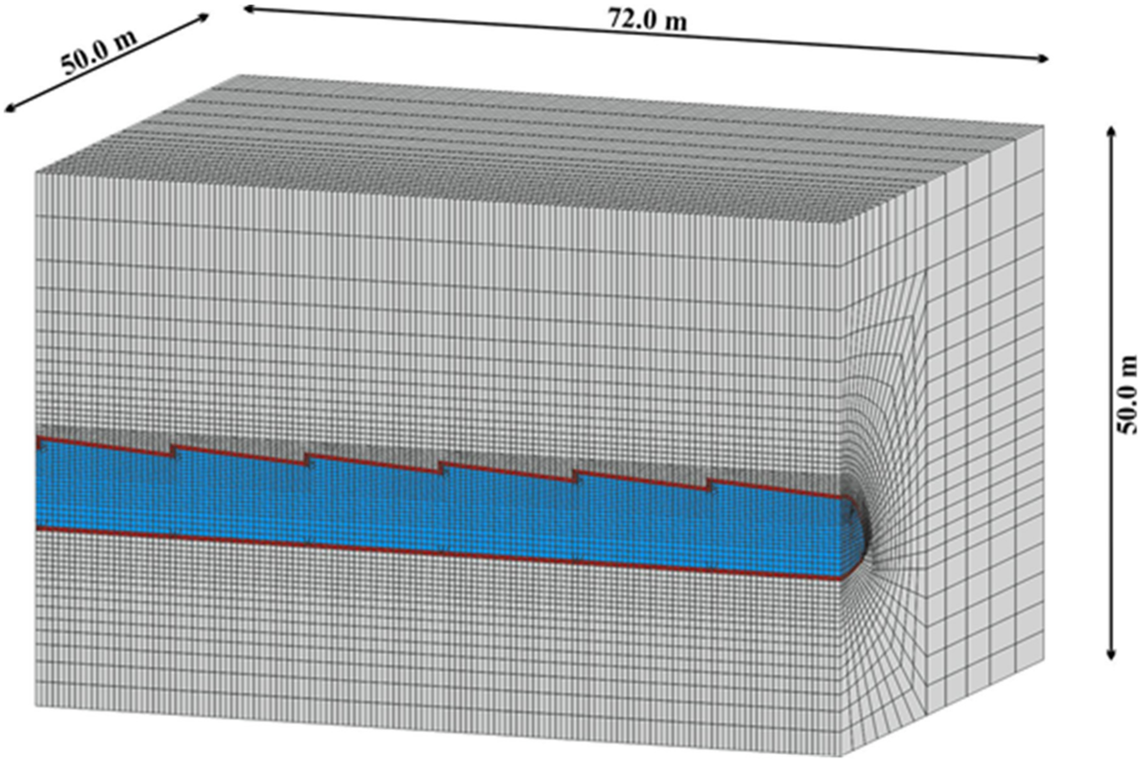

Figure 6. FLAC 3-D Model for a 12.0 m long pipe umbrella support system.

Figure 6 shows the basic geometry of the model. It only represents the top heading and top heading invert sections. The saw tooth shaped profile is generated as well.

The maximum finite element zone length in the tunnel area of the model is 0.5 m. This allows considering the additional working area ahead of the primary lining after finishing one excavation step. The mesh length divides the unsupported span in three zones in the longitudinal direction. This enables a good interaction of pre-support system (structural elements) with the ground in this area, which these investigations focus on.

The advance follows the ideas of the advance at Birgl Tunnel. One 2.0 m long step in the top heading invert section follows two 1.0 m long excavation steps in the top heading. The excavation of the top heading is always between 4.0 m and 6.0 m ahead of the top heading invert excavation and was excavated in 2 or 3 phases.

The maximum unbalanced force in the model decides about equilibrium during stability calculation. When this value was lower than 0.4 kN, the next step was automatically started.

5.2 Input Parameters

Essential for a realistic result of a simulation are realistic ground properties. Additional shear tests have been performed in the laboratory, and back analyzed.

The shear test of sample 109.1 was back calculated with a 1 to 1 model. The mechanical model “strain-hardening/softening plasticity model”, is applied for describing the sample’s properties. The calculation is controlled like the real test to obtain a good reproduction of the test procedure. Figure 7 shows the result of the shear test 109.1 in comparison to the result of the back calculation. The back calculation reproduces all significant characteristics of the real test at the right load level. Small differences between reproduction and reality seem to be negligible because other tests varied as well showing the possible influence of the natural spread in this ground type.

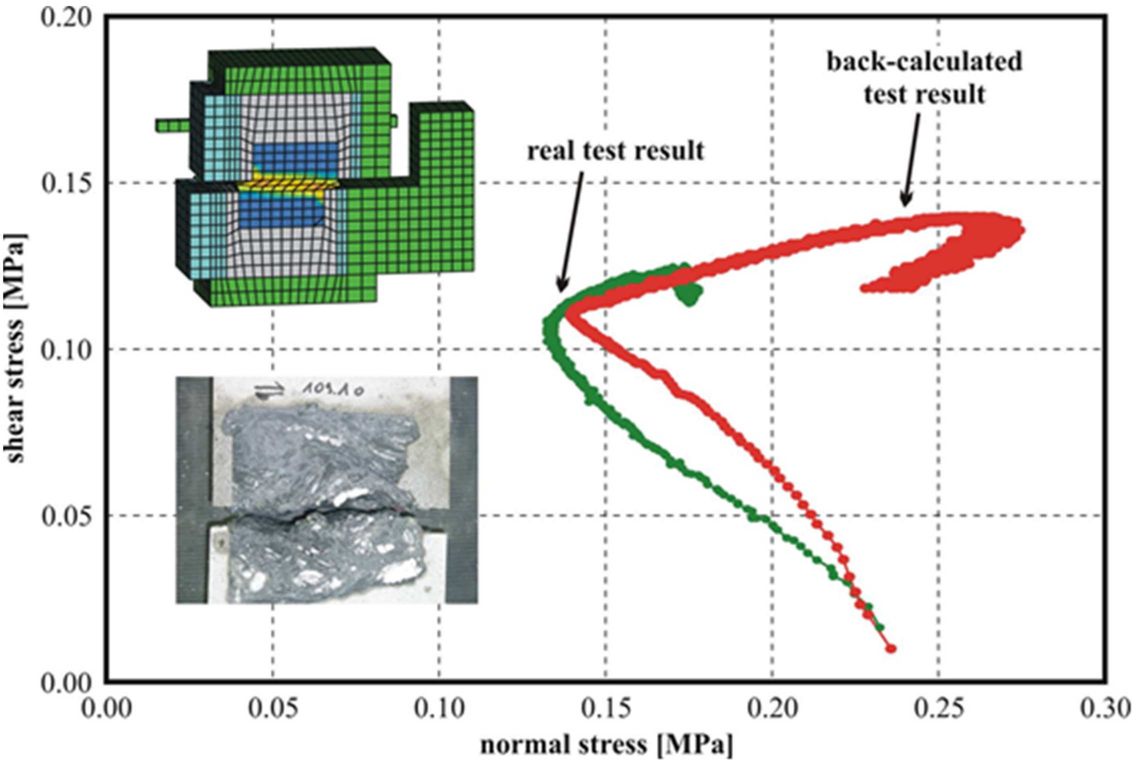

Figure 7. Comparison of real and back-calculated shear test results of sample 109.1

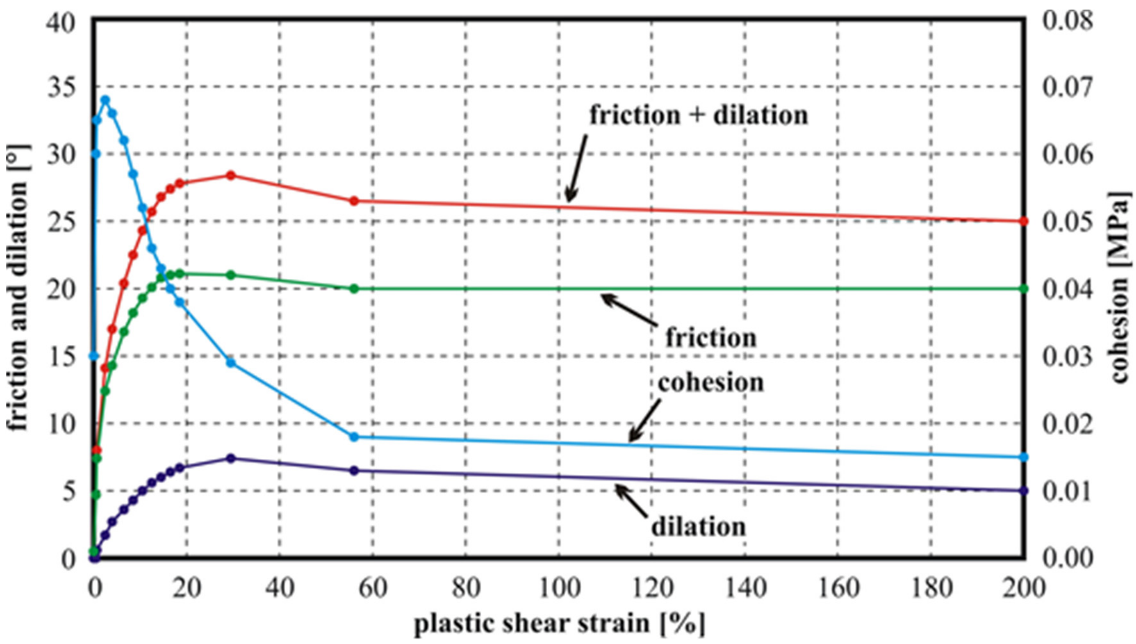

Figure 8. Development of the parameters friction, dilation, and cohesion over the plastic shear strain

Figure 8 shows the resulting ground properties related to the plastic shear strain. All parameters cohesion, friction, and dilation agree in its characteristic development with those shown in Figure 13 of C.D. Martin (1997). The cohesion mobilizes at first. At these strains, the friction and dilation have less effect. The influence of friction and dilation increases when the cohesion loss starts. In later stages of the test, the influence of dilation also decreases, and all three ground-describing parameters converge to its residual values.

The shotcrete in combination with lattice girders is used as primary lining in the model. The final thickness of the primary lining is 30 cm in the top heading and installed in 2 steps. The first layer is 15 cm thick and installed after each 1.0 m long excavation round in the top heading. The second 15 cm thick layer is installed after finishing the excavation and supporting routines of 2.0 m. Hence, the final thickness of the primary lining is achieved at a maximum distance of 3.0 m behind the face.

The strength and stiffness properties of the shotcrete change depending on the shotcrete age. Each excavation and support step are associated with a certain amount of time so the age changes due to the performed sequences in the calculation. The used development for these parameters is described in Aldrian W. (1991) and Müller M. (2001).

Cable structural elements represent the radial bolts in the numerical simulation because radial bolts are mainly loaded in the axial direction. When determining the input parameters, it was assumed that if failure occurs the ground surrounding the radial bolt fails because this is the weakest of all involved materials. For this reason, the grout parameters were changed to parameters dependent on ground properties of the surrounding ground. Due to the described and implemented mobilization and de-mobilization of the ground properties friction and cohesion both values were read out from the nearest gridpoint for each piece of the structural element before calculating stability.

Face bolts are implemented as pile structural elements in the calculation. The expected vertical deformations of the face during calculation are the reason for using pile elements instead of cable elements even if face bolts are also mainly loaded in the axial direction. The load controlling input parameters for the grout are also determined by assuming that the ground is the weakest part in case of failure.

The pipes of the pipe umbrella support system are represented as pile structural element. The axial distance between two installed pipes is 35 cm at the installation position. The pipes are in-situ grouted after installation so contact between ground and pipes is created before the construction re-starts. The deformations of the ground load the pipes via the surrounding grout. The weakest part in this combination is the ground so the determination of the grout input parameters are also calculated by using the parameters of the surrounding ground.

5.3 Results of calculations

All types of pre-support methods are aimed at preventing local failure associated with the open span. Face bolts are installed to stabilize the face itself, while spiles or pipe umbrella systems are applied to stabilize the perimeter of the tunnel. The numerical model was set up to show failure in case the pre-support is not installed Figure 4 in Appendix G presents one of these control calculations. In this case, the spiles were not installed in the normal excavation and support sequence resulting in a primary failure of the perimeter followed by a secondary shear failure of the face. In case face bolts are not installed during the standard routine the face shows shear failure without the failure mechanism at the perimeter. Failure modes in both cases are pretty similar, thus in reality the immediate cause would be difficult to assess.

The publication of Appendix G also shows two stable excavation sequences with spiles and pipe umbrella support respectively. The weight of installed steel is adequate – a comparable reinforcement grade – and as a result the final settlements are very similar. But when an unexpected change of the ground conditions appears – in this case simulated by a decrease of cohesion to two-third in a 1 m long vertical section – the tunnel supported by spiles collapses, while the pipe umbrella supported tunnel remains stable, although the final deformation increases due to the local face failure.

Several authors [Bae et al 2005, Hefny et al 2004, & Kim et al 2004] used three-dimensional numerical studies for investigations on pipe umbrella systems. The support system was modelled as a homogenized area at the outer perimeter of the tunnel. This does not fit to the proposed geotechnical model because a closed shell with increased cohesion does not only transfer loads in the longitudinal direction, but also perpendicular to the tunnel axis. So, there is another supporting effect created that should decrease settlements during calculation. Appendix H also deals with this issue and the comparison of the simulation results clearly shows that there is huge difference. Using a pile model as proposed results in similar deformation values with comparable mechanisms as measured during construction while the homogenized model results in 3-times lower settlement values and a flatter settlement characteristic. This shows that a pipe umbrella cannot be reproduced by a homogenized model, the results are on the un-safe side regarding stability and underestimate the subsidence. This geotechnical model would better fit to horizontal jet-grouted columns that form a closed shell ahead of the face.

Figure 10 and 12 in Appendix H present simulations with pipe umbrella support. The evaluation of the settlement-isolines shows a clear difference between the inside and the outside of the pipe umbrella. The outside is supported, reducing loads from the overloaded face region. The inside – the face itself – settles independently more than the surrounding ground. This may even lead to a gap between the face and the pipe umbrella as can be observed by the narrow shadow line in Figure 9.

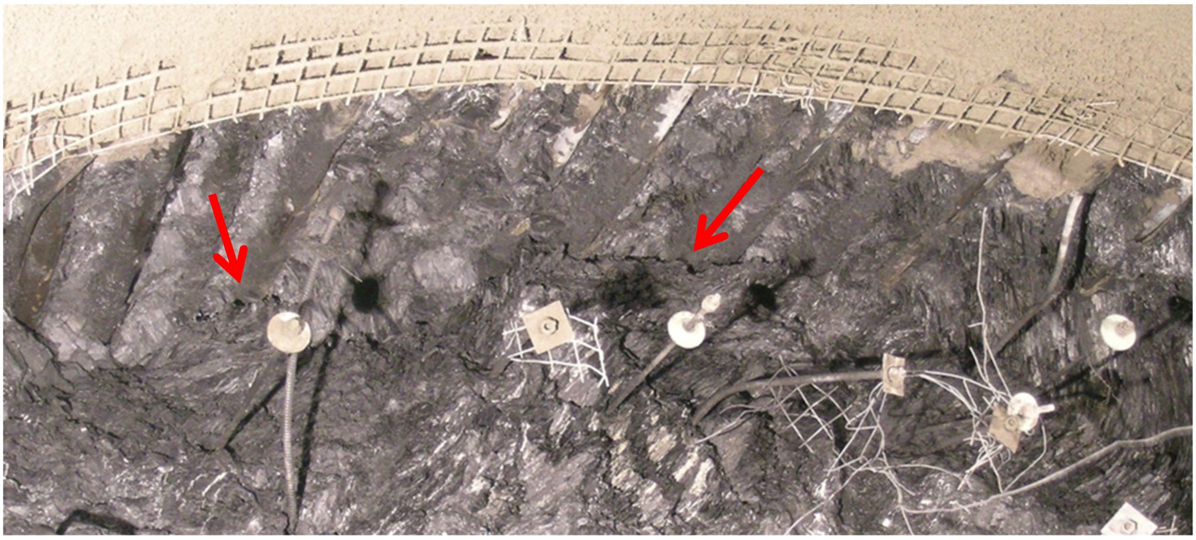

Figure 9. Gap between face and pipe umbrella supported ground.

The measured settlement values of the in-situ measurement campaign and the calculated ones of the numerical calculation are in good agreement regarding deformation size, position, and bending characteristics. All involved, important mechanisms are well reproduced. This could be achieved with comprehensible methods to determine all input values, which is a good basis for further calculations to evaluate changes of support parameters.

A point that is always under discussion is the influence of pipe umbrella systems on the subsidence. Results of calculations in Appendix I give a clear answer to this question. With increasing reinforcement grade due to pipe umbrella pipes the calculated settlement values decrease as well.

5.4 Summary of Numerical Simulations

The results of the numerical calculation confirmed the geotechnical model defined in chapter 4. Although, it must be underlined at this point, that the quality of the results increased tremendously after implementing adequate ground parameters, which were determined by the calibration and/or back-calculation of the shear test. So, modelling the correct ground behavior is a key issue for adequate results in weak ground conditions. It showed that the published homogenized model does not properly reproduce a pipe umbrella support system in numerical calculations. The use of this model results in unsafe stability conditions and an underestimation of expected ground deformations.

By using an adequate implementation of ground and support measures – as described above – it is possible to reproduce failure scenarios and the in-situ measured ground-support interaction. The calculated and measured ground-support interaction agree with respect to deformation magnitude, place of deformation as well as on the main characteristics, allowing a realistic reproduction of major mechanisms.

This validated model allows to investigate the influence of pre-support measures and a comparison of pre-support methods as well as changes of their design parameters. The calculations showed that omitting pre-support measures results in failure of face and perimeter respectively. Face bolts are the optimum solution to stabilize the face. Both, spiles and a pipe umbrella support are able to stabilize the open span at the perimeter but in case of weak ground a pipe umbrella system should be preferred because little changes of the ground conditions may result in failures when using spiles. The face may fail, and the tunnel stays stable due to the far-reaching load transfer when using a pipe umbrella system. Last but not least the results of the calculation proofed that a pipe umbrella system decreases construction induced deformation depending on the grade of reinforcement.

6 Conclusion

Since the mid 1990ies pipe umbrella support systems are increasingly used to support tunnels during excavation. The main application areas are shallow, urban tunnels, tunnels constructed in fluvial or marine deposits, excavations through fault zones, portals and re-excavation of collapsed tunnel sections. The design was mainly performed by experience from earlier projects because simple design rules or clear support effect descriptions did not exist.

This changed over the last two decades because of intensive in-situ measurement campaigns and further research activities on this basis. A geotechnical model could be defined, and the following support mechanism determined:

- The pipes divide the open span at the heading in smaller unsupported sections. Small arches can be created between the pipes so a possible overbreak volume stays small.
- Pipe umbrella systems transfer the loads from overloaded sections around the heading to less loaded sections in longitudinal direction. These are the ground ahead of the face and the already installed support in the excavated section.
- Each excavation step induces relaxation processes resulting in a movement of the ground to the excavation position. Due to the much lower ground stiffness compared to steel, a relative movement between pipes and ground is generated. The resulting shear force acts as confining stress, leading to an improvement of the ground strength ahead of the face.

These support mechanisms lead to support effects that can be associated with pipe umbrella support:

- Support of the open span (excavation step length plus working area)
- Protection of overloaded face region due to reduced loads
- Reduces unexpected overbreak
- Reduces excavation induced settlements and deformations respectively
- A stable tunnel perimeter in case of local face instabilities

To ensure these support effects during construction the design parameters bearing capacity of pipes, distance between pipes and foundation length in the ground must be defined adequately.

The results of bending tests showed that the regular pipe is not relevant regarding bearing capacity. These tests identified the connection as most important element because it may be – dependent on the chosen type – the weakest link in the support system. The maximum distance between pipes is defined by the criterion that local arches can be created also at the end of the pipe umbrella due to increased spacing of the pipes. The foundation length depends on the shape of an expected failure body at the heading; so ground properties, the height of the front excavation and additional measures like a face wedge control this parameter.

The time has passed, in which the relevant pipe umbrella parameters are designed according to experience. Nowadays, analytical solutions as well as numerical calculations are available to obtain reliable results. In the authors opinion analytical solutions should be still preferred because input parameters are easier to define and the result stays controllable. The number of input parameters make the problem more difficult for numerical calculations. Especially in soft ground, where the material is overloaded due to the excavation process, either adequate samples, proper laboratory tests and/or clear rules are missing to transfer the necessary parameters into the numerical models. As a consequence, a validation of results cannot be performed before construction.

6.1 Notes for Specifications

Aside from length of installed pipes and axial distance of pipes, tender documents define an umbrella pipe by an outer diameter, a pipe wall thickness, and a steel grade nowadays. This is definitely not enough because this definition does not include the connection type which is one of the important elements controlling the capacity of the pipes. It also does not define the way of installation as discussed in Appendix A and Appendix D. Pipes are bent during loading, so the steel grade should not have a higher yield stress than 360 N/mm^2 to 400 N/mm^2 because the curvature necessary to activate higher yield stresses lead to high deformations during tunneling as well.

Therefore, the following alternative is suggested:

Design drawings are the perfect tool to specify a proper pipe length, an axial distance between pipes, and a foundation length.

The following definition is proposed for the pipe umbrella pipe:

- Outer diameter (typically 88.9 mm to 168.0 mm)
- Minimum elastic moment at the weakest section (connection area) for a purely elastic design

or alternatively

- Minimum ultimate moment at the weakest section (connection area). This value should be at least 1.3 times the elastic moment of regular sections. This definition fits to an elastic design that allows the creation of plastic hinges in the connection areas.

- A preferred steel grade (e.g. S355) or a maximum yield stress (<400 N/mm^2)

Note: It is recommended that the defined bending moment values must be proven by a laboratory test report or an Inspection Certificate 2.2 (ISO 10474:2013) before installation

This must be supplemented by the way of installation; three options are available:

- Into pre-drilled holes: this can only be recommended in stable and un-deformable boreholes.
- Into cased holes: this way of installation can be used in all ground types but cannot be recommended when subsidence should be kept to a minimum.
- As self-drilling system with lost casing: this way can be recommended for all ground conditions and it has the smallest risk for additional deformations during installation.

6.2 Optimization Possibilities

In the last decades the technical development of drilling machines in tunneling allowed some changes in support installation. Around year 2000 a typical pipe length was 12 m and the installation of bigger diameters needed longer. Nowadays the drilling machines can install pipes to a length of more than 30 m and the installation speed is the same for both outer diameters 114.3 mm and 139.7 mm. Due to these changes and due to technical developments of connections material as well as time can be cut down with correct definitions.

Appendix J is going into more detail regarding this issue. Here only the results are highlighted for 139.7 mm outer diameter pipes:

- A change from 12m long to 18m long pipe umbrellas saves 17.8 tons of steel pipes for a 100 m long tunnel.
- Due to the higher elastic capacity of squeezed connections, a change from threaded connections to squeezed connections saves 58 tons of steel for a 100 m long tunnel.
- A change from 12m long to 18m long pipe umbrellas saves 36.7 hours of installation time for a 100 m long tunnel.
- A change from threaded connections to squeezed connections saves 62.5 hours of installation time for a 100 m long tunnel.

6.3 Construction relevant notes

Firstly, the pipe umbrella pipes must be in the correct alignment after installation. Therefore, the starting point must be measured and marked by geodetic survey. Afterwards so-called pin holes should be drilled. During installation the steel pin that is mounted on the front of a drill arm is inserted. This eases the stabilization of the drill arm at the starting position and protects the drilling machinery during installation. To get the correct drilling direction the drill arm must also be orientated correctly so the right position of the drill arm end is indicated with a geodetically controlled laser. The driller must move the drill arm until the laser light corresponds to the drilling axis. To ease this process for the worker, larger white plates (targets) with a mark for the drill axis are recommended instead of small geodetic targets in the drill axis because the laser light can be found, and the drill arm orientated easier and quicker (Figure 10). When the drill arm is positioned and orientated correctly the drilling process can start.

Figure 10. Different types of targets at the end of a drill arm. Both work but the left one has advantages because the bigger area eases to find the laser light

Secondly, the installation process is finalized with a grouting process. The goal of this grouting process is to fill the annular gap between the ground and the pipe to get an immediate load transfer when the excavation starts. So, regularly arranged injection holes with rubber valves are an essential part of pipe umbrella pipes. Additionally, the inside of the pipe is filled with cement suspension. Common pipe umbrella supported ground does not allow cement suspension to penetrate into the matrix, but in case of open joints or pores the cement suspension can intrude, and ground improvement is automatically performed. To fulfill these points a one-time filling of pipes from the mouth is sufficient. Of course, before the filling starts, the gap on the outside between the installed pipe and the temporary shotcrete lining at the starting point must be sealed. Typically, the filling is performed with pressure and volume stop criteria. Note: the pressure should never be higher than 8 bar at the pipe mouth because the cement suspension can bypass back on the outside of pipes and create a bubble between the ground and the temporary shotcrete at the face. This may destroy the temporary support of the face. Only in special cases it does make sense to use for example double-packers to perform a planned ground improvement. Such a case could be raveling ground with a high volume of pores. It is recommended to define the program for the ground improvement in the tender documents as well, otherwise an unambiguous financial offer including the ground improvement time cannot be given by the contractor.

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Appendix A

Title: Development of State-of-the-Art Connection Types for Pipe Umbrella Support Systems.

Author(s): G.M. Volkmann

Published: 15th Australasian Tunneling Conference 2014, Sydney, NSW, Australia, 17th – 19th of September 2014

Summary: As stand-alone publication the first part of this article describes state-of-the-art in forepoling, its mode of action, and gives an overview about design considerations and installation methods. This is followed by a description of the three available connection types and its performance during bending. The publication ends with a site example illustrating the installation equipment and performance when using a squeezed connection for a pipe umbrella.

Development of State-of-the-art Connection Types for Pipe Umbrella Support Systems

G M Volkmann

Business Development Tunnelling and Geotechnical Applications, DYWIDAG-Systems International GmbH, Alfred-Wagner-Strasse 1,4061 Pasching, Austria.

ABSTRACT

Pipe umbrella support systems- also called canopy tube systems - have been used successfully for tunneling in challenging ground conditions since the1970s. Historically, the drilling unit was the limiting factor for pipe umbrella installations - less attention has been paid to pipe couplings and their influence on labor time and load-bearing capacity.

This contribution investigates the influence of the type of pipe connection on the load bearing characteristics of a connected pipe and discusses associated influences on design calculations and installation performance. Standard thread connections reduce the effective cross-section and thereby load-bearing capacity in the connection area. To achieve design parameters, over- dimensioning of the un-weakened pipe section is the only way to overcome this limitation. Recently, two new, alternative connection types were designed and developed for connected pipe umbrella pipes. The first one is the nipple coupling, consisting of fitting with a threaded connection that is pressed and welded into the ends of standard tubes. The second type is the squeezed connection, which mechanically connects non-threaded pipe ends, achieved by force-fitted squeezing from the drill rig. Both coupling types have technical as well as operational advantages compared to the common thread couplings, which are the weakest point in the support system.

In the following sections significant technical differences are explained, design values given and a site example presented to illustrate the operational advantages of the newly developed coupling type.

INTRODUCTION

Many subsurface infrastructures constructed in and around urban areas, are often associated with weak ground conditions and shallow overburden. For this reason, conventional tunnelling needs additional measures to ensure stable conditions in the working area during construction. Commonly used pre-support measures are face bolts (face dowels), spiles (short forepoling), and pipe umbrellas. The application of these systems increases the stability during construction resulting in decreased construction-induced deformations.

Over the last decades, technological developments have led to an increased use of the Pipe Umbrella Support System. This support concept perfectly fills the gap between conventional spiling (short forepoling) and cost-intensive pre-support systems like ground freezing or jet-grouting techniques. This method supports potentially unstable ground ahead of the face and in the working area, provides a high degree of flexibility that can be easily adapted to the encountered conditions while increasing safety in the construction.

For years, the design of pipe umbrella systems was commonly performed empirically by using experience gained from earlier projects due to a lack of knowledge about the system and support behaviour (ground-structure interaction).

