Shallow Tunneling in a Tectonic Mélange : Rock Mass Characterization and Data Interpretation

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ABSTRACT:

The characterization of complex rock masses, such as fault zones and tectonic mélanges, for tunneling requires non-traditional techniques due to the extremely heterogeneous nature of both the ground conditions and the rock mass behavior. By using a consistent characterization process recently introduced in Austria the basic key parameters for this type of rock mass have been defined. Examples of complex behavior are given using the state-of-the-art techniques in displacement monitoring and data evaluation from a recently completed shallow tunnel in the Austrian Alps.

1. INTRODUCTION

When tunneling in difficult ground conditions many different types of problems may arise during the excavation. Common problems typically range from consistent overbreak, excessive deformations, severe water inflows, or in extreme cases large overbreaks, or a complete tunnel collapse. For the tunnel engineer to properly respond to a critical situation he must be able to understand and predict the rock mass behavior and how this behavior interacts with the designed excavation and support system.

Typically during tunnel design, the intact rock properties and joint behavior are determined in the laboratory, while the rock mass characteristics are determined from field investigations and subsurface exploration programs. This information is then evaluated to determine a homogeneous rock mass strength and the support and excavation methods are developed using various classification schemes (Q, RMR,GSI, etc) [1,2,3,4]. These results are commonly supported with numerical simulations to evaluate the ground response and/or support loads for different support and excavation methods. One of the key items that is missing in many classification methods is the connection between the tunnel behavior, the rock mass behavior, and the potential failure mechanisms.

The geologic characterization, as well as classification of mélange rock masses is not uniquely defined and is still debated today. Mélanges commonly occur in accretionary complexes and in other tectonic settings such as foreland basins or fault zones [5]. Typical shear zone mélanges, the type encountered during this project, contain an apparent chaotic structure but when fully analyzed the structural relationships become more clear with respect to multiple deformation phases in the brittle regime [6]. These multiply deformed rock masses can be called polygenetic mélanges after Raymond [7]. As a generality, the term block-in-matrix or bimrock as defined by Medley [7,8] will be used throughout this paper to discuss mélange type rock masses.

This paper briefly describes a rock mass characterization procedure recently introduced in Austria [9,10] and its application using a case history from a shallow tunnel recently excavated in a tectonic mélange. Then the observed displacements for a section of the tunnel are discussed considering the encountered geological conditions and advanced data evaluation techniques [11]. By combining these data short term prediction and support optimization can be achieved.

2. ROCK MASS CHARACTERIZATION

One of the most difficult subjects in rock engineering is estimating the rock mass behavior. Schubert et.al. [9] introduced a procedure for developing the rock mass model beginning with the feasibility study and continuing through the construction of the project. This procedure was incorporated into a new guideline published by the Austrian Society of Geomechanics in October 2001 [10]. Figure 1 shows a flow chart describing a systematic procedure for developing the excavation and support design for tunnels beginning with rock mass characterization.

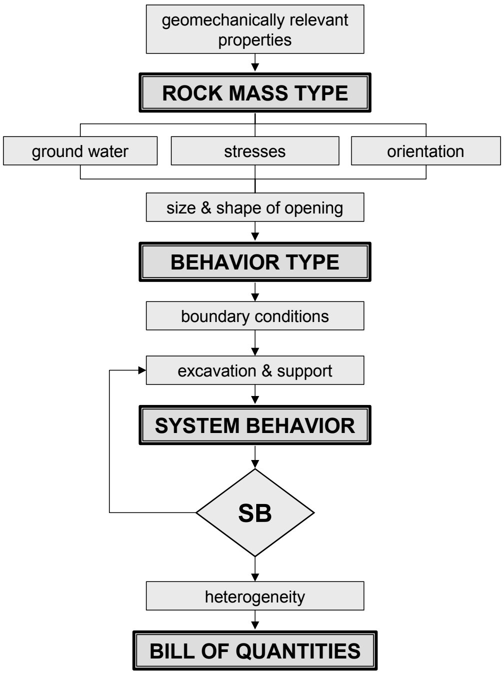

Figure 1. Flow chart outlining the basic procedure for excavation and support design for tunnels. After [9].

