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The Use of Monitoring Data and Geologic Documentation as a Basis for Defining Rock Mass Behavior Types for Tunnelling

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THE USE OF MONITORING DATA AND GEOLOGIC DOCUMENTATION AS A BASIS FOR DEFINING ROCK MASS BEHAVIOR TYPES FOR TUNNELLING

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ABSTRACT

The use of geodetic surveying to measure the absolute 3-D tunnel displacements has provided new opportunities to evaluate the system behavior and interpret the rock mass behavior associated with tunneling. For a meaningful case history evaluation it is necessary to have consistent and quality documentation covering the excavation and support sequence, the geological conditions, as well as the displacement measurements. Combing this data allows the rock mass behavior type to be evaluated. In contrast to many available rock mass characterization or classification procedures, the procedure introduced by the Austrian Society for Geomechanics within the Guideline for the Design and Construction of Conventional Tunnels, focuses on site specific evaluations of the rock mass types, potential rock mass behavior types considering the system boundary conditions and influencing factors, then determines the potential system behavior for different excavation and support methods. Using case histories provides valuable opportunities to develop a data base on rock mass behavior types associated with different environments and excavation and support systems. The examples discussed in this paper demonstrate this procedure can be used to identify key geologic parameters and associated behavior types.

INTRODUCTION

The development of modern rock mass classification or characterization schemes for underground structures was driven by the need to develop simplified, cost effective methods for assisting the tunnel designer in quantifying the ground conditions, and its potential behavior during excavation. It was stressed by Bieniawski (1989) that the rock mass classification systems were not intended to replace analytical or numerical studies, field observations, and measurements, nor engineering judgment, but were to serve as aids during design.

Many of the popular classification systems in use today including the Geological Strength Index (GSI) (Hoek (1994) Hoek and Brown (1997)), the Q-System (Barton et al. (1974) Barton and Grimstad (1994), Barton (2002)), and the RMR (Bieniawski (1973, 1979, 1989)), are based on determining a range of rating values for the rock mass that can be used either to estimate "rock mass parameters" or support requirements. Sections with similar ratings are grouped into regions where the excavation and support requirements are essentially similar. The rating value is also supposed to indicate similar rock mass behaviors.

However, none of the above mentioned classification methods explicitly discuss, or guide the user in determining, the deformational characteristics of the rock mass when the induced stress state approaches or exceeds the local rock mass strength. Unfortunately, it is under these conditions that tunneling is most difficult and many problems arise. In general, when the local rock mass strength is exceeded and larger deformations occur the ambiguous term "squeezing" is often used to describe the behavior. This is unfortunate because in different rock mass conditions, different failure modes occur that are related to the stress state and the rock mass textures, structures, and kinematics in the zone surrounding the excavation. It is these failure modes and the resulting rock deformations that interact with the support system and are then observed in the excavation. In order to optimize the excavation and support methods in these situations, it is necessary to understand how the rock mass is deforming and tailor the excavation and support accordingly. To identify changes in the rock mass behavior it is necessary to systematically monitor the excavation behavior, most preferably in a manner in which spatial measurements are systematically made and not just relative measurements.

With the rapid improvements in optical geodetic surveying systems over the last 20 years it is now possible to measure the absolute three dimensional spatial deformations during the tunnel excavation with high levels of accuracy. Rabensteiner (1996) discusses the principles of the monitoring systems commonly utilized in Austrian tunneling and increasingly applied around the world. This improvement in the knowledge of the 3-D system behavior (the interaction of the rock mass with the excavation and support methods), resulted in new interpretation and evaluation techniques, including Varvrovski (1988), Schubert and Budil (1995), Steindorfer (1996), and Sellner (2000). These techniques are summarized in Schubert et al. (2002).

The high quality of data that can be acquired with current surveying systems provides new opportunities to evaluate the system behavior from past projects, which allows for an interpretation of the rock mass behavior to be made. By combing detailed geologic face mapping, focused on identifying the key rock mass textures and structures that influence the deformation, with the spatial distribution of the deformation characteristics, rock mass behavior types can be defined. Additionally, this allows the spatial influence of major geologic features on the deformations to be identified.

EVALUTION PROCEDURE

The following evaluations of the deformation measurements and geologic data are based on the "Guideline for the Design and Construction of Conventionally Excavated Tunnels" compiled and published by a working group of the Austrian Society for Geomechanics (ÖGG, 2001). Schubert et al.(2001) describe and summarize this guideline with case histories from tunnel projects in Austria.