This has changed over the last decade because of scientific investigations which helped to clarify the mode of action and the influence on the system behaviour during construction. Rules for analytical as well as 30 numerical design approaches were developed and are in use nowadays (Volkmann, Button and Schubert, 2006; Volkmann and Schubert, 2010).

Motivation

Initially, pipe umbrella support systems were considered special support measures and the installation was performed by special machines with long feeds (for example Casagrande Drill Rigs) so the supporting pipes could be installed in one piece. Consecutive technical developments for conventional drilling machinery have allowed the installation of large r pipe diameters with standard drilling machinery. As the feed length of these machines is limited, it was necessary to install the pipe umbrella pipes piecewise, which were connected by standard threaded connections.

The overall pipe performance with standard threaded connections has one big disadvantage: the section mod ulus of the pipe is tremendously reduced in the coupling section. To overcome this weak point in the support system, the safety factor for the design must either be increased by a factor of more than two or the correct geometry including possible inaccuracies of pipes and threads is used for rigorous design calculations. Due to this fact, a standard cut threaded connection limits the support efficiency. Consequently, more than 50 percent of the regular steel cross-section stays unused during construction, so the cost efficiency is very low.

For example this issue was countered by the introduction of an additional steel (re)bar centered in the supporting pipe; however, this measure is not very efficient because the steel bar has a relatively small outer diameter so the increase of the overall section modulus is nearly negligible.

Further development led to the so-called “nipple connection” which, counters this weakness accordingly. This connection type is as strong and as stiff as a regular pipe under bending. For many project conditions, this optimum pipe connection solution is not required, so a simpler type of connection was developed without decreasing the safety conditions during construction - the so-called 'squeezed connection', which will be explained in the main part of this publication.

STATE-OF-THE-ART

In their book “Tunneling with Steel Support”, Proctor and White (1964) discussed the use of wooden “spiles” as a forepoling method for traversing weak and ravelling ground. Since that time, several slightly different concepts have evolved, all with the goal of providing additional support above and directly in front of the working tunnel face to suppress local instabilities and collapses. Concurrently to technical adaptations, terminology has also evolved. However, some pre-support methods are not delimited from each other by clear definitions, or different names are used for the same system.

For example the pipe umbrella support system is also referred to in worldwide literature as “Steel Pipe Umbrella” (Oreste and Peila, 1998), “Umbrella Arch Method” (Kim, Koo and Bae; 1996), “Pipe Fore-Pole Umbrella” (Hoek, 2003), “Long-Span Steel Pipe Fore-Piling” (Miura, 2003) or “Steel Pipe Canopy” (Gibbs, Lowrie, Keiffer and McQueen, 2002). All these terms describe a support system that consists of pipes, which are installed from the actual face into the ground ahead prior to excavation. Typically, umbrella pipes are 12.0 m to 18.0 m long and arranged along the outer periphery of the tunnel. The outer diameter of the pipes ranges from 70 mm to 200 mm.

Mode of action

Internal forces within the pipes are almost non-existent after installation, comparable to other passive support measures like rock bolts or steel support. The newly installed pipes are not affected by previous activities, whereas every construction process after the installation that causes a load transfer and/or deformation starts to activate the supporting effect of the pipes. Deformations are mainly developed by the excavation process during tunneling and their three-dimensional displacement characteristics rule the activation of the supporting effect after the installation of each pipe umbrella.

Each pipe is founded in the ground ahead of the face as well as on the primary lining behind the open span in the longitudinal and radial direction. So the strength and stiffness of the pipe umbrella support depends on both the ground properties and on the time dependent strength and stiffness properties of the primary lining (shotcrete, steel beams, etc.).

The supporting measure of a pipe umbrella can be divided in three different effects:

(1) the subdivision of the unsupported area in the open span of the working area,

(2) the radial supporting effect, and

(3) the longitudinal supporting effect (Volkmann and Schubert, 2007).

The co-action of these effects results in the support of the working area and parts of the face region. Loads created in this section are transferred by each single pipe in the longitudinal direction to its foundations; the ground ahead and the primary lining. Due to this load transfer, the working area is effectively supported and a stable construction can be ensured.

Design considerations

The design of pipe umbrella support systems follows the assessment of the projects geotechnical characteristics, the ground behaviour and loading conditions. Utilizing detailed in-situ data and three-dimensional numerical calculations it is possible to evaluate the entire system response for common construction conditions.

Three-dimensional numerical calculations must be modelled in detail to archive realistic results. These calculations are time consuming because of the complexity of modelling as well as the variations in ground properties during loading. Alternatively, the design can be done utilizing analytical approaches. In these approaches, each umbrella pipe is basically calculated as beam. This beam is loaded by the expected ground loads and founded on springs representing the strength and stiffness properties of the primary lining on one end and the ground on the other (fig. 1). A realistic loading condition based on back-calculations of in-situ measurements is described in Volkmann and Schubert (2010). The use of the recommended load is limited to regular three-dimensional stress redistributions around a tunnel heading. Consequently, in case of very shallow conditions, the load assumption must be more conservative.

Umbrella pipes are primarily loaded by bending, so it is important that the design is realized as an elastic system. Due to the fact that the curvature in the plastic range (ultimate load) increases dramatically, necessary deformations for the activation of the ultimate load would result in deformation values comparable to a non-realistic collapse scenario.

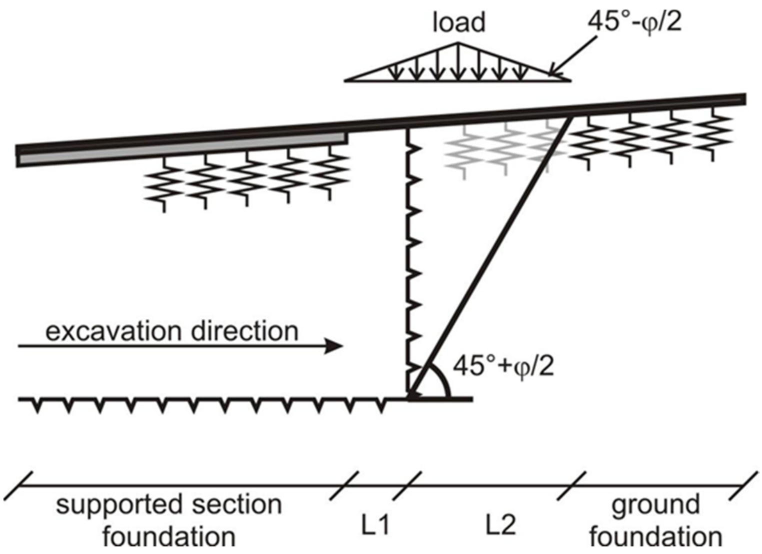


FIG 1. Proposed static model for an analytical design, including load and abutment characteristics.

Installation Methods

In principal either conventional drill jumbos or special drill rig machines can be used to install the supporting pipes. But, from a geotechnical point of view there are two different methods for installation: the pre-drilling system and the cased-drilling system. When using a pre-drilling system, the borehole for the installation is drilled first and in a second step the pipe is installed in this meanwhile unsupported borehole. The significant difference is that when using the cased-drilling system, the pipe follows directly behind the drill bit providing immediate support for the installation hole (Volkmann, 2004).

Due to these installation characteristics, the stress around the drilled holes can either close the annular gap between the pipe and the ground or the deformations can happen unhampered. The second case may result in a partly or full closure of the drilled holes (compare to tunnels in weak ground that induce deformations) (fig. 2). This closure effect creates a stress relaxation ahead of the face which is combined with additional, and mostly unwanted, subsidence. Furthermore, the supporting tube cannot be installed when this closure effect reaches a certain extent. So in weak ground conditions – the main scope for pipe umbrella applications – a cased-drilling system with a lost casing should be used to eliminate risk for higher settlements or non-correct installed support measures (Volkmann and Schubert, 2006). Cased- drilling systems are mostly installed by conventional drilling machinery so the supporting pipe is installed in pieces and these pieces must be connected to each other before drilling. This brings us to the main focus of this publication – performance characteristics of pipe umbrella pipe connections in tunnelling.

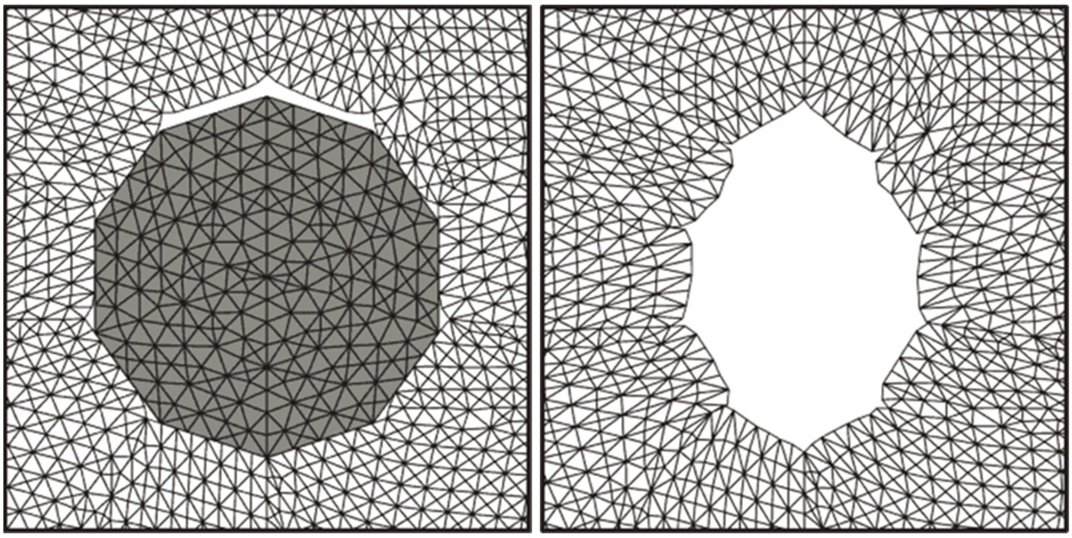


FIG 2. Result of a numerical study in UDEC on the deformation characteristic of the cased drilling system (left) and the pre-drilling system (right). Both holes were 150 mm in diameter before stability calculation.

PIPE CONNECTIONS

The self-drilling installation of cased pipe umbrella drills is accomplished by subsequent installation and connection of single pipe pieces. Thereby, the specified final length of the pipe umbrella can easily be installed even under limited space conditions, as shown in figure 3.

FIG 3. Piecewise installation of a pipe umbrella drill under limited space conditions in a conventional top heading-benching excavation.

As installation methods for pipe umbrella support systems are derived from classical overburden drilling applications, standard pipe connections used for pipeline or infrastructure casings have been typically utilized. These standard threaded connections consist of a female and male tube end, with a mechanically cut inside and outside thread. However, this default connection type does not feature the essential requirements for a pipe umbrella pipe connection, such as the ability to withstand excessive installation forces and provide a stiff and strong connection between two tubes.

These limiting factors have resulted in intense research and development efforts to develop alternative, optimized connection types for pipe umbrella support systems, which will be described further on.

Standard Threaded Connection

As mentioned before, standard threaded connections are generally not well suited for connecting pipe umbrella pipes. By mechanically removing a certain portion of the steel tube for a thread (fig. 4, a), the effective cross-section is reduced. This fact drastically decreases the load-bearing capacity and stiffness in the connection area. Hence, to achieve certain given pipe umbrella design parameters, an over-dimensioning of the un-weakened pipe section is a practical way – though highly inefficient –to overcome this limitation.

Standard threaded connections are therefore only recommended for the installation of measurement instrumentations and for ground-improving injection works; installed tubes show a constant inner diameter.

a)

b)

FIG 4 a) standard threaded connection and b) threaded nipple connection.

Nipple Connection

Nipple connections consist of threaded connection fittings which are pressed and welded into the ends of standard pipes (fig. 4, b). This connection type provides an elastic design load as well as stiffness properties equal to an un-weakened pipe. By using this connection type, default design parameters are constant over the entire length of installed, connected pipe umbrella drills.

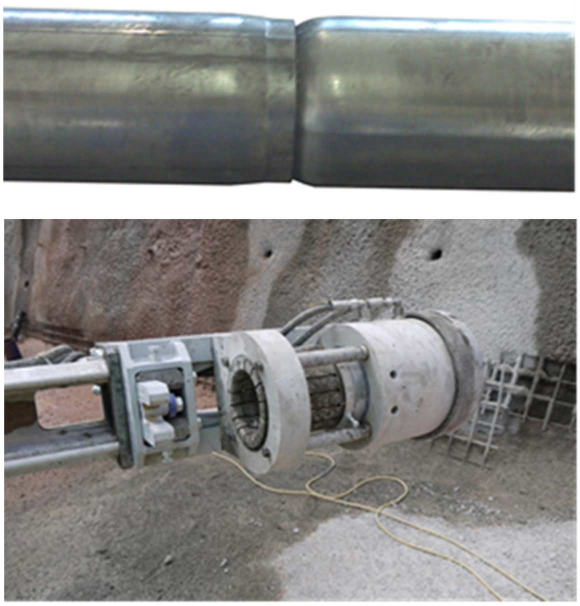
This connection type is recommended for applications where the static load-bearing capacity is required to achieve stable conditions, and the limitations of settlements are an integral part of the design. Hence, the inner cross-section of the pipe umbrella drills is reduced in the connection area, featuring an installation in combination with sacrificial, single-face drill bits.

Squeezed Connection

The latest development in the field of pipe connections is the squeezed connection, this connection type results from the attempt to provide a tough and easy-to-connect alternative to conventional threaded systems (fig. 5 a). By means of the squeezed connection, non-threaded pipe ends are mechanically connected in terms of force-fitted squeezing using a boom-mounted press (fig.5 b).

This connection type features a higher elastic design load in the connection area compared to standard un-weakened pipes as well as a slight reduction in stiffness compared to the regular pipe. Besides simple design and handling, this connection type provides operational benefits due to decreased time intervals required for pipe connection. Similar to the nipple connections the squeezed connections also reduce the inner cross-section of the pipe umbrella pipes so again an installation in combination with sacrificial, single-face drill bits is required.

a)



b)

FIG 5 - a) squeezed connection and b) squeezing console mounted on a drill boom.

PERFORMANCE CHARACTERISTICS

Bending Tests

According to the actual loading conditions, a bending test is the most appropriate testing procedure to evaluate the strength and stiffness properties of umbrella pipes and its features. So, bending tests were performed with grouted and un-grouted steel pipe samples. An outline of the dimensions of the testing apparatus is shown in figure 6. The relative displacement of three measurement points determines the sample's deflection. This value was used as a feedback command controlling the test procedure.

Back calculations from on-site data and geotechnical model show the necessity to design pipe umbrellas in the elastic range. However, higher loads leading to a failure of the pipes cannot be excluded. Thus, the test's focus was set to the elastic range. Nevertheless, the samples were loaded until failure or up to the point where a significant drop in resistance occurred.

The results of these test revealed some important points for construction and design. A perfect grouting procedure does not significantly influence the load-bearing capacity of pipe umbrella pipes. Grouting seems to be more important for the stress transfer between the ground and the pipes. Injection holes (for injection valves) neither influence the strength nor the stiffness characteristics in the elastic loading range.

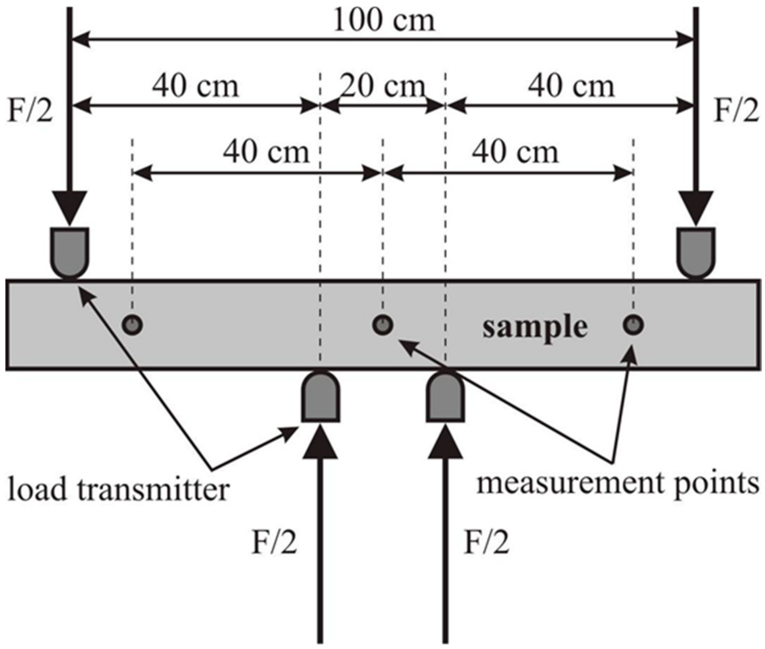


FIG 6. Bending test arrangement.

Comparison of Connection Types

Figure 7 shows results of the bending tests on 114.3 x 6.3 mm pipe umbrella pipes with different connection types. Due to the simple static system of the bending test, an analytical performance analysis based on a bi-linear material behaviour of steel can be calculated (black & grey line). The transition from elastic to plastic is marked by a red dot. At this point, the outer fibre of the steel pipe reaches the steel yield point. This represents the design load without considering any safety factors.

The green line represents a regular, grouted pipe. As expected, the results are a little stiffer and stronger compared to the analytical calculation.

The test sample with a standard threaded connection (red line) shows a much weaker performance during bending (more deflection due to load). In this case, the ultimate load is at a tension force of about 100kN, which is at the level of design load for the regular pipe. For this test, calibrated pipes were used. Assuming that default pipes would be used for a standard threaded connection, failure would occur at even lower loads resulting in a consecutive drop in load-bearing capacity.

As already described, the performance of the nipple coupling sample is comparable to the behaviour of the regular pipe. So in the design stage, the section modulus of the regular pipe can be used without any adjustment.

The squeezed connection (blue line) shows a much stronger behaviour than the standard threaded connection, although its elastic behaviour is more ductile than a regular pipe. The ultimate load is significantly higher than the design load point which increases the safety during construction for a given pipe specification.

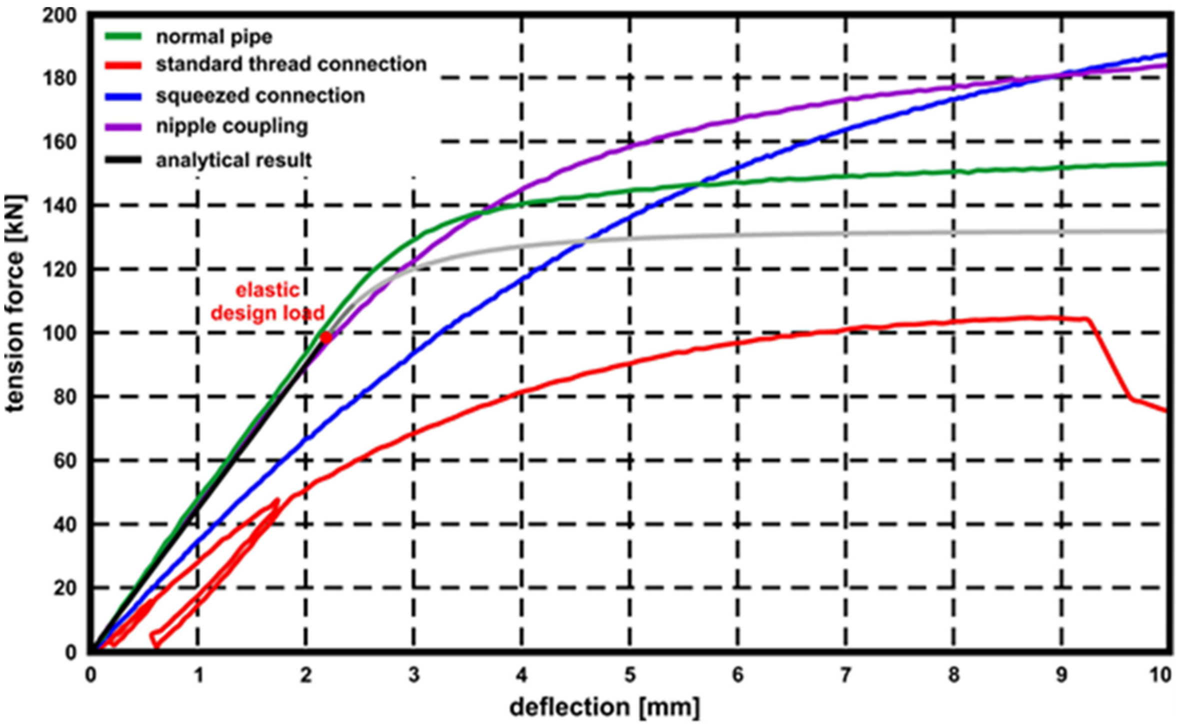


FIG 7. Results of bending tests for 114.3x6.3 mm pipes with different available pipe umbrella coupling types.

Evaluation of performance characteristics

Test results explicitly show that the influence of the connection type is essential for the support effect of a pipe umbrella. The selection of the best connection type is dependent on project requirements. A designer does typically not have access to test results of all available pipe types, so the following formulations will be explained to allow realistic design values for strength and stiffness properties of the different connection types to be developed (table 1).

The standard threaded connection consists of a thread area and the two remaining steel pipe sections outside and inside of the thread. For an elastic calculation, the wall thickness of the pipe must be reduced to calculate the necessary values. The remaining wall thickness can be estimated by the original pipe wall thickness minus the height of the thread section (1.3mm to 1.6mm) divided by two (male and female part). For the actual calculation, the male part should be chosen as it is the lower value (safe side). This calculation does not consider that the inner edges of the thread act as notches.

As previously stated for tendering a nipple connection the original values of regular pipes can be used for elastic design calculations.

For a squeezed connection, relevant elastic design parameters can be estimated by decreasing the outer diameter by two times the pipe wall thickness and additional 4 mm (one pipe is squeezed towards its counterpart so the inner steel tube is the relevant part). For tunnels where settlements are no issue, it would also be possible to use the values of the normal pipe due to the plastic reserves of steel (compare fig. 7).

TABLE 1. Exemplary elastic design values for 114.3 x 6.3 pipes with different connection types.

Parameter	Regular pipe & nipple connection	Squeezed connection	Standard threaded coupling
Second moment of area [cm^4]	312.7	158.8	121.9
Section modulus [cm^3]	54.7	34.4	22.8
max. elastic moment [kNm]	19.4	12.2	8.1
max. elastic moment [%]	100.0	62.9	41.8
* values are calculated for an exemplary 114.3 x 6.3 mm pipe			

SITE EXAMPLE KORALMTUNNEL LOT 3

The Koralm Tunnel (KAT) (Austria), part of Austrian railway system, forms an important link of the Trans European Railway Network (TEN). Upon completion, this twin-tube tunnel with a length of 32.9 km will be one of the longest railway tunnels worldwide. The project is divided in 3 main construction lots. Lot 1 at the eastern end was constructed by using the principles of the New Austrian Tunnelling Method (NATM), while lot 2 the central part under the Koralpe with overburden up to 1,000m is driven with two tunnel boring machines (TBM). Lot 3

(KAT3), which is already partly excavated due to the Exploratory Tunnel Paierdorf, is constructed by the NATM as well as TBM depending on the tunnel section.

At the western portal in Carinthia (Austria) the northern tube is excavated under the support of consecutively installed 15m long pipe umbrella support. This section is situated at shallow overburden in clay and sandstones. The AT-114 pipe umbrella system was applied, which has an outer steel tube diameter of 114.3 mm with a tube wall thickness of 6.3 mm. When using this system, the supporting tubes are installed piecewise with conventional drilling machinery (Atlas Copco E2) and a squeezed connection was chosen for connecting the single tube pieces. Main reasons for the selection of this new connection type were the optimization of the pipe umbrella tubes with regards to cross-section and weight, as well as time savings and safety gains during pipe umbrella installation.

Preparation of drilling machinery

The day before pipe umbrella drilling, the squeezing units were mounted on the drill booms and connected to the drill rigs hydraulic system. The entire process needed about 5 hours including a performance test of the squeezing units and their hydraulic connections prior to mounting the completed system on the drill boom (fig. 8). Further optimization led to changing times between 3 and 4 hours.

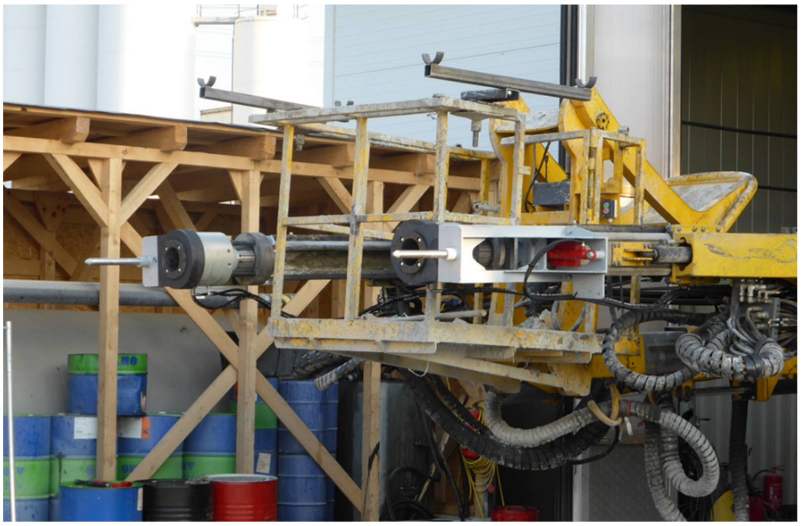


FIG 8. Both squeezing units mounted on the drill booms – drilling machinery ready for the installation

Installation of pipe umbrella support system

The drilling works started in the northern tube of the KAT3 in the night from April 2nd to 3rd. At the beginning the workers were introduced to the system and were instructed about the processes and their correct sequence for a successful installation routine. For this reason a 15m long pipe was installed with one drill boom and then another pipe with the other drill boom. These installation processes were finished after 40 minutes and 41 minutes respectively after the de-installation of the drill steel. Then both drill booms were used parallel and the lift in the middle were used to feed the extension tubes with the drill steel onto both drill booms (fig. 9). Approximately 24 hours later the first pipe umbrella were installed with 34 pieces of 15 meter long pipes.

With further experience with the system it was possible to typically install one pipe umbrella consisting of 34 pieces 15 meter long pipes in between 18 and 20 hours. So, the establishment of fast and safe connections for pipe umbrella pipes was proven in the course of the KAT3 project whilst time consumption for the working step “tube connection” could be reduced by more than 50% compared to typical installation times with threaded couplings.

FIG 9. Pipe umbrella installation with two squeezing units mounted onto the drill booms during drilling and a feeding lifter in the middle.

CONCLUSIONS

Following the successful application of pipe umbrella support systems over the past decades, one recent focus of R&D efforts was the optimization of pipe connections. By introducing new connection types, the design of pipe umbrellas can now be accomplished according to specific ground and application conditions – the proper connection type can be chosen based on elastic design loads, stiffness properties, as well as operational considerations.

These additional developments complement the knowledge base about umbrella support systems, so a realistic design and clear tender documents can be prepared. It was shown that the developed connection type and auxiliary equipment can improve both the structural behaviour and construction performance of pipe umbrella support systems, in particular ensuring a quicker installation, higher safety for the involved workforce and last but not least the high cost efficiency for the installed steel tube sections.

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Appendix B

Title: Rock Mass — Pipe Roof Support Interaction Measured by Chain Inclinometers at the Birgltunnel.

Author(s): G.M. Volkmann

Published: International Symposium on GeoTechnical Measurements and Modelling, Karlsruhe, 2003

Summary: This publication explains the applied inclinometer measurement system and its data acquisition system. Differences between a geodetic survey and the inclinometer system are discussed and advantages numbered. It also shows results of the measurement campaign regarding time dependent behavior as well as the behavior of a pipe umbrella support system.

Rock Mass — Pipe Roof Support Interaction Measured by Chain Inclinometers at the Birgltunnel

G.M. Volkmann

Institute for Rock Mechanics and Tunneling, Graz University of Technology, Austria

ABSTRACT

The increased use of pipe roof support systems in tunneling has not been followed by an in-creased understanding of the system behavior of this support system. In order to address this problem detailed investigations on the rock mass—pipe roof support interaction have been initiated. The knowledge about this system was increased by an on—site monitoring system which included continuously recorded chain inclinometer measurements combined with geodetic measurements. Characteristics of the measured settlements indicate that the design of the overlapping length should consider the bearing capacity of the rock mass.