2.1. Rock Mass Types

The first step involves defining rock mass types (RMT's). Rock mass types are based on lithology and discontinuity structure quantified by field observations and laboratory data and should convey geomechanically relevant information to the engineer. This is done by defining key parameters for each rock type that are easy to obtain and have the greatest influence on the rock mass behavior [12]. For example, in a bimrock key rock mass parameters include the block size and volumetric proportion, the relative strength and stiffness contrasts between the blocks and the matrix, dominant orientation of the matrix foliation. The key parameters should be represented with distributions to account for the spatial variability associated with most rock masses.

Rock mass types should be evaluated on a project specific basis even though many characteristics may be similar to conditions in other projects. Prior experience should assist in defining the RMT's and key parameters, but should not govern the site-specific assessment. For example, Liu et.al. [13] introduced an algorithm to quantify parameter relationship in tunneling to the data base system DEST (Data Evaluation System for Tunneling) [14,15]. This procedure allows statistical analyses to be performed on key parameters obtained during an excavation, the utilized excavation and support methods, and the observed tunnel behavior.

2.2. Rock Mass Behavior Types

Once the RMT's are defined the next step is to define rock mass behavior types (BT's). The behavior types are developed by considering the specific site conditions such as the initial and induced stress state, relative orientation between discontinuities and the excavation, groundwater conditions, and the shape and size of the opening and how these factors interact with the different rock mass types.

The BT's are developed by assessing the potential behavior, failure modes, and approximate displacements under the expected site conditions for an unsupported excavation. In our case study, behavior types are related to the proportion of observed blocky material in the excavation, the relative material strengths related to the stress conditions, the location of the stronger blocky material related to the excavation perimeter, and the effect of ground water on the material behavior.

To our knowledge no detailed work has been published on the effects of blocks on the rock mass behavior for underground excavations. It is suggested that the minimum and maximum block sizes be defined as 5% and 75% of the characteristic engineering dimension, respectively [8,16]. This would result in a definition of a minimum block size range of 50 cm to 7.5 m for a 10 m diameter tunnel. More research needs to be performed to confirm these relationships for the underground behavior.

Current research and data evaluation of tunnels in highly heterogeneous rock mass conditions indicate that the location of the blocky material plays a key role in the rock mass behavior. That the displacement magnitude generally increases with decreasing block volumetric proportion. However, this is not a direct relationship because the location of the blocks is very important to the resulting behavior. This will be discussed in section 3.6.3 using the case history

Blocks outside the excavation can have a significant influence on the excavation as most of the rock masses long term support capacity is located outside the excavation while blocks within the excavation perimeter are more important for the three dimensional effects associated with the advancing tunnel face, but do not add to the long term behavior of the excavation.

A severe problem can occur when very large more competent blocks act as aquifers. Blocks typically have a larger fracture permeability and storativity then the finer grained matrix. This can create significant water and seepage forces between the blocks, matrix, and the excavation. When the block is located just outside of the excavation this creates a high potential for a water pressure induced failure of the weaker matrix. This behavior can be considered one of the most critical situations and is often associated with more severe overbreaks or top heading collapses [17].

To evaluate the behavior types during the design phases of a project analytical solutions and numerical simulations are used to supplement experience and physical models to gain a better understanding of the potential failure modes. Descriptions of basic behavior types can be found throughout the literature [1,3,9] It must be stressed that the rock mass behavior must be representative of the unsupported rock mass and not consider the excavation or support methods.

2.3. System Behavior

The final step in the geotechnical design process is to define the system behavior (SB). The system behavior results from the interaction between the excavation and support methods and the rock mass behavior. Common methods to evaluate the system behavior include analytical methods proposed by Feder [18,19] or Hoek [4], and numerical simulations. Different excavation and support systems are evaluated for the project defined BT's and the results compared to the required project goals (limited surface settlements, maximum allowable deformations, etc.). If the project requirements are achieved, then the acceptable support method is further put through cost benefit analyses with other suitable excavation and support methods and a final design chosen for a given BT.

During construction it is necessary to compare the actual to the predicted tunnel behavior. Geologic face mapping is used to continuously update the rock mass model. This is combined with the evaluation of the monitoring data to further update the BT's related to the observed SB. Figure 2 shows a procedure to update the rock mass model during excavation. By combining these analyses the appropriate support scheme for the next excavation sequence can be chosen from the design options.

This method of analyses requires detailed knowledge of the current system behavior. A newly developed software package developed by Sellner "GeoFit" [20,21,22] can be used during this phase to rapidly evaluate and predict the effect of different support capacities and excavation sequences on the SB. This allows the appropriate excavation and support methods to be chosen and implemented in the forthcoming excavation steps.