One of the key steps in the design process for tunnels is to define potential behavior types for each rock mass type. The rock mass types are developed from the site geological and geotechnical investigation and should be considered site specific. A behavior type is defined as how the expected rock mass type would respond to the full excavation without support, considering influencing factors such as the initial stress state, ground water, orientation, etc. The process of determining the rock mass behavior types is the most important step in the geotechnical design, inaccuracies in this step can lead to insufficient or over-designed support systems or the choice of the wrong excavation method (for example TBM vs. Conventional Techniques). The behavior types also assist the construction team in evaluating monitoring results. Experience from other sites in similar geological conditions can be very helpful in determining both rock mass and rock mass behavior types, but a thorough evaluation should be performed for each project. This behavior is then used to evaluate the system behavior for different excavation and support methods, resulting in the geotechnical design.

The procedure during the excavation is quite similar to the design process and a flow chart is shown in Fig. 1. The geologic conditions are mapped at the face focusing on describing the rock mass with "key parameters" that can be evaluated by the geologist and have the largest influence on the rock mass behavior. The rock mass type is determined considering an extrapolation of the observed geology into the rock mass surrounding the tunnel. It is this zone that influences the deformations after the excavation passed, while the material observed at the face primarily influences the behavior in front of and the immediate vicinity of the tunnel face. Influencing factors such as the stress state and ground water are measured or estimated and the appropriate rock mass behavior type assigned to the section. The appropriate excavation and support methods are chosen and used for the next section. Monitoring data is evaluated to determine if the system behavior meets the requirements. If so then the correct interpretations and actions were made. If not, then further evaluations are performed to determine why there was a difference

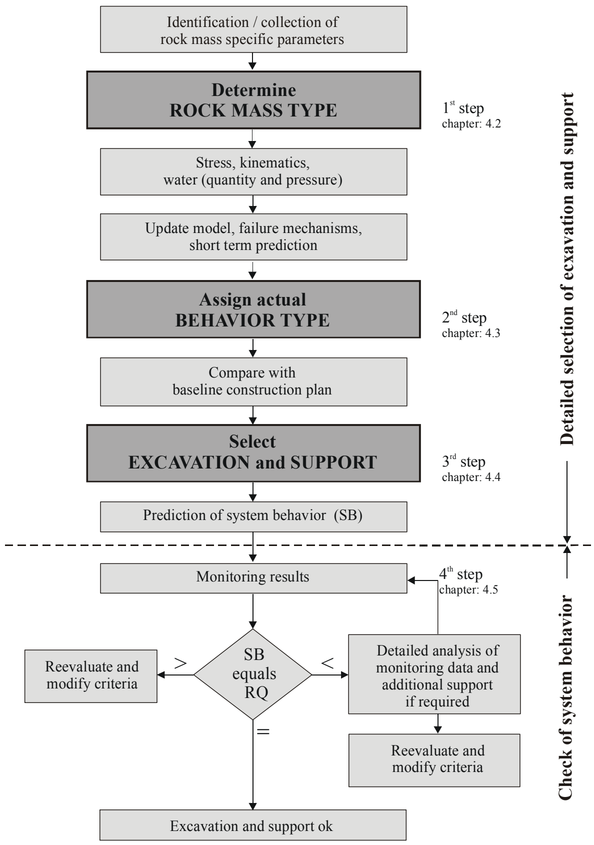

Fig. 1. *Procedure to evaluate the system behavior during excavation (ÖGG, 2001).*

EXAMPLES

One problematic rock type that is often encountered in Alpine environments is phyllite or similar foliated metamorphic rocks. The rock mass quality can very widely depending on the local geological and tectonic situation. Three examples are shown from deep tunnels in quartz phyllite. The third example is from a shallow tunnel in a tectonic mélange. In this rock mass the matrix material is composed of highly sheared phyllites and blocks consist of marble and quartzite.

The program GeoFit® Gruppe Geotechnik Graz (2003), developed by Sellner (2000) has been used to evaluate and plot the deformation measurements.

