

The Observational Approach in Tunnelling

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ABSTRACT: The observational approach in rock and underground engineering often is interpreted as “design as you go”, implying that there is no or only a very basic design available prior to start of construction. Spectacular cost- and time overruns are the consequence of a lack in preparation of the observational approach. Appropriate preparation includes a thorough investigation and ground characterization as a basis for selection of the alignment, assessment of possible failure modes, and assignment of appropriate construction methods to each section of the tunnel.

With a serious observational approach, the design is continuously reviewed during construction to account for the uncertainties, and updated if required. To optimally adjust the construction to the actual ground conditions, a sound preparation of the project is required.

Even if the preparations prior to construction are perfect qualified personnel is required on site to successfully implement the observational approach. They need to have a sound understanding of kinematical and mechanical processes to be able to make the optimal adjustments to the real ground conditions. Rock mass and site specific criteria for assigning construction methods to the ground conditions and requirements have to be developed. Advanced tools for data collection and interpretation assist in making the right decisions.

1 INTRODUCTION

A sound and economical tunnel design depends on a realistic geological model (Riedmueller and Schubert, 2001), a quality rock mass characterization, and the assessment of influencing factors such as primary stresses, groundwater, and kinematics. Despite this requirement it is still current practice to base the tunnel design primarily on experience, basic empirical calculations, and standardized rock mass classification systems (Bieniawski, 1974, 1989, Barton et al., 1974, Barton, 1998). Additionally, the on site decisions on excavation and support modifications are frequently based more on intuition than on analyses.

To overcome the shortcomings of the current practice during design and construction, a structured and coherent design procedure has to be followed. In the following the basic procedure during design and construction according to the Austrian guideline for the geotechnical design for underground openings (OeGG, 2001) will be briefly outlined. Following this approach, the design is developed step by step. The transparent presentation of all design steps during construction allows adjusting the construction methods to the actual ground conditions and requirements. Appropriate monitoring and data

evaluation methods are devised during the design, which can be modified as more information is available during construction.

2 DESIGN PROCESS

The basic procedure consists of 5 general steps to develop the geotechnical design, beginning with the determination of the Ground Types and ending with the definition of excavation classes. During the first two steps statistical and/or probabilistic analyses should be used to account for the variability and uncertainty in the key parameter values and influencing factors, as well as their distribution along the projects route (Goricki, 2001, Goricki et al. 2002). The probabilistic analyses are then continued throughout the entire process as necessary, resulting in both a risk analysis and a distribution of excavation classes on which the tender documents are based (Goricki et al., 2002).

The five steps to be followed are outlined below.

2.1 Step 1 – Determination of Ground Types (GT)

The first step starts with a description of the basic geologic architecture and proceeds by defining geo-

technically relevant key parameters for each ground type. The selection of parameters used should focus on such parameters, which are expected to dominate the behaviour of the rock mass and have a significant influence on the construction method, time and costs (Liu et al., 2001). A Ground Type is a group of rock masses having similar physical and/or hydraulic parameters. Not necessarily each lithological unit leads to a separate Ground Type, if the properties of different units are the same within acceptable limits. In general also alternating layers of different rock types are grouped in one GT. The number of Ground Types elaborated depends on the project specific geological conditions and on the stage of the design process.

Special care has to be taken when evaluating rock mass parameters. With empirical relationships, which frequently are based on ratings, completely unrealistic results for the rock mass strength and deformability are obtained under certain circumstances. A check on the plausibility of the results is thus advisable.

2.2 Step 2 – Determination of Ground Behavior

The second step involves evaluating the potential rock mass response to tunnel excavation considering Ground Type and local influencing factors, including the relative orientation of relevant discontinuities to the excavation, ground water conditions, stress situation, etc. This process results in the definition of project specific Ground Behaviours. Ground Behaviour in this context is defined as the reaction of the rock mass to the excavation of the underground opening without consideration of sequential excavation steps and support.

One starts with dividing the alignment into geotechnical units or sections, which exhibit same rock mass types, influencing factors and boundary conditions. The rock mass response to the excavation is then analysed in each section. The knowledge of the rock mass behaviour without the influence of construction measures is an important basis for the design of appropriate excavation and support methods.

The sophistication of analysis methods depends on the stage of the project and the complexity of the expected rock mass behaviour. In early project stages and for rather homogeneous rock mass conditions, closed form solutions (Feder, 1978, Hoek, 1999) will be sufficient, while for the detail design or strongly anisotropic materials appropriate numerical methods will have to be used. Systematically each section is checked against all possible failure modes. This in general requires applying different methods of analysis. For example jointed rock masses will show a tendency to severe overbreak up to chimney type failure in a low stress environment. The same rock mass may be perfectly stable under medium stress, while other failure modes, like

spalling or shearing will have to be expected under high stresses.