1 INTRODUCTION

Increasingly forepoling methods and/or soil improvements are utilized ahead of the face in tunnels with shallow cover. These should improve the behavior of the surrounding rock mass with respect to stability and deformation during the excavation process. To improve the stability of the area near the face, face bolts, spiles, and tubes often combined with grouting are in use.

Technological advances in pipe roof drilling systems have reduced costs and the installation time over the last years. This has resulted in the increased application of pipe roofs as a normal support system during shallow tunneling instead of being limited to special geotechnical problems.

This rapid increase in use has not been followed by an increased understanding of the interactions be-tween the support system and the surrounding rock mass. Therefore, there are only numerical simplifications or conservative rules for design. Currently dur-ing tunnel design, as well as on site, parameters for a pipe roof system e.g. tube length, overlapping length, distance between the tube axes and the strength of one tube are fixed by experience or an empirical approach. In order to improve design methods for these systems a detailed study has been initiated.

Numerical programs can solve a given geotechnical model very well, but an appropriate model can only be selected after calibrating it by measured settlement data from a tunnel site. That data set should have the following specifications: On one hand it should be as exact as possible and on the other hand it should describe the area ahead of and behind the face as well as on the surface. In order to understand and model the behavior of a pipe roof support system a monitoring program was developed to collect the necessary data to calibrate the geotechnical mod-el.

2 PROJECT AND GEOLOGY

The 950 m long Birgltunnel (Austria) is a double track rail tunnel constructed as a part of the upgrading of the “Tauernachse” between Salzburg and Villach (Austria) with an excavation area of around 130 m2.

A majority of the tunnel is situated north of the “Tauernnordrandstörung” in the “Grauwackenzone”. The west portal and an approximately 80 m long section of the tunnel are situated within the fault zone. This section was constructed by using the NATM (New Austrian Tunneling Method) utilizing a pipe roof support system. In the evaluated zone the over-burden raises form 30 m up to 50 m.

This part of the “Tauernnordrandstörung” consists of a clayey, cataclastic fault zone material with shear lenses composed of more competent blocks (3G & BGG, 2001). Due to the potential for encountering blocks the rock mass behavior was described as ranging from isotropic to highly anisotropic. The design rock mass parameters are shown in Table 1.

The design idea was to pass this weak rock zone with a stiff support system. To achieve this, a temporary top heading invert and a short bench were ap-plied. In the section discussed here, the top heading face was opened in three parts, with a 1 m advance supported with a pipe roof and additional rock bolts at the sidewalls. The stability of the face was ensured by a combination of shotcrete and up to five face bolts. After the top heading, a top heading in-vert was installed for a temporary ring closure at a maximum distance of 5 m. The ring closure followed the bench excavated with a maximum distance of 6 m (fig. 1). Using that system, the weak rock section was passed without any problems.

Table 1. Strength parameters for “Tauernnordrandstörung” in the area of Birgltunnel (3G & BGG, 2001)

	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

3 DATA ACQUISITION SYSTEM

Two methods were used to acquire the settlement data used in this evaluation. Chain inclinometers were used to measure the settlements in the crown both in front of and behind the face. Geodetic measurements were taken as part of the normal monitoring program and used to describe the absolute position of the inclinometer.

3.1 Geodetical measurements

Each measurement section of the top heading consists of three points (one in the crown, one on each sidewall) with a longitudinal distance between two sections of approximately four meters. The zero reading of the points was made within 12 hours of the excavation passing that section and then once a day.

Additional geodetical points were situated at the beginning of each chain inclinometer string.

3.2 Inclinometer measurement system

The system which was used at Birgltunnel is an in-place 20 m long chain inclinometer consisting of ten links (Boart Longyear Interfels, 2002). Three pipe roof fields in a row were equipped as shown in figure 2. The tilt meters measured the inclination continuously and transferred data every minute to the data acquisition system.

As neither the start point nor the end point of the inclinometer chain can be considered to be fixed points, the geodetic measurements are used to determine the spatial position of the beginning of the inclinometer.

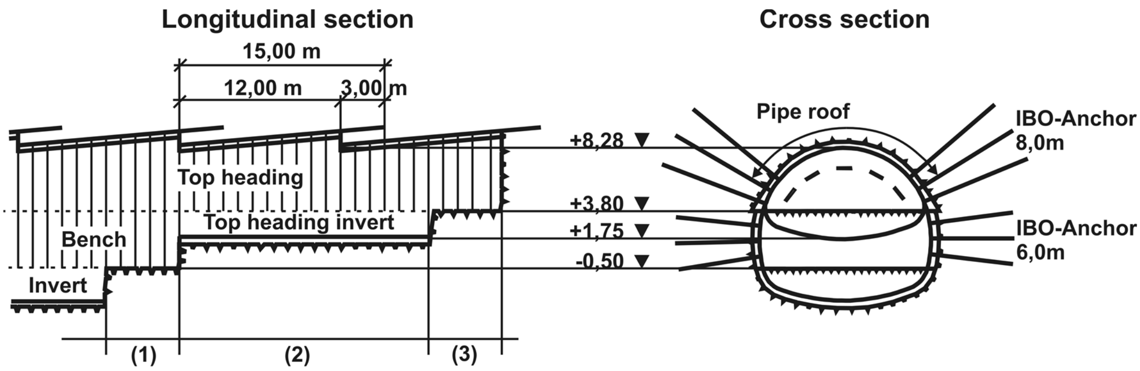

Figure 1. This figure shows on the left side the longitudinal section and on the right side the cross section of the excavation steps. *1 The maximum distance between the top heading face and the temporary top heading invert closure was 5 m.*2 The maximum distance between the face of the bench and the invert closure was 6 m.

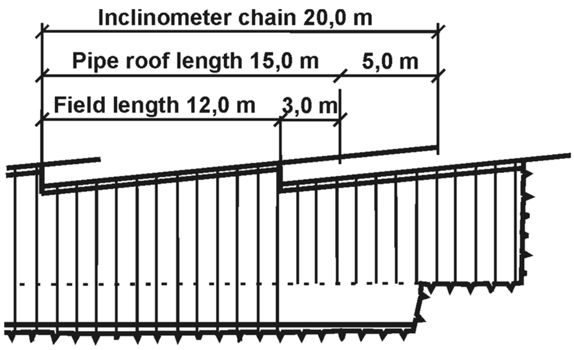

Figure 2. Position of the inclinometer chain relative to the pipe roof.

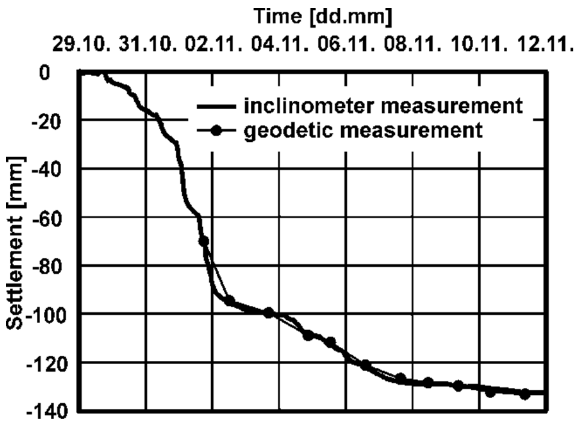

Figure 3. The inclinometer chain and geodetical measurements of the section 59.75 compared to each other shows that the results are equal with respect to the accuracy.

3.3 Problems during the data evaluation

During a study of technical literature, it was assessed that no comparable measurements were published. That raises the problem of how to fix the measured values in between the time of two geodetical measurements.

A comparison between a linear and an inclination dependent approach showed that using the inclination of the first or the last inclinometer string multi-plied by a linear factor gives better results for the vertical settlements over the measured time period. The data quality of the chosen system is shown in the time settlement line of figure 3. The errors be-tween the geodetical measurements and the inclinometer measurements can be minimized to a value lower than 1 mm. But the absolute accuracy of this inclinometer measuring system can never be higher than that of the geodetical system, because as dis-cussed before the vertical position was given by that system.

3.4 Advantages of the inclinometer chain measurements

In figure 3 it can also be seen that the settlement path of the inclinometer starts nearly 4 days earlier than the geodetical measurements. This advantage exists because the inclinometer measurement starts immediately after the installation and measures the inclination continuously for 20 m in front of the face position where it was installed (fig. 4). Due to the overlapping length of the inclinometer chains the settlements were measured for a minimum length of 8 m ahead of the face.

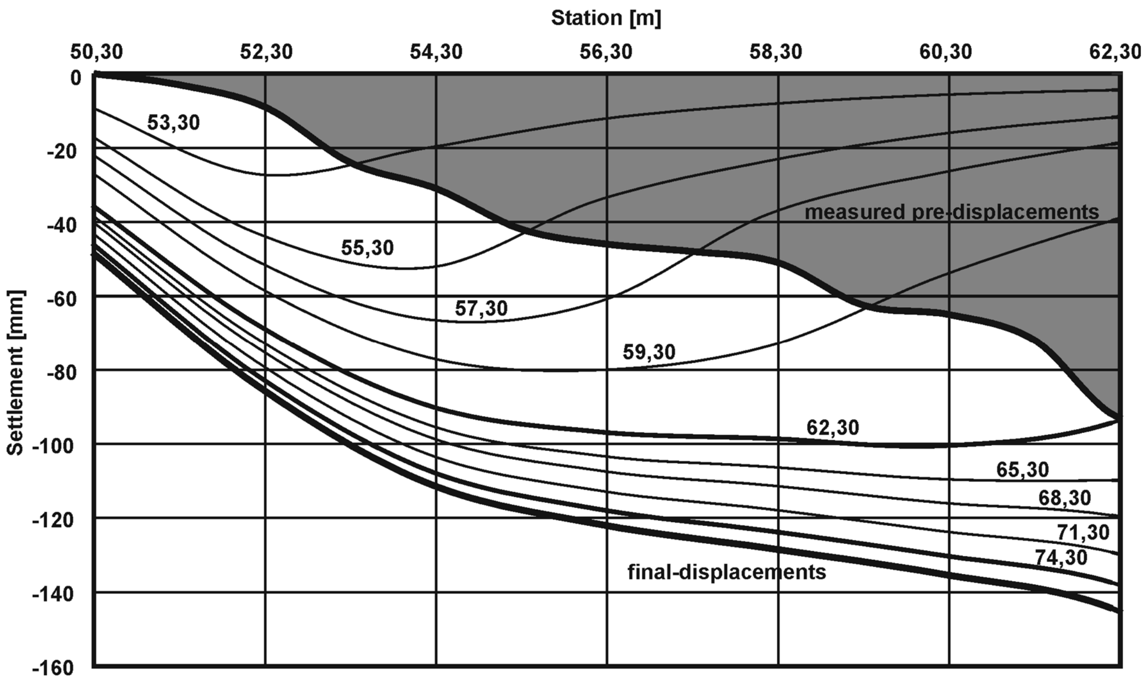

Figure 4. The deflection curve diagram shows the data of the inclinometer chain from station 50.3 to 70.3 in the discussed pipe roof field.

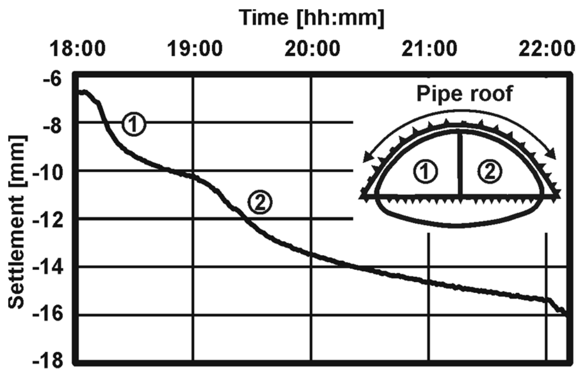

Figure 5. The time settlement line in that diagram shows the settlements two meters ahead of the new face position with a face area divided into two parts during one meter of excavation.

This arrangement allows the evaluation of the settlements ahead of the face in their absolute value. Additionally, the given settlement rates and their relationship to the face can be evaluated for the designation of the stress transfer length during tunneling. The measured rates were also used to get information about the system behavior of the pipe roof system during single excavation steps.

Besides the advantage that the pre-displacements can be measured, the precision of the dataset in time creates the possibility to observe the time dependent rock mass behavior. Another evaluation possibility is that every construction step can be analyzed with regard to its influence on the settlement increments (fig. 5).

Because of these advantages this monitoring pro-gram was chosen for the evaluation of the pipe roof support influences in tunneling.

4 RESULTS OF THE MEASURING CAMPAIGN

The evaluation of the measured dataset leads to a few significant results which are discussed in the following paragraphs.

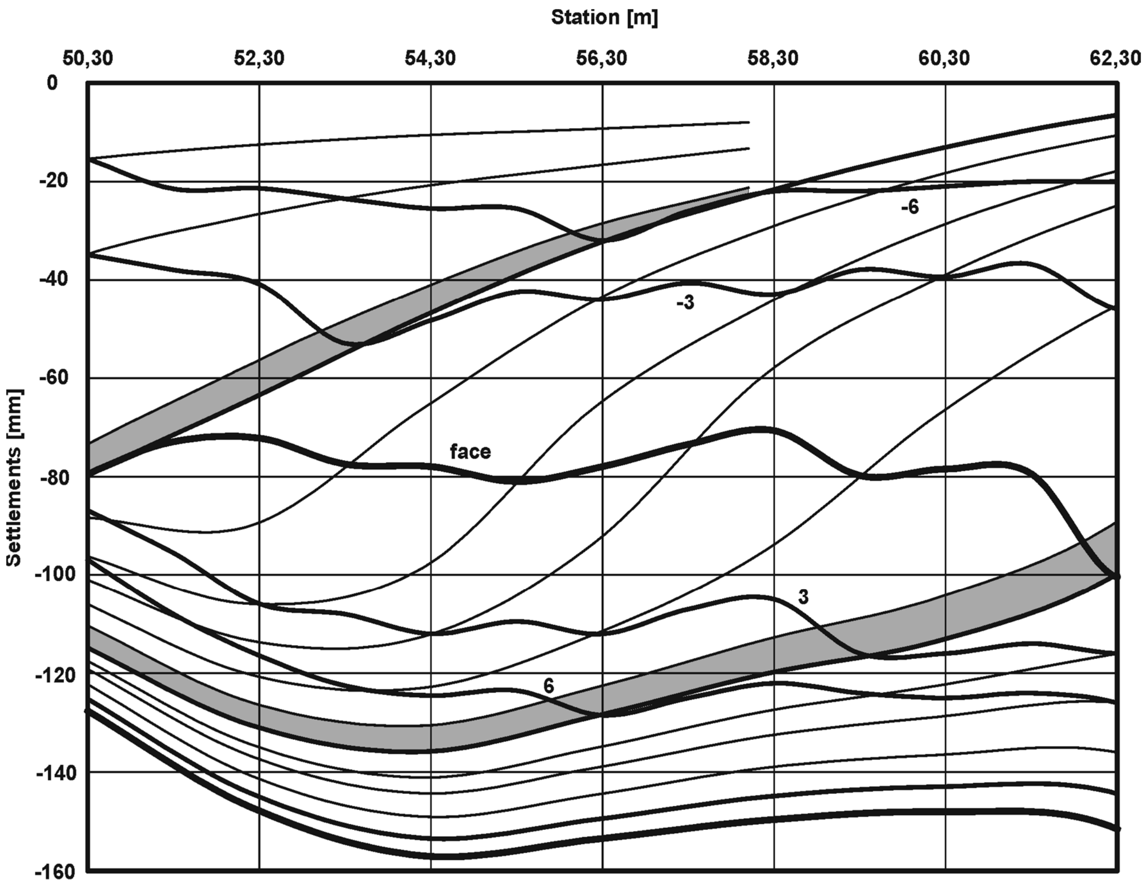

Figure 6. deflection curves diagram with the whole settlement path in the pipe roof field between station 50.3 and 62.3

4.1 Time dependent settlements

As shown in Figures 3 and 5 the stress transfer re-quires a given period of time until the settlement rate indicates stability. As this process occurs, the shotcrete's support strength increases. Therefore, the in-fluence of each phenomenon cannot be determined uniquely.

4.2 Ratio between pre-displacements and settlements behind the face

The overlapping length of the inclinometer chains makes it possible to add measured pre-displacements of the previous inclinometer chain to the measured settlements of the current inclinometer. This fact was used to evaluate the whole settlement path in the crown of the pipe roof field (fig. 6). The top three lines are measurements from the first inclinometer and the remaining lines are from the second inclinometer. The grey shaded area in that figure shows the time dependent settlements during the installation time of the pipe roof.

The “face” line divides the pre-displacements from the settlements behind the face. Over two third of the pipe roof field, the settlements ahead of the face are equal to those occurring behind the face (fig. 7).

4.3 Stress transfer area

The first measured settlements appear in a region around 15 to 18 m in front of the face. That is three times the height and around one and a half times the width of the top heading area. The line “-6” in Figure 7 displays the settlements which occur 6 meters in front of the advancing face. These settlements are up to 15 % of the whole measured settlements. The settlements occurring behind the face are nearly the same.

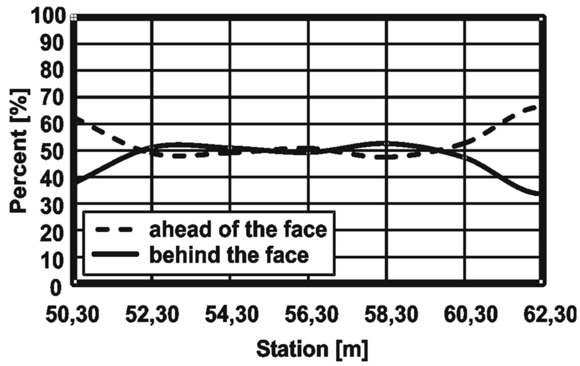

Figure 7. The percentage between the settlements ahead of and behind the face in one pipe roof field.

4.4 Characteristics in a pipe roof field

As described before, the percentage of the settlements ahead of and behind the face in the middle of the pipe roof field are nearly the same. At the beginning and at the end of every measured pipe roof field the pre-displacements are up to 65 % of the total settlements. The analysis illustrates that the pre-displacements increase in between the face and three meters ahead of the face. The foundation length of the pipes ahead of the face is also 3 m. This results from an increase in the loading level directly ahead of the face because of the installed pipes.

Relative to the high amount of settlements ahead of the face, the stiff shotcrete arch reduces the settlement amount for the first three excavations steps. On this account the total settlements in the pipe roof field are nearly the same at each position.

5 CONCLUSION

The increasing use of pipe roofs in tunneling calls for a scientific based design method. Therefore, a monitoring campaign was designed to evaluate the system behavior of the pipe roof support system during the excavation process. The measured dataset fulfills the chosen requirements for further theoretical investigations and shows significant characteristics for the settlement behavior in a pipe roof field.

In the evaluated section of the Birgltunnel (Austria) the excavation dependent vertical displacements in the crown start around 15 m ahead of the face and by the time the face passes around one half of the total measured settlements have occurred. At the be-ginning and at the end of the pipe roof fields the amount of the pre-displacements raises up to 65 % of the total settlements but the stiff arch at the drilling position of the pipe roof counters this effect, leading to similar total displacements. The evaluation of the settlement data indicates that the bearing capacity of the rock mass should be included in the design of the overlapping length of the pipe roof fields.

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Appendix C

Title: A Contribution to the Effect and Behavior of Pipe Roof Supports.

Author(s): G.M. Volkmann

Published: EUROCK 2004 & 53^{rd} Geomechanics Colloquium. Schubert (ed.) © 2004 VGE

Summary: This publication explains the applied inclinometer measurement system. In 2004 this measurement system was applied at 2 projects; the Birgl tunnel (Austria) and the Trojane tunnel (Slovenia). Key data of both projects are described. Furthermore, the observed settlement behavior of a pipe umbrella system and the influence of different installation methods are evaluated. The last focus of this publication are the possibilities to determine changes in the ground conditions ahead of the face.

A Contribution to the Effect and Behavior of Pipe Roof Supports.

Volkmann, G.

Graz University of Technology, Institute for Rock Mechanics and Tunneling

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 m2 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.

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

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

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.

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 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.

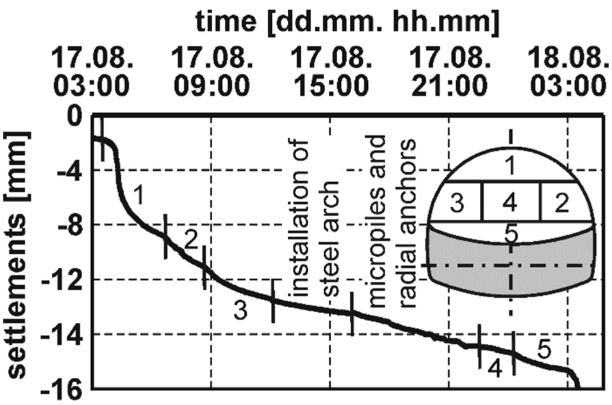

Figure 1. Measured time - settlement line for one excavation step in five phases with support installation

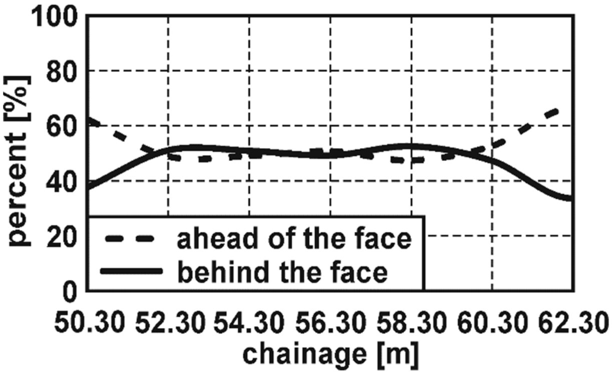

Figure 2. Settlement characteristic in a pipe roof umbrella field (Volkmann, 2003)

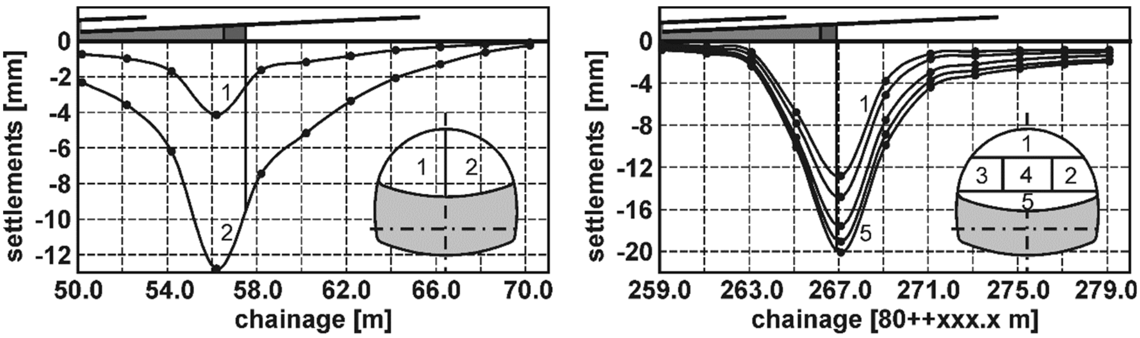

Figure 3. Deflection curves due to one excavation step; left side Birgltunnel, right side Trojanetunnel

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 sections where the installations indicated settlement amounts smaller than 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.

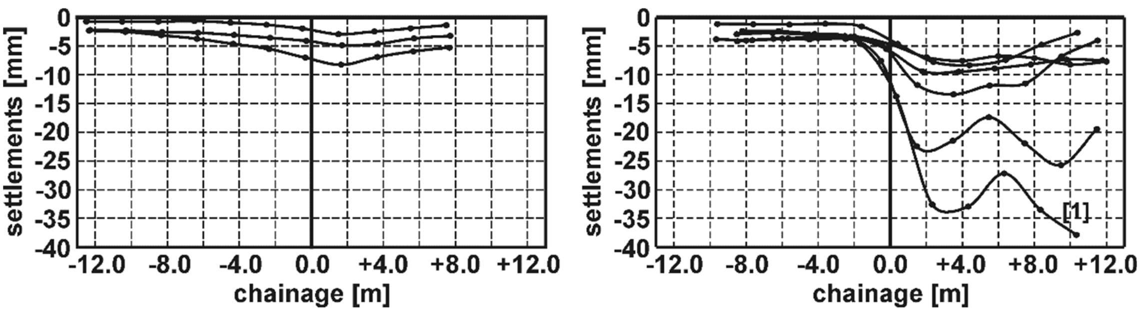

Figure 4. Settlements during the installations of different pipe roofs; left Birgltunnel, right Trojanetunnel

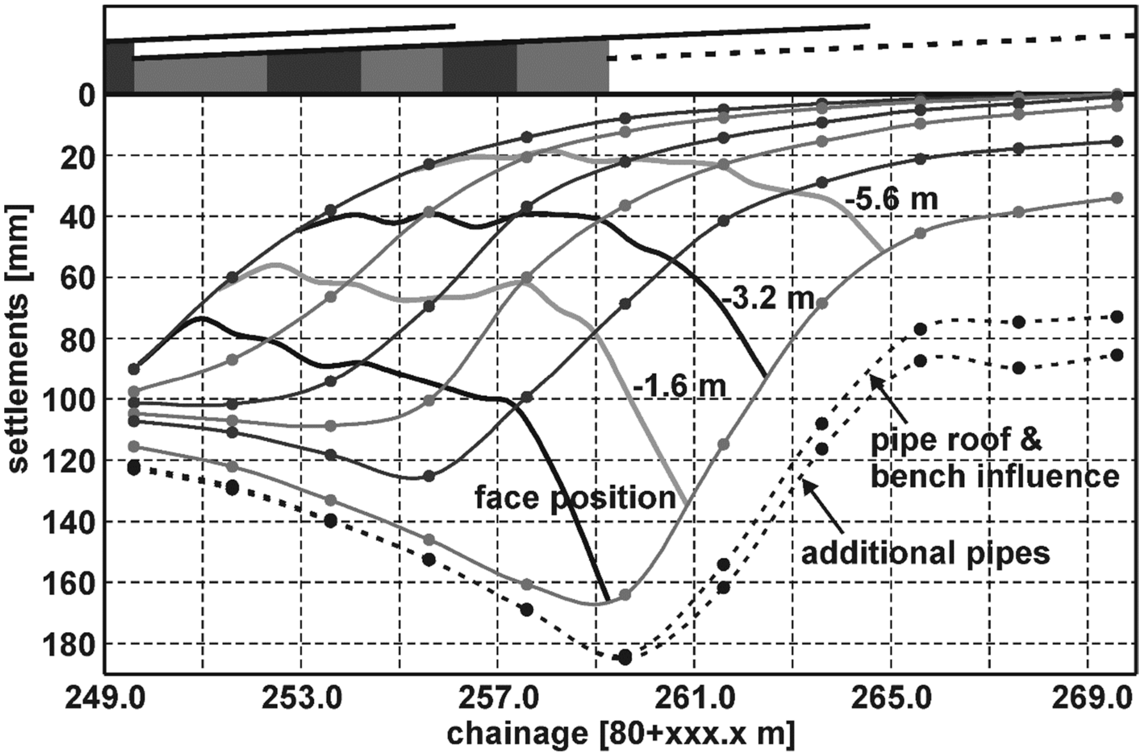

Figure 5. Deflection curves diagram showing every second excavation step including the installation of the following pipe roof umbrella with additional pipes

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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Appendix D

Title: Geotechnical Model for Pipe Roof Supports in Tunneling.

Author(s): G.M. Volkmann; W. Schubert

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Summary: After the introducing part this article defines a pipe umbrella system and describes the term system behavior. Then the measured behavior is described for three phases of a pipe umbrella excavation sequence and the associated ground displacements displayed in figures. The main part of this publication determines the geotechnical behavior including the three supporting effects associated with pipe umbrella support systems.