2.4. Displacement Monitoring

One major difficulty in tunneling is to interpret the SB. In order to evaluate the utilization of the chosen support, systematic evaluations of the tunnel behavior have to be performed. The results from traditional monitoring techniques (convergence tapes, inclinometers, extensometers, etc.) are difficult to interpret when they are installed from inside the tunnel because data is lost depending on the installation time, and in most cases they supply only relative displacements. Additionally, they are typically only used in selected locations rendering a true evaluation of the tunnel behavior practically impossible.

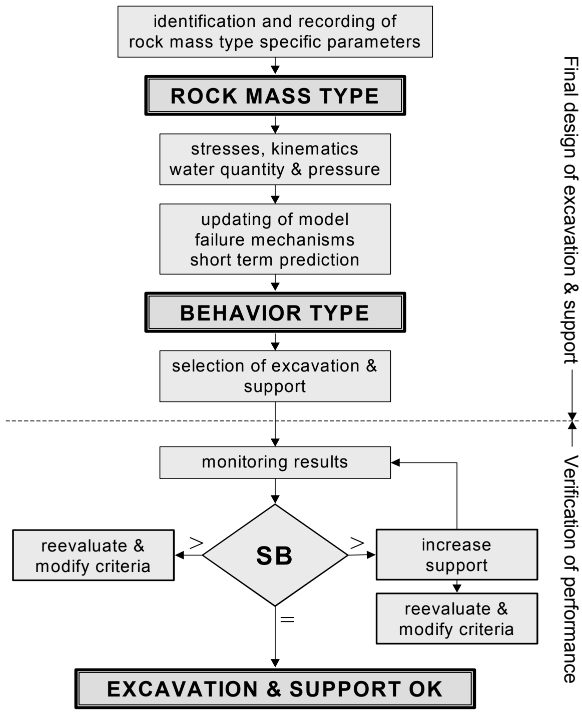

Figure 2. Flow chart of basic procedure for excavation and support selection and verification during construction. After [9]

Rabensteiner [23] discusses the advanced monitoring systems that have become common practice in Austria and are becoming increasingly popular around the world. Using modern surveying equipment, absolute tunnel displacements are monitored at approximately 10 m intervals, allowing the three dimensional tunnel deformations to be quantified. Rokahr [24] introduced a method to calculate the stress intensity in a shotcrete lining by evaluating the displacement measurements, and discusses common problems associated with the interpretation of the results [25].

This data has been used extensively in Austria to determine trends and relationships between different rock mass conditions and the resulting tunnel deformations [26,27,28,29]. By using this type of data, the SB which is directly observed and quantified can be used to perform a detailed evaluation of the rock mass behavior. This information develops the basis for the proposed procedure of excavation and support determination. The state-of-the-art in data evaluation methods are discussed in section 3 using a case history from a shallow tunnel in Austria.

2.5. *Numerical Simulations*

In order to evaluate the SB during design phases numerical simulations are increasingly been performed. The simulations often use simplified rock mass models to estimate the SB. One of the most useful applications of numerical models is the investigation of the SB with very simple heterogeneous rock mass conditions. For example, the effects of a single fault zone crossing the tunnel with different thickness' or orientations, as well as with different deformability compared to the "undisturbed" rock.

Grossauer [30] investigated these relationships supplementing investigations by Steindorfer [28] and Golser [29,31] on the effect of fault zone characteristics on the SB using the 3-D numerical modeling program Boundary Element Finite Element (BEFE) [32]. Elastic material properties were used for all of the calculations to acquire a general feeling for the behavior of the system. Figure 3 shows the effect of different stiffness ratio's on the stresses, displacements, and the displacement vector orientation when encountering a soft zone.

Pustow [33] performed several evaluations to study the stress redistributions in a bimrock. Again the program BEFE was used with elastic material behavior. Figure 4 shows how a stress concentration occurs in the stiffer blocks immediately next to the excavation that could result in overstressing and failure of the intact material.

Even though these calculations used very simple material behavior the general phenomenon is represented very well. This information can be used to help understand the complex stress redistributions that occur during the excavation in heterogeneous rock mass conditions.