Each characteristic behaviour identified is described with respect to applicable Ground Types, ground water conditions, failure mode or combined failure modes, and quality and quantity of displacements. In the Guideline eleven basic categories of Behaviour Types are listed. For the ease of communication, the behaviours evaluated should be assigned to one of the basic categories. Distinct delimiting criteria for each Behaviour Type evaluated have to be used. For example the volume of overbreak may serve as criterion for distinguishing between different behaviours within the basic category “discontinuity controlled overbreak”. The depth of expected failure zone may serve as a criterion to distinguish between the category “shallow stress induced failures” and “deep seated stress induced failures”. The delimiting criteria have to be shown in the geotechnical report. It is quite obvious, that combinations of failure modes can occur, for example overbreak combined with swelling, or shear failure combined with overbreak.

2.3 Step 3 – Determination of excavation and support and evaluation of System Behaviour

Based on the defined project specific Ground Behaviours, different excavation and support measures are evaluated and acceptable methods determined. Section by section construction methods and measures can be assigned. Once a range of applicable construction measures is assigned to each section, the expected system behaviours have to be assessed and described. The System Behaviour (SB) is a result of the interaction between the rock mass and the selected excavation and support schemes. This includes the definition of the spatial and transient development of displacements, and other relevant phenomena. This helps during construction identifying the “normal” tunnel performance and assessing the severity of deviations. Very rarely the initial assumption for the chosen construction measures will be the optimal one, so an optimization process is required. As there is always more than one solution to the same problem, generally more than one construction method has to be analysed. As this process may result in a frequent change of construction methods in heterogeneous ground conditions, homogenization of the methods is required to arrive at a reasonable overall program.

The evaluated System Behaviour has to be compared to the defined requirements. If the System Behaviour does not comply with the requirements, the excavation and/or support scheme has to be modified until compliance is obtained. It is emphasized, that different boundary conditions or different requirements may lead to different support and excavation methods for the same basic Ground Be-

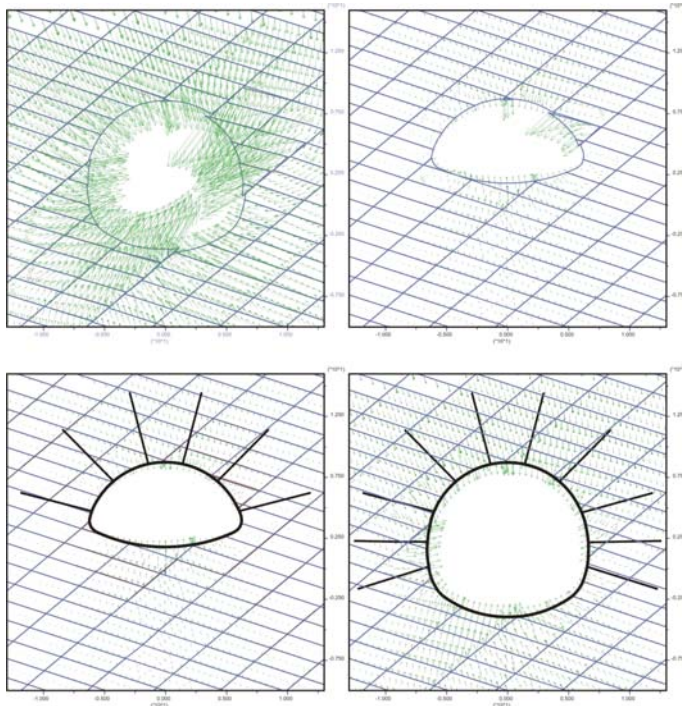


Figure 1. Result of a numerical simulation to determine the Ground Behaviour (upper left), the System Behaviour after dividing the face into two sequential excavation steps (upper right), the System Behaviour after installation of support in the top heading (lower left), and the System Behaviour after excavation of bench and installation of support (lower right)

behaviour within one project. A shallow tunnel in weak ground may serve as example. When built in open space, surface settlements will be a minor issue and the excavation and support can be optimised with respect to construction costs. In built up areas the excavation and support methods have to be designed to limit surface settlements. Methods of excavation and support and costs will be quite different in both cases.

Regulations with respect to safety factors, loads to be assumed, life cycle of primary support, etc. vary strongly in different countries. This also must lead to different designs for the same ground behaviour in different environments.

Once the acceptable excavation and support methods have been determined both risk and economic analyses should be performed to allow appropriate assessments during the tender process.

2.4 Step 4 – Geotechnical report – baseline construction plan

Based on steps 1 through 3 the alignment is divided into “homogeneous” regions with similar excavation and support requirements. The baseline construction plan indicates the excavation and support methods available for each region, and contains limits and criteria for possible variations or modifications on site.

The plan summarizes the geotechnical design and should contain information on the geological conditions, relevant geotechnical features, limitations (e.g. surface settlements, blasting vibrations, etc.), as well

as warning criteria and remedial measures for the case when acceptable limits of behaviour are exceeded.