Geotechnical Model for Pipe Roof Supports in Tunneling

G.M. Volkmann

ALWAG Tunnelausbau GesmbH, Austria

W. Schubert

Graz University of Technology, Institute for Rock Mechanics and Tunneling, Austria

ABSTRACT

Pipe Roof Support systems are increasingly being used in weak ground tunneling although the design is often only based on experience. The results of an intensive measurement campaign, including settlement measurements ahead of the face, were used to determine a geotechnical model for this pre-support system. The measured ground-support interaction was reproduced utilizing three-dimensional numerical calculations with FLAC-3D. This study confirmed the model created by the in-situ data and clarified the influence of the design parameters on both the supporting effect and the ability to control displacements. This enables a more informed decision in the design stage on whether a Pipe Roof System or more cost- and time-consuming support systems must be used to guarantee the project requirements during construction.

1 INTRODUCTION

The increasing population in metropolitan areas re-quires a concomitant upgrade of the infrastructure. The majority of the infrastructure is located subterranean, especially in congested urban areas. Due to the pre-existing structures on the surface, the construction of new projects is subjected to specific restrictions such as subsidence and/or noise limitations during construction. In urban areas, the ground generally consists of sedimentary soil and/or highly weathered rock mass. Both types of ground can be associated with major displacements during tunneling. In these cases, the project limitations control the entire design process, as compliance with the design requirements may require time and cost intensive additional support systems including ground freezing, jet grouted columns or pipe jacking.

An alternative support system is the “Pipe Roof Umbrella” System, which is also referred to in literature as “Steel Pipe Umbrella” (Oreste & Peila 1998), “Umbrella Arch Method” (Kim et al. 1996), “Pipe Fore-Pole Umbrella” (Hoek, 2003), “Long-Span Steel Pipe Fore-Piling” (Miura, 2003) or “Steel Pipe Canopy” (Gibbs et al.

2002). Compared to this system the previously mentioned pre-support systems are stiffer but pipe roof systems are less time and cost intensive. These facts have led to an increase in the use of the pipe roof method without an accompanying increase in the knowledge about the ground support interaction associated with this system.

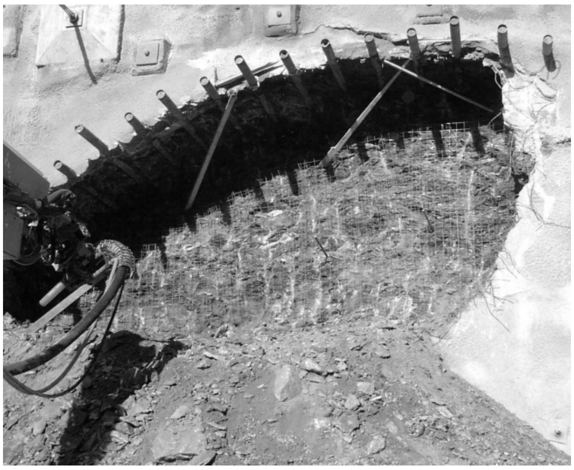

Figure 1. West portal of the Birgl tunnel (AUSTRIA) supported by a pipe roof

In order to obtain a better understanding of this support system in situ measurements, using inclinometer chains installed parallel to the pipe roof support, were performed. The measurements of the inclinometer chain were linked to the geodetical dis-placement measurements taken inside the tunnel and on the surface. These measurements display the longitudinal distribution as well as the magnitudes of settlements in the crown level of the tunnel (Volkmann et al. 2003). The outcome was a geotechnical model for pipe roof systems.

Additional laboratory investigations on the ground and the pipes were performed in order to obtain their strength- and stiffness parameters. Both the geotechnical model and the parameters, describing the behavior of the applied materials, were used for the numerical investigations.

2 TYPES AND DEFINITIONS

The pipe umbrella system is one among the group of pre-support systems. The terminology for pre-support systems is not clearly defined and in order to avoid confusion with other systems a brief description is given below.

The aforementioned terms are simply descriptions of the system itself. Steel pipes, and sometimes fiberglass pipes, are installed from the actual tunnel face to the front (fore-poling system) arranged like an umbrella or a canopy around the area to be excavated (Figure 1).

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 generally 12 m or 15 m. The excavated length underneath (length of a pipe roof field) ranges from 6 m to 12 m. When the end of a pipe roof field is reached, there is still a part of the pipe remaining in the ground ahead of the face. This length is called the "overlapping length" of the pipe roof system.

3 SYSTEM BEHAVIOR

The observation and the interpretation of movements caused by tunneling is one of the principles when using the New Austrian Tunneling Method (NATM) (Rabcewicz 1944, Rabcewicz 1975). The observations and their interpretation are used to control the ground support interaction. Specific evaluation techniques allow changes in the ground conditions in front of the face to be predicted (Steindorfer & Schubert 1997). The support system is continuously adapted to the actual ground conditions encountered during tunneling, leading to an economical construction progress. This is possible as the measurements represent the ground-support interaction and not the ground or the support reaction separately. The observed interaction is commonly referred to as System Behavior (Goricki 2003, ÖGG 2001).

The main problem with developing the geotechnical model for pre-support systems is that the geodetical survey only measures the system behavior in the already supported section of a tunnel, while the pre-support systems primarily influences the system behavior ahead of the supported tunnel. The observation of the system behavior, influenced by a pipe roof system, was enabled by performing measurements with horizontal inclinometer chains, installed in the tunnel crown.

4 MEASURED BEHAVIOR

4.1 Installation

Conventional drill jumbos or special machines can be used to install the pipes. From the geotechnical point of view, there are 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 providing immediate support for 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 un-supported hole. In weak ground a pre-drilling system has a higher risk for settlements than a cased-drilling system (Figure 2) (Volkmann 2004).

In weaker ground the stress transfer related to the drilling of the installation holes may cause the closure of the annular gap between the ground and the pipes before the excavation under the pipe roof starts. However, this closure does not result in any significant pre-stressing and hence the internal forces of the pipes are still practically zero. The initial loadings are not influenced by the method used to remove drill cuttings; water or air. However, one must consider that the use of water, when flushed through the annular gap and not within the pipe, may result in a decrease in the engineering performance of the ground. Additionally, the installed fan of pipes operates as a drainage system until the pipes are grouted (Sellner 2005).

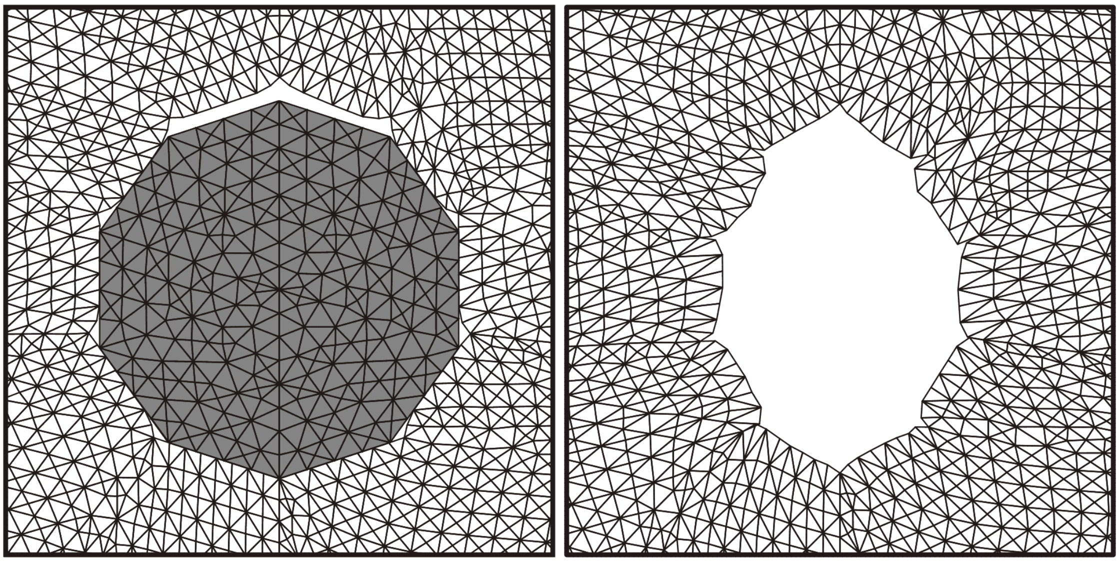

Figure 2. Result of a numerical study in UDEC on the deformation characteristic of the cased drilling system (left) and the pre-drilling system (right) (Volkmann & Schubert 2006)

4.2 The first meters

The space that is needed for the installation of the next pipe roof fan requires a constant widening of the profile during the excavation of a pipe roof field. The new installation generates a ring of shotcrete at the end, respectively the beginning of every pipe roof field. During the excavation of the first 1 to 3 rounds after the installation, the inclinometer measurements displayed smaller settlement magnitudes. This observation is explained by the stiff shotcrete arch that creates a foundation for the longitudinal arching effect during stress transfer processes.

4.3 Normal excavation rounds

The foundation effect of the stiffer shotcrete ring disappears with distance. The data from two case histories display that the following rounds of excavation induce a uniform settlement characteristic. This characteristic contains the magnitudes and positions of the measured settlement values relative to the face and the three-dimensional behavior in the al-ready supported sections (Figure 4 for example). It depends on the strength- and stiffness contrasts be-tween the ground and the installed support.

4.4 Last meters

Depending on the ground quality and the height of the tunnel, the effectiveness of the pipe roof foundation ahead of the face decreases with decreasing length. This decrease in effectiveness is associated with an increase of the settlements ahead of the face. Therefore, when the pipe roof support system is used to minimize displacements caused by the excavation a new pipe roof should be installed before the foundation effectiveness decreases. When the pipe roof system is only applied to increase the stability of the unsupported span, it is no problem to excavate further rounds as long as the stability of the face is guaranteed.

5 BASIC GROUND DISPLACEMENTS

The numerical results shown in the Figures 3a, 3b, 3c, and 4 are calculated with the program “Fast Lagrangian Analysis of Continua in three dimensions” (FLAC-3D). The geometric conditions, used for this example, simplify those from the project Trojane tunnel (Slovenia). The selected section, with a crown cover of 15.0 m, was supported by a pipe roof system to help minimize surface displacements with-in a major fault zone (UCS << 1.0 MPa) where dis-tress to surface structures had been observed. The input parameters were determined by laboratory tests (ground, pipes); as well as taken from literature (ground, shotcrete) (Zlender 2003, Aldrian 1991, Müller 2001). Further information on these calculations can be found in Volkmann et al. 2006. The displacements displayed in the Figures 3a, 3b, 3c, and 4 result from excavating only one 1.0 m long excavation round.

5.1 Radial displacements

The displayed cross sections are 2.0 m ahead of the face (Figure 3a), in the middle of the unsupported span (Figure 3b), and 2.0 m backwards in the sup-ported section (Figure 3c). All three figures display that the orientation of the incremental displacement is almost normal to the shape of the tunnel in the area where the pipe roof is installed.

The maximum displacement vector in Figure 3a is about 3.0 cm but is situated in the area excavated later. The maximum displacement in the zone where the pipes are installed is 2.1 cm.

The displacement vectors in the unsupported span (Figure 3b) have nearly the same magnitude (2.0 cm) and are practically uniform around the upper section of the tunnel perimeter. The displacements only in-crease to 3.3 cm in the region below the installed pipes (lower sidewall).

Due to the very stiff support system in the sup-ported section behind the face, the displacements decrease to 0.5 cm only 2.0 m behind the unsupported span (Figure 3c).

5.2 Distribution of displacements in the longitudinal direction

Elastic numerical calculations normally result in 30 % of the total displacements ahead of the face (Steindorfer 1998). When the stress level is higher than the strength of the ground near the tunnel the dis-placement distribution changes considerably, resulting in significantly higher displacements ahead of the tunnel face. The strength weakening characteristics of the material associated with the post peak behavior control these changes. This fact agrees with the measured data from the Trojane tunnel where up to 80% of the total displacements occurred ahead of the supported section in the tunnel.

Figure 4 exemplifies a calculated displacement distribution in the longitudinal section. The calculated values increase to 2.5 cm approximately 1.0 m ahead of the face while the displacement vectors in the supported section are lower than 0.5 cm. The settlement values start decreasing with distance to the face ahead of the position where the maximum amount occurs.

This example displays that the displacements are orientated against the excavation direction in the sections ahead of the face, except the last 2.0 m ahead of the face. This area moves vertically or slightly in the excavation direction. In the unsup-ported span and in the first supported meter the dis-placements are orientated towards the face. The previously supported region again displaces against the excavation.

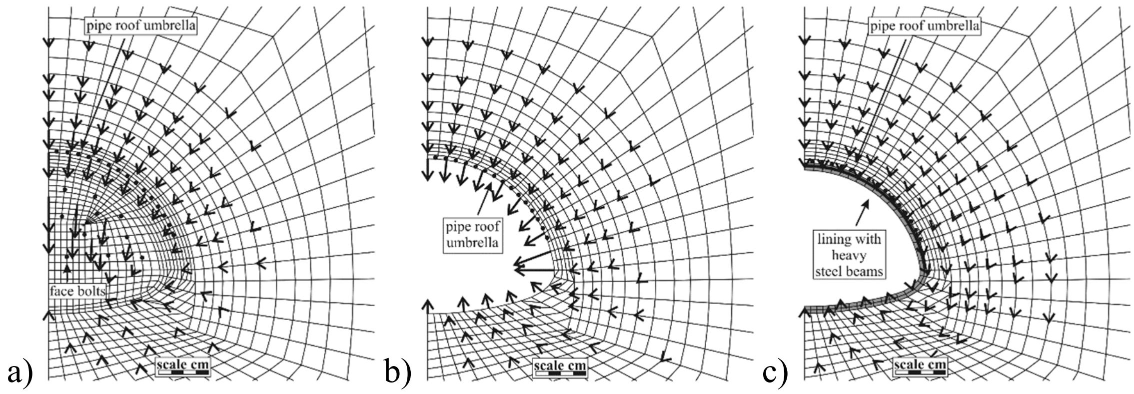

Figure 3. Three cross sections displaying the displacement vectors normal to the tunnel axis; a) 2 m ahead of the face, b) in the unsupported span, c) 2 m backwards in the supported section (one excavation round only).

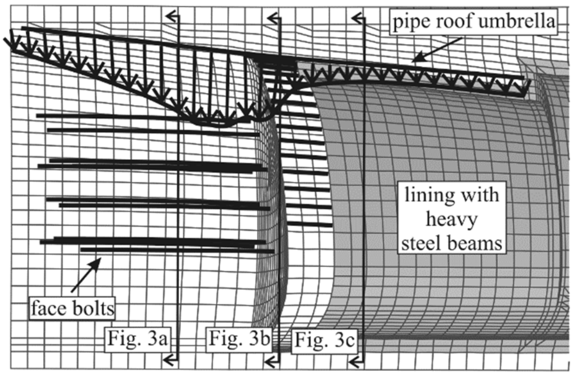

Figure 4. This longitudinal section displays the vertical and longitudinal displacements in the crown (one excavation round only).

6 GEOTECHNICAL BEHAVIOR

The internal forces of the pipes are almost zero after the installation, comparable other passive support measures (rock bolts). The stress transfer, due to the previous excavation steps, has an influence on the ground and on the already installed support. The newly installed pipes are not affected by previous activities, whereas every construction process after the installation that causes a stress transfer starts to activate the supporting effect of the pipes. The dis-placements mainly develop by the excavation process during tunneling so the three-dimensional dis-placement characteristic rules control the activation of the supporting effect after the installation of the pipe roof.

Each pipe is founded in the ground (ahead of the face) as well as on the lining (behind the face) in the longitudinal- and radial direction. The strength and stiffness of the pipe roof therefore depend on both the ground properties as well as on the time dependent strength and stiffness properties of the lining (shotcrete, steel beams).

6.1 Radial supporting effect

Figure 5 displays the calculated result of the ground pipe interaction after every 2nd excavation step. The pipes bend primarily at two positions with ongoing excavation steps under the pipe roof as long as the pipe foundation effectiveness does not decrease ahead of the face. One position in this example is 3.0 m ahead of the face. The second one is 1.0 m behind the face. These sections indicate the positions where the pipes operate as support and where the pipes transfer the loads to the ground and the lining. The supported section is between chainage 47.0 and 50 (for bottom line) while on both ends of this area the loads are transferred to the foundation regions. The position ahead of the face corresponds to the orientation change of the displacements in Figure 4. The loads are only transferred in the longitudinal direction and the pipe roof support does not create any arching normal to the tunnel axis. It is therefore necessary to model each pipe individually in the numerical simulation; because in numerical calculations both a stiffer homogenized region as well as the use of shell elements would additionally cause an arch effect normal to the tunnel axis. Both simplifications could lead to an underestimation of the displacements and/or to an overestimation of the stability conditions.

This pipe roof supporting effect is affected by the bending of the pipes, so the second moment of the area defines the activation speed of the supporting effect by bending. Pipes with a larger diameter activate the supporting effect faster at similar displacements compared to smaller diameters. The pipe wall thickness, on the other hand, defines the critical moment alternatively the maximum bending.

This example displays that a pipe roof system supports the face region, the unsupported span, and sometimes a short zone at the beginning of the lining (fast excavation). The supporting effect decreases in these critical sections the risk against possible local or global failures associated with the tunnel face.

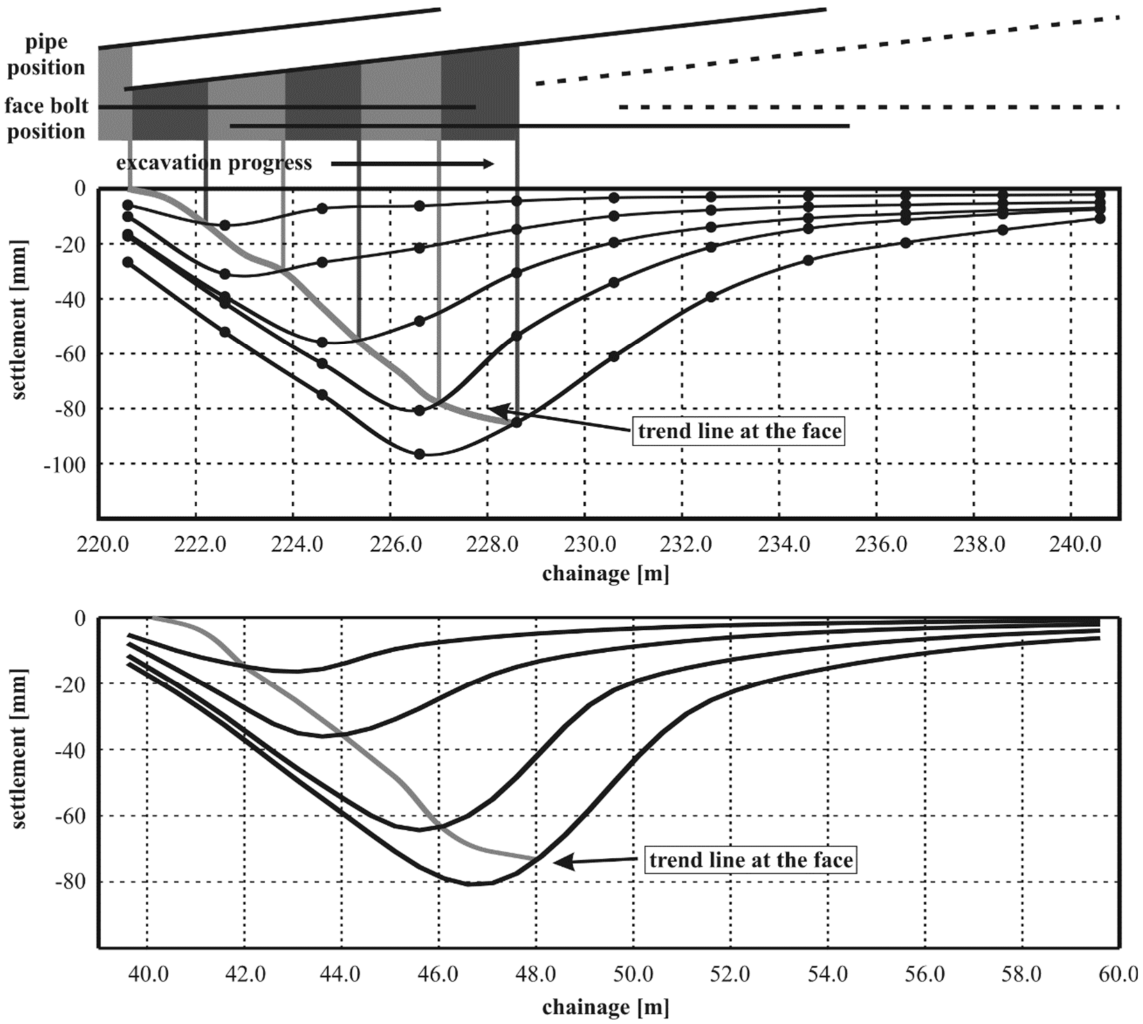

Figure 5. This deflection curve diagrams display measured settlement values from the Trojane tunnel (upper diagram) and results of a three-dimensional numerical calculation (lower diagram) after every 2nd excavation round.

6.2 Longitudinal supporting effect

The stiffness of a steel pipe is much higher compared to the stiffness of ground that needs additional sup-port by a pipe roof system during tunneling. The relative movements in the longitudinal direction there-fore create a second supporting effect of pipe roof systems during excavation. The pipes are subjected to longitudinal compression. This influences the stress distribution ahead of the face positively so the displacements in the ground decrease ahead of the face.

This effect is affected by the area that can be used for the transfer of interaction forces so the outer pipe diameter defines the effectiveness of this supporting effect.

6.3 Pipe gap in the unsupported span

Both previously mentioned support effects are al-so influenced by the number of pipes. This design parameter and the diameter of the pipes define the remaining gap in between the pipe roof pipes. A local arching effect in between the pipes (Stöckl 2002) increases the stability in the unsupported span and decreases the overbreak volume as long as this local arch can be formed.

A comparison of calculated results can be found in Volkmann & Schubert 2006. Those results dis-play both the effect of larger pipe diameters and different numbers of pipes on the displacement characteristic.

6.4 Validation of numerical results

Complex three-dimensional numerical calculations are influenced by all input parameters. These include geometric conditions, material properties, installed support systems, and time dependent changes of strength- and stiffness properties.

The comparison of the measured and calculated displacements (Figure 5) for this case history dis-plays good agreement ahead of the face, in the un-supported and supported sections. This fact pro-vides evidence that the relevant mechanisms, which occurred during the excavation of the Trojane tunnel, could be reproduced in the numerical calculations.

7 CONCLUSION

The advantages of being cheap and less time consuming during the installation, increased the use of Pipe Roof Support systems over the last decades even though neither formulas nor clear rules for the design of this pre-support system were determined. An intensive monitoring campaign, including settlement measurements ahead of the face at the tunnel level, enabled the determination of the geotechnical model for this support system. Using this knowledge design parameters of the Pipe Roof System were investigated in numerical calculations.

The results of the numerical investigations demonstrate that the pipe roof system supports the critical section around the heading by transferring the radial loads to both the ground in the front as well as the lining in the supported section. This sup-port effect increases the safety in the working area against local or global failures.

Compared to the stiffness of the ground, the high stiffness of the steel pipes influences the stress distribution positively ahead of the supported section. This effect and the radial support around the heading decrease the settlement amounts.

Numerical calculations enable the adaptation of the pipe roof support system by a knowledge-based change of the design parameters: overlapping length, pipe dimensions, and number of pipes. So, the calculated system behavior can be optimized with respect to the project requirements.

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Appendix E

Title: Back-Calculated Interacting Loads on Pipes of Pipe Umbrella Support Systems

Author(s): G.M. Volkmann, Florian KRENN

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Summary: The tunnel Vomp-Terfens was additionally supported by a pipe umbrella in the investigated section. This section was also equipped with an inclinometer measurement system, so bending lines of the pre-support system are known. A back-calculation is presented resulting in internal loads as well as interacting loads between ground and pipe. Exemplary results are shown, and the additional results gained by this back-calculation are pointed out.

BACK-CALCULATED INTERACTING LOADS ON PIPES OF PIPE UMBRELLA SUPPORT SYSTEMS

Günther M. VOLKMANN

ALWAG Tunnelausbau Ges.m.b.H., Wagram 49, 4061 Pasching/Linz, AUSTRIA

Florian KRENN

GEOCONSULT India Pvt.Ltd., Plot 473, Ind.Est., Udyog Vihar, Ph.V, 122016 Gurgaon, INDIA

Keywords: pipe umbrella support, canopy tube support, soft ground tunneling

INTRODUCTION

Geomechanical design according to the Austrian Guideline (ÖGG, 2001) distinguishes between rock mass behavior and system behavior. Whereas the rock mass behavior (or ground behavior) describes the behavior of a fictitious full-face excavation without support measures and consequently failure modes of an unsupported rock mass, the system behavior considers the interaction of ground and installed support measures.

The geotechnical monitoring performed during tunneling observes the system behavior; especially in soft ground tunneling since it is never intended to reach the limit state of ground failure during any construction phase. Therefore, all analyses are performed based on a monitored system behavior, meaning the contribution of installed support measures to the system behavior has to be assumed with simplified structural models.

This paper involves conclusions drawn from on-site monitoring results. This includes the evaluation of geodetic surveys on the surface and horizontal in-place inclinometer chains installed in a pipe umbrella support. The objective of this paper is to analyze the mode of loading and its effect on the overall system behavior for pipe umbrella supports during tunneling.

PROJECT DESCRIPTION

The Brenner corridor is part of the new North-South European rail link for high-speed trains and combined transport services connecting Munich in Germany and Verona in Italy. A part of this route follows the Lower Inn Valley in Austria, which is narrow and densely populated with towns lined up along the Inn River. For this reason, 83 percent of the 41 km long Lower Inn Valley route will run underground. The new route includes the tunnel Vomp – Terfens, which is about 8 km long and is constructed through different types of ground; competent rock and soft ground (quaternary sediments such as ground moraine and gravel).

The applied support measures varied from 150 mm steel fiber – reinforced shotcrete in the hard rock sections (dolomite) to 300 mm reinforced shotcrete lining with a pre-support system consisting of face bolts and double layered pipe umbrellas in the sections with low overburden and layers of potentially unstable gravel.

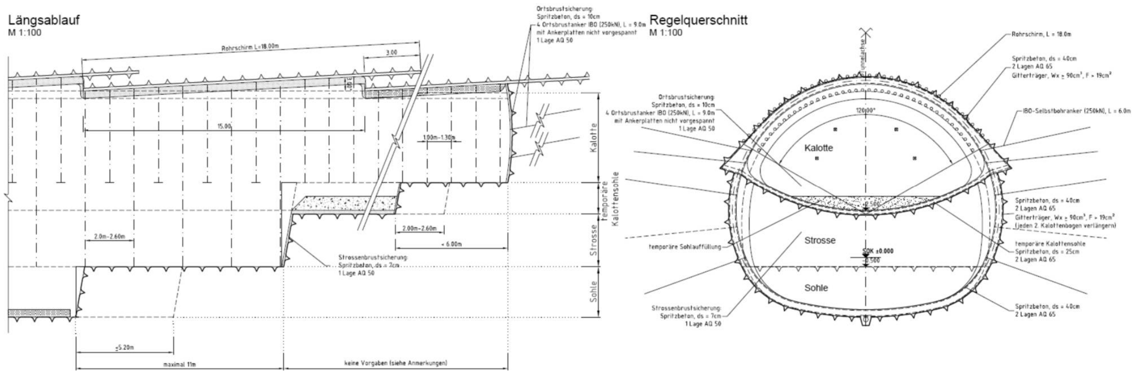

Figure 1. shows the installed support system

Excavation Concept and Support System

The support system adopted was defined on site in so-called "excavation & support sheets", which are based on support classes defined in the final design drawings by the designer. The sequential excavation method was applied for the excavation in the analyzed stretch. The basic support combines a primary shotcrete lining with two layers of wire mesh and lattice girders as reinforcement. Self-drilling radial bolts are additionally installed at the tunnel shoulder to support the surrounding ground. The excavation is done in three stages – namely top heading with top heading invert, bench and invert. The stability of the face is ensured by sector excavation of the face and face bolting with self-drilling bolts. As advance support, a pipe umbrella system was installed (average pipe length 18 m with 3 m overlap).

The features described above reflect the typical requirements for tunneling in soft ground; short ring closure, face stabilization, immediate shotcrete sealing on exposed surfaces and additional support at the perimeter of the working area to avoid local instabilities.