Example 1

The tunnel is a 2-lane road tunnel with a diameter of 11.5 m excavated with a top heading-bench-invert sequence by drill-and-blast. The rock mass consists of quartz phyllonite and gneissic phyllonite of various qualities depending on the tectonic situation. The laboratory investigation during the design stage resulted in the following strength parameters. A uniaxial strength ranging between 10 and 45 MPa depending on the degree of tectonisation, samples were tested at an angle of 10° -15° from parallel with the foliation. Young's Modulus was between 15 and 30 GPa with a very low Poisson ratio, negative to 0.2. These types of values have been measured frequently in our lab in foliated phyllitic rocks, and are considered representative. Triaxial and direct shear tests resulted in a peak friction angle of 45° for small strains and a residual friction angle of approximately 26°, the latter is a reasonable long term value for this rock mass. Laboratory tests from fault gouge material were not performed due to difficulty in retrieving samples from drill cores. The residual friction from shear tests provides a good estimate for cataclasite fault gouge, however when the fault is described as containing high amounts of clay, both the frictional strength and stiffness decreases and the structures influence of the deformational characteristics increases.

Figure 2 shows the geological conditions observed during the excavation. The rock mass consists of hard quartz-phyllites, affected by both ductile deformation and brittle faulting. The foliation dips approximately 80° to the left at an angle of approximately 30° with the axis. Quartz lenses are distributed throughout the section and are parallel to either the foliation or transitional shear zones (brittle-ductile). The shear zones form rhombiodal lenses that are considered the primary structures together with the foliation. A weak zone associated with a small fault that crossed the tunnel from left to right existed just to the right of the excavation boundary. The overburden in the section is approximately 600 m, the primary initial stress is presumed to be oriented 15 to 25° from vertical due to both the mountain topography and the anisotropic nature of the rock mass.

Fig. 2. Photo of the geological conditions for example 1.

Figure 2 also shows the type of support used in this region. The excavation was advanced in 1 m steps with a primary support consisted of steel ribs and 25 cm of mesh reinforced shotcrete. Rock bolting was performed 2 m behind the face with 15 anchors, and a second round later if necessary, doubling the anchor density.

Figure 3 shows a monitoring section approximately 1 m ahead of the section shown in Fig. 2. Two plots are shown on this figure, one to highlight the cross sectional displacements and the other to show the longitudinal displacements. The displacements are at 1:15 scale with the excavation, and range from 10 cm in the crown and right sidewall to 17 cm for the left side wall. The lower points, installed after the bench excavation displace less then 2 cm. In this example it can be seen that there is an anisotropic response, which is typical for this type of rock.
Following the blasting and mucking, the left side, near the intersection of two of the shear zones, gradually began to fail, as the loose material was scaled in preparation for shotcreting the process continued. This situation is similar to spalling in hard brittle rock. After installation of the support this zone continued to deform and had the largest displacements

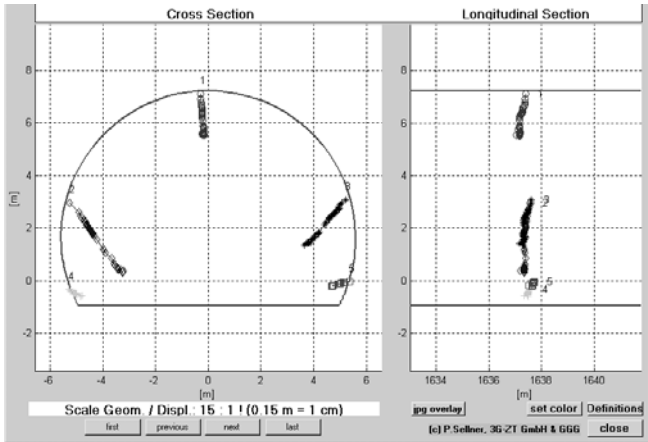

Fig. 3. Measured Displacements for example one.

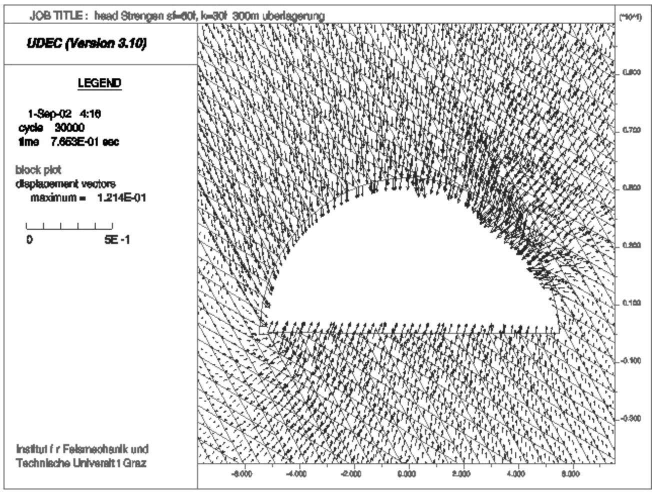

Fig. 4. Results of a UDEC calculation for the general rock mass behavior types for the discussed tunnel.

