CHARACTERIZATION OF PHYLLITES FOR TUNNELLING

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Abstract: Rock mass characterization and the development of a geotechnical model form the basis for tunnel design. Many characterization procedures focus on determining the engineering parameters necessary for empirical or numerical evaluations of the required tunnel support, this is only one step in the characterization process. The *Guideline for the Geomechanical Design of Conventional Tunnels* recently introduced by the Austrian Society for Geomechanics (2001), focuses on characterizing the rock mass behaviour and potential failure mechanisms considering the site specific influencing factors and boundary conditions. Case histories from tunnels in phyllites in which systematic, 3-D absolute displacement monitoring was carried out have been used to identify the influence of the rock mass structure and the rock mass quality on the system behaviour. This allows the key parameters for this type of rock mass to be identified and their influence on the behaviour quantified. Several examples are shown to demonstrate this philosophy.

Keywords: Phyllites, Characterization, Rock Mass Behaviour, Tunnelling, Squeezing Ground

1. INTRODUCTION

The most important steps during the planning and realization of a tunnel project are to develop a realistic geologic model and based on that model, a geotechnical model that can be used to evaluate which excavation and support methods provide for the most economic and safe construction. For any given rock mass there is a group of physical characteristics that can be quantified, through testing and observations, that governs how that particular rock mass will respond to the construction activities given the local boundary conditions. These characteristics can be described with engineering parameters such as deformability; strength; discontinuity characteristics, spacing, orientation; stress; permeability; etc.. There are many papers available that focus on characterizing specific rock mass types, for example Gokceoglu and Aksoy (2000) clay bearing, densely jointed weak rocks, Ramamurty et al. (1993) phyllites, Habimana et al. (2002) cataclastic rocks, but these as well as other investigations primarily focus on characterizing the engineering parameters in a laboratory environment or compare the rankings from empirical classification procedures and attempt to judge their applicability for the rock mass in question.

While the empirical classification methods have been applied to hundreds of tunnel projects there are still inherent weaknesses with these procedures in certain rock mass conditions, Riedmueller and Schubert (1999). Additionally, case history evaluations that are based on these systems tend to report either a success or failure and rarely discuss the actual system behaviour in terms of failure kinematics. Therefore, they are limited in the knowledge that they provide to readers about the rock mass or system behaviour.

Laboratory investigations are a vital and necessary step in the characterization procedure, however they are only a part of the process. The goal of the geotechnical model is to describe the rock mass behaviour, at the scale of the excavation, along the tunnel alignment. In this sense, there is very little published information quantifying or describing the rock mass behaviour for different rock types and the influence of different structures or variations in the rock mass quality on the behaviour at the scale of the excavation.

The Austrian Society for Geomechanics recently introduced a guideline, OEGG (2000), that provides a consistent and transparent procedure for the design and construction of conventional tunnels. This Guideline differs from the empirical rock mass classification and tunnel design procedures (Q, RMR) in several ways. The first is that project and rock mass specific key parameters are used to characterize each Rock Mass Type, instead of a few parameters universally applied to all existing geological conditions.

Secondly, the procedure focuses on characterizing the Rock Mass Behaviour associated with the full area and/or volume of the planned excavation. This step is used to identify what, if any, failure modes are possible for a given rock mass type considering influencing factors such as the stress state, discontinuity orientation and location, groundwater, etc.. Identified failure modes should be ranked in a hierarchical fashion and dealt with probabilistically.

The final difference is that construction and support methods are designed to counteract the identified rock mass behaviour (failure modes), always considering the project requirements and boundary conditions. The system behaviour is confirmed during construction by systematically monitoring deformations and comparing the predicted to the observed behaviour considering the encountered geologic conditions. In this way, the support recommendations for a given rock mass type can be optimised during construction based on a continuous updating of the geotechnical model and the relationship between the geological conditions and the observed system behaviour.

This procedure has been used to evaluate case histories from tunnels excavated through phyllites in which absolute 3-D displacement measurements were consistently recorded. This data was then compared to the encountered geologic conditions and the utilized excavation and support to identify the influence of different structures and rock mass quality on the system behaviour. Examples are shown from two deep tunnels with similar rock mass properties but different structural conditions.