It must be stated that the impact of a stiff advance support such as a pipe umbrella is strongly dependent on the load transfer to its abutments. These abutments are the ground ahead of the face and the primary lining. The face bolting and the temporary top heading invert have to be given special attention during design and construction since these measures provide additional strength and stiffness to both sections where the pipe roof transfers its loads into (Volkmann & Schubert, 2007).

GEOTECHNICAL MONITORING

One key element of the “New Austrian Tunneling Method” is the geotechnical monitoring of the system behavior. The interpretation of the monitoring data provides a feedback about the ground - support interaction during tunneling. The performance is fine as long as the observed system behavior complies with the expected one. If deviations are identified, the underlying mechanisms have to be analyzed and evaluated in order to allow an adequate response.

Geodetic Survey

A common system for collecting the monitoring data is a geodetic survey. The system behavior of shallow tunnels is usually controlled by both, surface displacements above tunnel axis and displacements of surveying targets in the already supported tunnel section. The displacement data is normally collected daily during construction times.

The interpretation of this survey data is done by using different kinds of display formats. Typically used plots are; displacement versus time plots, deflection curve diagrams (displacement versus chainage of tunnel drive), trend lines and cross-sectional diagrams (jellyfish diagrams). Krenn discusses some findings and interpretations for this type of data taken during construction of tunnel Vomp – Terfens (Krenn et al. 2008).

Inclinometer Measurement System and Inclinometer Measurements

A supplementary survey system was specially developed for controlling the system behavior at tunnels supported by a pipe umbrella. This measurement system consists of 10 two-meter long inclinometer links connected to each other resulting in a continuous inclinometer chain. The inclinometers are installed into a roof pipe of the pipe umbrella or in an extra pipe drilled parallel to the others but a little higher. Thus, the equipment is protected from construction activities. The positioning of the measurement equipment enables the measurement of changes in pipe inclination between the single inclinometer links. The pipe bending line is thereby monitored at the tunnel crown level, up to 20 m ahead of the working face. One geodetically surveyed target at the starting point of the inclinometer chain fixes the pipe position in the vertical direction so that settlements can be calculated by using the inclinations. During construction, the inclinometers stay in-place and measurements are stored automatically in pre-defined time intervals to a data acquisition system positioned either at a tunnel sidewall position or in an office. This way the construction can proceed without being interrupted by taking measurements.

Earlier measurement campaigns showed that by the time the pipe umbrella support and the inclinometer chain is installed, the construction induced stress redistribution is already finished. For this reason, the pipes stay without loading until the following excavation and supporting processes induce deformations. Starting at the installation time each following deformation of the surrounding ground induces an interaction with the pipes as well as other support elements. Due to this interaction, the pipes are bended and loaded respectively. This leads to the finding that the loads acting on a pipe umbrella pipe develop due to consecutive loading processes. These loading processes cannot be compared to any assumed dead load body. A dead load body can only be assumed when a pipe umbrella system must stabilize the tunnel after failures at the face or the periphery of the tunnel have occurred, as the analytical back-calculations of Möhrke (1999) show. This leads to a very conservative design for normal construction processes.

The result of the data analysis presents the development of settlements due to the ground – support interaction in the inclinometer section. The continuous analysis of this data helps in understanding the complex stress redistribution processes around the heading. Furthermore, it can be used for a continuous adaptation of the support elements to optimize the construction process with respect to the ground and the project requirements. Volkmann & Schubert (2005) describes more details about this topic.

Figure 2 presents an exemplary settlement development for the last excavation steps before the next pipe umbrella support was installed at the inclinometer section in the tunnel Vomp – Terfens. It can be seen that the abutments (sections with more curvature) of the pipe umbrella pipes move concurrent with the advance of the tunnel. The pipes support the section in between these two points in the radial direction (Volkmann & Schubert, 2008).

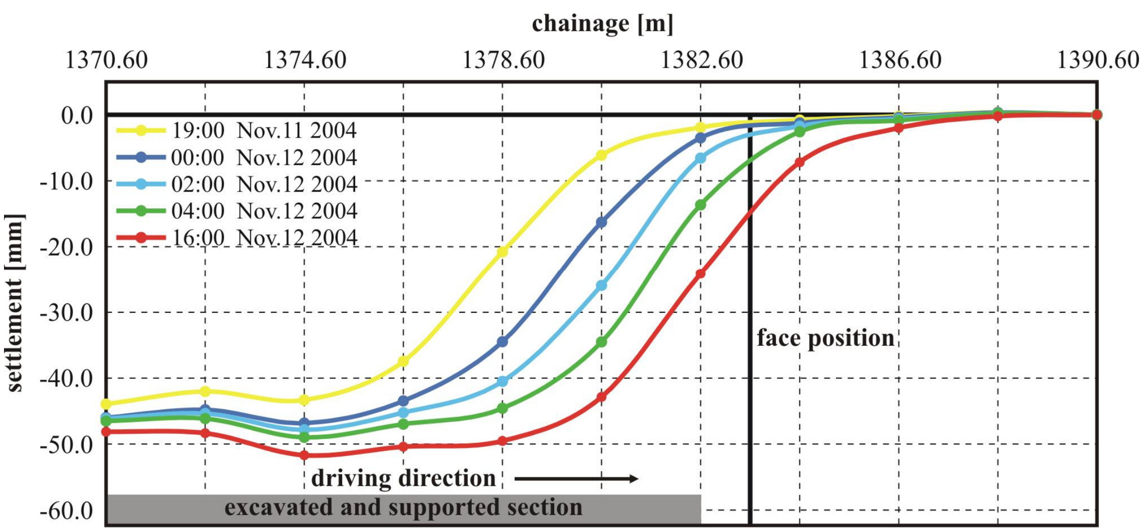

Figure 2. This deflection curve diagram shows the settlement development in the crown due to proceeding excavation. It includes the pre-settlements that happened before the inclinometer chain was installed.

METHOD OF BACK-CALCULATION

The data, shown in the last chapter, allows observing the deformation of the installed roof pipe at pre-defined times during construction. The stress redistribution due to the excavation process is the main cause of the ground deformations, however, the measured deformations may also be effected by the installation of other support elements (i.e. bolts, piles) at locations with weak ground conditions (Volkmann & Schubert, 2005). As a result, the deformations apply a load on the pipe. This continuously changing load defines the bending line of a pipe, which can be back-calculated by using common structural engineering equations. To perform this calculation, the following assumptions must be made:

- The measured bending line of a pipe is continuous between the measured points.
- The structural properties of a pipe are known and do not change longitudinally.
- The pipe bending is due to the load applied by ground movements and Equation 1 can describe this interaction.
- The pipe only bends in the measured direction and not perpendicular to this direction.

The following equations were used for the back-calculation of the internal forces.

$$q = A * y^3 + B * y^2 + C * y + D \qquad \text{Equation 1}$$

$$Q = -\int q * dy + C_1 \qquad \text{Equation 2}$$

$$M = -\iint q * dy\, dy + C_2 \qquad \text{Equation 3}$$

$$\kappa = \frac{1}{E * I} * M \qquad \text{Equation 4}$$

$$\varphi = \int \kappa * dy + C_3 \qquad \text{Equation 5}$$

$$w = -\int \varphi * dy + C_4 \qquad \text{Equation 6}$$

q	interacting load [kN]
y	position in longitudinal direction [m]
Q	shear load [kN]
A, B, C, D	variables defining the interacting load [-]
M	bending moment [kNm]
C1, C2, C3, C4	constants defined by boundary conditions [-]
κ	curvature [-]
φ	inclination [rad]
E	modulus of elasticity for steel [MPa, MN/m^2]
w	bending line [m]
I	geometrical moment of inertia [m4]

The boundary conditions differ at the ends of the pipe; the end on the tunnel side is already clamped by the primary lining while the opposite (deepest) end is embedded in weak ground. For this reason, the bending moment (M) as well as the shear load (Q) is assumed to be zero at the deepest end. This assumption still leaves the pipe inclination (φ), the interacting load (q) and the inclination that describes the change of the interacting load at this position (q') as variables. The pipe inclination has a significant effect on the results, but the effect of the other two variables is small so at the beginning of the back-calculation these two values can be assumed to be zero as well.

Using these assumptions, the system of equations can be solved by iterations. Due to the expected degree of freedom for the interacting force, the system of equations was solved stepwise for 4-meter segments (3 measurement points) with an overlap of 2 meters.

This way of calculating the internal forces does not produce a unique result; in fact, there are endless results, so a rule must be established to calculate a result which is close to an authentic one. The minimum energy necessary for bending the pipe was used as decision criteria to create a smooth bending line.

$$E = \sum Q * \varphi + M * \kappa \qquad \text{Equation 7}$$

Following these rules resulted in the ground – pipe interaction as well as the internal forces of the pipes with a smooth bending line defined by the minimum energy needed (Figure 3).

EXEMPLARY RESULT OF BACK-CALCULATION

The following example is calculated using the bending line measured when the pipe umbrella field was excavated just before the next pipe umbrella was prepared for installation. Figure 3 shows the 11 measured points as well as the calculated bending line. It can be seen that both data sets fit perfectly to each other and the calculated line has a logical shape between the measured points.

The bending moment (Figure 4) has two maxima; one is about 0.6 m ahead of the face position (5.55 kNm) while the second one is in the already supported tunnel section about 3.2 m behind the face position (5.45 kNm). This result is similar to the results presented in Krenn et al. (2008). The induced deformations load the pipe up to 13 % of its yielding moment under elastic conditions (~42.5 kNm). It is important to note that for design purposes the load of rupture for thread connections must be considered. Specific information on this topic is discussed in Volkmann & Schubert (2008).

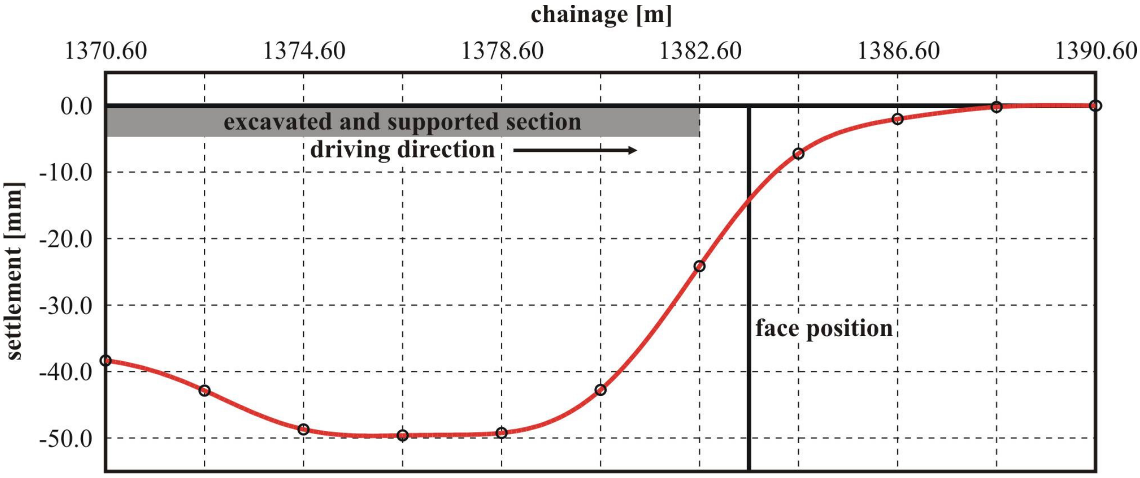

Figure 3 shows the back-calculated bending line and the in-situ deformations at the measured points

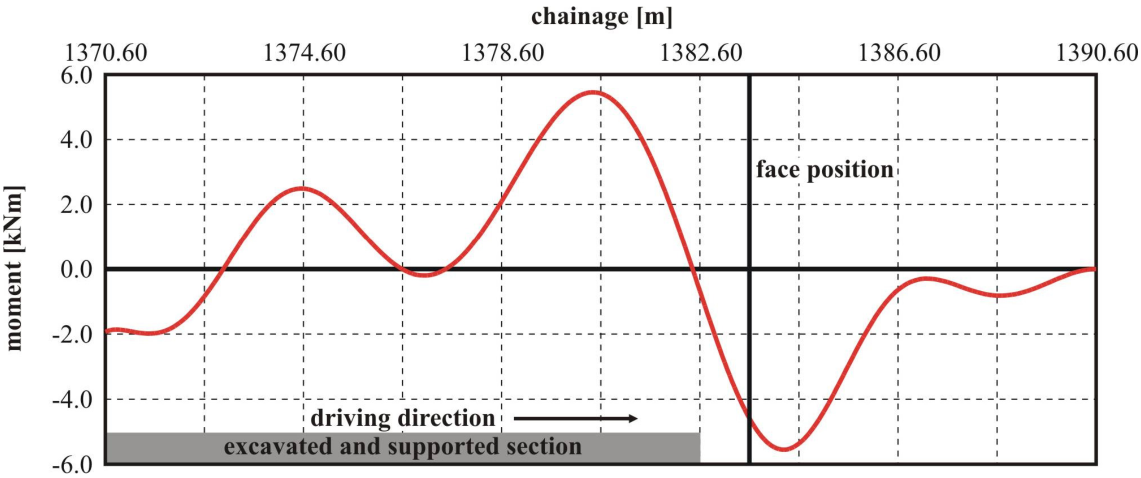

Figure 4 shows the bending moment of the back-calculation.

Figure 5 presents the development of interacting forces along the pipe. The main peak is 0.5 m ahead of the face and has a value of 4.1 kN orientated upwards. At this section, the pipes support the working area, which includes, in the calculated case, unsupported span as well as a nearly 2.0 m long section ahead of the working face. The two adjacent peaks in the diagram mark areas where loads are transferred to the ground (2.5 kN) and the primary lining (3.0 kN). The oscillating characteristic of the interacting force indicates that the pipes are only embedded in its abutments as the refined structural model in Krenn et al. (2008) already indicates by the springs, which transfer the loads to its abutments. For a further refinement of an analytical approach, strength and stiffness parameters of the abutments must also be included in this way of back calculating the interacting forces. This would lead to an analytical formulation that can calculate stability conditions as well as deformation values when using a pipe umbrella as advance support.

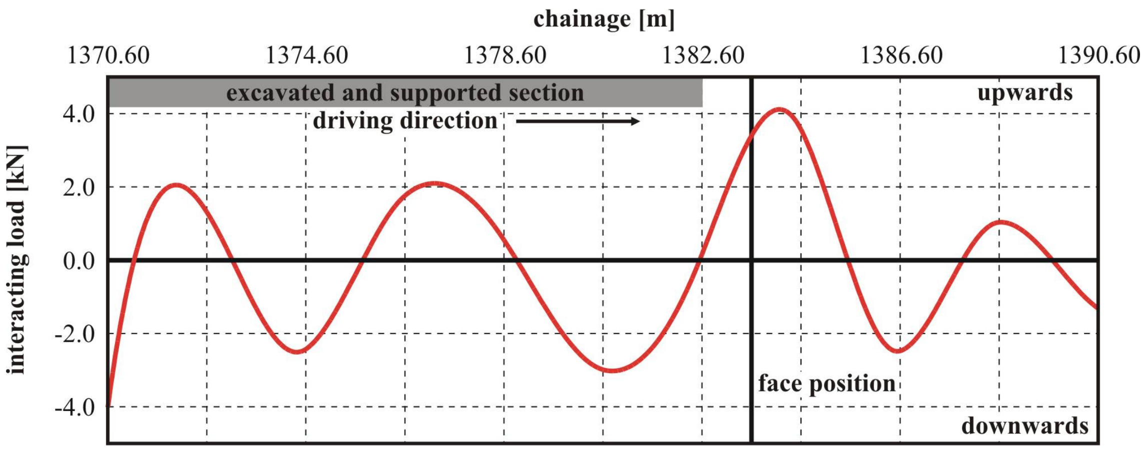

Figure 5 shows the interacting loads between ground, support, and pipe.

SPIRIT AND PURPOSE OF IN-PLACE INCLINOMETER MEASUREMENT

This measurement equipment has control and exploratory functions during construction. The basic evaluation of deformations allows controlling the system behavior during excavation and supporting processes in a very detailed way. Possible changes can be detected, and causes identified. This allows adapting the support system accordingly at an earlier stage than with currently used observational systems. This is because deformations can be measured up to 20 m ahead of the face at the tunnel crown level and the data can be stored in pre-defined time intervals, sometimes as short as minutes (Volkmann & Schubert, 2005).

The additional information, given by back calculating internal forces and interaction load between ground, support and pipe umbrella system, is:

- Knowing the bending moment allows controlling the actual utilization of the pipe umbrella support system. The possibility to adapt the system accordingly leads to a higher degree of safety during construction.

- The interaction load presents the actual load transfer areas, which is important to understand the ongoing stress transfer processes whenever problems occur.

- A comparison to results of the design stage validates the chosen support system or may identify discrepancies. Both cases are a control mechanism even though when finding discrepancies adaptations of support elements may be needed.

- The additional load that is transferred from pipes to primary lining is known and can be included in stability evaluations of primary lining or its foundation.

- The additional load transferred to the ground ahead of the working face has an influence on the stability of the face. By considering the additional load at this area for definitions of face support, unexpected instabilities may be avoided.

- The length of the pipe foundation ahead of the face is the major factor for defining the overlapping length for pipe umbrella support systems. An overlap length that is chosen too short may lead to an increase in deformations or even face instabilities unless extra face support blocks the mechanisms.

CONCLUSION

Tunnels in environmentally sensitive areas combined with weak ground conditions require special attention during design and construction. Thus, 3-dimensional geodetic surveys in the tunnel as well as on the surface are used to monitor the system behavior during construction. Additional measurement techniques can complement the monitoring program. The complete data helps in understanding the complex stress redistribution processes and allows identifying critical changes in the system behavior.

Furthermore, special evaluation techniques can answer specific design questions. The presented back calculation of interacting forces between ground and pipe umbrella pipes determines the position of its abutments, the supported section, and the pipe's degree of utilization. This knowledge enables the designer to review his assumptions and may lead to standard load assumptions for complex structural system like pipe umbrellas.

ACKNOWLEDGEMENTS

The fact that the Contractor “ARGE Tunnel Vomp – Terfens” installed the horizontal inclinometer and provided the data (together with the surveying joint-venture “ARGE GEODATA-AVD”) is greatly acknowledged.

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Appendix F

Title: A load and load transfer model for pipe umbrella support.

Author(s): G. M. Volkmann, W. Schubert

Published: EUROCK 2010, Rock Mechanics in Civil and Environmental Engineering – Zhao, Labiouse, Dudt & Mathier (eds) © 2010 Taylor & Francis Group, London, ISBN 978-0-415-58654-2

Summary: This publication describes the geotechnical behavior as well as the back-calculation of internal forces and interacting forces between ground and pipe. The main part of this publication presents an analytical load and load transfer model for pipe umbrella pipes.

A load and load transfer model for pipe umbrella support

G. M. Volkmann

DYWIDAG-Systems International GmbH, Pasching/Linz, Austria

W. Schubert

Institute for Rock Mechanics and Tunneling, Graz University of Technology, Graz, Austria

ABSTRACT

Pipe umbrella support systems are used in weak ground conditions. The main reason for using this system is the supporting effect in the working area, which leads to safe construction conditions. Due to the complex stress transfer processes around the heading there is still a lack of knowledge regarding this system. Based on high quality measurements in the past years back-calculations of in-situ data were performed. Exemplary results of these back-calculations will be shown, and typical characteristics described. By using this knowledge, calculations during design can be performed that define the structural properties for the supporting pipes and changes in the load redistribution.

1 INTRODUCTION

The increasing population in metropolitan areas re-quires continuous extension of the infrastructure. The majority of the infrastructure in congested urban areas is located subterranean. In these areas, the ground generally consists of sedimentary soil and/or highly weathered rock mass. Both types of ground can be associated with stability problems that are overcome with additional support during construction.

Face bolts are the most effective support to stabilize the face, while the perimeter of the open span can be supported by different system. One of those is the pipe umbrella system that is also referred to as “Umbrella Arch Method”, “Pipe Fore-Pole Umbrella”, “Long-Span Steel Pipe Fore-Piling” and “Steel Pipe Canopy” in literature.

In order to obtain a better understanding of this support system in situ measurements, using inclinometer chains installed parallel to the pipe umbrella support, were performed. The measurements of the inclinometer chain were linked to the geodetic dis-placement measurements taken inside the tunnel and on the surface. These measurements display the longitudinal distribution as well as the magnitudes of settlements in the crown level of the tunnel.

The evaluation of these data sets helps not only in controlling construction processes, but also in under-standing the geotechnical mechanisms around the heading when using pipe umbrella systems. Many details are displayed that can be used for further analytical and numerical investigations.

2 GEOTECHNICAL BEHAVIOR

The internal forces of the pipes are almost zero after the installation, comparable to other passive support measures like rock bolts. The stress transfer, due to the previous excavation steps, has an influence on the ground and on the already installed support. The newly installed pipes are not affected by previous activities, whereas every following construction process that causes a stress transfer starts to activate the supporting effect of the pipes. The spatial displacement characteristic developing during excavation controls the activation of the supporting effect after the installation of the pipe umbrella.

Each pipe is founded in the ground (ahead of the face) as well as on the lining (behind the face) in the longitudinal and radial direction. The strength and stiffness of the pipe umbrella system therefore de-pend on both the ground properties as well as on the time dependent strength and stiffness properties of the lining (shotcrete, steel beams).

When using a pipe umbrella system, the mode of action can be divided in 3 main parts: the local arching effect in the open span, the radial support effect, and the longitudinal load transfer (Volkmann & Schubert 2007).

Immediately after the excavation, the tunnel perimeter is subdivided in smaller open areas because of the prior installed pipes. The shortest length for an arching is no longer the length of the excavation step; it is the distance between the pipes now. Due to that fact the local arching effect can be created much easier in critical ground conditions.

In ground conditions typical for pipe umbrella supported tunnel sections, the area where the pipes are loaded is the open span, and the adjacent area ahead of the face. The pipes are loaded by the de-formations (mainly radial) and the supporting effect works against these deformations so the ground in the open span and the adjacent area ahead is sup-ported in the radial direction. Due to the stiffness of the support system, these loads can be transferred from the most critical sections to its foundations.

Pipe umbrella systems are also used to decrease the deformations induced by the tunnel construction. This effect can be explained because the stiff fan of pipes reduces the relaxation around the heading in the longitudinal as well as radial direction. A verification of this support effect shows that the heading tends to fail (Figure 1).

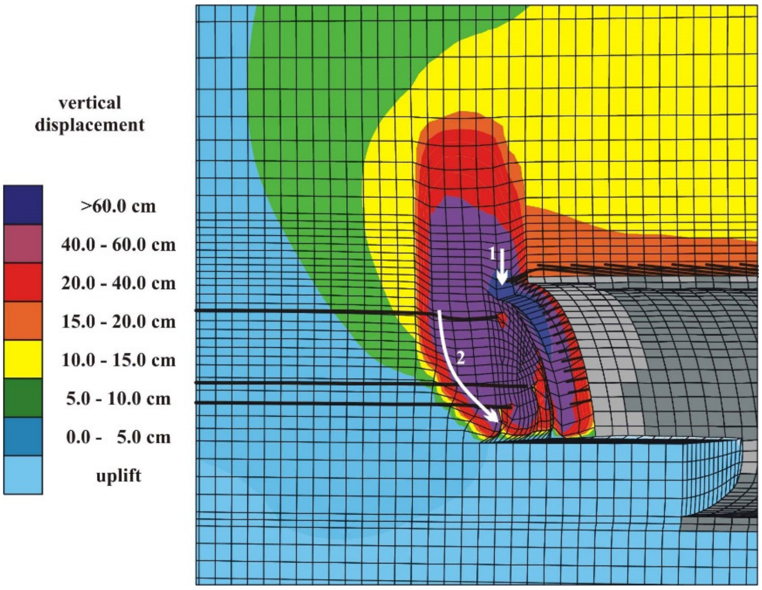

Figure 1. Failure mechanism without pre-support at the shape of the working area; it starts at the crown and develops to the sidewall; in a second step the face fails (Volkmann & Schubert 2009).

3 BACK-CALCULATION OF IN-SITU DATA

In the past years the excavation processes were monitored in different pipe umbrella supported sections with online in-place inclinometer chains. The instrumentation is normally installed parallel to the roof pipe and the monitored inclinations stored in a data acquisition system. Typically, the inclinometer links are 2m long and connected to each other. This con-figuration allows measuring settlements of single points exactly every two meters in pre-defined time intervals (usually 1-minute interval). During construction the continuous evaluation of these data give construction relevant information in critical tunneling sections (Volkmann & Schubert, 2005)

Furthermore, the high quality of the data enables to calculate pipe bending lines by using the equations 1 to 6. During this process equation 7 is used to find the result with minimum energy resulting in a smooth bending line. This additional evaluation tool analyzes the load transfer processes around the heading in de-tail and enables a higher quality of control and optimization during construction (Volkmann & Krenn, 2009).

$$q = A * y^3 + B * y^2 + C * y + D \qquad (1)$$

$$Q = -\int q * dy + C_1 \qquad (2)$$

$$M = -\iint q * dy\, dy + C_2 \qquad (3)$$

$$\kappa = \frac{1}{E * I} * M \qquad (4)$$

$$\varphi = \int \kappa * dy + C_3 \qquad (5)$$

$$w = -\int \varphi * dy + C_4 \qquad (6)$$

q	interacting load [kN]
y	position in longitudinal direction [m]
A, B, C, D	variables defining the interacting load [-]
Q	shear load [kN]
M	bending moment [kNm]
C1, C2, C3, C4	constants [-]
κ	curvature [-]
φ	inclination [rad]
E	modulus of elasticity for steel [MPa, MN/m2]
w	bending line [m]
I	geometrical moment of inertia [m4]

$$E = \sum Q * \varphi + M * \kappa \qquad (7)$$

One of the most important results of the back calculation is the development of interacting forces. It clearly indicates the position of supported sections and loaded sections (abutments of the pipe) (Figure 2).

The central part of the abutment ahead of the face can often be observed at an angle of 45+φ/2 from the bottom of the front excavation (Figure 3). Typically, this abutment is 2m to 3m long and more pronounced in weaker ground when deformations ahead of the face are large. The supported section starts at the nearest point of the abutment ahead of the face and ends approximately at the front end of the installed support. The material under the pipe umbrella moves in this section (face region) more than the material above the pipe umbrella so it can no longer be used as abutment. This interpretation agrees with results from 3-D numerical calculations discussed in Volkmann & Schubert (2006).

The second abutment in the already supported section starts very close to the front end of the primary lining. The distance is rarely longer than 0.5m and the length of this abutment is mostly about 2m but can also reach 4m. Stiffer ground conditions result in a more pronounced load transfer to this abutment.

The data shown in Figure 2 were recorded at shallow depth (~1.2D cover) but the magnitude of sup-porting forces does not indicate any gravity con-trolled failure. The back-calculated magnitude rather refers to a slice of ground that is squeezed out by the longitudinal stress component over the entire length of the supported section. The back-calculation of the observed data indicates that the outer shape of this body can be described by an angle of 45-φ/2.

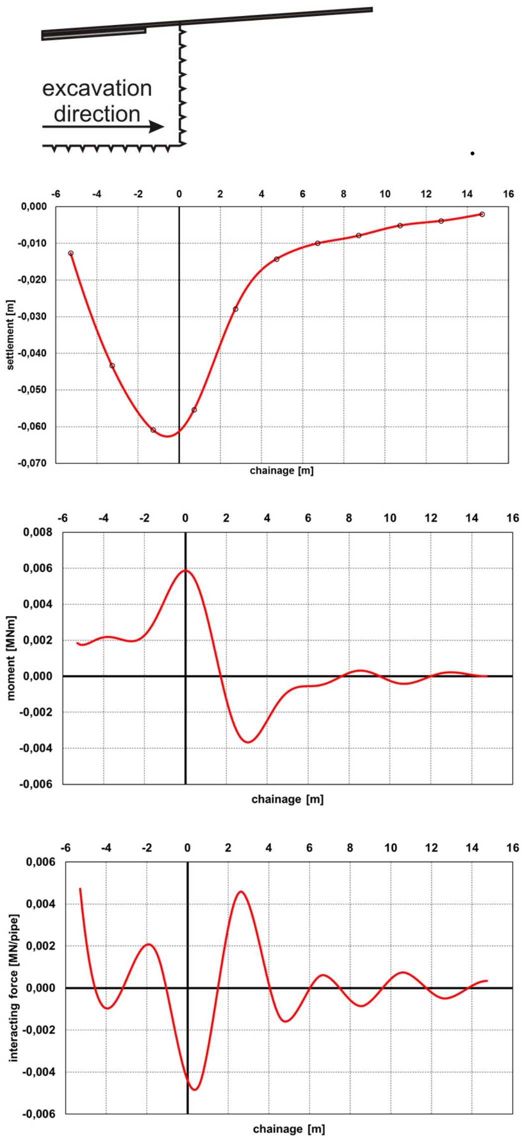

Figure 2. Example for a back-calculated in-situ measurement data set. The upper diagram displays the measured settlement points and the calculated bending line, the middle one the back-calculated moments in the pipe and the lower one dis-plays the back-calculated interacting forces between ground and pipe.

