New Evaluation Methods in Pipe Roof Supported Tunnels and its Influence on Design during Construction

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ABSTRACT: Pipe roof support systems are becoming increasing popular as cost and installation times have decreased with improved technology. The design of these systems is still based primarily on experience and the system behavior is still not well understood. Horizontal chain inclinometers were applied at a shallow tunnel in weak rock to provide a quasi real-time feedback of the pipe roof performance aimed at optimizing the excavation and support process to minimize surface displacements. The evaluation methods utilized, what they show, and how they were able to contribute to the adaptive design process are briefly explained in the paper. This system shows very promising results for monitoring and evaluating pipe roof supported tunnels.

1 INTRODUCTION

The Trojane tunnel was constructed in Carboniferous, tectonically sheared rocks, characterized by numerous joints, shear zones, and a complex schistosity. In critical sections above the tunnel including a small village, the current road connecting Ljubljana and Celje, as well as a natural gas pipeline the surface settlements had to be kept within predefined limits.

During the construction of the northern (right) tunnel tube under the village of Trojane (Figure 1), the vertical displacements of surface structures were measured and were within the allowable limits, between 4 cm to 6 cm and only in rare cases reached 8 cm. Differential displacements were 1:800 or less.

Geotechnical conditions in the southern (left) tunnel tube were much less favorable for the construction. Individual shear zones typically exceeded 50 cm in thickness and separated materials with contrasting deformability. As the tunnel approached the village of Trojane, the monitored settlement characteristics were different than observed during the excavation of the north tube; with the influence of tunnel excavation extending over a broader region and increased settlements. Due to these observations, the construction method within the weak rock category (SCC2) was modified. The monitoring program was also modified to provide a more comprehensive overview of the displacement field both on the surface and at the tunnel level. The major changes included monitoring the 3D displacements of the surface points and the use of a horizontal inclinometer installed just above the tunnel level to measure the settlements above the crown both ahead of and behind the face position. The use of the horizontal inclinometer started after a design review initiated when one of the surface structures (Building C28) developed significant cracking. It was later found that this damage result from an inadequate foundation, Likar & Jovičić (2004). This inclinometer system was used for an 80 m section which passed beneath a popular tourist restaurant.

This paper addresses the evaluation methods utilized with the horizontal inclinometer to understand the behavior of the pipe roof support system and how the data was used to modify the construction method during the excavation.

2 PROJECT OVERVIEW

The excavation of the 2.900 m long twin Trojane tunnels, located on the motorway section AC A10 Ljubljana-Celje (Slovenia), was recently completed. The tunnel diameter was approximately 11 m and was constructed using principles of the New Austrian Tunneling Method (NATM). The excavation was advanced with four heading utilizing a conventional excavator. Each advance was performed in multiple phases (max. 9) depending on the rock mass conditions and settlement restrictions. The primary support was installed immediately after the excavation of each phase, which consisted mainly of steel ribs, wire mesh reinforced shotcrete and rock bolts.

The excavation advancing from the east portal in both tunnel tubes encountered the most demanding section, the Trojane village. Difficult ground conditions, low overburden and the presence of the urban development above the tunnels all congregated at this particular section. A comprehensive monitoring program was designed and included displacement measurements of roads, buildings, pipelines, electric cable towers and other communal infrastructure. Tunnel deformations were monitored optically and extensometers and measuring anchors were installed in selected locations. This program enabled the surface and subsurface deformation field caused by tunnelling to be evaluated and allowed appropriate changes to the construction to be made to meet project requirements.