2.5 Step 5 – Determination of excavation classes

In the final step of the design process the geotechnical design must be transformed into a cost and time estimate for the tender process. Excavation Classes are defined based on the evaluation of the excavation and support measures. The excavation classes form a basis for compensation clauses in the tender documents. In Austria the evaluation of excavation classes is based on ONORM B2203-1 (2001). In other locations the local or agreed upon regulations should be used.

The distribution of the expected excavation classes along the alignment of the underground structure provides the basis for establishing the bill of quantities and the bid price during tender.

3 CONSTRUCTION

Even with a good geological and geotechnical investigation and an up to date design, the adjustment of excavation and support to the local conditions has to be done on site in order to achieve an economical and safe tunnel construction. The uncertainties in the ground model increase with increased overburden and the complexity of the geological conditions. Considerable effort and expertise is required to continuously update the ground model, predict ground conditions ahead of the face, identify possible failure modes, assign appropriate excavation and support methods, and predict and verify the system behavior. The increased information gathered during construction allows a more precise ground characterization, and thus an optimal adjustment of the construction method to the ground behavior and required system behavior.

Many serious problems during tunneling arise from so called unexpected geological conditions. This may involve the late detection of faults and fault zones, but also the inflow of ground water. To minimize damages and losses due to such conditions, efficient and continuous site engineering is required.

To allow successful implementation of an observational approach, several technical and organizational conditions have to be fulfilled. During construction a similar procedure is followed, as in the design stage. This includes the identification of the designed Ground Types, assessment of the associated Ground Behaviours, and assignment of appropriate construction methods to the specific conditions. This process is supported by the evaluation of the results of the continuing investigation and monitoring process. On site the implementation of a

monitoring program targeted to the expected behavior must be implemented in an appropriate density. Processing, evaluation and interpretation of monitoring results has to be done sufficiently rapidly to allow mitigation measures to be implemented in time. Last, but not least, the site organization shall allow an efficient decision making process and a rapid implementation of required measures.

3.1 The role of on-site engineering

Although the general nature of the ground may be known prior to construction, an accurate prediction of the internal structure is impossible. Thus the ground model has to be continuously improved during construction. Monitoring and data collection have to focus on the specific problems associated with project. An important part of the on site activities of geologists and geotechnical engineers is the prediction of the ground conditions in a representative volume ahead of the tunnel face and around the tunnel. Only if a relatively accurate model exists, can appropriate excavation and support methods be selected, and the expected system behavior be predicted. A second very important task is the monitoring of the system behavior after excavation, and the assessment of its compliance with the prediction.

3.1.1 Geological tasks

On many sites the site geologist is used only to document the actual geological conditions. This usually is done in the form of face maps, with a later compilation into longitudinal sections and plan views. This may help in defending or supporting claims, but is not sufficient to allow for a reasonable adaptation of the construction to the actual conditions. To fulfill the requirements of an observational approach, the geologist has to continuously update the geological model, incorporating the observations on site. As many of the decisions during construction have to be made prior to the excavation, like round length, overexcavation, lining thickness, etc. the geologist also has to predict the ground conditions ahead of the face and in a representative volume around the tunnel. To enhance the accuracy of the prediction, a continuous observation of trends of certain parameters is required. For efficient data management and evaluation data base systems with advanced statistical and probabilistic features can be used (Liu et al. 1999). Basis for the geological modeling ahead of the face in general is the observation of trends of structures, recorded in the excavated section. Traditional manual face mapping increasingly is supported by up to date 3D image systems (Gaich et al. 2004, 2005). Figure 2 shows an example of a 3D model of a tunnel face with measurements of discontinuity orientations taken from the image. In this way an unbiased evaluation of the geological situation is possible. In contrast to hand

sketches and discontinuity orientation measurements with the compass, the information gathered from the images is complete and accurate, as the images are calibrated and scaled. Orientations of joints can be measured from joint planes or joint traces. The evaluation software also offers options to measure bedding thicknesses, joint bridges, areas and distances.

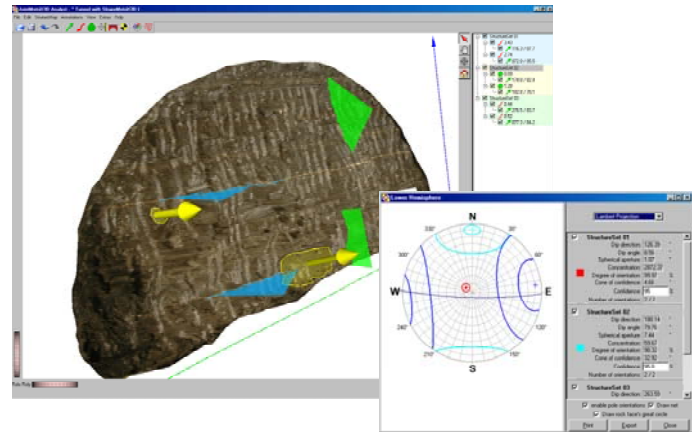


Figure 2. 3D image of a tunnel face with measurements of discontinuity positions and orientations, and statistical evaluation of discontinuity data with JointMetrix3D Analyst

A series of evaluated images of successive excavation faces allows predicting the rock mass structure and quality of a representative volume ahead of the face and around the tunnel by extrapolation. This, combined with the structural geological evaluation can lead to a pretty reliable geological model, forming the basis for predicting ground behavior, which again is the basis for the determination of required construction measures.