4 LOAD AND LOAD TRANSFER MODEL

The observations described in the last chapter can of course also be used to determine an expected load and load transfer to the abutments in a design stage.

One possible way is shown in Figure 3. The definition of the load acting on each pipe is one of the key features. It is dependent on ground properties, the axial distance between single pipes and the sup-porting length. In this case the shape of the load is described by the back-calculated equivalent dead load body. The supporting length is composed of two parts; L1 and L2. L1 is the excavation step length plus a working area that is necessary during construction (usually 0.3-0.5m). Figure 3 shows the most conservative version for L2. This length could be reduced if required by the chosen way of construction sequence or additional face support.

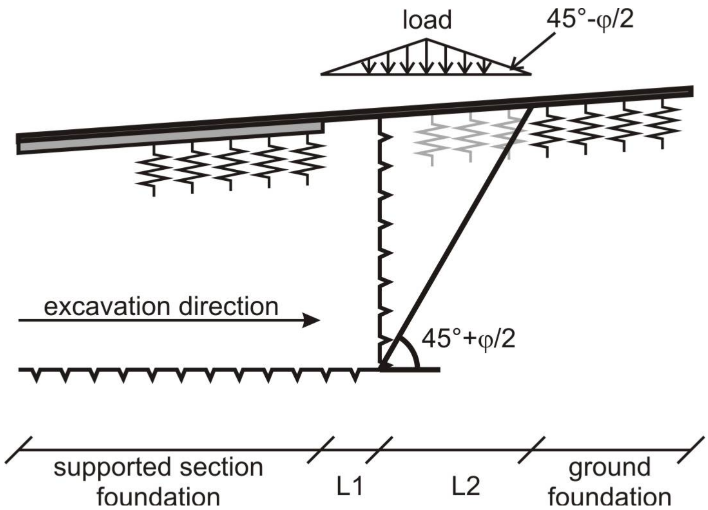

Figure 3. Simplified static model for analytical calculations including the load and the abutment characteristics.

Ahead of the supported section each pipe loads the ground similar to a strip foundation so the de-formations and interacting forces respectively can be calculated by formulations commonly used in soil mechanics. The stiffness of the abutment in the al-ready supported section can be calculated as pro-posed by Oreste & Peila (1998) for example. In a first step, uniformly distributed loads at both abutments may give good results as well.

After choosing structural properties for the pipe and an adequate connection type (Volkmann & Schubert, 2008) a bending line can be calculated, and the input parameters varied until stable conditions are achieved. The magnitude and position of the interacting forces influences other design parameters as well.

The optimum overlapping length regarding settlements can be defined when the abutment length ahead of the face is known. Whenever the actual length falls below this value during construction the effectiveness of the pipe umbrella system decreases, and the loads cannot be fully transferred to less loaded section. This leads to an increase in settlements at the end of a pipe umbrella field as documented in Volkmann & Schubert (2003).

The transferred loads in general should also be considered for the strength properties of the primary lining and the design of the face support.

5 CONCLUSIONS

Unfavorable conditions must be managed in weak ground tunneling. At these conditions instabilities may occur at the face as well as at the perimeter of the open span. Face bolts are mostly used to mitigate the first problem while forepoling methods are able to support the open span.

Due to the complex stress transfer process around the heading it is a challenge to define the load acting on these forepoling elements. This publication presents a load model based on back calculations from in-situ measurements and a method to calculate the load distribution in forepoling elements.

The introduced method decouples the forepoling elements from the tunnel construction and just calculates a loaded steel element with springs as foundations. The resulting deformation mainly depends on the stiffness and strength characteristics of the fore-poling element and the stiffness characteristics of its foundations. These are the ground ahead of the face and the already installed support behind the face.

By using this method, the following results can be achieved:

- The utilization of the forepoling elements, which helps in the decision-making process for the structural properties of the forepoling elements and its connections.
- The foundation length ahead of the face, one of the main parameters to define the design parameter overlapping length in the longitudinal direction.
- The additional load that is transferred to the ground ahead of the face. This result may lead to adaptations of the face support.
- The resulting additional load that acts on the first meters of the primary lining. This is necessary in-formation to design the primary lining and to analyze its foundations.

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Appendix G

Title: Effects of Pipe Umbrella Systems on the Stability of the Working Area in Weak Ground Tunneling

Author(s): G. M. Volkmann, W. Schubert

Published: Proceedings of SINOROCK2009, Rock Characterisation, Modelling and Engineering Design Methods, 19th to 22nd May 2009, Hong Kong, China

Summary: This publication deals with results of numerical simulations in FLAC-3D. It focuses on the failure mechanisms without pre-support measures as well as the support effects of spiles and pipe umbrella systems in homogeneous ground conditions. Both systems result in stable construction conditions but when little unexpected changes in the ground conditions appear, this result changes and pipe umbrella systems show clear advantages compared to spiles.

Effects of Pipe Umbrella Systems on the Stability of the Working Area in Weak Ground Tunneling

G. M. Volkmann

ALWAG Tunnelausbau Gesellschaft m.b.H., Pasching/Linz, AUSTRIA

W. Schubert

Graz University of Technology, Institute for Rock Mechanics and Tunneling Graz, AUSTRIA

ABSTRACT

Weak ground conditions are common in urban areas. In such ground conditions certain potential for instabilities in the working area exists. Therefore, additional support elements supplement the standard support system to achieve stable conditions during construction. The most known additional pre-support system is the face bolt, ensuring face stability. A second group comprises different pre-support techniques, which primarily support the working area. This group of pre-support methods comprises the spiling method (forepoling) and the pipe umbrella method (long-span forepoling). By using 3-dimensional numerical calculations, these two methods are compared regarding their effectiveness as support element in weak ground tunneling. Failure can be controlled with both methods. The pipe umbrella method has advantages at frequently changing ground conditions or unexpected changes of the ground properties.

1 INTRODUCTION

The need of infrastructure increases the number of tunnels constructed in urban areas. Cities are often situated on flat areas with fluvial deposits prevailing. Weak or frequently changing ground conditions are common. Due to the housing on the surface, both stability requirements and settlement limitations rule the design of the support system.

Instability of the face and unsupported span are common problems in weak ground. Additional support is used to ensure stability. These support methods include the spiling method (forepoling), the pipe umbrella method (long-span forepoling), the pipe roof method, jet-grouted columns and freezing techniques. Besides the fact that these support techniques increase the stability conditions at the heading during construction, they also reduce the excavation-induced deformations depending on their stiffness and mode of load transfer to the surrounding ground. Therefore, it may happen, that pre-defined maximum settlement amounts and not the stability criteria govern the decision for the most effective support system.

Logically, additional support elements increase time and costs for construction. For this reason, these support elements must be chosen painstakingly. By unexpected changes of the ground's quality in urban areas, damages may appear. Repairing these damages do commonly cost a lot more than the tunnel construction itself. Furthermore, such unexpected events have a negative influence on the public opinion.

While damages induced by increased surface deformations are often uncritical, the extent of loss is usually more serious in case of tunnel collapses.

For this reason, the overall stability of sequentially excavated tunnels shall be investigated by using numerical calculations. The focus in this comparison is the influence on the stability of 2 forepoling methods; spiling and pipe umbrellas.

2 SPILE AND PIPE UMBRELLA PRE-SUPPORT

Proctor and White (1964) discussed in their book "Rock Tunneling with steel supports" the use of wooden "spiles" as forepoling method for traversing weak and raveling ground. Since that time, different forepoling methods have been developed; all characterized by the installation prior to the excavation with the goal of stabilizing the working area.

In today's tunneling, solid and hollow steel bars (or tubes) substituted wooden spiles (Figure 1) representing the simplest forepoling method. These bars do normally have an outer diameter up to 50 mm. After drilling or pushing them into the ground at the outer contour of the working face, the annular gap can be grouted to achieve a better load transfer. The typical bar length for this method, which is commonly known as forepoling or spiling, is between 4.0 m and 6.0 m,

While forepoling is installed after each excavation step, a pipe umbrella is installed to support several consecutive excavation steps (6 - 12). Other commonly used terms for this method are long-span forepoling or the canopy tube method. The pipes drilled as support elements typically show an outer diameter range from 70 mm to 200 mm; the pipe length is mostly 12.0 m or 15.0 m. The supporting pipes can be installed with both special machines and conventional drill jumbos. The following excavation starts after grouting the inner area of the pipes and the annular gap between ground and pipe. The cross-section increases until the far end of a pipe umbrella. This creates the necessary space for the next installation process. The result is the so-called sawtooth-shaped excavation profile, which is typical for pipe umbrella supported tunnel sections.

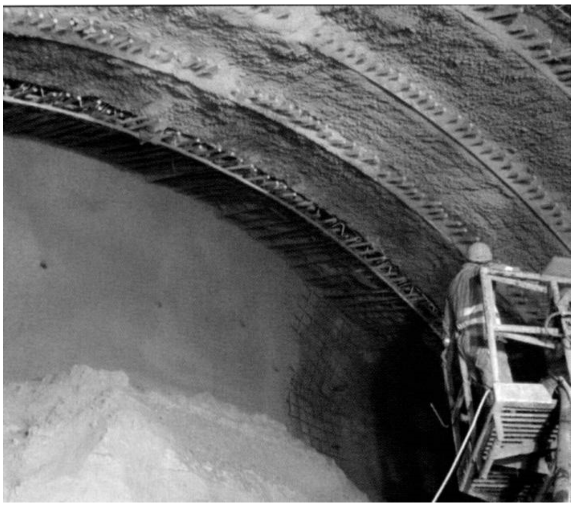

Figure 1. Spiles (steel bars) installed through lattice girders in every excavation step.

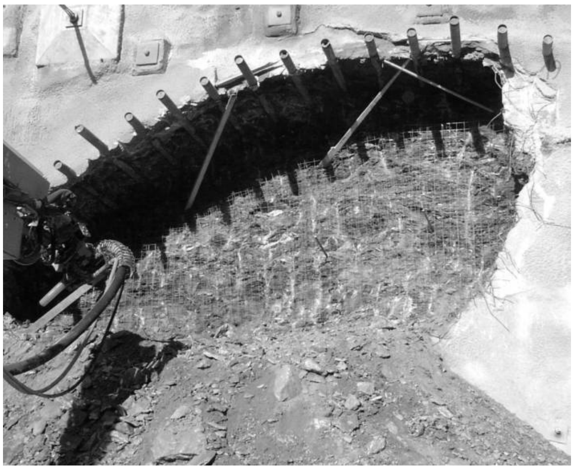

Figure 2. Pipe umbrella support at the Birgl tunnel portal (Austria)

3. BASICS - NUMERICAL MODEL

All calculations shown in the following are performed by using the program FLAC-3D. Different types of laboratory tests were used to determine the ground properties. The shotcrete strength and stiffness were changed due to its age after each step of the sequentially calculated tunneling process. Pile structural elements represent the steel bars and the pipes in the numerical model because observations on site did not show a closed grout arch around the perimeter of the tunnel (Figure 3).

Further details about the model, the ground properties and the validation on this numerical study can be found in Volkmann & Schubert (2006a) and Volkmann & Schubert (2006b).

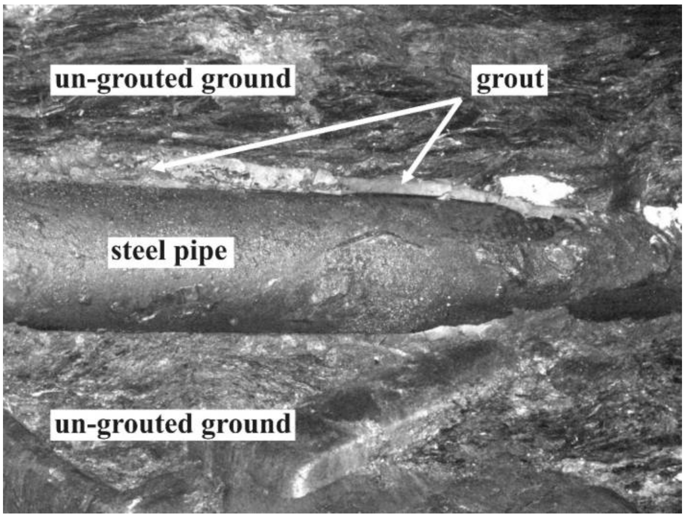

Figure 3. Detail of a pipe showing the grouted annular gap between ground and pipe and the un-grouted weak ground around the pipe.

4. EVALUATION OF STABILITY CONDITIONS

Before defining a proper support system, areas at risk for failures and its failure mechanisms must be detected. The determination of possible failures at the heading is done with a 3-dimensional model. Due to influences of the boundary conditions, the failure mechanisms were analyzed in the middle of the calculated model. Up to this excavation step, the tunnel is supported with lattice girders, shotcrete and three radial bolts in the sidewall as primary lining. Face bolts and a number of spiles ensured the stability of the heading.

To investigate the stability of the face, the recurrent installation of the pre-support measures was stopped in the sequentially calculated excavation process. Only a few excavation phases later - the top heading area was excavated in two phases – the material at the face collapsed by shearing into the already excavated tunnel. This showed the necessity of a support element ensuring the stability of the face area. Face bolts are the cheapest and most effective support element for the stability of the face. Therefore, the following calculations included recurrently installed face bolts. In the calculated cases, 3 pieces (due to symmetry only the right side is calculated) with a length of 15.0 m installed every 12.0 m ensured the stability of the face.

Failure in weak ground can also occur in the unsupported span. To prevent this type of failure spiles were installed recurrently until the middle of the model. Figure 4 shows the result of a calculation where the excavation advances without spile support. The unsupported span collapses when the ground (foundation) under the forepoling is excavated. Primary failure occurs at the crown level and develops to the sidewall region in the unsupported span. As this failure develops, a second failure occurs at the face. By having a look on the development of deformations in figure 4, the probability for a daylight collapse is very high at this scenario.

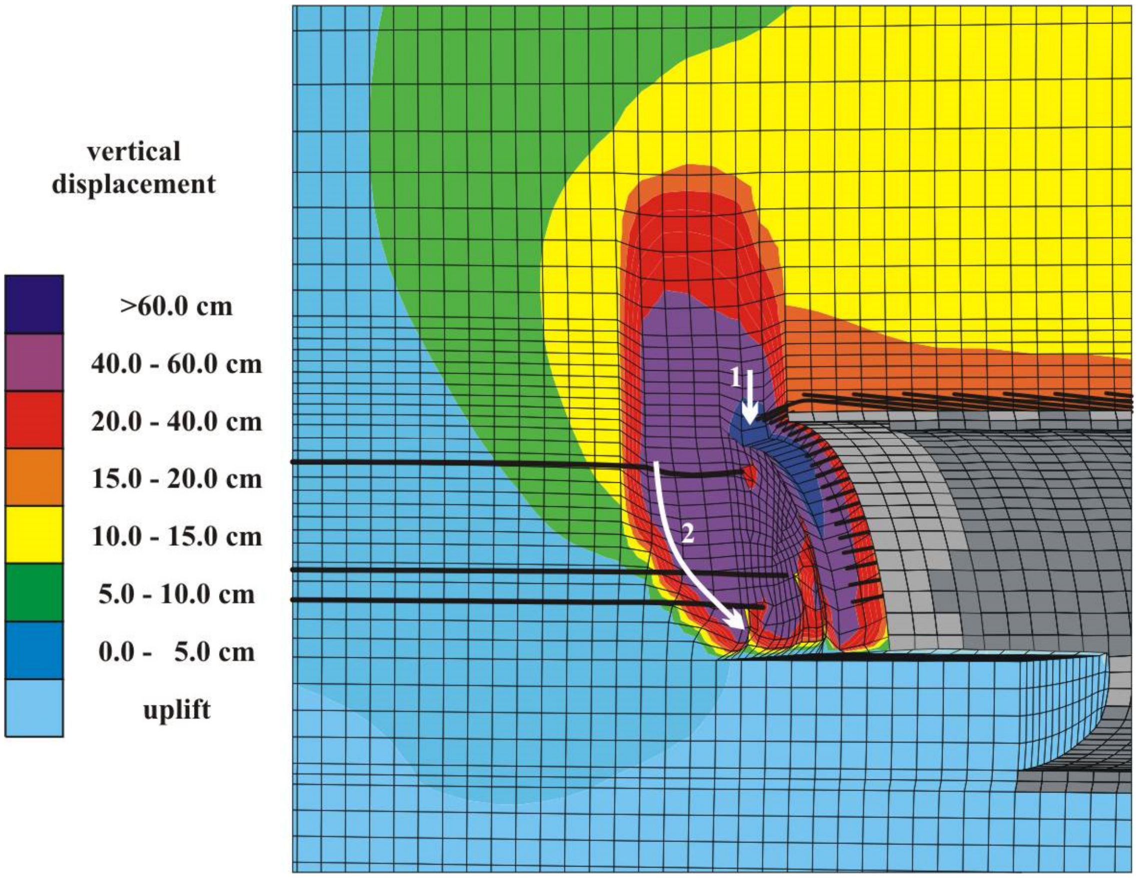

Figure 4. Failure mechanism without pre-support at the shape of the working area; it starts at the crown and develops to the sidewall; in a second step the face fails.

The failure mechanism, marked with number 1 in figure 4, did not appear at the previously discussed failure. For this reason, the mechanisms causing this failure will be described in detail to extract differences.

The excavation step shown in figure 4 is sequentially excavated in 2 phases. The upper top heading was excavated first. Then the stress redistribution and deformations were calculated respectively. During this calculation, the crown stayed stable although the material operating as front foundation of spiles was already excavated. The unsupported span is modeled in 3 zones in the longitudinal direction. Though this step stayed stable, the stress redistribution caused an overloading in shear strength but also tensile strength in the middle of those 3 zones.

This step was followed by excavating the lower top heading. The stress redistribution and accordingly the deformations that were calculated due to this excavation phase caused the failure shown in figure 4. At the beginning of the calculation, the material ahead of the face moved towards the tunnel. This movement squeezes the material of the unsupported crown into the tunnel. All 3 zones consequently got overloaded in shear and tensile strength in this section. This failure propagates to the sidewall and in the radial direction. The stresses transferred at those regions are redistributed to regions farther to the excavated area. Due to this effect, the dead load forces, transferred to the material at the face bottom, increase. This additional load causes a shear failure (No 2 in figure4) at the face now although there are face bolts to stabilize the face.

5 EVALUATION OF PRE-SUPPORT MEASURES

Spiles are used to stabilize the unsupported span in weak ground. It is a simple and very effective support method, which can quickly be adapted to the actual ground conditions. During the past decades, the pipe umbrella method also became very popular for stabilizing the earlier described failure mechanism. To assess the effectiveness of the different systems, both additional pre-support measures are compared in the following sections. 20 spiles installed every meter create the forepoling while the pipe umbrella system uses 15 pipes. The simulated spile is a hollow bar (38.0 mm x 4.0 mm) with 4.0 m in length. This results in approximately 268 kg steel per tunnel meter. The pipe umbrella pipes have 114.3 mm as outer diameter and a pipe wall thickness of 6.3 mm. The umbrella is recurrently installed every 12.0 m with a pipe length of 15.0 m. This system results in approximately 315 kg steel per tunnel meter. These numbers are valid for the calculated half model.

Both systems lead to a stable excavation (figure 5 and 6). The final settlement amount is a little higher at the tunnel crown when using the steel bars than with the pipe umbrella.

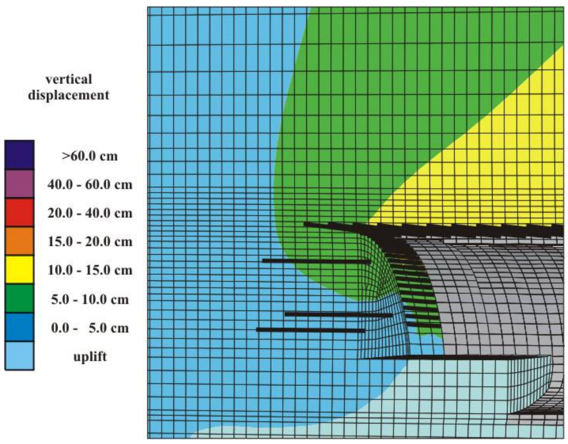

Figure 5. Detail of the numerical calculation showing a stable tunnel with spiles as pre-support method

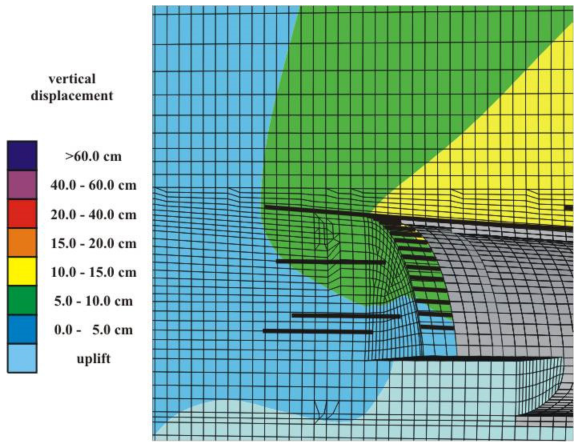

Figure 6. Detail of the numerical calculation showing a stable tunnel by using a pipe umbrella method

6 UNEXPECTED CHANGES OF GROUND CONDITIONS

The ground conditions are sometimes inhomogeneous in urban areas. Additionally, the probability for unknown features in the ground is due to historical, very often unknown reasons high. For example, Ayaydin (2001) reported about a collapse during the excavation of the Metro in Istanbul. During their investigations, they found that an old well caused this collapse (but local fillings could also cause serious problems).

These unexpected cases are nearly impossible to detect during the exploration phase of a project. For this reason, a weaker zone was introduced in the model and the effectiveness of both systems checked when passing this feature. In the following example, the excavation passes a vertical layer perpendicular to the tunnel axis. Its thickness is 1.0 m. The cohesion values are decreased to two-thirds of the original values in this layer. With the lower cohesion, it is expected that the ground fails before the excavation can be finished and support installed.

The calculation shows that the first instabilities occur at the face with both pre-support methods after excavating the upper half of the top heading. Approximately, one-third of the weak material in the face failed. After excavating the lower part of the top heading, the calculation showed that the one-meter thick layer failed nearly at the entire face region. Thus, both pre-support methods must support a span of nearly 2.5 m.

The spiles are bent to their capacity and the ground support is not sufficient at the face. A complete collapse of the ground at the working area is the result. In this case, the calculation cannot clarify, which effect caused the failure, but the spiling cannot support the 2.5 m span.

While the face also collapses when using the pipe umbrella support, the tunnel remains stable (Figure 7). The deformations increase but the calculated moment in the pipes reads at about 45 % of the maximum capacity, when stability is achieved. While the foundation ahead of the failed face section was overloaded by using spiles, the much stiffer pipes can traverse the same region without causing apparent problems. This fact indicates the importance of a proper foundation length ahead of the construction when using pipe umbrellas.

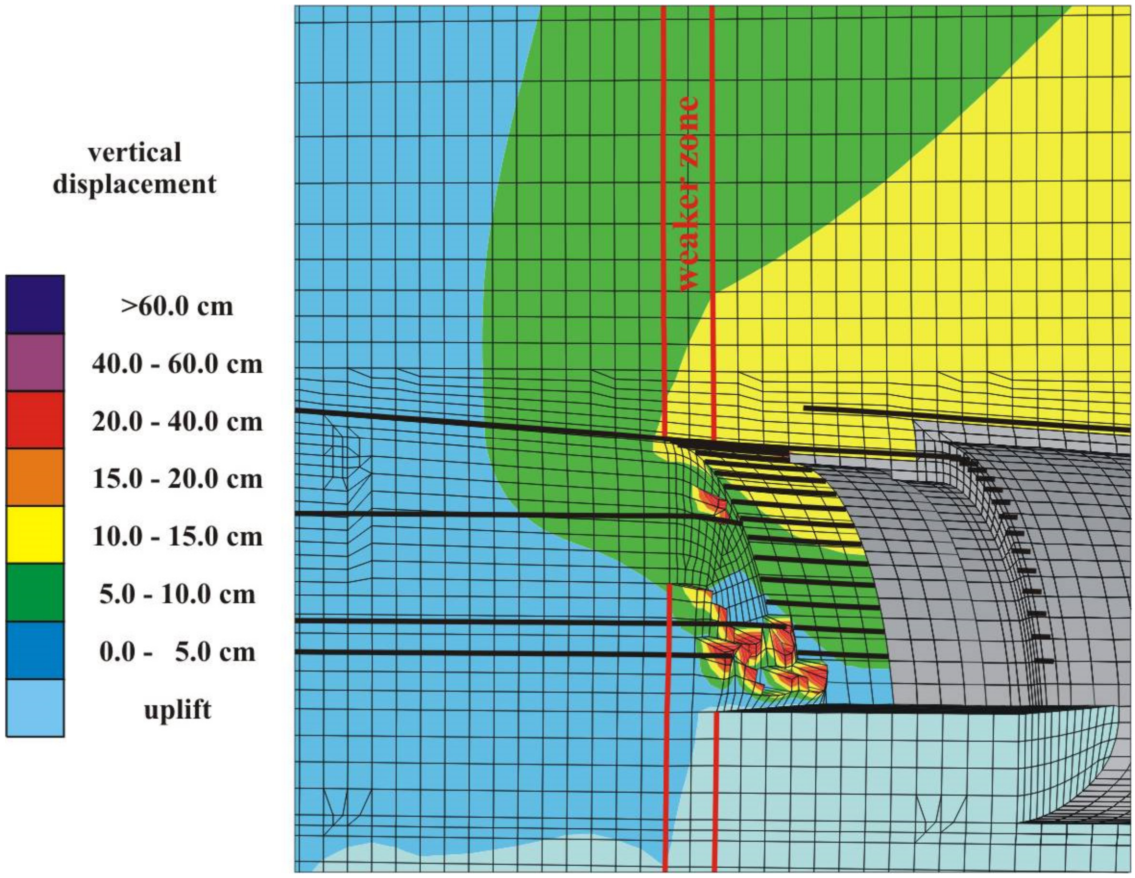

Figure 7. This figure shows the result of the calculation with an unexpected change of the ground conditions for a one-meter thick vertical layer (red). Although the face collapses, the pipe umbrella method stabilizes the ground movements around the tunnel periphery.

The calculated result is similar to one of the cases, which Möhrke described in 1999. During excavation works at the Greek railroad between Platamon and Leptokaria, a face collapse occurred. He reported that the installed pipes prevented a propagation of the collapse and the tunnel remained stable.

The main reason for the significant difference in stability is the magnitude of section modulus of the compared systems. In the studied case, with similar weight of steel per tunnel meter, the section modulus for one spile is 4.1E-7 m4. Hence, the sum for all 20 spiles is 8.2E-6 m4. Compared to these values, one pipe has a section modulus of 6.8E-6 m4 and all installed pipes give a section modulus of 1.0E-4 m4. As can be seen by these values the section modulus for the pipe umbrella method is more than 10 times higher than that of the spiles.

7 CONCLUSION

Conventional tunnel construction in weak ground conditions usually faces the problem of instability in the working area. Combinations of common support components counter these problems. The most effective measure against face instability is the face bolt. Various methods are available for stabilizing the excavated tunnel; Freezing or jet-grouting techniques create are very stiff support but they are expensive as well. The simpler spiling and pipe umbrella method are in most cases sufficient to stabilize the tunnel. Of course, these methods are less effective against deformations, but also less expensive. Although the experience with those methods increases, there are no clear rules for the decision-making process based on project demands.