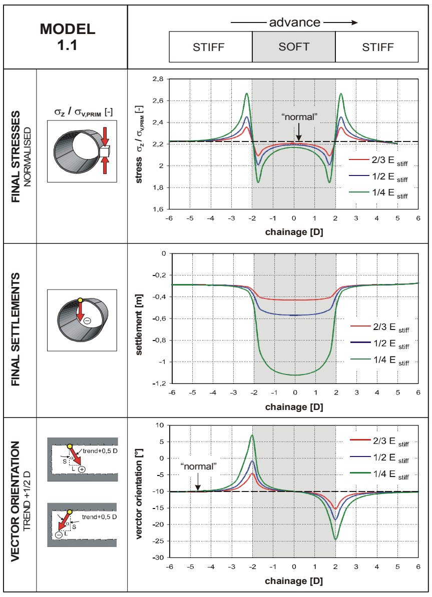

Figure 3. The effect of tunneling through a soft zone with different stiffness ratio's on the stresses, displacements, and vector orientation. After [30].

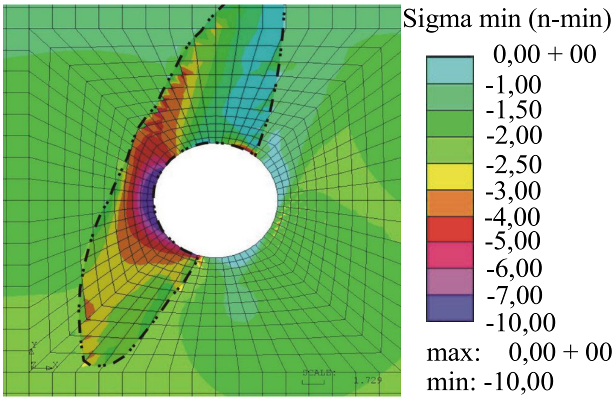

Figure 4. Stress Concentration in a stiff block intersected by the tunnel excavation. After [33]

3. CASE HISTORY: TUNNEL SPITAL

3.1. *Location*

The tunnel Spital is one of three tunnels designed for the modernization of the S-6 motorway through the Semmering region in Eastern Austria. The tunnel is 2.5 km long and consists of two 10 m diameter tubes separated by approximately 50 m over a majority of the length. The first 2044 m were excavated using the NATM and the remaining 456 m were constructed using cut and cover techniques. The overburden averages 50 m with a maximum of approximately 90 m.

3.2. *Site Geology*

The tunnel is located within a unit of the "Semmering-Unterostalpine" geologic sequence. The geological situation is characterized by a tectonic mixture (bimrock) composed of predominantly quartzitic, carbonatic, chloritic phyllites and gouge material with embedded blocks

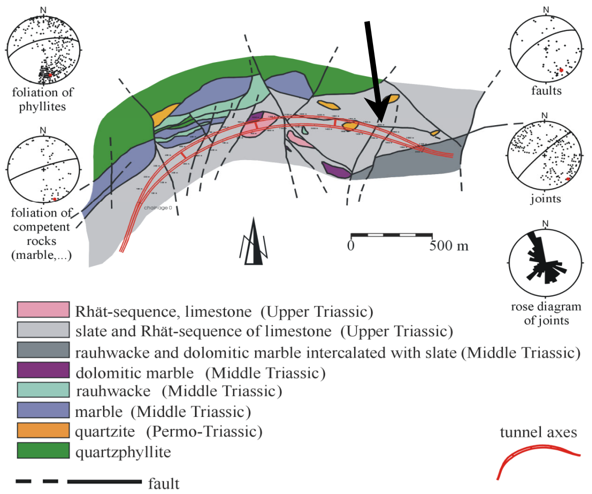

Figure 5. Simplified geologic map. The arrow points to the region discussed in the following sections. After [33].

of marble and quartzite. Figure 5 shows a simplified geologic map of the tunnel alignment. Significant block sizes range from several meters to more then 500 m in length. This chaotic tectonic mixture was generated from the surrounding host rocks during thrusting associated with the alpine orogeny and was further disaggregated during the strike slip faulting. This bimrock can be defined as a polygenetic mélange after Raymond[7].

The mélange is dominated by a north to north east shallowly to moderately dipping foliation formed during thrusting. However, local folding around the blocks results in continuous variations in the local dip and dip direction. The blocks are typically lenticular in shape and orientated with their long axis sub-parallel to parallel to the foliation. The original structures formed during thrust tectonics are overprinted by brittle strike slip faulting along steeply dipping east-northeast trending strike slip faults associated with the Mur-Murztal fault zone. This combination of tectonic events has created a extremely heterogeneous rock mass exhibiting varying degrees of rock mass strength with a spatially complex distribution.

