World Tunnel Congress 2013 Geneva
Underground – the way to the future!
G. Anagnostou & H. Ehrbar (eds)

ISBN 978-1-138-00094-0

Gotthard Base Tunnel – 3-dimensional numerical calculations for part of the Clavaniev Zone considering geotechnical measurement data

C. Volderauer[(1)], R. Galler[(2)], T. Marcher[(3)]

[(1)]*until 31 August 2012, Chair of Subsurface Engineering, Montanuniversität Leoben, Leoben, Austria*

since 3 September 2012, Department of Geotechnics, ILF Consulting Engineers ZT GmbH, Rum, Austria

[(2)]*Chair of Subsurface Engineering, Montanuniversität Leoben, Leoben, Austria*

[(3)]*Department of Geotechnics, ILF Consulting Engineers ZT GmbH, Rum, Austria*

ABSTRACT: This paper discusses the numerical analysis of a specific part of the deep seated Gotthard Base Tunnel, the longest railway tunnel in the world. A section of the western tube situated in the Clavaniev Zone was simulated with the finite element program ABAQUS. Geological and geotechnical documentation from the design stage and the construction stage as well as daily and monthly reports of the construction site were provided by AlpTransit Gotthard Ltd. This documentation has been used to perform 3-dimensional analyses with implementation of support elements in a realistic construction sequence. For the simulations different constitutive laws with a focus on time-dependent ground behaviour were used. Preliminary calculations have shown that measureable tunnel deformations are over predicted by far compared to measurement results when using standard FE-boundary conditions. To obtain more realistic results, a depth-dependent stiffness approach, triggered with triaxial test data and analytical calculations, was used in 3-dimensional simulations. This approach made it possible to achieve good compliance between simulation results and measured radial deformations of several cross sections and measured deformations of RH-Extensometers which were used at the Gotthard Base Tunnel for the first time.

1 Introduction

Major tunnelling projects with very high overburdens were built in Switzerland within the last decades. One famous example is the Gotthard Base Tunnel, the longest railway tunnel in the world. For a research project, with the project partners being the Chair of Subsurface Engineering, Montanuniversität Leoben and ILF Consulting Engineers ZT GmbH, geological and geotechnical documentation from the design and the construction stage as well as daily and monthly reports of the construction site and results from measurements were provided by AlpTransit Gotthard Ltd. for a specific part of the Gotthard Base Tunnel. AlpTransit Gotthard Ltd is the constructor of the Gotthard axis of the New Rail Link through the Alps with base tunnels through the Gotthard and Ceneri.

The aim of this research project was to investigate whether the measurement results from the construction site can be matched with the results from 3-dimensional FE simulations, also taking into consideration time-dependent material behaviour based on information available during the design and the construction stage.

2 Gotthard Base Tunnel

2.1 Geological and geotechnical parameters regarding the research investigating section

For the actual research investigation a 35 m long section of the western tube of the new Gotthard Base Tunnel situated in the Clavaniev Zone (CZ) between the northern intermediate Tavetsch formation (TZM) and the Aare Massif was investigated. The section of interest is at the end of the Clavaniev Zone at the transition to the Aare Massif (Figure 1), starts at chainage 2090 and ends at chainage 2125. This section was chosen based on the available documentation of:

- monitoring of stratification and cleavage;
- measuring data of deformations in the tube due to an intensive monitoring program;
- geotechnical parameters, gathered during design and construction stage;
- excavation and support measures;
- measuring data of the installed reverse-head extensometers (RH extensometers).