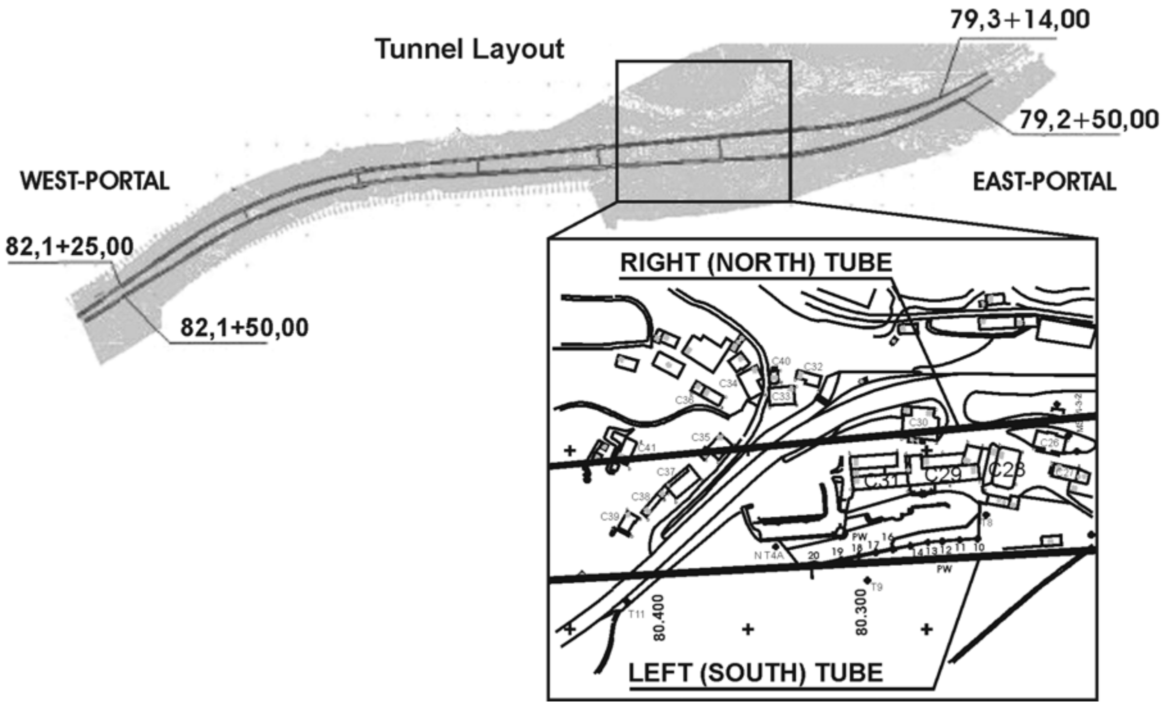


Figure 1. Layout of the Trojane Tunnel highlighting the discussed section in Trojane village

The geological conditions encountered during the construction were dominated by a meta-sediment sequence including mudstone, claystone and sandstone. The rock mass was affected by tectonic deformations associated with alpine thrusting resulting in a dismembered formation, Raymond (1984), characterized by sudden and frequent transitions between lithological units, which were often highly sheared (resulting in a schistose structure) and weak. Local shear zones separating the lithological units varied in thickness from a few centimeters to greater then 50 cm. These conditions result in potential instabilities and anisotropic deformation characteristics, which makes the excavation of the tunnel particularly difficult.

3 MEASUREMENT SYSTEMS

The surface and absolute tunnel displacements were measured geodetically. Originally, the surface settlements were measured on all structures in the vicinity of the tunnel, along the tunnel axis (10 m intervals), and perpendicular to the axis every 20 m (extending between 20 m and 30 m to either side in 5 m intervals). Due to the observed change in behavior in the discussed section the total surface displacements were measured instead of just the settlements, the interval along the axis was decreased to 5 m, while the cross sections remained the same. Tunnel displacements were measured at measuring sections every 10 m with 7 targets per section. During the construction the measurement frequency increased to twice per day both in the tunnel and on the surface.

To evaluate the utilization of the pipe roof system, an in-place horizontal chain inclinometer was installed at the beginning of each pipe roof field, Volkmann (2003). Ten 2 meter long inclinometers were used to provide a monitoring length of 20 m. The inclinations were measured at 1 minute intervals and processed twice a day to determine settlement characteristics. Additionally, the real time display at the data logger allowed the stabilization characteristics to be observed during the excavation and support cycles. In order to track the absolute position of the inclinometer the onset of the chain inclinometer was measured during the regular geodetic survey. This allows the total settlements to be determined instead of just the relative settlements. The absolute magnitude of the measured settlements is limited to the geodetic survey accuracy, however the relative settlements along the inclinometer string were found to be accurate to within 0.02 mm after processing.