3.1.2 Geotechnical tasks

The information gathered by the site geologist is further processed by the geotechnical engineer, forming the basis for decisions on construction method, monitoring layout and reading frequency, to name a few tasks only. To allow decisions to be taken in time, all data recording and evaluation has to be done quasi in real time, and the relevant data have to be always available to all parties involved in the construction. Internet based information platforms can be used for that purpose, allowing also off-site experts to keep track with the information flow. Based on the geological model, the geotechnical engineer has to update the ground model by assigning properties to the geological features. Then the ground behavior (ground reaction on excavation without support) for the section ahead is evaluated, possible failure modes identified, and excavation and support methods assigned. To support the geotechnical modeling, the monitoring results of the previously excavated sections can be used. In a next step, the system behavior (combined behavior from ground and construction measures) is predicted and compared to the requirements, like serviceability,

compliance with limitations (subsidence, vibrations, etc). Based on the recommendation of the geotechnical engineer the Engineer under consideration of contractual aspects fixes the construction measures. In case those deviate from those recommended by the geotechnical engineer, the expected system behavior has to be re-evaluated.

The geotechnical engineer also has to determine the detailed monitoring layout and program, which should be targeted to capture the expected behavior. Once the expected system behavior is determined and the monitoring conducted, the observed behavior is compared to the predicted one. Deviations from the normal or predicted behavior have to be assessed, and in case of unacceptable developments mitigation measures proposed. Warning and alarm criteria and respective mitigation measures are laid down in a geotechnical safety management plan, jointly developed by the designer and geotechnical engineer on site.

3.1.3 Advanced analysis of displacement monitoring data

The geological modeling preferably is supplemented by an analysis of the monitored displacements. It has been shown, that the trend of the spatial orientation of the monitored displacement vector can be used to identify changes in the rock mass quality ahead of the tunnel face (Schubert et al. 1995, Steindorfer 1998, Grossauer et al. 2003). Figure 3 shows the results of a series of numerical simulations, where the development of the stresses, displacements, and displacement vector orientations for a tunnel crossing a weak zone are shown.

Figure 4 shows an example of an Alpine tunnel, where the displacement vector orientation trend (L/S) significantly changes already when the face is several tens of meters ahead of a fault zone. At this project, the normal displacement vector orientation in quasi homogeneous ground was in the range of 4° to 9° against the direction of the advance. From about station 1100m a deviation of the vector orientation from the normal range can be observed. The peak of the deviation is reached right at the transition between sound rock and fault zone. With further progress of the excavation through the fault zone, the trend of the displacement vector orientation first tends to the normal range again. When the heading is within the fault zone, the trend of the displacement vector orientation deviates to the opposite side of the normal range, indicating the stiffer rock mass behind the fault zone. For extended fault zones, the displacement vector orientation generally returns to the “normal” range again, until the influence of the boundary to the more competent rock mass is indicated by another deviation. This information can be used to estimate fault zone extensions.

As a general rule it can be stated, that the higher the stiffness contrast between faulted rock and