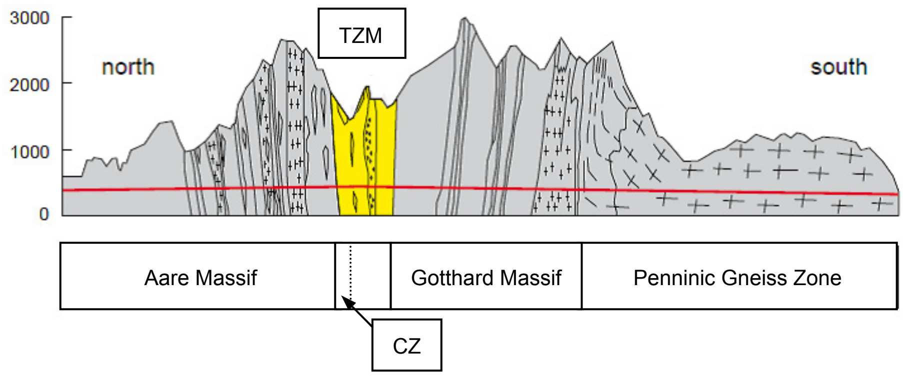


Figure 1. Schematic geological section of the Gotthard Base Tunnel (according to Kovari 2009)

The Clavaniev Zone and the intermediate Tavetsch formation consist of gneisses, slates and phyllites in layers of decimetres to decametres. The quality of these rock masses varies from intact to more or less kakiritic.

A laboratory testing programme in the design stage was performed to investigate the strength and deformation properties of specimens retrieved from a 1700 m long exploratory borehole (Vogelhuber 2007). By means of this borehole the expected squeezing conditions in the intermediate Tavetsch formation and the Clavaniev Zone were investigated. During the construction stage laboratory tests were performed on specimens retrieved from boreholes from the tunnel face (Anagnostou et al. 2008).

2.2 Construction method

Full face excavation of the tunnel was carried out using tunnel excavators with lengths of rounds of 1.34 m to 1.0 m. The excavation radius was between 5.89 and 6.24 m with an over-excavation of 0.5 and 0.7 m to accommodate the predicted deformations. To handle these deformations, two overlapping rings of sliding steel arches (TH 44/70) were installed as yielding support elements after each round, with a spacing of 0.66 or 0.5 m. After convergences subsided, these two rings were rigidly connected by closing friction loops. Furthermore up to 26 rock bolts, each with a length of 8 m, were installed over the whole circumference of the tunnel after each round. About 30 meters behind the face a shotcrete lining of 0.5 m was applied. To provide a safe face, 50 to 60 pieces of 12 to 18 m long IBO-bolts with an overlap of 6 to 9 m and a 0.1 m thick shotcrete lining were installed.

2.3 Measurement programme

Optical measurements were performed with 5 to 7 points per cross section, with the distance between measuring sections varying from 4 m to 13 m. RH-Extensometers (Thut et al. 2006), a new development in the field of geotechnical monitoring for determining the pre-displacements in front of the face, were installed in the Gotthard base tunnel for the very first time. The core extrusion was measured with 7 pieces of 24 m long RH-Extensometers at the axis of the tunnel. Each set of RH-Extensometers overlapped 4 to 8 m with the preceding ones to provide a continuous measurement of the core extrusion.

3 Finite element model

3.1 Model design

For 3-dimensional numerical simulations a model was generated with a size of 100 m in depth, 200 m in height and 118 m in length, with a total of 135,000 hexagonal linear elements using the software ABAQUS. The overburden of the selected area is about 960 m, by choosing a specific weight of 25 kN/m³ a primary stress level of 24 MPa was calculated. For numerical simulations a lateral pressure coefficient of K_0 = 1.0 was used. At the bottom of the model, vertical and horizontal boundaries were applied, at the sidewalls of the model, movements in horizontal directions were set to zero. The advance was modelled by deactivating elements of the circular tunnel using the “Model Change” function of ABAQUS. In the initial stage all support elements were deactivated and re-activated again continuously with ongoing advance.

Three ground layers normal to the tunnel axis have been implemented in accordance with the geological section (Figure 2a). The first two layers are part of the Clavaniev Zone, layer 3 is characterized by the more competent Aare Massif.