Figure 2a shows a typical time settlement line recorded at a single inclinometer chain for one excavation step. The time at which each phase began is highlighted. This display method allows the settlements at each inclinometer to be compared as well as evaluating the settlement and stabilization process of the different excavation and support phases. Figure 2b shows a deflection curve diagram for one excavation step. The new face position is marked with the solid line, the position of the latest support ring is marked with the dotted line, and the relationship between the excavation location and the pipe roof fields are also shown. This diagram allows the distribution of the settlements both ahead of and above the excavation to be evaluated for each phase of the excavation process. The influence of the support activities are included in the phase three settlements, but can also be displayed as single deflection lines if necessary.

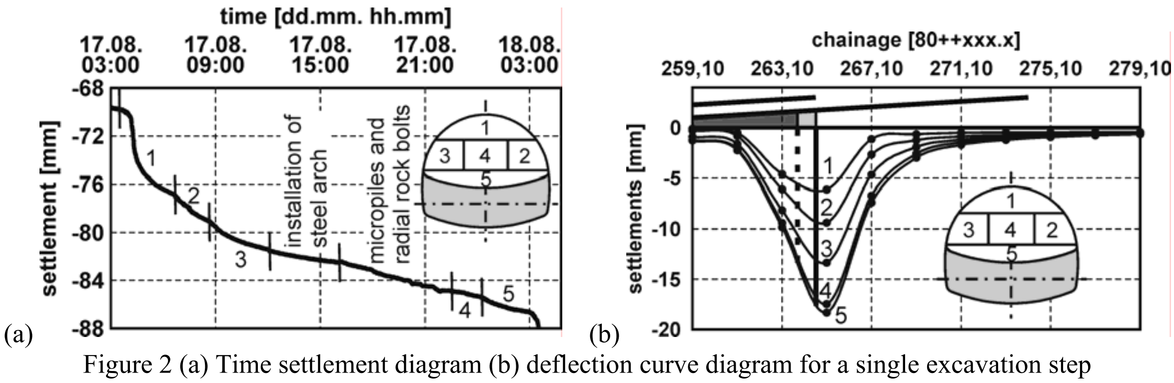


Figure 2 (a) Time settlement diagram (b) deflection curve diagram for a single excavation step

Figure 3 shows an example of deflection curves developed from the inclinometer data. In this example the first 10 excavation steps in this measuring section are shown. The excavation steps from the previous inclinometer position are not shown. In order to develop the deflection lines for the shown inclinometer it was necessary to extrapolate the initial settlements from measured surface settlements and observed settlement trends. However, if two inclinometer chains are used with a correct overlapping length the total settlement path can be measured. The displacements occurring ahead of the face and those occurring within the excavation are separated by the face position trend line. To assist in predicting changes in the rock mass and thus the system behavior, trend lines at varying distances ahead of the face are constructed by connecting the accumulated settlements at predetermined distances from the face position, in this example the trend line 3.2 m ahead of the face position is shown. Settlements between measurement points were extrapolated using a spline function. This diagram can be separated into two sections for evaluations. First, the development of the cumulative settlements ahead of the face position along the tunnel axis can be observed. Second, the total settlement profile along the tunnel axis is determined.