The investigations discussed in this work compared the effectiveness of the spiling method (forepoling) and the pipe umbrella method (long-span forepoling) in supporting the working area against failures. It could be shown that both systems are very effective in supporting the perimeter of the working area. Nevertheless, when facing inhomogeneous ground conditions, the probability of failure is higher with the spiling method. The lower section modulus and the shorter foundation length ahead of the face mainly cause this difference. Due to this result, a pipe umbrella method should be preferred when sudden unexpected changes in the ground cannot be excluded.

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Appendix H

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

Author(s): G. M. Volkmann, W. Schubert

Published: Proceedings of the Golden Rocks 2006, The 41st U.S. Symposium on Rock Mechanics (USRMS): "50 Years of Rock Mechanics - Landmarks and Future Challenges.", Golden, Colorado, June 17-21, 2006. Copyright 2005, ARMA, American Rock Mechanics Association

Summary: After specifying common pre-support measures at the perimeter of the tunnel, this article focuses on results of FLAC-3d numerical simulations. The implementation of different geotechnical models identifies the already discussed one as the right one because it results in the same deformation characteristics and magnitudes. Results of the same calculations but without pipe umbrella support show a huge increase in deformations.

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

Volkmann, G.M. and Schubert, W.

Graz University of Technology, Institute for Rock Mechanics and Tunneling, Graz, Austria

Button, E.A.

Department of Earth Sciences, ETH Zurich, Zurich, Switzerland

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 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 nowadays is much used 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 easily adapted 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 pipes were performed to develop input parameters for numerical simulations. 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 in front of 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 (figure 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 shaped profile. Their lengths can vary, but typically are 12 m or 15 m long. After grouting the inner annulus and the annular gap 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.

- 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".

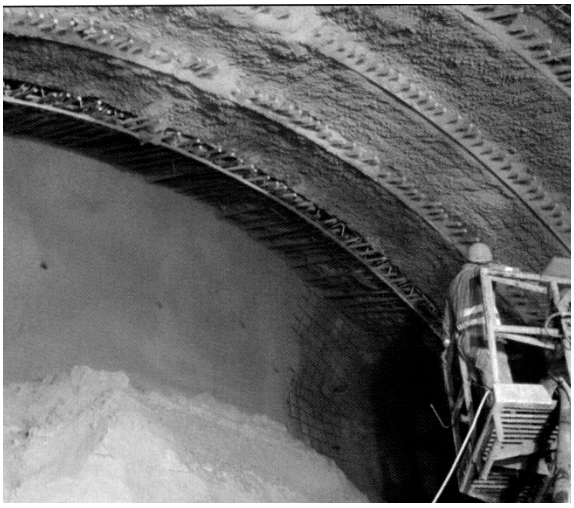

Figure 1. Spiles installed through the lattice girder to the face in every excavation step.

This paper will focus on the pipe roof umbrella system, which will be explained in more detail. There are currently two methods 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. We call this the cased-drilling method. There appear to be two distinct advantages to this installation method compared to 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 is in contact with the rock mass only in the area around the drill bit. This minimizes 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.

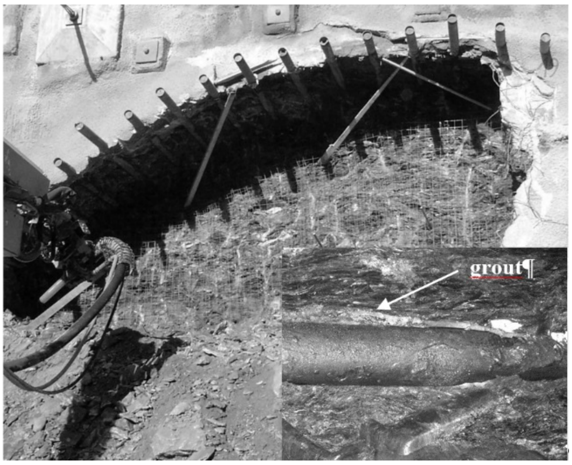

Figure 2. Portal pipe roof umbrella at Birgltunnel (Austria); the detail shows grout filling the annular gap.

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 a 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 works. This aspect is important for the 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. The pipes are usually not grouted with high pressures as they are 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 may differ from that one described in the publications up to now.

3. IN SITU MEASUREMENTS

The movement initiated by the construction of a conventional tunnel is normally measured 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. Deviations from the expected behavior have to be analyzed and if necessary an adaptation of the support system is made.

Pre-support systems like a pipe roof umbrella are primarily acting ahead of the face and in the non-supported area behind the face. The Measurement of displacements by geodetic survey is not possible in those areas. To obtain information on displacements in those sections an additional measurement system is required.

An in-place 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.

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 m2 and 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 ratio [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 losing 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 geotechnically reasonable and showed promising results 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 values in pre-defined time intervals without interrupting the construction process. For the 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. During the grouting process of the pipe roof pipes the inclinometer casing and the instrumentation were 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.

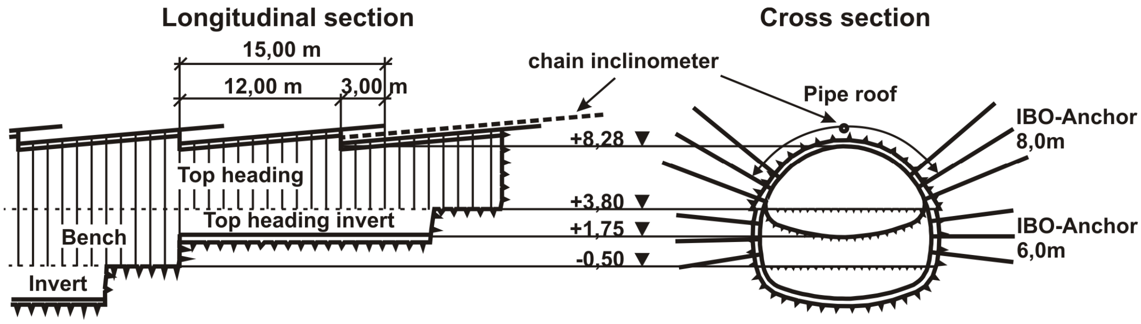

Figure 3. Position of the chain inclinometers [1]

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 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 to the tunnel displacements.

3.3. Data evaluation

Compared to the geodetic survey, which was performed once or twice a day, for the inclinometer we chose 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 are shown as a time-settlement 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 dots. 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 is discussed in detail in relation to the numerical simulations in Section 4.3.

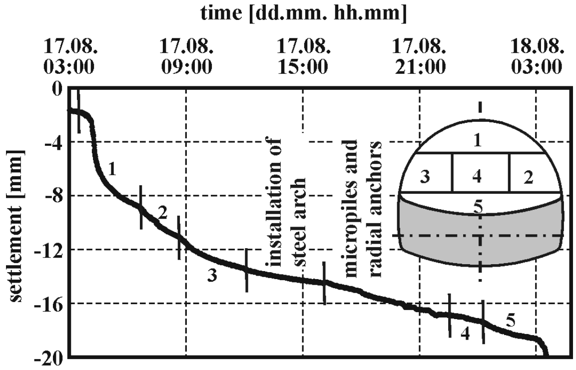

Figure 4. Time settlement line including the installation of radial rock bolts [11].

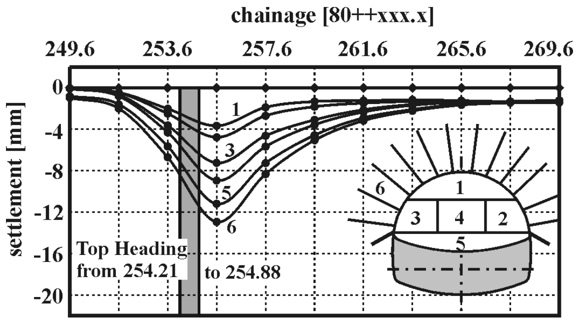

Figure 5. Deflection curve of one excavation step [12].

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. When the unsupported span and the new face areas are supported with shotcrete the settlement rate decreases. 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 (to measure both horizontal and vertical displacement components) the influence 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. During this time the stabilization process continues (figure 4) and the settlement velocity decreases.

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 Trojane tunnel. 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 at 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 a typical deformation characteristic in this section directly. The excavation and support system can be adjusted to the ground conditions ahead of the face whenever changes to the typical characteristic appear [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 used for the numerical investigations. For this discussion the measurements and numerical simulations are related to the Trojane tunnel discussed above. Two different models are discussed. One utilizes the pile elements in FLAC-3D to simulate the pipe roof and the other is a comparison to the homogenation method where a shell of improved ground is used instead of the pile elements.

4.1. Numerical Model

In order to decrease the boundary effects in the excavation direction the model had 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 only one half of the tunnel was simulated.

Rock mass properties were determined for the pre-peak behavior with the values reported by Zlender [8] for the Trojane tunnel. 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 installing the pipe roof pipes, related to the pipe installation angle. 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.

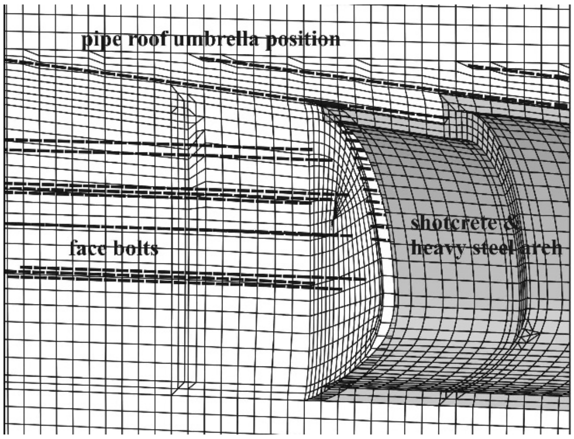

Figure 6. The numerical model before the excavation takes place

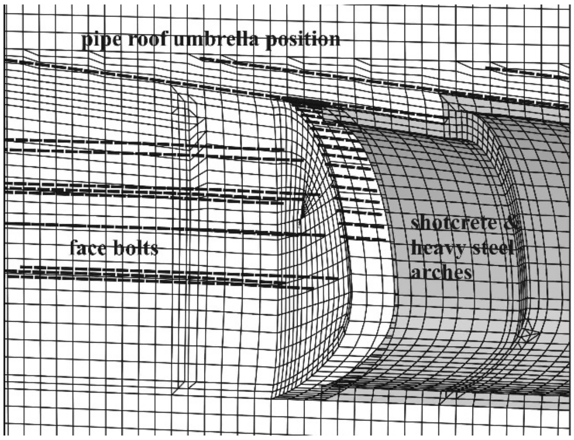

Figure 7. The modeled situation when the excavation has reached equilibrium is done.

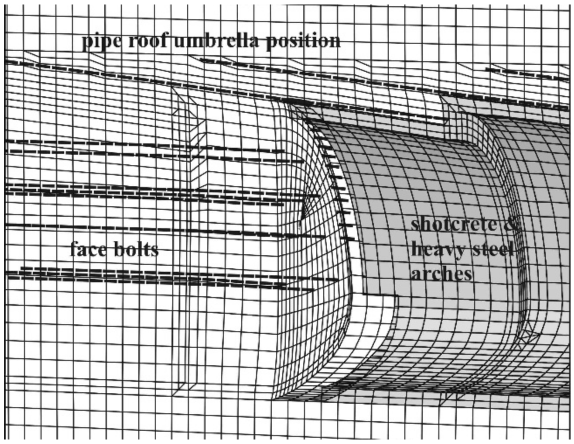

Figure 8. The first supporting phase used in the simulation.

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 behavior and vice 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 showed that the drilling of the pipe roof holes ahead of the face in this weak rock also produces movements [20]. This led to the conclusion 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 uppermost 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 Trojane tunnel the results of the numerical calculation show a similar result as those 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 Trojane tunnel 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 settlement magnitude begins to increase indicating the zone directly influenced by the excavation. 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 smoother than in the real case. Both of these results seem to 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.

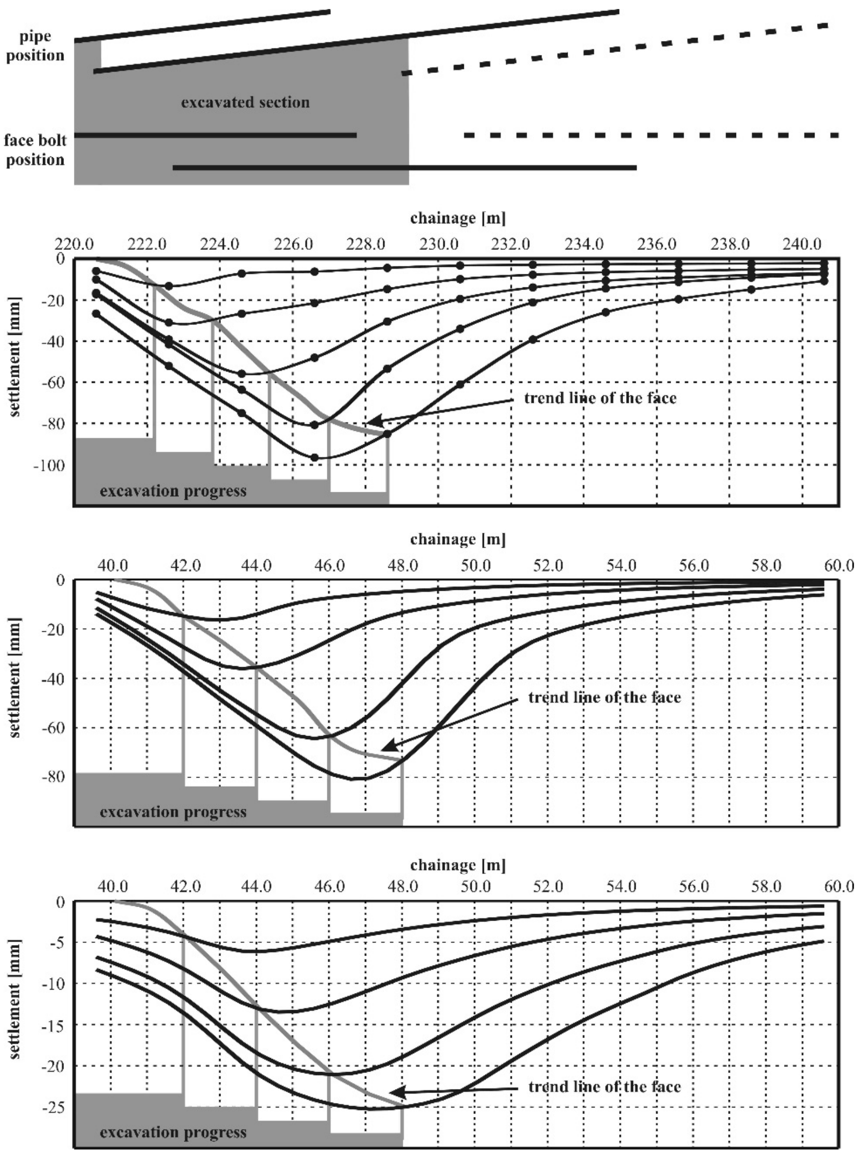

Figure 9. deflection curves diagram for the in-situ measurements (above), the structural element model (middle) and the homogenized case (below).

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.

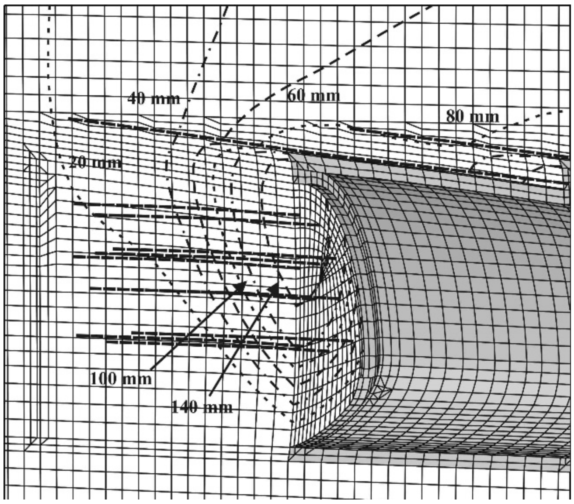

Figure 10. Settlement-isolines for the pre-supported case

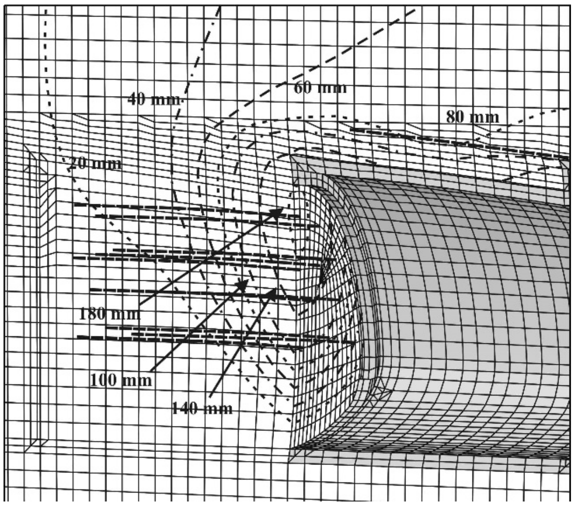

Figure 11. Settlement-isolines for the case without pre-support

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 weaker ground conditions. A reason for this effect is that the bending of the pipes mobilizes the supporting effect of this system.

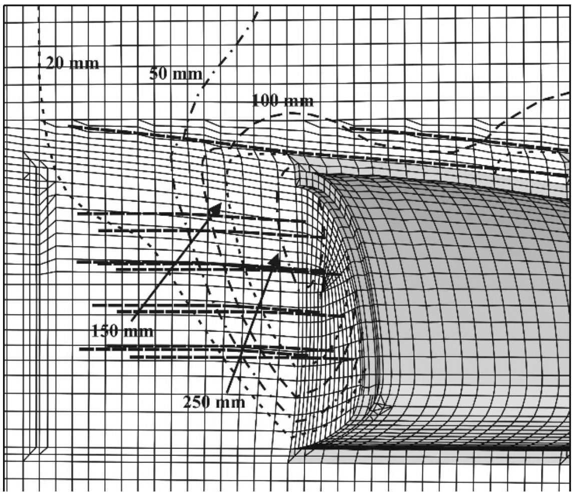

Figure 12. Settlement-isolines for the pre-supported case with lower residual cohesion

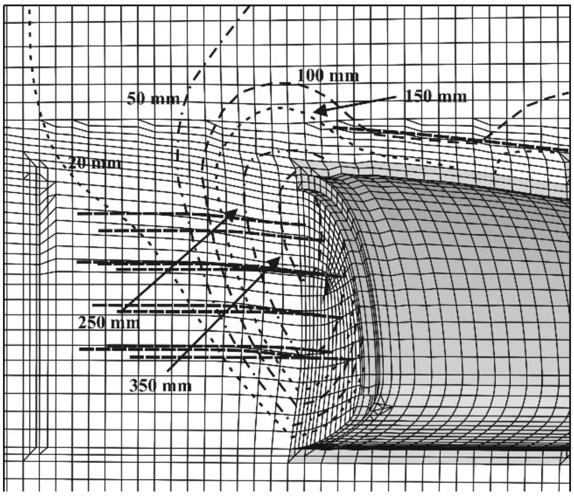

Figure 13. Settlement-isolines for the case without pre-support with lower residual cohesion

5. GEOTECHNICAL MODEL

For tunneling in weak ground, the pipe roof umbrella system is supporting the excavation in the radial direction around 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.

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 and 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

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

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; the stronger the pipe should be to develop the same supporting effect. The diameter of the pipes seems to control the rate at which the support effects mobilize, with larger diameters the supporting effect develops faster due to increased bending resistance.

The overlapping length of the pipe roof pipes in the longitudinal direction depends 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 projects, which are sensitive to subsidence, the stability and the movements due to unsupported holes for the pipes should be investigated [20]. If 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 only 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 verify 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 adapted 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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Appendix I

Title: CONTRIBUTION TO THE DESIGN OF TUNNELS WITH PIPE ROOF SUPPORT

Author(s): G. M. Volkmann, W. Schubert

Published: In Proceedings of 4th Asian Rock Mechanics Symposium, ISRM International Symposium. eds. Leung C.F. & Zhou Y.X., ISBN 981-270-437-X World Scientific Publishing Co. Pte. Ltd., Singapore, 8th – 10th November 2006

Summary: After defining pipe umbrella support systems, the two projects, which were observed with online inclinometer-chain measurements, are introduced. Observations and benefits during construction are discussed and finally a parametric study with numerical 3-dimensional simulations compared. This comparison shows clearly that a pipe umbrella support decreases construction induced deformations. As a result, the essential parameters to control deformation reduction are the pipe dimensions and the installed number of pipes.

CONTRIBUTION TO THE DESIGN OF TUNNELS WITH PIPE ROOF SUPPORT

G.M. VOLKMANN and W. SCHUBERT

Graz University of Technology, Institute for Rock Mechanics and Tunneling; Graz, Austria

(e-mail of corresponding author: volkmann@tugraz.at)

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 m2. 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 the surface are determined in a local coordinate system during excavation. Such surveys are normally executed daily.

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 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.

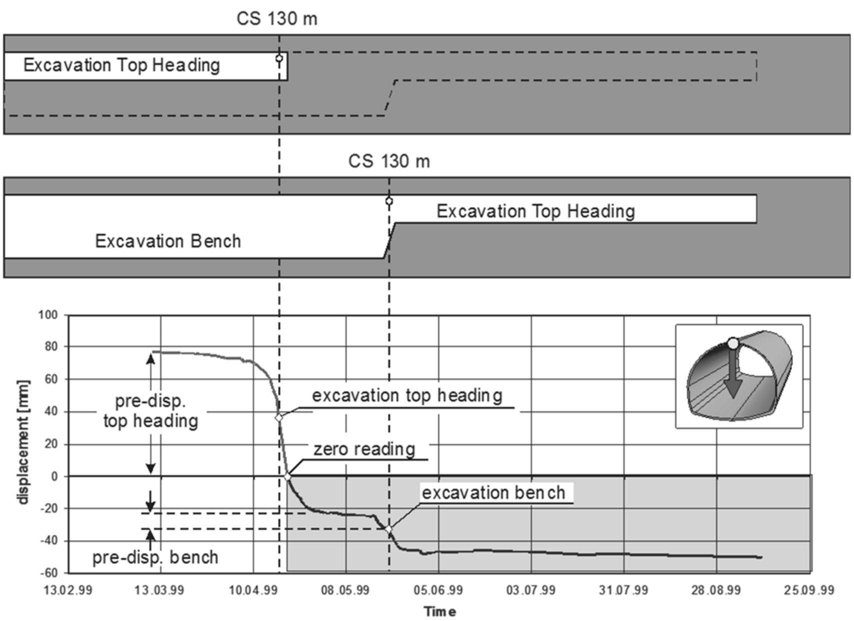

Figure 1. Time-settlement diagram: the grey shaded area in the diagram highlights the part of displacements that can be measured by a geodetic survey at the tunnel level (Sellner, 2000)

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).

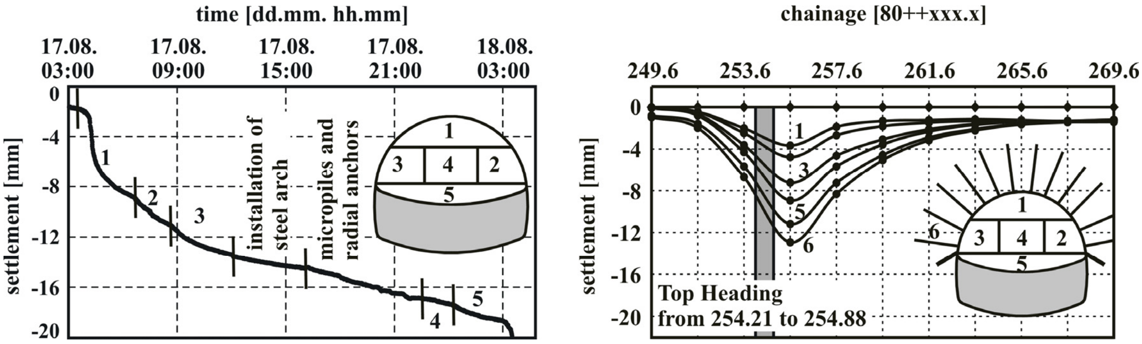

Figure 2. Time settlement diagram (left) and single deflection curve diagram (right) for one sequential excavation round

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.

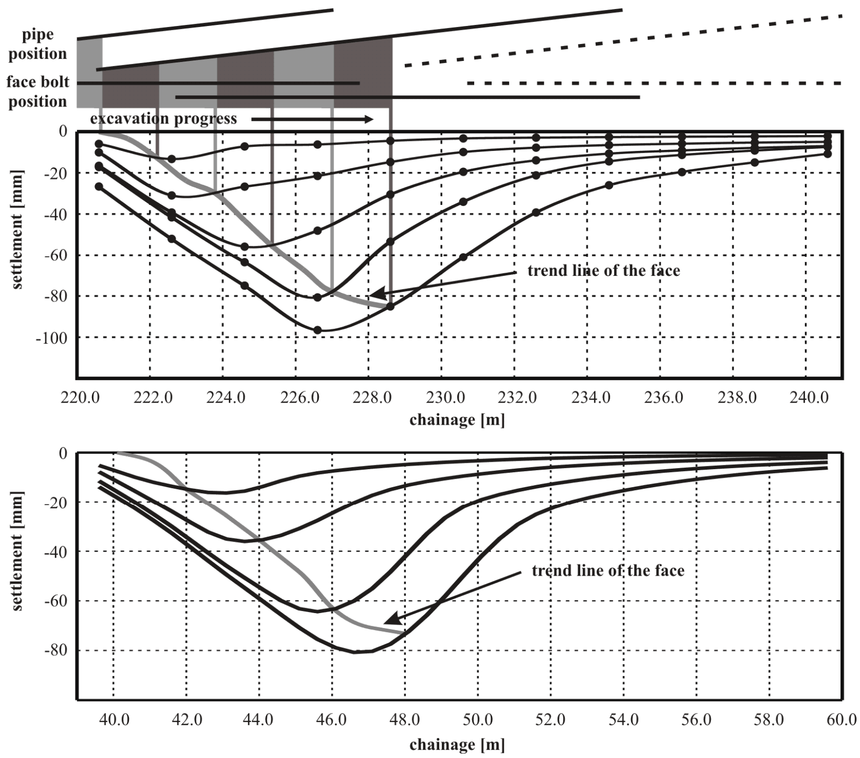

Figure 3. In situ deflection curve (upper) and simulated deflection curve diagram (lower) without pre-displacements.

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.

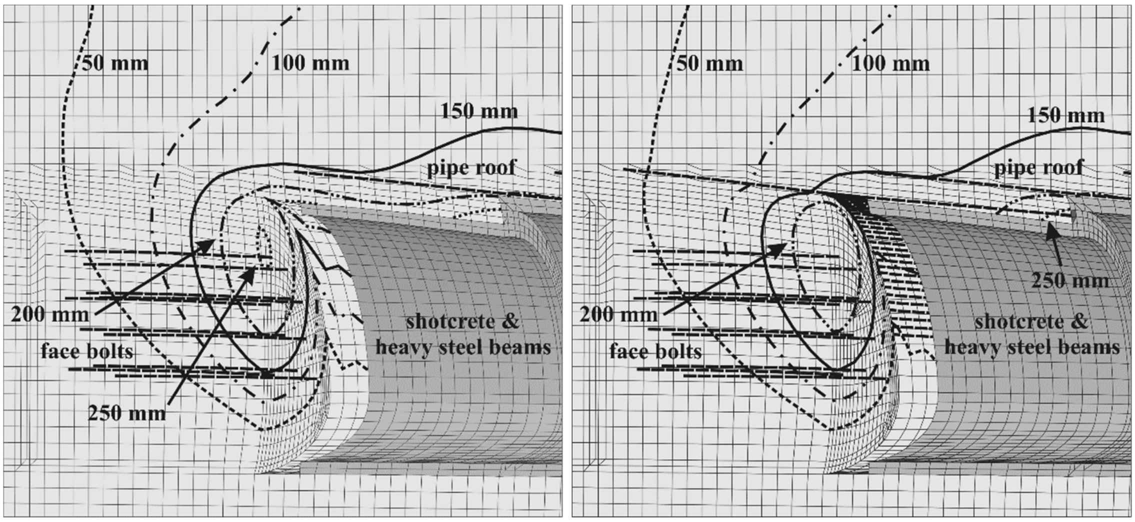

Figure 4. Calculated settlement values without (left side) and with pre-support (30 pieces of 114.3 x 6.3) (right side)

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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Appendix J

Title: Optimization Potential Regarding Safety, Material, and Installation Time for Pipe Umbrella Installation Methods

Author(s): G. M. Volkmann, D. Glantschnegg

Published: Proceedings of the 16th Australasian Tunnelling Conference 2017, 30th of October – 1st of November 2017, Sydney, Australia

Summary: The technical developments of drilling machinery and pipe umbrella connections allow some optimization for the pipe umbrella system during design. These optimization possibilities can be analyzed regarding applied material, necessary time, or safety issues. It can clearly be presented that changing to current techniques is connected to advantages for all involved parties.