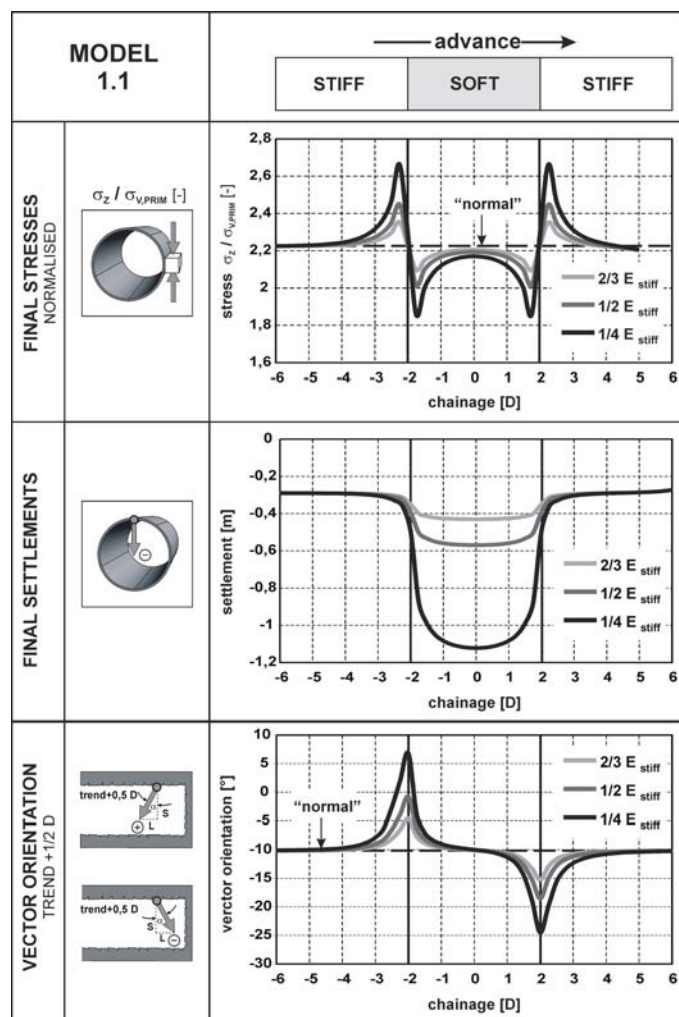


Figure 3. Distribution of stresses along the sidewall of a tunnel when the tunnel penetrates a weak zone (top). Development of displacements, when the tunnel penetrates a weak zone (center). Trend of displacement vector orientation (bottom). The different lines show the influence of different stiffness contrasts between soft and stiff rock (Grossauer, 2001)

neighboring rock mass, and the longer the fault zone is, the larger is the deviation of the displacement vector orientation from the normal range. It has been shown by Grossauer (2001), that this is valid up to a certain critical length of a fault zone. As for fault zones with an extent of less than about three to five tunnel diameters, a certain arching between the more competent rock masses can be observed, also the displacement magnitude within the fault zone is smaller than in a fault zone with a large extension.

Displaying the spatial displacement vector orientation in stereographic projection, the orientation of faults outside the tunnel profile can be determined with some accuracy. Naturally also the virgin stress field and anisotropy of the rock mass influence the displacement vector orientation. Thus for each project the range of “normal” displacement vector orientation will be somewhat different.

3.1.4 Prediction of displacements

Once the geological-geotechnical model for the region ahead of the face has been established, the support and excavation measures can be determined, and the expected displacement development predicted. Sellner (2000), based on research conducted

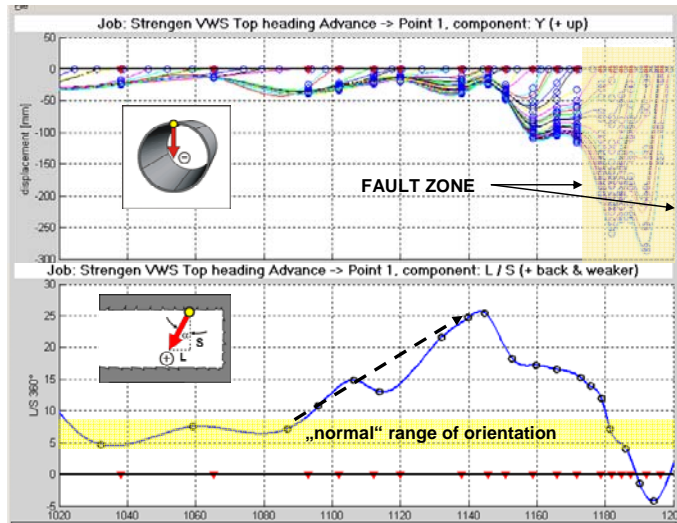


Figure 4. Deviation of the displacement vector orientation from the “normal” several diameters ahead of a fault zone. When the excavation passes the fault zone the deviation in the opposite direction indicates stiff rock mass ahead again.

by Sulem et al. (1987) developed software (GeoFit[®]) allowing the prediction of displacements considering varying excavation sequences, advance rates and different supports. With this tool it is not only possible to predict the development of the displacements and the final displacement magnitude, but also the effect of different supports. Figure 5 shows such a prediction of the displacement development for a shallow tunnel in a tectonic mélangé. The excavation was done in a top heading-bench-invert sequence.

The shotcrete-rock bolt support is supplemented by a temporary shotcrete invert in the top heading.

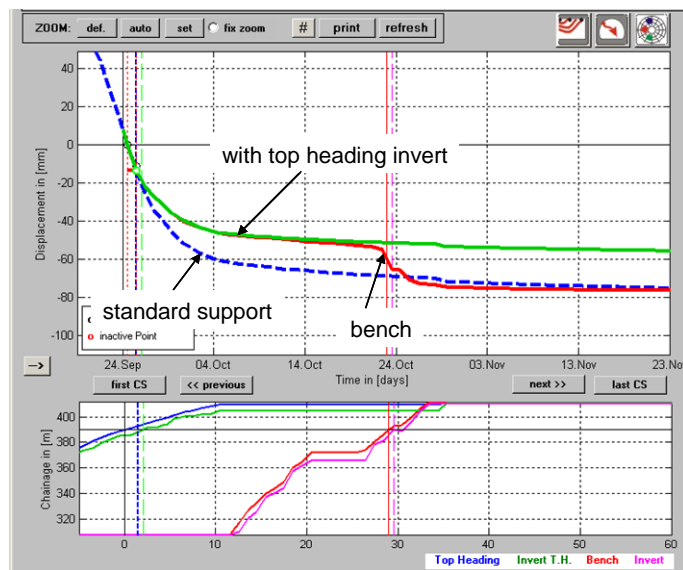


Figure 5. Predicted development of the displacements for a top heading-bench-invert excavation sequence. In the top heading a temporary shotcrete invert is installed