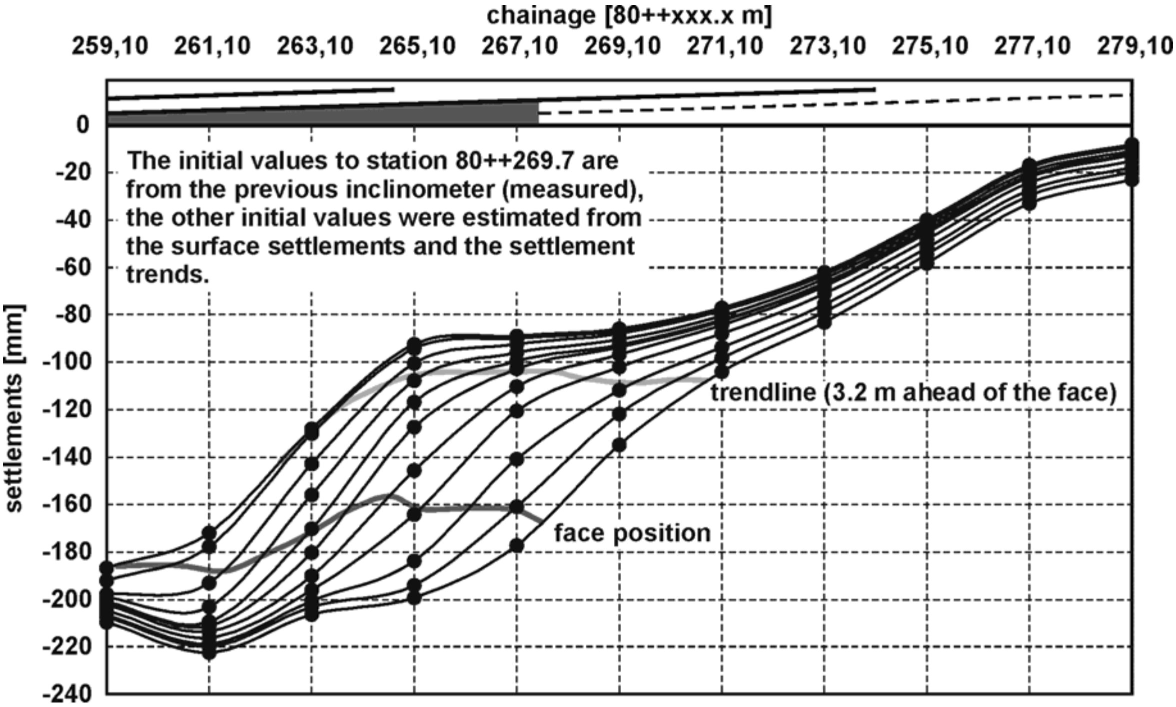


Figure 3. Total settlement deflection curves developed from the inclinometer measurements

4 ADDITIONAL EVALUTION POSSIBILITIES

The high quality of the inclinometer measurements allows for very precise evaluations of the settlements induced during the excavation and support processes. Having measurements every minute, spaced at 2 m intervals and correlating the beginning and ending times of the construction process it was possible to determine how each perturbation to the system influenced the observed settlement characteristics. For example, it was observed that the drilling process, whether for rock bolts or for the pipe roof system, influenced the settlement characteristics. In the time of these installations the settlements increased depending on the rock mass conditions (Figure 2a). These are very important considerations when trying to back calculate material properties from deformations when the actual construction process can not be adequately taken into consideration.

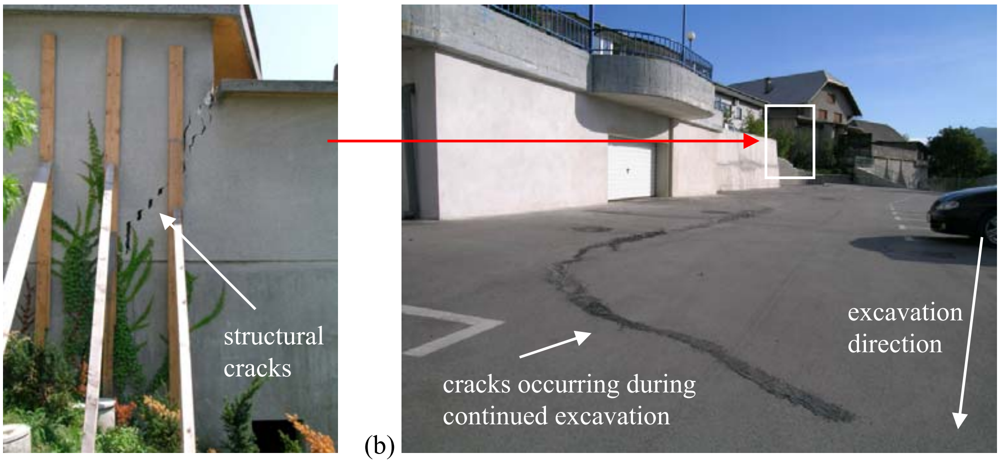
Evaluating the time settlement lines allowed the time dependent aspects of the stabilization process to be directly observed and correlated to the local rock mass conditions that were observed at each location. When the excavation process stopped for extended periods of time and no construction activities were taking place it is possible to determine if the rock mass is prone to creep or if the observed settlements with time are due to the excavation process at considerable distances from the face position and the gradual load transfer through the lining.

One of the most hazardous situations in shallow tunneling involves face instabilities and their potential to expand to a global failure and extend to the surface. It was found that the data allows the differentiation between local face stability and face stability induced by a more global failure in the ground by comparing the distribution and time characteristics of the settlements before, during and after local failures. This is not possible with daily measurements within the tunnel that are limited by the installed support or on the surface where there may be time delays in the ground response.