3.3. Laboratory Data

In order to quantify the relative strength parameters of the different materials several samples were tested in direct shear at the beginning of the excavation. During the initial site investigation the strengths of the phyllites were reported to be 28° with cohesion values around 100 to300 kPa. Subsequent testing of samples acquired during the excavation revealed a range from 16° to 26° for the phyllites and 28° to 35° for the blocky material. The major difference in the behavior was that the blocky material were much stiffer and consistently dilated while the phyllites were contractual. Figure 6 shows the results of two multi-failure state shear tests, one on a chlorite phyllite and the other on a quartzite block. The more brittle nature of quartzite is clearly seen in this diagram.

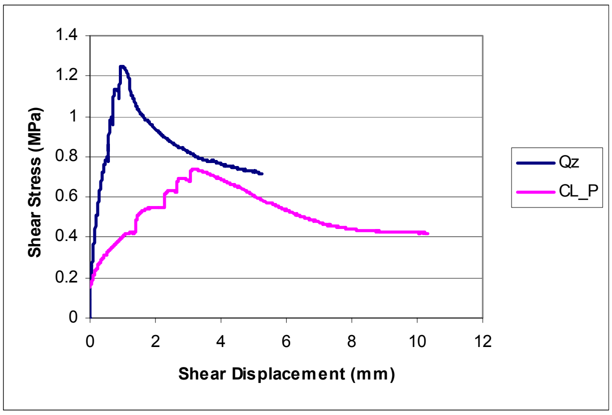

Figure 6. Comparison between the shear behavior of the blocky material and matrix. Multi-failure state shear test.

3.4. Data Evaluation and System Behavior

As mentioned in section 2.4, 3-D displacement measurements were routinely made during tunnel excavation. This allows for the absolute movements of the tunnel to be monitored instead of relative displacements as with other monitoring techniques. Techniques for data evaluation that are commonly used in practice have been routinely published since 1995 [26,27,28,29,30,31], and are continuously under development. The following examples comment on these techniques while a detailed summary can be found in [11].

3.5. Example Evaluation

The fault zone identified in Figure 5 influenced both excavations. Only the south excavation will be discussed for brevity. Figure 7 shows a horizontal and longitudinal cross section of the encountered geological conditions between stations 1700 m to 1800 m. Phyllites = P are shown in green and gray, quartzites =Q in orange and marble= M in blue.

3.6. Visualization Techniques

In order to display the monitored displacement data, several techniques have been developed that provide direct useful data when interpreted. The following data visualization techniques will be discussed:time histories

- displacement time histories
- displacement vector plots
- deflection and trend lines

3.6.1. Time Histories

A displacement time history is the simplest method of plotting the displacement data. For an individual

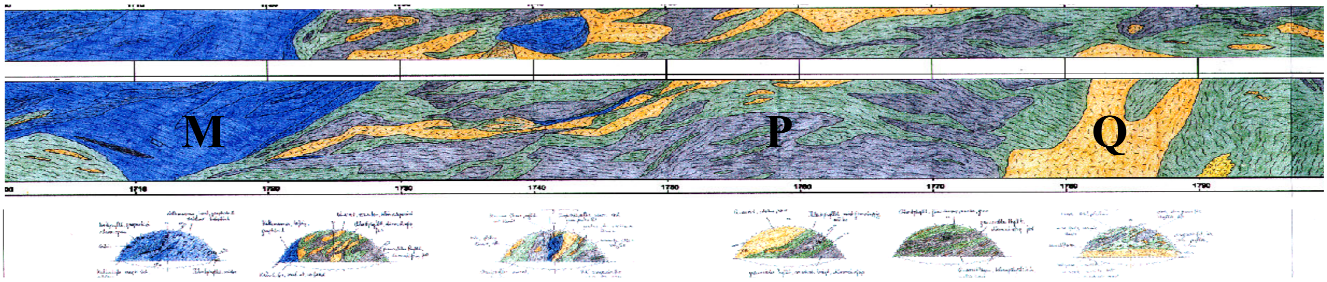

Figure 7. Documented Geology from station 1700 to 1800. Marble = M, Quartzite = Q, Phyllites =P.