Based on an assumed construction progress, the development of the displacements for the top heading without temporary invert is predicted (dashed line). Then the temporary invert is added, showing in a decrease of displacements. In a third step the additional displacement caused by the bench and invert excavation are predicted. This approach allows an assessment of the effectiveness of various support types and the influence of the construction sequence on the development of displacements in a very early stage. With some experience, the displacement development can be predicted already a couple of hours after excavation, if readings are taken in sufficiently short intervals. If it for example shows, that displacements would be in an unacceptable range, support can be increased, and the efficiency immediately simulated.

During excavation, the measured displacements can be compared to the predicted ones (Figure 6). The big advantage of such a tool compared to traditional plots of the displacement history only is the fact that also for unsteady advance a clear assessment of the normality of the system behavior is possible.

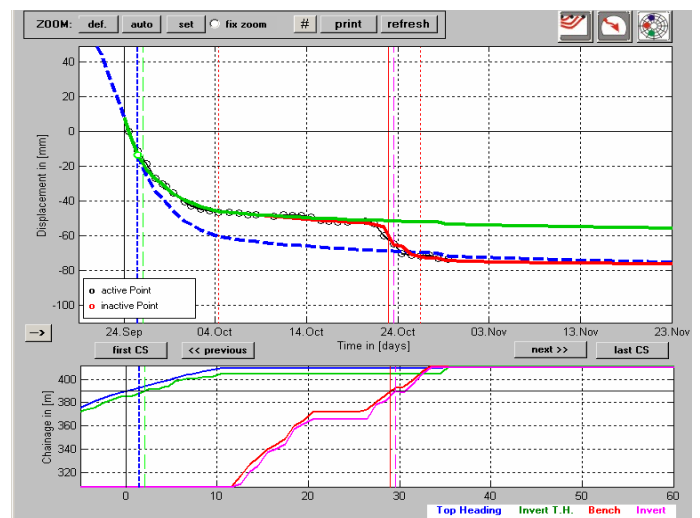


Figure 6. Comparison of the measured displacements (circles) to the predicted ones (full lines)

It has been shown in many applications that the empirical formulations used in GeoFit[®] for the prediction of displacements very well reflect the ground reaction. Deviations from the predicted displacement development thus can be attributed to unusual behavior or failure in the ground – lining-system. A reasonable application of the software however needs quite some experience.

The previous example shows an excavation with a very steady advance rate. In such cases the interpretation of displacement history diagrams is pretty simple, as the displacement rate should continuously decrease with increasing distance between face and measuring section. More challenging is the assessment of the normality of the system behavior in case of an unsteady advance. Figure 7 shows an example, where a weak zone in the ground to the left of the

tunnel led to overstressing, showing in a pronounced deviation from the predicted displacement development. If the comparison between predicted and measured displacements is done routinely, such deviations can be easily detected, and mitigation measures implemented in time.

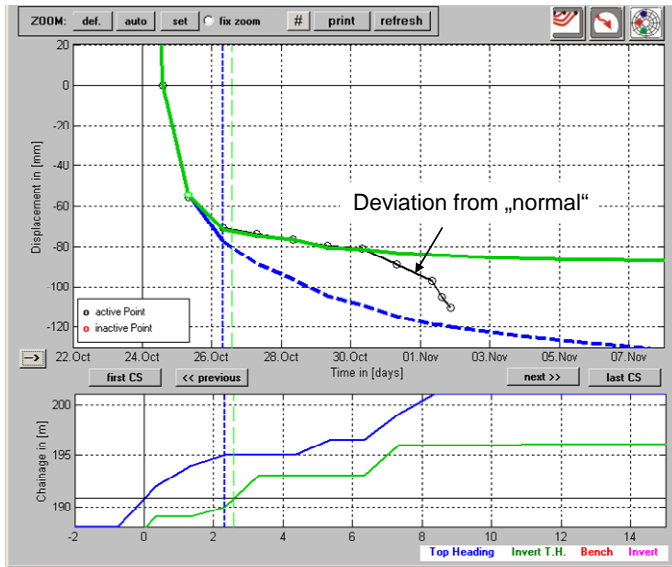


Figure 7. Typical example of the deviation of the system behaviour from the predicted development

3.1.5 Check of lining stresses

For shallow tunnels in urban areas usually a pretty stiff lining is used to minimize ground deformations. As such linings tend to fail in a brittle mode at low levels of deformation the evaluation of the dis-

placements only does not provide a reliable indication of the state of stress. The results of 3D optical monitoring can be used to evaluate the strain development.