Figure 4 shows the results of a UDEC, Itasca (2000), calculation that was performed for the general evaluation of rock mass behavior types for this tunnel, the calculation was not meant to back calculate the results from this monitoring section but were designed to show the general behaviors observed during the excavation. Material properties from the initial site investigation were directly used for the calculation. The geometry is opposite of that observed in this section because the tunnel advances from both directions and the geometry is representative for the other drive. The behavior shown is opposite to the discussed example. It can be seen that the relationships between the displacements is similar to that observed during the excavation, with the largest values occurring in the zone where the foliation is parallel to the tunnel periphery. The deformations primarily result from dilation perpendicular to the discontinuities as well as shearing along the steep foliation.

Example 2

Example 2 discusses a different section in the same tunnel as described in example 1. Figure 5 shows the geological documentation recorded during the excavation. The material was described as a being faulted with frequent shear bands parallel to the foliation. The right side was described as primarily consisting of cataclasite material. A fault up to 60 cm thick, exited the excavation 6 m before this section running practically parallel to the tunnel. The weak material on the right side is associated with the zone surrounding this fault. The central zone consists of tightly folded material between larger shear bands. Two shear bands are located on the right side of the excavation that dip toward the tunnel excavation. Major joints are not observed in this material, but there is a distributed shearing associated with the foliation and associated shear bands (typically 15 to 30° from the foliation) that results in a significant loss of tensile (cohesive) strength of the rock mass, as well as folding between the individual shear zones. The overburden in this region is approximately 630 m.

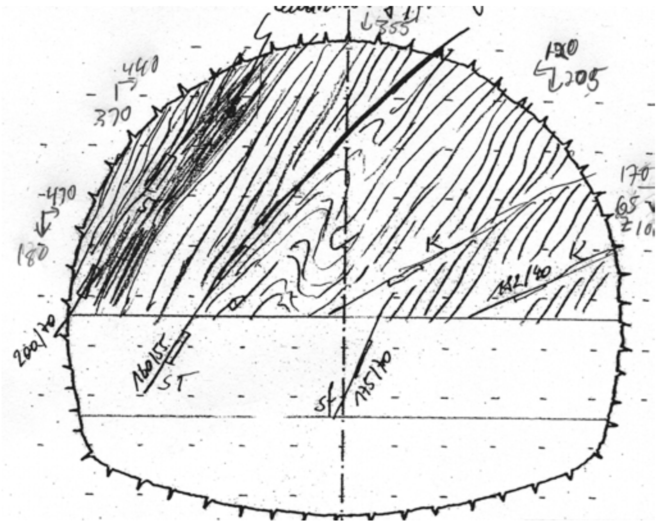

Fig. 5. Geological sketch recorded during the excavation of the discussed section.

The support method in this region is significantly different then the first example. A ductile support system was utilized consisting of four rows of deformation elements located approximately in the middle of the lower side walls and just above the springlines. A layer of 25 cm of mesh reinforced shotcrete combined with heavy radial bolting was used for support. The deformation elements used in the tunnel are similar to that used in the Galgenberg tunnel (Schubert, 1996). Moritz (1999) in his thesis optimized the deformation elements resulting in a system called lining stress controllers (LSC's), as well as a method to tailor their strength, number, and the quantity of slots to the rock mass deformations and shotcrete strength behavior. More recent discussions on the LSC's can be found in Button et al. (2003) and Schubert, (2004).

Figure 6 shown the measured displacements for the monitoring section associated with Fig. 5. The behavior in this section is different then shown in the previous example, through the general rock mass structure is similar. This is due to the different rock mass quality, geometrical relationships with the influencing structures as well as the support system. It was mentioned in the introduction that optical geodetic measurements allow the system behavior to be determined. Evaluating the rock mass behavior is an interpretation from these measurements.