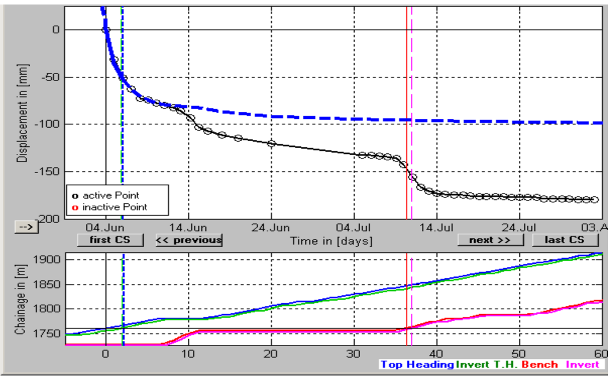

Figure 8. Displacement Time history (upper graph) for station 1761 m. The predicted displacements resulting from the top heading are shown with the dashed line. The excavation progress is plotted on the lower graph.

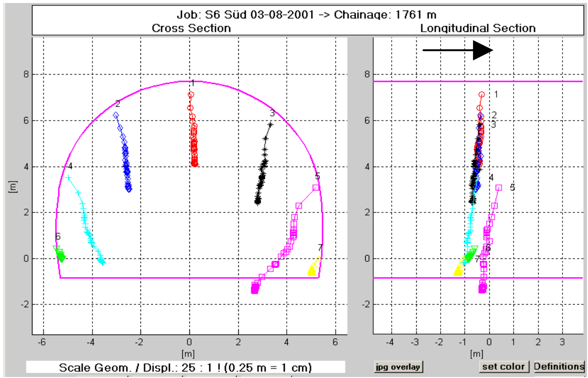

Figure 9. Displacement vector plots showing the horizontal and vertical components (left plot) and the vertical and longitudinal components, e.g. displacement vector orientation (right plot). The arrow shows the excavation direction.

measuring section the displacement magnitude is plotted versus the time. Figure 8 shows a time history for the vertical displacement of the lower left sidewall at station 1761. Each component of a single measurement point must be plotted separately with this display method. These diagrams allow the stabilization process to be observed, e.g. the displacement rates should continuously decrease.

It is common practice to plot the excavation phases at the bottom of the diagram to distinguish between naturally accelerating displacements (potential instability) and excavation induced displacements. Some applications allow one component of all of the measurement points to be shown on one diagram.

Figure 8 was generated to show the predicted displacements for the top heading excavation (blue dashed line) using the program GeoFit [22]. This display allows the influence of the bench excavation on the displacements in this plot to be distinguished more clearly.

3.6.2. Displacement Vector Plots

Displacement vector plots are a convenient way to show two of the displacement components as a single vector. Figure 9 again shows the displacements measured at station 1761.

This type of plot begins to allow the engineer to evaluate the system behavior as the displacement kinematics are clearly shown. This information is combined with the geologic data to evaluate the influence of the rock mass structure on the observed displacements.

The longitudinal cross section is used to plot the 2-D vector orientation. This is the ratio between the settlement and the longitudinal displacement. It has been shown [28] that in a given rock mass there is a “normal” orientation that is approximately 10° against the direction of the excavation, but can vary depending on the given site conditions.

When approaching a weaker zone the stiffer material cannot transfer the same amount of load ahead of the face causing an increase in the stress level directly ahead of the face, which subsequently causes the displacement vector orientation to point against the direction of the excavation. The opposite is true when approaching a softer material. These types of behavior were shown in Figure 3.The stiffer material absorbs more load and the softer material begins to relax resulting in the rotation of the vector orientation into the direction of the excavation. Comprehensive discussions of this phenomenon are given in [28,29,31].

3.6.3. Deflection and Trend lines

The previous two visualization methods only show the data for a single measurement section. In order to evaluate the tunnel behavior over larger distances it is common to plot deflection lines. Deflection lines are also called influence lines as they show the influence of subsequent excavation steps on sections already excavated. Deflection lines are developed by plotting the tunnel chainage on the horizontal axis and the measured displacements with time on the vertical axis. They can be used to plot the displacements along the tunnel axis at any given time or interval.