Optimization Potential Regarding Safety, Material, and Installation Time for Pipe Umbrella Installation Methods

G.M. Volkmann and D. Glantschnegg

DSI Underground Austria GmbH, Alfred-Wagner Strasse 1, 4061 Pasching, AUSTRIA

E-mail: guenther.volkmann@dsiunderground.at

ABSTRACT

Pipe umbrella support systems have been successfully used for tunnelling in challenging ground conditions since the 1970's. Due to ongoing developments of the available systems, drilling machinery, and other necessary equipment this type of pre-support system is used at shallow tunnels in weak ground conditions on a regular basis nowadays. Pipe umbrella pipes are installed stepwise subparallel to the tunnel alignment by connecting pipes to each other. So, the process of installation requires coupling and uncoupling steps of drill steel as well as pipes. Actual drilling machinery allows to install longer pipes and additional automation units change the installation process.

These developments allow changes in the design of pipe umbrella support systems leading to a significant potential for saving material as well as time in the installation process. Furthermore, the implementation of state-of-the-Art coupling types and automation units decrease the risk for injuries leading to more efficiency due to lower physical as well as mental fatigue. These advantages will be explained in detail and presented in the article.

Keywords: Canopy Tube, Pipe Umbrella, Installation Length, Pipe Coupling, Safety, Mechanization and Automation.

1. INTRODUCTION

Increased tunnelling activities within developed areas resulted in the modernization of urban as well as regional infrastructure in soil and weak rocks. A safe and economical construction is always desired, even though ground conditions may not be optimal. In urban areas, this often results in critical sections, which are additionally supported with pre-support systems such as lagging boards, spiles, pipe umbrella support or cost intensive and time-consuming methods like ground freezing or jet grouting. All these measures are utilized to ensure stability of construction as well as to protect surrounding infrastructure from damages.

Due to technical developments of drilling machinery over the past decades, the pipe umbrella support method is one of the pre-support concepts that is increasingly used in conventional tunnelling and has even been included in TBM support systems. The installed pipes support potentially unstable ground during excavation in the working area and ahead of the face. The support method provides a high degree of flexibility and can be adaptable readily to the encountered conditions. Principles of this method are quite easy, hence the potential regarding optimization possibilities is rarely used even though material costs, construction time, and safety issues are important facts all around the world.

2. PIPE UMBRELLA SUPPORT (CANOPY TUBE)

Pipes with an outer diameter lower than 200 mm (not exactly defined) are installed from the actual face to the front using either special machines (e.g. Casagrande drilling rigs) or conventional underground drilling machinery. Pipe umbrella pipes can be installed into pre-drilled holes, cased holes where the casing is removed after installation, or as self-drilling system where the supporting pipes follow directly behind the sacrificial drill bit. The pipes are aligned around the upper shape of the later excavated tunnel creating an “umbrella” or a “canopy” above (Figure 1). Due to the method of installation, the system requires a widening of the cross section resulting in a sawtooth profile. Installed pipe lengths can vary but are typically in the range of 12 m, 15 m, or 18 m length with a pipe wall thickness between 5.0 mm and 12.5 mm. After grouting the inner annulus and the annular gap, the excavation advances under the supporting pipes. After a pre-defined length of excavation, the same procedure recurrently starts using the space created by the sawtooth profile for the installation. For this system, a few names are used worldwide: e.g. steel pipe umbrella system [2], umbrella arch method [3]; pipe forepole umbrella [4], long-span steel pipe fore-piling method [5], steel pipe canopy [6].

The implementation of pre- support systems as additional support measure may be caused by tunnel stability reasons, expected surface settlements, increase of safety, expected uncertainties or also public acceptance. In case a design decision results in the choice of a pipe umbrella system, the pipe design is commonly performed with analytical solutions or with complex 3-dimensional numerical calculations. Exemplary analytical solutions can be found in Oreste and Peila [7] as well as Volkmann and Schubert [8]. 3-dimensional numerical calculations are a little more sophisticated. Parameters for the entire model must be selected very carefully to get reasonable results that picture the later construction process. The result of this design process should be a minimum elastic moment that can at least be carried by each part of the pipe umbrella system. This results in a definition independent from pipe dimensions, steel grades, and system features like couplings, injection holes, etc. Further down, a detailed explanation will be provided why this way of definition is crucial to achieve a proper support system during construction.

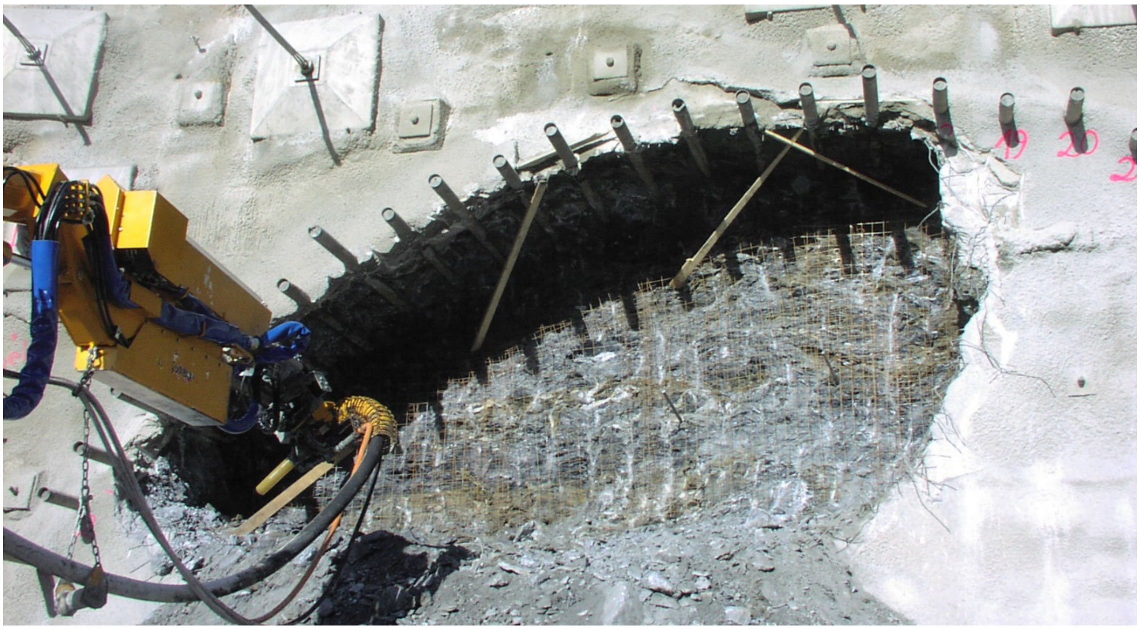

Figure 1 Portal pipe umbrella at Birgltunnel (Austria) [1]

2.1. Pipe umbrella example

To be able to provide a reference for later comparisons, a simple pipe umbrella example is illustrated in figure 2. This example is based on the top heading section of a 12m diameter tunnel. The top heading section is supported by 40 pipes with a length of 12m. Pipe umbrella pipes are installed with an axial distance of 400 mm at the point of installation. Due to the required widening, the maximum axial distance results in 456 mm above excavated sections. The overlap in the longitudinal direction is defined to be greater than 3 m resulting in eight 1 m long excavation steps and a necessary remaining pipe foundation ahead of the face. This exemplary pipe umbrella consists in sum of 480 m of steel pipes, which are installed in 3 m long pieces so 120 couplings must be fixed during installation. The over-excavation due to the sawtooth profile results in app. 70.5 m3.

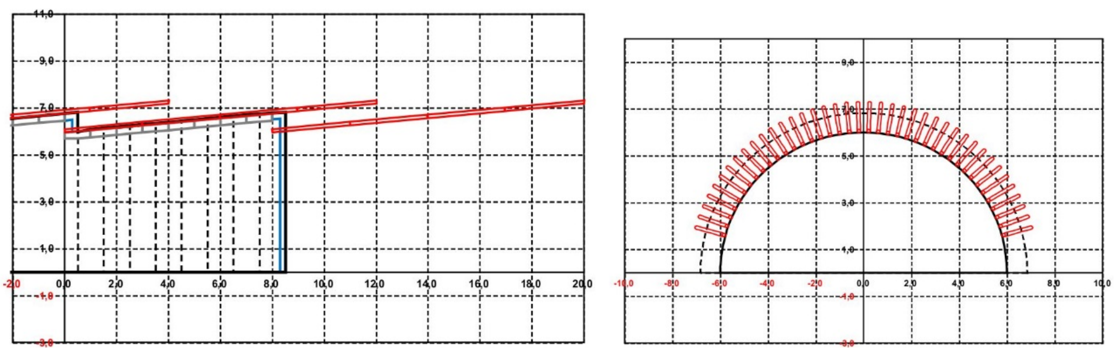

Figure 2 longitudinal and cross section of an exemplary pipe umbrella.

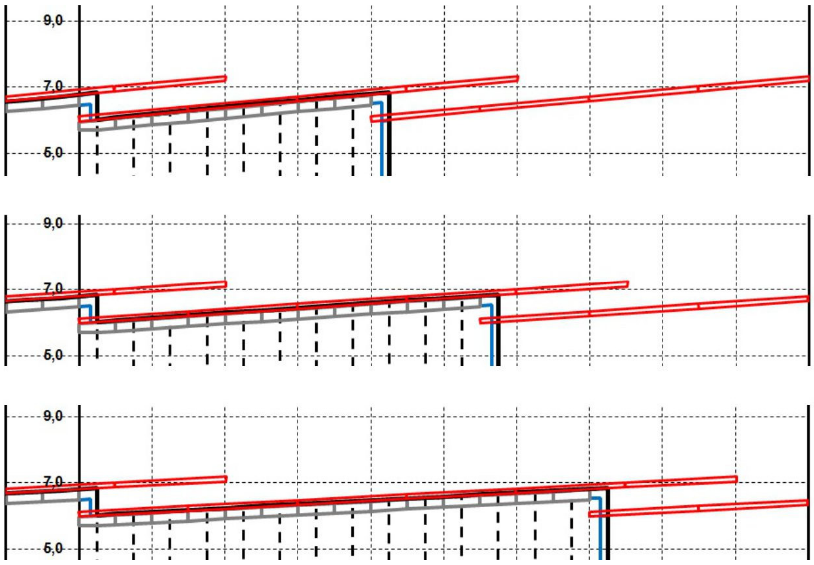

Figure 3 longitudinal sketch of 12 m, 15 m, and 18 m installation length.

3. OPTIMIZATION POTENTIAL REGARDING MATERIAL

The example presented in chapter 2.1 is a typical design for European highway or railroad tunnels of the last 20 years. Nowadays, technical developments of drilling machinery, installation methodologies, and the increased accuracy in boom (drill arm) orientation allows to install pipes with a length up to 18 m on a regular basis without major deviations. In proper ground conditions, it is even possible to install pipes up to a depth of 30 m or more (exceptional cases). Due to the typical boom length, the maximum length of pipe pieces is 3m, resulting in common installation lengths of 12 m, 15 m, or 18 m. As can be seen in Figure 3, the overlap in the longitudinal direction and the axial distances are constant while the parameters shape of sawtooth, pipe length, inclination, and number of excavation steps change.

Due to the ration installed pipe length - overlapping length, material consumption can be decreased by increasing the installed pipe length (table 1). The change from 12 m to 15 m decreases the necessary pipe quantity by nearly 10 % while a change to 18 m results in savings of nearly 15 %. When using a 139.7x6.3 pipe, a 15% reduction would result in nearly 180 kg less steel per running meter tunnel plus a small decrease of excavated material, which must not be mucked out or refilled by shotcrete.

Another feature can be used to optimize the utilized material. As mentioned above, pipe umbrella pipes are installed piecewise, so the pieces must be connected to each other during installation. Typically, these couplings are the weakest link of the pipe umbrella system. [9] Therefore, the regular pipe dimension (outer diameter and wall thickness) cannot be used for static calculations, it must be the weakest point of the system – the connection area. Consequently, a tender document specification not defining a coupling type is ambiguous without defining the minimum elastic moment – as recommended above. Nowadays, there are three different couplings types in use worldwide with different characteristics regarding resistance against bending. [10] There is the standard thread connection, the squeezed coupling and a so-called nipple connection.

Table 1 Comparison of material for 12 m, 15 m, and 18 m long pipe umbrellas.

Pipe umbrella length	Excavation length	Axial pipe distance	Maximum axial distance	Pipes installed	Pipe per rm tunnel	Over-excavation	Over-excavation per rm tunnel
12 m	8 m	400 mm	456 mm	480 m	60.0	70.4 m³	8.8 m³
15 m	11 m			600 m	54.5	95.3 m³	8.7 m³
18 m	14 m			720 m	51.4	120.1 m³	8.6 m³

Table 2 Comparison of mechanical properties of different pipe couplings.

Pipe type	Coupling type	W [cm³]	W [%]	I [cm⁴]	I [%]	M_{el}* [kNm]	M_{el} [%]	M_{pl}* [kNm]	Weight [kg/m]
139.7x6.3	squeezed	58.4	100	344.0	100	20.7	100	>40.0	20.7
	none (tube)	84.3	144	588.6	171	29.9	144	n/a	
139.7x10.0	cut thread	53.6	92	345.3	100	19.0	92	n/a	32.0
	none (tube)	123.4	211	861.9	250	43.8	212	n/a	
139.7x4.0	nipple	56.2	96	392.8	114	20.0	97	n/a	13.4
	none (tube)							n/a	

* Values must be proven by the manufacturer with certificates.
M_{el} … maximum bending moment [kNm] in the elastic material range
M_{pl} … Maximum bending moment [kNm] when using plastic reserves of the steel material

Standard threaded connections are generally not well suited for connecting pipe umbrella pipes. By mechanically removing a certain portion of the steel tube for a thread, the effective cross-section is reduced. This fact drastically decreases the load-bearing capacity and stiffness in the connection area. Because of dangerous problems when using standard threaded connections during construction, the so-called nipple connection was developed. Nipple connections consist of threaded connection fittings, which are pressed and welded into the ends of standard pipes. This connection type provides an elastic design load as well as stiffness properties equal to an un-weakened pipe. The latest development in the field of pipe connections is the squeezed connection, this connection type results from the attempt to provide a tough and easy-to-connect alternative to conventional threaded systems. By means of the squeezed connection, non-threaded pipe ends are mechanically connected in terms of force-fitted squeezing using a boom-mounted press.

The influence of the installed coupling type can be seen in the technical values of Table 2. In this table, the squeezed coupling is taken as reference (100%) because it is the newest and from a handling point of view the safest. For this coupling type, a quite common pipe dimension (139.7x6.3) is taken as reference. To achieve a comparable calculated elastic moment with a standard thread the pipe wall thickness must be increased from 6.3 mm to 10.0 mm. This change increases the weight of a pipe meter more than 50%. By using the most efficient coupling type – the nipple coupling – the coupling is no longer the weakest link so 35% of steel can be saved without losing support strength. These values show the huge potential of optimizing material utilization with correct technical definitions for pipe umbrellas. So, for all standard situations in tunnelling a squeezed coupling can be recommended due to its effectiveness and economic reasons while a nipple connection should be used at all tunnel sections where prevention of damages on the surface is of highest importance.

4. OPTIMIZATION POTENTIAL REGARDING TIME

As mentioned above, pipe umbrellas can be installed with special or conventional drilling machinery. Typically, special drilling machines have only one boom while conventional tunnel drilling machines are normally equipped with two booms and a basket. This results in most cases in reduction of installation time when utilizing common drilling machinery.

The increase of installation length has an influence on time requirements as well. As can be seen in table 3, the number of installed pipe meters per running meter of tunnel decreases with increasing installation length. This results in different installation times; at 12 m length 150 minutes, at 15 m length 136 min and at 18 m length only 128 min. So, we have a decrease of installation time by 10% when changing to 15 m and by 15% when changing to 18 m installation length. There is also a small decrease of coupling processes per tunnel meter, but there is a more important point regarding couplings that will be explained in the following paragraph.

Table 3 Comparison of times for 12 m, 15 m, and 18 m long pipe umbrellas.

Type	Excavation length	Pipes installed	Pipes per rm tunnel	Installation time per rm *	No of couplings	Coupling per rm tunnel
	[m]	[m]	[m]	[min]	[-]	[-]
12 m	8	480	60.0	150	120	15.0
15 m	11	600	54.5	136	160	14.5
18 m	14	720	51.4	128	200	14.3

* Values are taken from [11]

Pipe umbrella pipes are connected by two methods during installation; thread based connections and non-thread based squeezed connections. The thread geometry on tubes has smaller tolerances and the manufacturing process is more extensive than tubes produced for squeezed connections. Threads must also be protected against damages and dirt before installation. Due to these disadvantages, a certain percentage of thread connections require additional handling time during connection. This is mainly caused by dirt or small damages of the thread. A squeezed connection is not prone to these effects due to its robust nature. In table 4 the given example of chapter 2.1 is analysed regarding different connection times for both connections including the known delay times for a thread connection. The analysed data shows that the connection time is less than half when using a squeezed connection on-site during installation.

Table 4 Influence of connection type on installation time.

Type	No. of connections per tunnel meter	Single connecting time	Connecting time	Single delay time *	No. of difficult connections	Delay time	Total connection time
	[-]	**[min]**	**[min]**	**[min]**	**[%]**	**[min]**	**[min]**
Standard thread	15	3.5	52.5	10	5	7.5	60
Squeezed coupling		1.5	22.5	-	0	0	22.5
Time savings			**30**			**7.5**	**37.5**

* Experience: 5% difficult connections (outliers) which require additional handling time

Finally, it must be mentioned that mechanized or automated processes run in general faster than manual ones so any kind of mechanization or automation during pipe umbrella installation assists in speeding up the entire process. An example for a comparison of installation times is given in Figure 4. The presented data opposes a manual installation process to a partly mechanized and automated installation process. The comparison is based on 15 m long pipes connected with standard thread connections and installed with a conventional drilling machine with 2 drill arms and one loading basket. Obviously, 40% of installation time can be saved when using a state-of-the-Art AT - Automation Unit on each boom. When comparing the equipment utilization and critical paths, the loading basket becomes critical for manual installtion, while the automated installation indicates all three positions (basket, left and right drill arm) are nearly equal regarding the time-critical path.

5. OPTIMIZATION POTENTIAL REGARDING SAFETY

During manual installation processes, which are still a standard procedure worldwide, two steps must be emphasized regarding safety: uncoupling of drill steel and coupling of pipe umbrella pipes.

The manual un-coupling of drill steel is a working step with inherent risks because a steel wrench is fixed on the drill steel and afterwards the drifter counter rotates till the leverage is blocked by the drill arm. Only in this position the drifter is disconnected from the already installed drill steel. The critical point related to safety is that the wrench does not have any fixation on the drill arm or drill steel so in case of abrupt movements or in case the wrench is not fixed properly when the drifter rotates the wrench may slip or disengage and be ejected potentially damaging machinery or causing injury. This process can be defused using an AT – Threading Unit, which includes an open-end wrench that can be remote controlled to unscrew drill steel in a controlled way from the shank adapter or another drill steel. The same system is also integrated in the AT – Squeezing Unit as can be seen in figure 5. The wrench is simply activated on the remote control, moves by hydraulic cylinder upwards and fixes the already installed drill steel in its position. As soon as the drifter starts to counter rotate the disconnection starts. In this case, no persons must be close to the area where machinery parts rotate so the risk for injuries does not exist at this process.

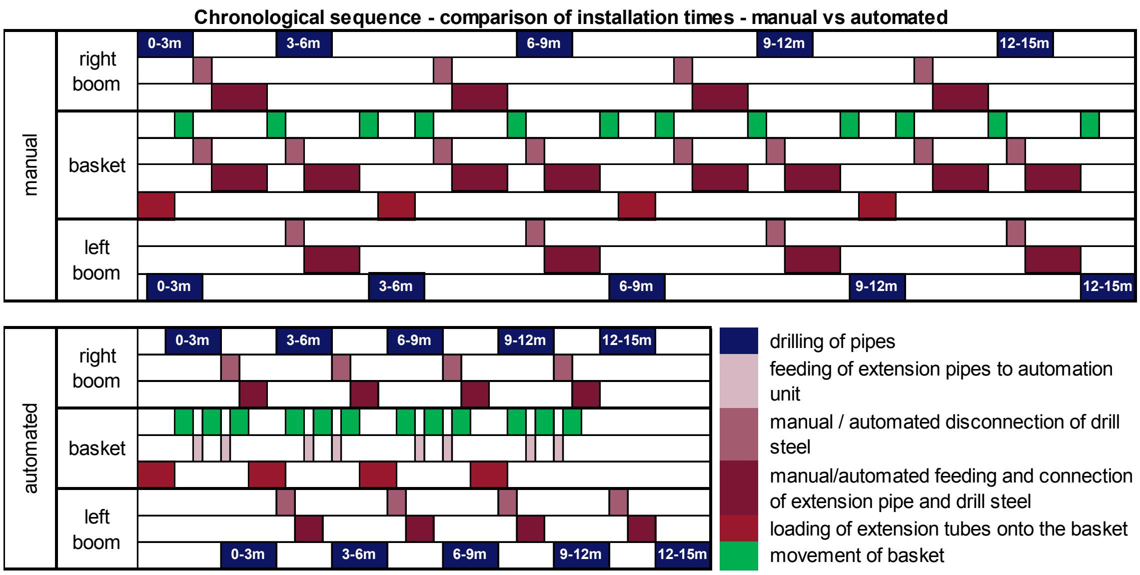

Figure 4 Chronical sequence – comparison of installation times – manual vs. automated [12]

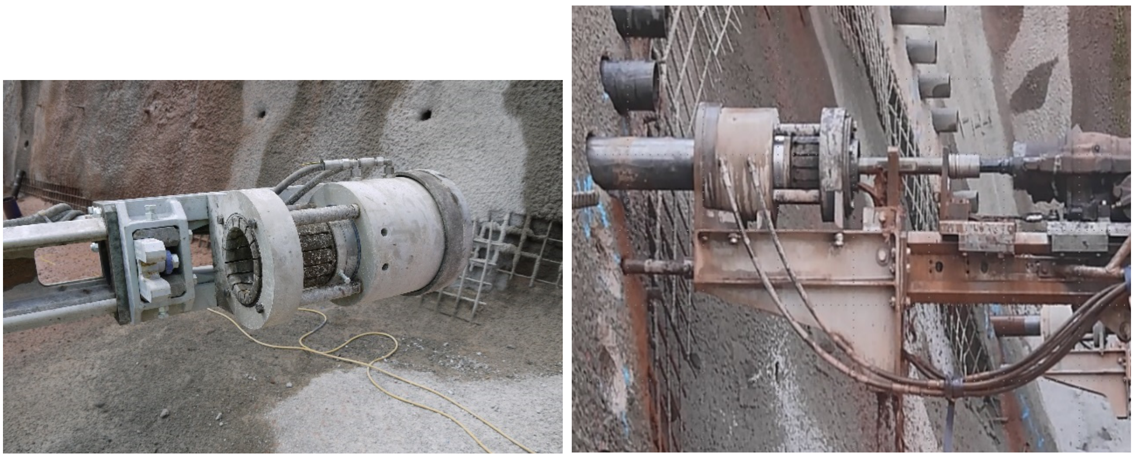

Figure 5 AT – Squeezing Unit with remote-controlled hydraulic wrench.

During the pipe coupling process with a standard thread connection, a worker situated in the basket moves the front end of a new pipe to a rear end of an already installed one. The two threads must be clean when the pipe with the outside thread is slightly moved into the inside thread. By rotating the pipe manually and then with the aid of a chain pipe wrench the pipe ends are connected to each other. Dirt in the threads or any movement of the drill arm during the entire installation process can lead to problems during connection because the threads get jammed, requiring manual loosening, cleaning, and repetition of the entire process resulting in more exposure. When using a squeezed connection without an AT – Automation Unit, a worker in the basket moves the reduced pipe end into the already installed pipe end. After moving himself away from the squeezing unit, he gives the signal for squeezing and the hydraulic cold forming process is activated by remote control. During the connection process, no rotational parts are exposed so there is less risk to harm workers and the connection process needs only about two seconds. The tolerances are large enough that dirt or small deviations between the drill arm and installed pipe does not interfere the coupling process so no more intervention is required.

A pipe umbrella installation process comprises many different installation steps. These steps are performed on the ground, in the basket, and with both drill arms. The workers need to be skilled and well-rehearsed, the number of workers at the face must be selected carefully, otherwise installation times and costs increases tremendously. As an example: A 3 m long pipe umbrella pipe with the dimension 139.7x8.0 has a weight of 80 kg and the necessary drill steel another 35 kg. So, when workers feed these two pieces from the basket to the drill arm they handle and position about 115 kg manually at a height of 6-8 m with limited headroom.

Table 5 Installation of an extension tube and drill steel. manually vs. automated

Installation step	Manual installation	Automated installation
Pipe is loaded onto the basket (mostly tube deposits)	Manually, workers on the ground	
Drill steel is loaded on the basket into the pipe	Manually, workers on the ground	
Disconnection of drill steel	Manually with wrench	Remote controlled with hydraulic wrench
Pipe is fed to loading device	n.a.	By moving the basket plus remote control
Pipe is fed to drill arm	Manually lifted from basket to drill arm	Remote controlled from hydraulic arms to drill arm
Re-connection of drill steel	Manually	Remote controlled
Connection of steel pipes	Manually with aid of chain pipe wrench	Remote controlled with threading unit or squeezing unit
Drilling	Mechanized by drilling machine	

The procedure analysis for installing an extension pipe clearly shows that heavy pieces must be moved from and to unusual positions during manual installation (table 5). Doing this work over a shift clearly leads fatigue and exhaustion resulting in inefficiency and a higher risk for injuries. Today, such a manual process is not necessary as can be seen in the second column. The automated installation shows that all processes after moving the pipe plus the drill steel onto the basket can be mechanized and remote controlled. Resulting in significant advantages in both time and safety during the process as well as in subsequent tasks due to lower physical and mental fatigue.

The required grade of mechanization or even automation must carefully be defined for each project depending on regional regulations. When discussing automation units, safety issues are the main concern but other advantages as shown in figure 4 can be achieved as well.

6. CONCLUSION

A large number of circumstances in tunnelling call for additional pre-support measures to supplement the support concept. The Pipe Umbrella System or also called Canopy Tube System is one of the increasingly been used methods of pre-support because its application area starts at relatively hard ground conditions and is only limited by flowing or ravelling ground conditions. The system is very flexible and adaptable to changing situations and can be installed by commonly used drilling machinery. Its optimization potential is very often not used in worldwide tunnelling therefore the discussed points are highlighted in the following.

Simple Changes in the way of installing pipe umbrella systems like 18 m instead of 12 m long pipe umbrellas or alternative coupling types may have a major impact on material consumption and installation time. Safety is an important issue as well so in the following the main optimization points are highlighted for an exemplary 100m long pipe umbrella supported tunnel (pipe dimension 139.7x6.3).

- 17.8 tons of steel tube savings when installing 18 m instead of 12 m long pipe umbrella pipes.
- 58 tons of steel tube savings when utilizing a squeezed coupling instead of thread connections.

- 36.7 hrs of installation time when installing 18 m instead of 12 m long pipe umbrella pipes.
- 62.5 hrs of installation time when utilizing a squeezed coupling instead of thread connections.

- Significantly lower risk of injuries or damages when using AT - Threading Units or AT - Squeezing Units to uncouple drill steel or elongate steel tubes.

- Less fatigue and exhaustion, higher efficiency, and less risk for injuries by using an adequate grade of mechanization or even automation.

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