2. CASE HISTORIES

2.1 Inntal Tunnel

The first case history is from a double track rail tunnel constructed in the early 1990’s. The tunnel was excavated using a top heading bench sequence. Support for this section consisted of steel ribs shotcrete and rock bolts. The overburden in the discussed sections is approximately 300 m. The rock mass consisted of quartz phyllite with a UCS that ranged from 23 MPa to 53 MPa depending on the orientation to the foliation relative to the loading direction. Indirect tensile tests resulted in a tensile strength ranging from 1,6 MPa to 6 MPa. Tests were not performed on weak samples associated with fault gouge due to limited recovery and preparation difficulties. This example shows how a fault zone begins to effect the displacement characteristics before it is observed in the excavation. Figure 1 shows a summary of the encountered geologic conditions for a 100 m section in which the tunnel passed from a good quality rock mass into the beginning of a major fault zone. The foliation (light grey) in this example dips in the excavation direction at approximately 30°. Joints are shown with solid black lines, while faults are shown with dashed lines.

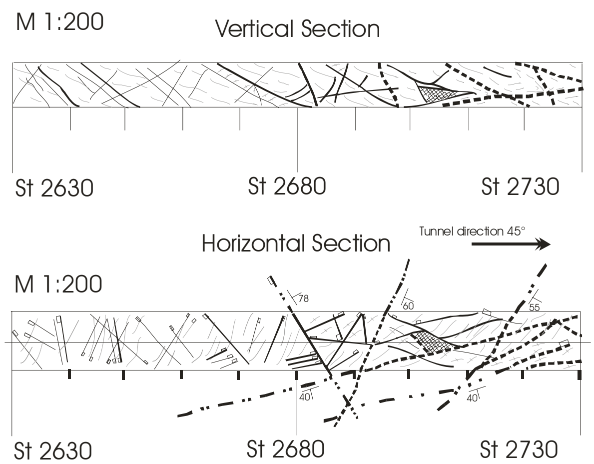

Figure 1. Simplified documented geology.

Figure 2 shows the monitored data at a measuring section located at chainage 2645. All of the following displacement plots are created with the software package Geofit®, Gruppe Geotechnik Graz, (2003) In this region the joint intensity was increasing, as shown above, but did not have a influence on the measured displacements. This behaviour is typical for the good quality rock mass. The displacements at this section ranged from 77 mm at the crown position to 26 mm at both side walls. This was a typical magnitude and characteristic for the better quality rock masses encountered during this excavation.

Figure 3 shows the next monitoring section which was located at chainage 2680. This position is just before the fault zone enters the excavation from the right side. The displacements increase at all of the monitoring points compared to the last monitoring section. Proportionately the largest change occurs at point 3 located at the right sidewall, which is closest to the fault zone.

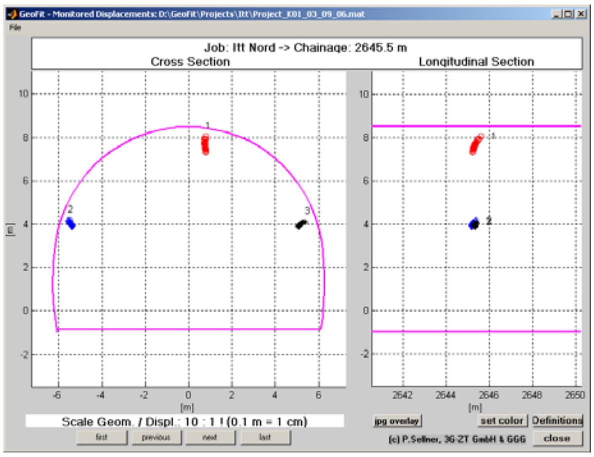

Figure 2. Measured displacements station 2645.

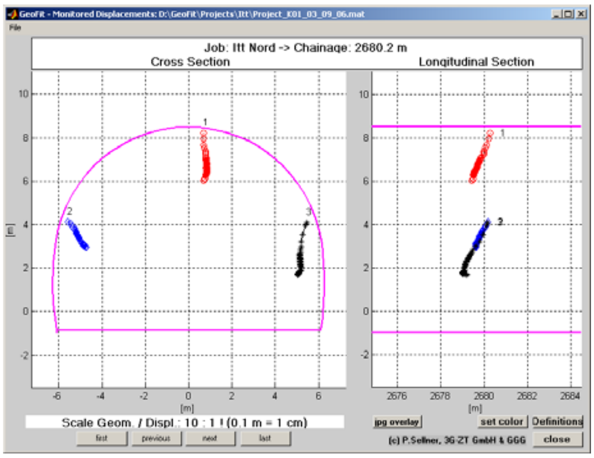

Figure 3. Measured displacements station 2680.