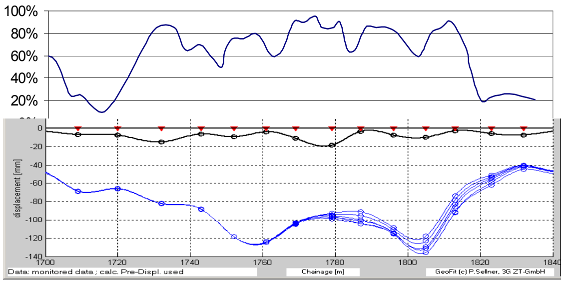

Figure 10. Deflection lines (crown point) showing stabilized displacements and creep in the heavily faulted material. The percent of matrix in the top heading from the observed geology is shown in the upper graph.

Figure 10 shows 5 days of vertical displacement for the crown together with the observed matrix percentage from the top heading excavation from station 1700 to 1840. Matrix material was defined as the various phyllites, while the blocky material was defined by quartzite and marble.

There is a good correlation between the increase in the displacements and the percentage of matrix in the top heading from station 1715 to 1750 and with the decrease in displacements between stations 1810 and 1830, but as mentioned earlier this factor alone does not control the displacement characteristics.

What is not indicated by the above plot is the location and size of the observed blocks. There was a large block located beneath the top heading excavation that began on the right side and extended just across the excavation. It was in the excavation for approximately 20 m from station 1770 m to 1790 m (the top of the block is shown in the horizontal cross section of Figure 6). This block reduced the initial displacements by providing a solid foundation for the installed support. When the block was removed by the bench excavation. This additional foundation was removed and the entire left side of the tunnel settled (Figure 11 upper graph). This was most profound where the entire block extended under the left support (station 1787), approximately 5 cm of additional displacement occurred the first day and 5 cm more over the next 16 days. This is compared to the right side where the support still rested on the block and no additional displacements occurred due to the bench excavation. This resulted in a rotation of the entire tunnel lining causing the shotcrete in the right springline to crack over a distance of 25 m.

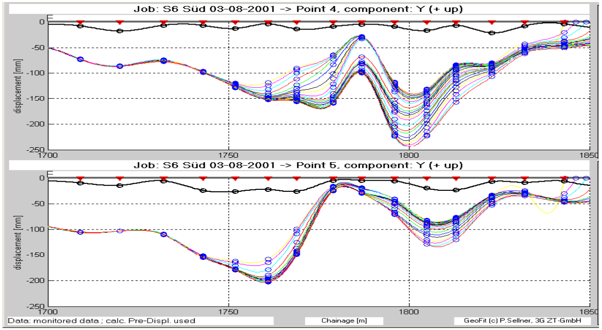

Figure 11. Deflection lines for the lower left(4) and right (5) side wall points showing typical heterogeneous displacements associated bimrocks.

Another way to use the deflection line plot is to show the changes in the displacement vector orientation along the tunnel axis. Figure 12 shows the displacement vector orientation for the lower left sidewall point developed 12 m behind the face.

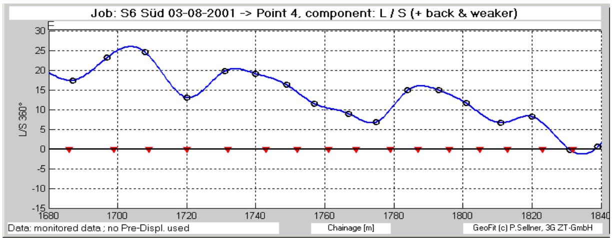

Figure 12. Displacement vector plot for lower right side wall point. Increases indicate softer material ahead of the excavation, while decreases indicate stiffer material.

The displacement vector orientation plot has been shown to be a possible tool for the short term prediction of rock mass conditions ahead of the excavation[28,30,31]. When the displacement vector changes orientation it indicates a change in the conditions ahead of the face. Increases against the excavation direction indicate softer conditions, while an increase in the excavation direction indicates stiffer condition ahead of the excavation.

It is important that the vector orientations for each point are inspected using trends developed at different distances from the face. Trends developed from shorter distances are more sensitive to small changes while larger distances tend to smooth the data.

4. CONCLUSIONS

Consistent data monitoring and evaluation during tunneling in difficult ground conditions allows the system behavior to be analyzed and short term prediction to be made. This allows the engineer to optimize the excavation and support methods for the encountered ground conditions, resulting in a more economic and safe excavation. This information is also used to analyze and compare the rock mass behavior to the observed rock mass conditions resulting in improved understanding of the factors governing the rock mass behavior.

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