With an appropriate material model, considering time dependent hardening and strength development, as well as the effects of shrink, temperature and creep, the actual stress level in the lining can be evaluated (Schubert, P. 1988, Aldrian 1991, Rokahr et al. 2002, Hellmich et al. 1999, Macht 2002, Tunnel:Monitor, 2006).

The evaluation of stresses in the lining not only can be done after monitoring results are available, but also can be predicted on the basis of the predicted development of the displacements. This is particularly important in cases, where the tunnel has to be constructed in weak ground under high primary stresses. Damages to stiff linings are a common problem under such conditions. A change of the lining characteristic from a stiff lining to a ductile lining can be advisable. Figure 8 shows a prediction of the lining utilization on the basis of predicted displacements. It can be seen, that with a stiff lining, the capacity of the shotcrete would be exceeded after approximately one day. Integrating ductile elements into the lining (Moritz, 1999), leads to a reduction in the lining stresses to a maximum of about 50 % of its capacity.

Appropriately using the tools available for the processing and interpretation of the monitoring data allows a continuous control of the tunneling process, thus minimizing the “surprises”.

4 CONCLUSIONS

The uncertainties associated with underground construction call for continuing design during construction. A continuous adjustment of the excavation and support methods to the actual rock mass conditions contributes to safe and economical tunneling.

A prerequisite for successful application of such an observational approach is an appropriate basic design, which should incorporate means and tools to cope with difficult conditions. Another must is the implementation of an adequate monitoring system, allowing the acquisition of accurate data in due time. The huge amount of data obtained during excavation needs to be processed, evaluated and interpreted. For an efficient decision process the results have to be available practically in „real time“, which requires equipping the site with advanced software for data management and evaluation. Quite some progress was made in this respect over the last decade.

Interpretation of geological, geotechnical, and monitoring data due to the complexity of the ground and the interaction between ground and construction still relies a lot on education and experience. Responsible owners account for this by hiring qualified

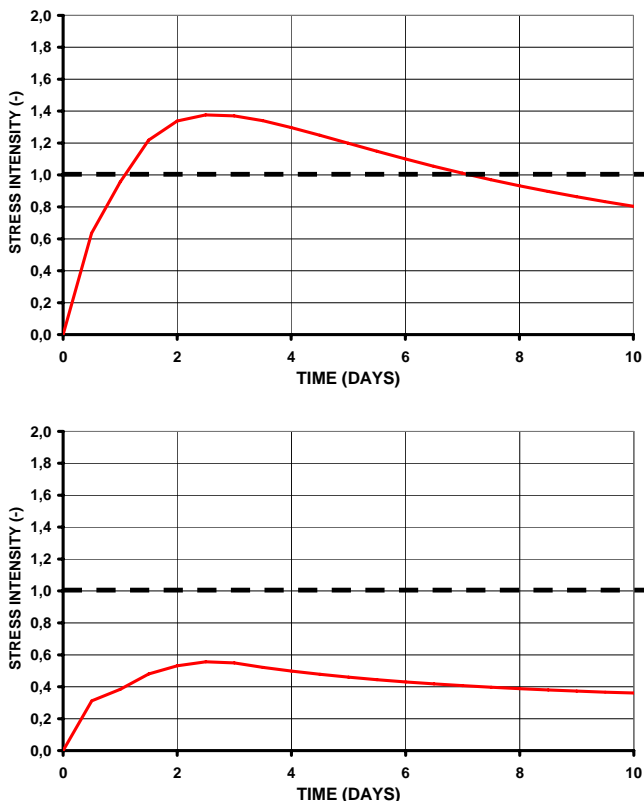


Figure 8. Development of lining utilization factor for a stiff lining, using predicted displacement development (upper diagram), and lining utilization for a ductile lining (lower diagram)

geotechnical personnel for the site assistance. Not only can qualified staff contribute to reduce accidents and damages, but can also identify opportunities to make the construction smooth and economical by optimally adjusting construction methods to the encountered ground. Hard- and software for the collection, processing and evaluation of monitoring data have enormously improved over the last decade. Last but not least, the contractual setup has to allow the continuous optimization of the construction.

Internet has made it possible to involve also off-site experts at comparatively low costs in real time. All data can be made available on a server, which allows following up the construction from any part of the world. This way, offsite experts can give advice on a sound basis at any time required without the necessity to visit the site. It is quite clear that data transmitted in such a way cannot replace the personal impression. Thus for the time being, competent staff on site is still a very important element of a successful implementation of an observational approach in tunneling.

Advances in modeling ground and system behaviours, as well as monitoring techniques have made tunneling less risky. The observational approach has changed from a “design as you go” procedure with many surprises and costly delays to an engineering approach with a flexible response to the actual ground conditions.