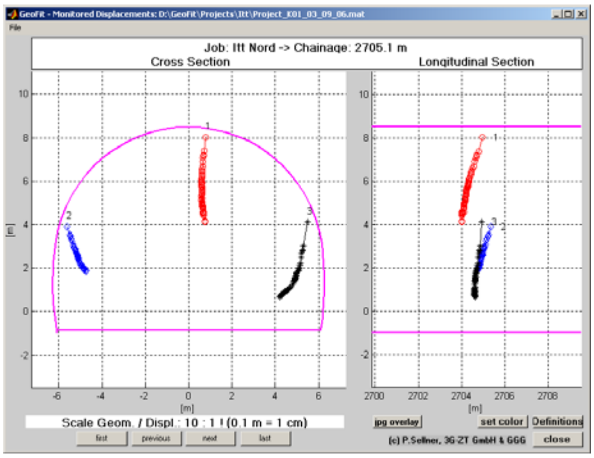

Figure 4. Measured displacements station 2705.

The last monitoring section shown for this example is located at chainage 2705, Figure 4. At this location the fault zone is in the central section of the excavation and a zone of clayey cataclasite is located in a zone between intersecting faults, hatched zone in Figure 1. In this case the displacements have again increased compared to the previous section indicating a continued decrease in the rock mass quality associated with the multiple faults located in this region.

2.2 Strengen Tunnel

The next examples come from a two lane road tunnel excavated in phyllites that are were very similar to those encountered in the first example. In this case the foliation dips between 60° and 80° and crosses the tunnel axis at approximately 30°. In the discussed zones the overburden is initially around 590 m and increases to over 630 m. The laboratory investigation resulted in UCS values ranging from 15 MPa to 35 MPa (foliation 15° from the loading direction) the major difference in the strengths were related to mineralogical differences. Tests were not performed at multiple orientations due to limited sample availability. With these strength values and the stress state simply due to the overburden stress induced failures were expected and observed in locations with decreased rock mass quality. The excavation method was drill and blast. Support consisted of steel sets, shotcrete, and rock bolts. When displacements increased or were highly anisotropic ductile support elements were used within gaps left in the shotcrete lining.

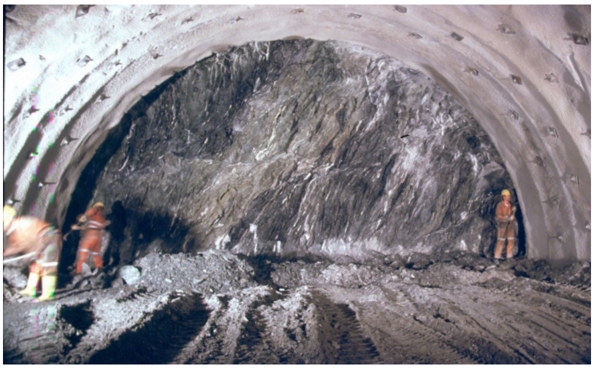

Figure 5. Example of the rock mass structure associated with example 2.

Figure 6 shows the displacement tendencies for a typical cross section that is only influenced by the anisotropic nature of this type of rock mass. Several joints and small faults were observed at the face but these do not have a significant influence on the system behaviour at this location. Displacements ranged from slightly less then

30 mm at the crown and right side wall to approximately 75 mm at the left side wall.