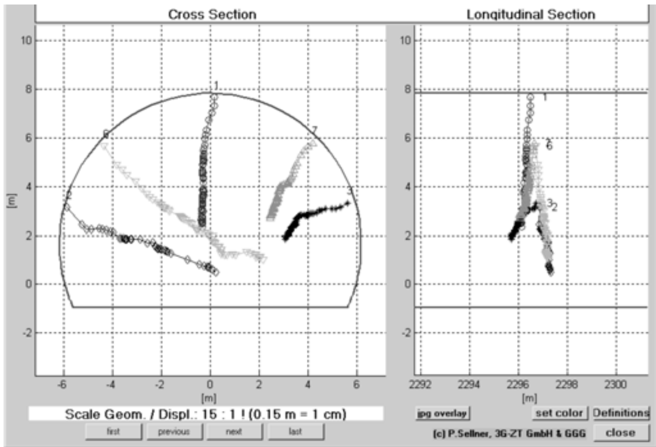

Fig. 6. Measured displacement for example 2.

In this example deformations range from approximately 200 mm at point 3 to 540 mm at point 6. These are not the final magnitudes as the bench excavation has yet to reach this station. There are several interesting characteristics in these deformations. First is that due to the deformation elements, the lining system behaves as individual panels, with axial shortening occurring at the deformation elements. The axial load in the shotcrete is "controlled" by the strength-strain characteristics of the deformation elements. The elements allow zones with different displacement characteristics to deform in a more natural manner, resulting in a more homogeneous stress distribution in the lining without the large bending moments typically associated with anisotropic behavior. This improves the lining performance and capacity.

The change in behavior observed in points 2 and 6 are due to the failure kinematics of the rock mass. As seen in fig. 5 the foliation is steeply dipping to the left, the horizontal movements in the deformation vectors are associated with distinct shearing along the foliation that result in rock slabs being forced into the excavation, while the surrounding material continues to move downwards. The later appearance of this phenomenon at point 6 shows the progressive nature and deepening of this failure mechanism. A similar behavior can be seen in Fig. 4, though the observed magnitude is 6 fold compared to the numerical example. It should also be noted that depending on the exact location of the measurement prism in relation to the local failure kinematics, different behaviors may be interpreted from in the measurements, especially if multiple failure mechanisms are occurring jointly and simultaneously.

It can be seen that point 3 has a large longitudinal displacement, 65 mm, as well as initially a predominantly horizontal displacement vector, this is caused shearing along the observed shear zones that dip towards the excavation. Though not seen in the displacement plots, blocks formed by the intersection of these zones and the foliation have been squeezed upwards into the tunnel excavation, observations in this region have shown a heave of between 250 and 600 mm. These displacements reduce the confining stress around the tunnel lowering the frictional resistance as well as provide new space for the adjacent zones to move. This results in a time dependent response that is related to the change in kinematics and the resulting change in the stress state and not necessarily related to a “viscous” behavior.

<u>Example 3</u>

The example 3 is from a shallow tunnel recently completed in the Semmering region of Eastern Austria, approximately 100 km to the south west of Vienna. The rock mass is composed of highly sheared phyllites surrounding blocks of limestone and dolomitic marbles in addition to quartzite forming a block-in-matrix rock mass, Medley, (1999). Blocks range from the centimeter scale to over 500 m, and are highly fractured or faulted depending on their size. Initially the rock mass was created during north northwest thrust faulting associated with nappe emplacement, resulting in the mixing of the marbles and quartzites with the phyllitic materials, the foliation dips moderately to steeply in this direction. Later, strike slip faults associated with the Mur-Muztal fault zone of the Eastern Alps overprinted the original structures with steeply dipping brittle faults, as well as disaggregated larger blocks. This resulted in the complex geologic conditions observed during the excavation.

The large competency contrasts between the weak phyllites and the hard blocks resulted in many challenges for the contractor and the tunnel engineer. Mixed face conditions occurred in which an excavator was required to remove the matrix material, while blasting was required to break apart the blocks. This created logistic problems, as well as some construction delays. For the engineer, the rapid changes in the deformational characteristics of the rock mass make it difficult to continuously apply the same support type, making constant adjustments and short term prediction of the rock mass conditions ahead of the tunnel face absolutely necessary for a safe and economic construction, Schubert et al. (2003) discuss methods for using the geodetic measurements for making short term predictions in this type of rock mass. The large variation in deformation magnitudes can lead to overstressing of stiffer zones which resulted rapid movements due to brittle failure or to large overbreaks.