REFERENCES

- Aldrian, W. 1991. Beitrag zum Materialverhalten von früh belastetem Spritzbeton. Ph.D. thesis, Mining University Leoben
- Barton, N., Lien, R., Lunde, J. 1974. Engineering classification of rock masses for the design of tunnel support. *Rock Mechanics* 6(4). 189-236.
- Barton, N. 1998. NMT support concepts for tunnels in weak rocks. In A. Negro Jr., A.A. Feirreira (eds.), *Tunnels and Metropolises; Proc. intern. symp., Sao Paulo*: 273-279. Balkema.
- Bieniawski, Z.T. 1974. Geomechanics classification of rock masses and its application in tunnelling. *Adv. in rock mech.* 2(A). 27-32.
- Bieniawski, Z.T. 1989. Engineering rock mass classifications. 251 pp. New York: Wiley.
- Feder, G. 1978. Versuchsergebnisse und analytische Ansätze zum Scherbruchmechanismus im Bereich tiefliegender Tunnel. *Rock Mechanics* (6). 71-102.
- Button, E., Schubert, W., and Moritz, B. 2003. The Application of Ductile Support Methods in Alpine Tunnels. *Proc. 10th ISRM Congress*, 163-166
- Gaich, A., Pötsch, M. Fasching, A., Schubert, W. 2004. Contact-free measurement of rock mass structures using the JointMetriX3D system. *International Journal of Rock Mechanics and Mining Sciences*, Vol. 41, No. 3, 414
- Gaich, A., Schubert, W., Poetsch, M. 2005. Three-dimensional rock mass documentation in conventional tunnelling using JointMetriX3D. *Underground Space Use: Analysis of the Past and Lessons for the Future. Proceedings of the 31st ITA-AITES World Tunnel Congress, Istanbul, 2005*, 59-64.
- Goricki, A. 2001. Classification of Rock Mass Behaviour based on a Hierarchical Rock Mass Characterization for the Design of Underground Structures. Doctoral Thesis, Graz University of Technology
- Goricki, A., Schubert, W., Steidl, A., Vigl, A. 2002. Geotechnical Risk Assessment as the Basis for Cost Estimates in Tunneling. *Felsbau* 20, Nr. 5, 24-30, VGE Essen
- Grossauer, K. 2001. Tunnelling in Heterogeneous Ground. – Numerical Investigation of Stresses and Displacements. Master thesis at the Institute for Rock Mechanics and Tunneling, Graz University of Technology, Austria.
- Grossauer, K, Schubert, W & Kim, C.Y. 2003. Tunnelling in Heterogeneous Ground – stresses and displacements. In: *Proceedings of 10h Int. Congr. On Rock Mechanics*. 437-440. South African Institute of Mining and Metallurgy.
- Hellmich, Ch., Macht, J., Mang, H. 1999. Ein hybrides Verfahren zur Bestimmung der Auslastung von Spritzbetonschalen. *Felsbau* 17, No.5, 422-425
- Liu, Q., Brosch, F.J., Riedmüller, G., Klima, K., Schubert, W. 1999. Application of a data base system during tunnelling, *Felsbau* 17 (1), 47-50
- Macht, J. 2002. Hybrid analyses of Shotcrete tunnel linings: Assessment and online monitoring of the level of loading, Doctoral Thesis, Technical University Vienna
- Moritz, B. 1999. Ductile support system for tunnels in squeezing rock. In Riedmüller, Schubert, Semprich (eds), *Schriftenreihe Gruppe Geotechnik Graz*, Volume 5
- OeGG, 2001. Richtlinie für die Geomechanische Planung von Untertagebauarbeiten mit zyklischem Vortrieb, Austrian Society for Geomechanics
- ONORM B 2203-1, 2001. Underground Works - Works Contract – Part 1 Cyclic Driving. Austrian Standards Institute
- Rokahr, R., Stärk, A., Zachow, R. 2002. On the art of interpreting measurement results; *Felsbau* 20, No. 2
- Schubert, P. 1988. Beitrag zum rheologischen Verhalten von Spritzbeton, *Felsbau* 6, Nr.3, 150 – 155
- Schubert, W., Budil, A., 1995. The Importance of Longitudinal Deformation in Tunnel Excavation. *Proceedings 8th Int. Congress on Rock Mechanics (ISRM)*, Vol.3, 1411-1414. Rotterdam: Balkema.
- Sellner, P. 2000. Prediction of displacements in tunnelling. In Riedmüller, Schubert & Semprich (eds), *Schriftenreihe der Gruppe Geotechnik Graz*, Heft 9.
- Steindorfer, A., 1998. Short term prediction of rock mass behaviour in tunnelling by advanced analysis of displacement monitoring data. In Riedmüller, Schubert & Semprich (eds), *Schriftenreihe der Gruppe Geotechnik Graz*, Heft 1.
- Sulem, J., Panet, M., Guenot, A. 1987. Closure analysis in deep tunnels. *Int. Journal of Rock Mechanics and Mining Science*, Vol. 24, 145-154.
- TUNNEL:MONITOR, 2006. www.tunnelmonitor.com