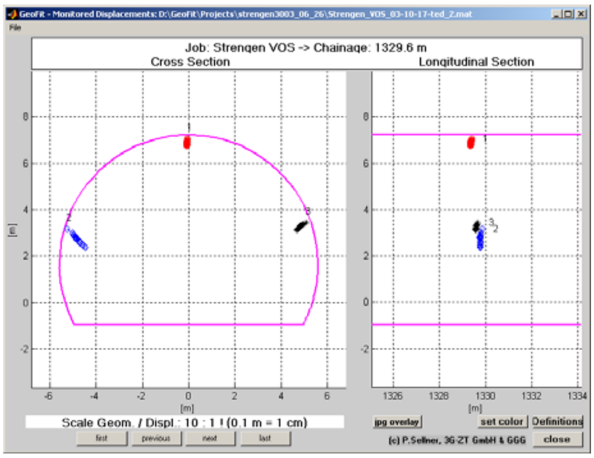

Figure 6. Displacement characteristics for a section not influenced by faulting.

Figure 7 shows the displacements characteristics for a section with a similar rock mass structure, however in this section a fault with up to 10 cm of gouge material was extended from the crown region to the lower left side wall dipping at approximately 45°. It can be seen that in this case the displacements show the same geometric trends with a larger displacement magnitude. In this case the largest deformation was around 200 mm. The deformation of the other two points was approximately 50 mm for the crown and 80 mm for the right side wall.

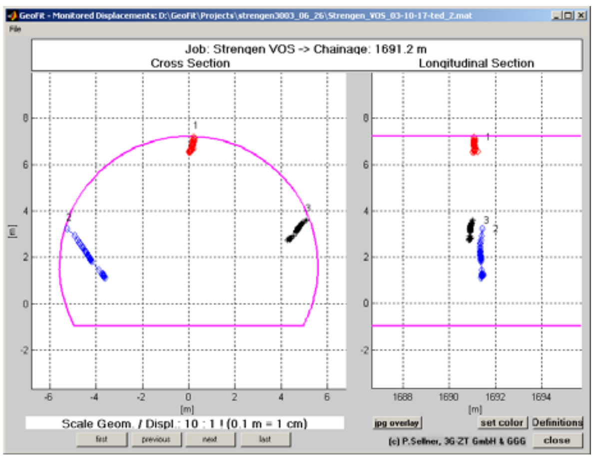

Figure 7. Displacement characteristics for a weaker rock mass associated with a single fault

Figure 8 shows the system behaviour for a section in which a small fault zone consisting of two parallel faults with up to 20 cm of gouge were located just inside of the excavation in the lower right corner. The zone bounded by these faults was described a soft and weak. This zone has a significant influence on the deformational characteristics at this monitoring section. Deformation magnitudes ranged from approximately 30 mm at the crown to 190 mm at the right side wall.

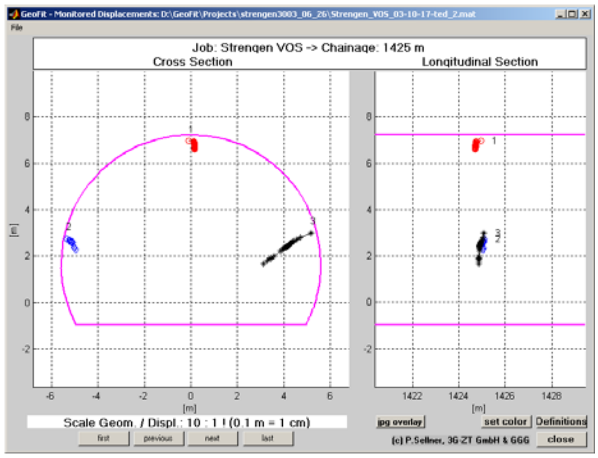

Figure 8. Displacement characteristics influenced by a small fault zone.

The last example is associated with a major fault zone that was composed of several parallel faults with up to 30 cm of gouge. This fault zone was located in the left side of the excavation extending from the crown region towards the lower left side wall dipping at 30° to 40° and was semi parallel to the tunnel axis and extended beyond the excavation boundary. Additional, smaller shear zones were located in the right section of the excavation. It should also be noted that the support in this section consisted of ductile elements placed in gaps left in the shotcrete lining. With this support system, large deformations can be accommodated while maximizing the utilization of the shotcrete lining without exceeding its capacity Schubert (1996), Moritz (1999), Button et al. (2003).

The deformational characteristics of this section are quite different then those previously shown. Several failure modes were occurring in this region that led to highly anisotropic deformations that continued to occur over time. The horizontal deformational behaviour of the lower left side wall is associated with shearing along the foliation. The change in orientation, as well as a significant “time-dependent” trend was attributed to a change in the kinematics associated with a fault bounded block located in the invert region being squeezed upwards into the excavation, decreasing the confining stress which in turn results in increased displacements in the surrounding rock mass. Zones with significant invert heave were observed throughout this region.