A thorough discussion about the rock mass behavior types in mélange rock masses is given in Button et al. (2002, 2003). Depending on the block proportion (BP), three general rock mass behavior types have been defined. These are matrix dominated, block influenced, and block dominated . It is important to consider the influence zone surrounding the tunnel for determining the block proportion as these are the zones that are deforming. This requires that principles from structural geology be used to condition statistical descriptions of the block proportion, this is especially important during the design phases. In addition to the block proportion, important considerations include the size and spatial distribution of the blocks for block influenced zones and of the matrix zones in block dominated regions. A block dominated zone starts to approach the behavior of a faulted rock mass. A brief example will be given here for a matrix dominated zone.

Figure 7 shows the mapped geological conditions for the top heading excavation, 85% of the entire section was composed of phyllitic matrix rocks, 70% was composed of chlorite phyllite, 15% violet phyllite, and 15 % was composed of quartzite blocks, mostly in the bench excavation on the right side. The matrix dominated zone was approximately 40 m in extent striking approximately 30° to the tunnel axis. The overburden was approximately 45 m in this section.

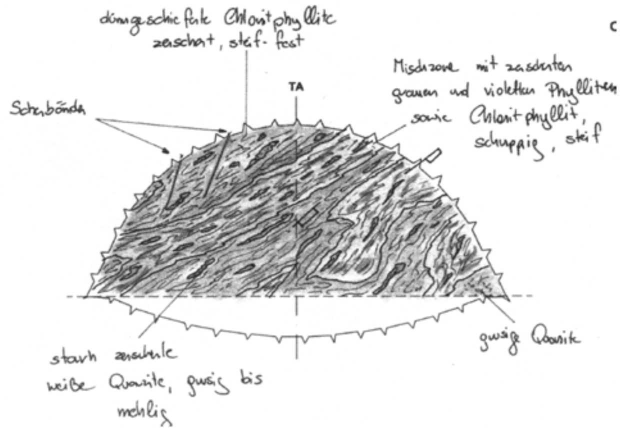

Fig. 7. Geological documentation of the top heading excavation for example 3.

The top heading was excavated in sections with a layer of shotcrete applied to the face immediately after opening. Steel ribs in combination with mesh reinforced shotcrete and rock bolting composed the primary support, a temporary invert was installed 1 m behind the face to achieve a quick ring closure. The bench excavation followed with a minimum separation of 30 m, averaging around 40 m.

Figure 8 shows the measured displacement for the section shown in Fig. 7. Deformations ranged from 280 mm on the lower right to 195 mm in the crown region. The right side having a slightly larger deformation then the left side. This is most likely due to the slight anisotropic nature of these rocks. Button and Blueml (2002) describe shear tests on these materials that show a distinct behavior difference when shearing with the foliation (contractive) or normal to the foliation (dilative) at the stress level of interest. The relatively homogeneous response with a low ratio between the horizontal and vertical displacements is typical for matrix dominated zones in low stress environments associated with relatively shallow tunnelling. If the anisotropic nature of the matrix dominated rock mass increases then there is a tendency for the behavior to also respond anisotropically.

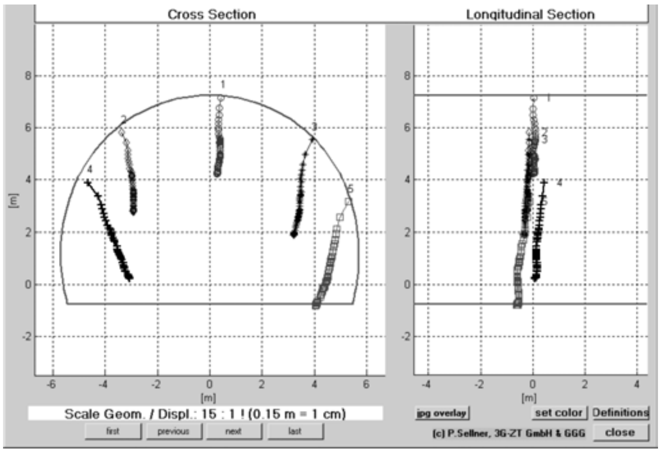

Fig. 8. Measured displacements for example 3.