It is also possible to compare the measured surface deformation and settlement at the tunnel level in both time and space. This can be used to evaluate how quickly deformations travel through the overburden when major changes in the deformation characteristics are observed.

5 GEOTECHNICAL SUPPORT ADAPTATION

As the southern excavation approached the critical section under the village of Trojane some of the building began to show initial signs of distress. During continued excavation the damage continued to increase resulting in an adaptive increase in the primary support methods. The strongest primary lining was introduced at the most critical section that began immediately below the building C28 (Figure 1), which suffered the most extensive damage. Figure 4a shows some of the damage to this building, which occurred very rapidly, while Figure 4b shows cracking in an adjacent parking lot which occurred very gradually with the continued excavation, the contour of the cracking followed the old topography where the fill material for the parking lot was thicker.



(a) (b)

Figure 4 (a) Damage to the side of building C28 showing the cracking due to differential foundation settlement (b) cracking in the adjacent parking lot following original topography

The primary support was reinforced in several ways in rock class SCC2. The excavation and support step was reduced to 0.8 m. The top heading face was reinforced with 35 15 m long IBO rock bolts with a capacity of 250 kN. Additionally, 15 13 m long fibreglass bolts with a capacity of 250 kN were added at the perimeter of the top heading excavation. The overlapping for the face anchoring was 5 m. The tunnel face was supported immediately after each excavation phase with a 15 cm thick shotcrete layer. Two sets of I-shaped steel segments, type IPE 180, with elephant foot bases were installed for each step. In total there were 24 250 kN IBO rock bolts, applied radially per excavation step. They were 6 m long on the left side and 9 m long on the right side for optimization in the anisotropic rock mass. A pipe roof was used, one row of the pipes, 114 mm in diameter and 15 m in length. The primary shotcrete lining (MB25) had one layer of mesh reinforcement and a thickness of 35 cm. Micropiles, 64 mm in diameter and 6 m long were installed at sides under the elephant foot foundations to help distribute the load of the top heading lining. The technological sequence included closing of the lining after the excavation of the top heading with a temporary invert and the introduction of the final shotcrete lining at the invert after the excavation of the bench. These measures effectively lowered the settlements, which primarily occurred only during the excavation stages, to about 140 mm at the crown.

Once the excavation restarted with the inclinometer monitoring it was observed that when the radial rock bolts were drilled the settlement characteristics changed. To investigate the utilization of the rock bolts measuring anchors were installed and it was found that they were not contributing significantly to the reinforcement; it was therefore decided to eliminate the rock bolts. The roof stiffness ahead the excavation face and above the lining was produced with overlapping injected steel pipes (normally one or in critical regions two layers depending on the results of the measurement program). Initially the overlapping length was 5 m this was increased twice, first to 7 m then to 8.4 m due to observations that the settlements increased whenever the excavation passed the overlapping region. The final overlapping ensured the excavation was always beneath two rows of pipes. With this support system the stiffness and bearing capacity of the primary lining was enough to prevent large displacements in the tunnel, which not exceed 6 cm or max. 8 cm.

The bench and invert excavation followed the top heading at a distance of between 10 m and 20 m with excavation steps of 0.8 m. This procedure was important so we did not allow the rock mass to relax during the final ring closure with the primary lining. Tunnel displacements during these phases were typically less then 1 cm, indicating the effectiveness of the top heading support and the rapid ring closure during the bench-invert excavations.

6 CONCLUSION

Shallow tunneling in urban environments often includes restrictions for the induced surface settlement magnitude. Pipe roof support systems are increasingly being used in conventional tunneling to reduce surface settlements and increase face stability in difficult ground conditions. Horizontal chain inclinometers installed with the pipe roof provide a tool to evaluate the performance of these systems. It was shown that different display and evaluation methods can be used to understand the settlement behavior both ahead of and behind the tunnel face. This information can then be used by the designer to optimize the support system for the encountered ground conditions and the project requirements. Additionally, this data can be used to understand the rock mass and system behavior during tunneling reducing the potential for unexpected events.

ACKNOWLEDGEMENT

The authors would like to thank D.A.R.S. (Slovenian Highway Department of Construction) for the cooperation during this project and the permission to publish the results of this campaign.

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