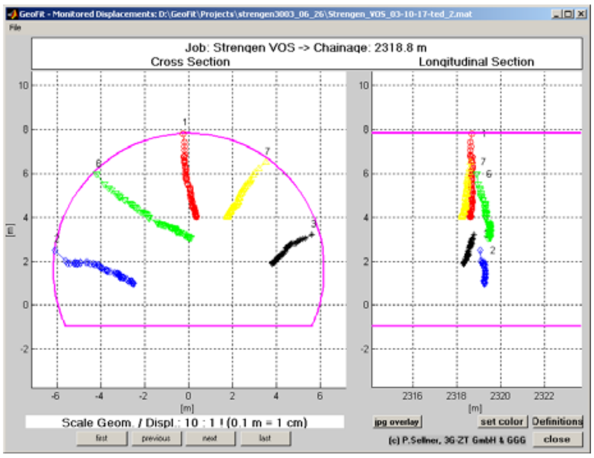

Figure 9. System behaviour influenced by a major shear zone

3. DISCUSSION

The examples presented above showed that there is a wide variety of deformational characteristics associated with foliated metamorphic rocks, including but not limited to phyllites. The systematic analysis of several tunnel projects using the procedures outlined in the Guideline for the Geomechanical Design of Conventional Tunnels, OEGG (2001) has led to the following results.

3.1 Key Parameters

In order to optimise the site investigation and to determine the rock mass types key parameters must be defined. Basic key parameters for characterizing foliated metamorphic rock types, such as phyllites consist of the following:

- Anisotropic strength and deformability, associated with the foliation
- Foliation orientation persistence (i.e. folding)
- Fault and joint characteristics
- Mineralogy (percent and distribution of phylosilicates and clay minerals)

These key parameters provide the starting point for characterizing different phyllitic rock mass types. There may be additional parameters which can be used to delimit different rock mass types depending on the site specific conditions.

3.2 Influencing Factors

In order to determine the rock mass behaviour types it is necessary to identify what factors will have the largest influence on the rock mass behaviour. For phyllites and other similar foliated metamorphic rocks the following criteria should be considered:

- Relative orientation of foliation to the excavation
- Fault orientation and spatial characteristics
- Stress level
- Ground water
- Excavation size and shape

These are the basic influencing factors that should be considered when evaluating the rock mass behaviour types for phyllites. Depending on site specific conditions others may also be applicable. It must be stressed that in this stage no support methods are considered. One item that also should be considered is whether there is a potential for activating global slope movements, this is especially important for portal regions, as well as hill slope tunnels.

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3.3 Behavior Types

As demonstrated with the examples shown above there is a wide variety of deformational characteristics associated with this type of rock mass and the inherent spatial variability in geology. Usually, behaviour types can be generalized into basic types, Schubert et al. (2001), OEGG (2001) such as gravity controlled failures associated with overbreak or block sliding, stress and geometry induced failures, or swelling ground. These behaviours can be sub-divided based on failure modes, the expected deformation magnitudes or rates. It is in this stage that different scenarios should be developed considering all of the potential spatial relationships to evaluate the different characteristics which provides a reference for both evaluating the system behaviour during the design phases as well as interpreting the system behaviour during construction.

4. CONCLUSION

In order to develop a cost effective and safe tunnel design it is necessary to have a realistic geotechnical model. This model is then used to evaluate and compare different excavation and support methods within a consistent and transparent procedure. The most cost effective and safe method should be chosen that meets all of the stated project goals and boundary conditions. By basing the evaluations on the rock mass behaviour and potential failure modes the design can be tailored to the site specific conditions and the construction optimised.

Examples were given of different system behaviours observed in tunnels constructed in phyllites. The differences were attributed to different geological conditions and spatial relationships between influencing structures and the excavation. Evaluations from several tunnels in phyllites has allowed the basic key parameters and influencing factors to be identified. Rock mass behaviour types must be determined based on scenarios developed from the geologic model. These scenarios should be treated probabilistically during the design phase and used during the excavation to assist in interpreting monitoring results.

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