<u>Example 4</u>

Example 4 is used to demonstrate that as the stress level increases, especially the horizontal stress, with tunnel depth the behavior changes from a low ratio of the vertical to horizontal displacements as shown above to a more radial displacement trend. This example is from the Inntal tunnel. The rock mass was a sheared phyllite associated with a major fault zone that extended for over 2000 m of the tunnel. The overburden was around 300 m. Different rock mass qualities existed throughout the fault zone. The example shown in Fig. 9 is from a zone in which the conditions were practically homogeneous, i.e. no major changes in rock mass quality in the zone immediately surrounding the tunnel. The foliation dips gradually in the direction of the tunnel drive. It can be seen that the deformations are practically radial, though there is a difference in the longitudinal deformations that reflects changes in the rock mass quality ahead of the advancing face as discussed by Schubert and Budil (1995) and Steindorfer (1996). This type of behavior is more common when the anisotropy has been destroyed by faulting at the stress level of interest and typically can be associated with large extensive fault zones in deep tunnels

DISCUSSION

Rock mass characterization systems that rely on a few selected parameters to characterize the rock mass can have limitations in certain rock mass types. One of these rock types is phyllite. A study has been undertaken in which case histories from several tunnels in different types of phyllites have been evaluated. The evaluations were based on the documented geology the monitored displacements, and the boundary conditions. The excavation sequencing and support system was also considered as this can have a significant effect on the behavior characteristics. When the stress level is low compared to the long term strength the largest problems arise in brittle faults where consistent overbreak is a problem. As the stress level increases this loosening is not as common until a stress level is reached that stress induced failures combined with local structures influence overbreak volumes. At higher stress levels, large deformations are a reality, the low ratio between the tensile (cohesive) strength and the frictional component typically results in extended deformation periods. Making the estimation of the over excavation very important to avoid reshaping. Additionally, highly anisotropic responses are quite common, with magnitudes differing 10 fold in one measurement section. As the weak rock mass zones increase in extent there is a tendency for the anisotropic deformations to begin to homogenize. Phyllites can be a very difficult material to tunnel in especially when the stress level is greater then the strength. All types of individual failure mechanisms occur in this general rock mass, which leads to a wide range of behavior types. Exhaustive analyses should be performed to identify what types of situations are possible and how they may deform given the geologic architecture and boundary conditions in the area of the tunnel.

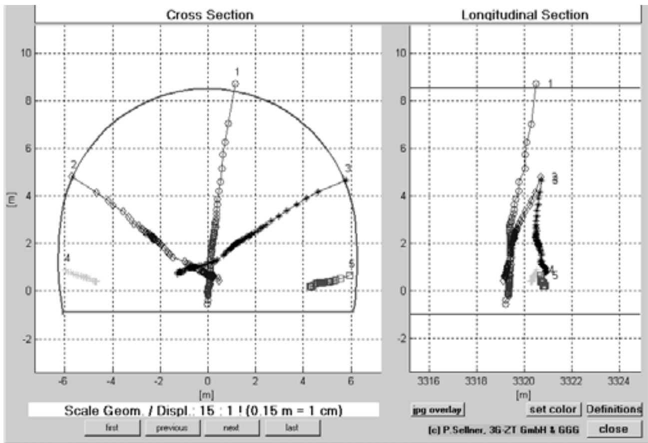

Fig. 9. Deformation measurements from the Inntal Tunnel.

In phyllitic rock masses, as well in laboratory samples, it is important to characterize the rock mass, or rock, texture at the scale of interest. And consider how local changes or distribution in the texture influence the observed response. In this context, the spatial relationships between the excavation periphery and the rock mass structures play a significant role in how the excavation will deform, and therefore the optimum support system for enhancing stability. Changes in the spatial relationships can lead to vastly different behavior types and failure modes, due to different induced stress fields and kinematics.

A solid foundation in mechanics and structural geology, combined with results from physical, analytical and numerical modeling are necessary for a quality interpretation of the system behavior. The importance of quality site documentation cannot be stressed enough for making the most out of the information available in case histories. This information has the ultimate benefit for the owners and contractors who must pay for and construct the engineers design. A better understanding of the rock mass behavior and its characteristics allows the excavation and support system to be optimized for the expected rock mass conditions.

SUMMARY

Four examples were discussed that highlighted how absolute geodetic measurements of the tunnel deformations can be used to assess rock mass behavior types. The identification of different failure modes and how they influence the system behavior provides valuable information for both interpretating data during the excavation, as well as improves design strategies for future projects in similar conditions. A site specific evaluation should be performed for each project using this type of information as a guide and not as a substitute for thorough engineering evaluations.

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