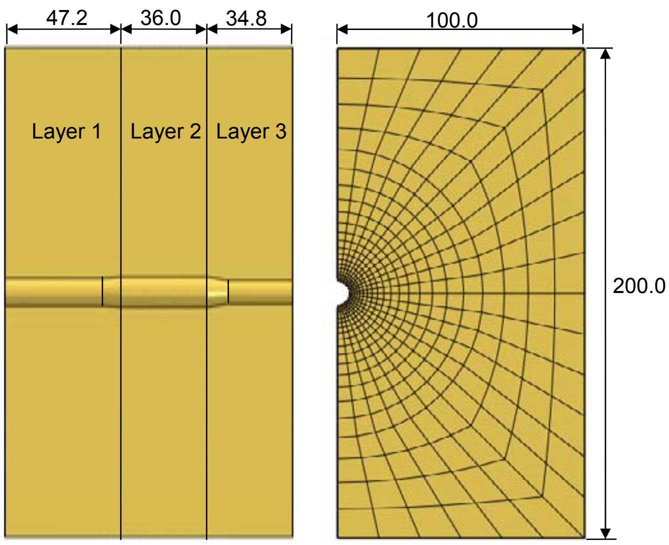


Figure 2. a) Front view of the model including dimensions; b) side view of the model including dimensions and mesh

At the end of layer 1, the tunnel was expanded from 5.89 m to a radius of 6.24 m over a length of about 7 m and at the beginning of the Aare Massif reduced again to 5.18 m over a length of about 10 m (Figure 2a). Figure 2b shows a cross section of the tunnel and the applied mesh.

3.2 Implemented constitutive laws

For numerical simulations linear-elastic ideal-plastic constitutive laws were used. First numerical simulations were performed with a Mohr-Coulomb failure criterion, using ground parameters gathered during the design stage. The rock mass parameters, cohesion (c), friction angle (ϕ) and dilatancy (ψ) are given in table 1.

Table 1. Ground parameters using a Mohr-Coulomb failure criterion

Layer	c [kPa]	ϕ [°]	ψ [°]
1	500	26	5
2	300	26	5
3	1100	29.8	7

Afterwards a Drucker-Prager failure criterion was used. Models were simulated for the Drucker-Prager criterion with parameters for a compression cone and for an approximation cone to obtain comparable results with the Mohr-Coulomb failure criterion. The parameters, cohesion (d), friction angle (β) and dilatancy (ψ) are shown in Table 2 for the different ground layers for the approximation cone of the Drucker-Prager failure criterion.

Table 2. Ground parameters for a Drucker-Prager failure criterion

Layer	d [kPa]	β [°]	ψ [°]
1	740	35.9	8.2
2	445	35.9	8.2
3	1570	29.5	11.4

3.3 Use of a depth-dependent Young's modulus

Observations show an increase of Young's modulus of rock mass with increasing distance to the tunnel (Asef et al. 2002). This depth dependency is even more pronounced in weaker rock masses (Verman et al. 1997). As results of first numerical calculations using the initial stiffness properties did not match the measurement data, a depth-dependent Young's modulus was implemented in the numerical model for the different ground sections. Basic data for this approach were results from triaxial tests performed at the Institute of Geotechnical Engineering of the ETH Zürich (Anagnostou 2008) with lateral pressures of 2, 5 and 9 MPa for Layer 2. With these test results the Young's modulus without lateral pressure was determined using the assumptions given in Asef et al. (2002) and Verman et al (1997) according to equations 1 and 2.

$$E_{\sigma 3(t)} = E_{\sigma 3(t=0)} \frac{200 \frac{\sigma_{3(t)}}{\sigma_{cj}} + b}{\frac{\sigma_{3(t)}}{\sigma_{cj}} + b} \quad (1)$$

with:

$$b = 15 + 60 e^{-0,18\,\sigma_{ci}} \quad (2)$$

where $E_{\sigma 3(t)}$ is the Young's modulus at a given lateral pressure $\sigma_{3(t)}$, $E_{\sigma 3(t=0)}$ is the Young's Modulus without lateral pressure, σ_{ci} is the unconfined compressive strength of intact rock and σ_{cj} is the unconfined compressive strength of jointed rock.

Afterwards Young's moduli for lateral pressures of 15 and 20 MPa (equation 1 and equation 2) were calculated using equation 1 and 2. Young's moduli for ground layers 1 and 3 were gathered with extrapolation from ground layer 2, based on the original Young's moduli used in the design stage. The calculated Young's moduli and the applied lateral pressures can be seen in Table 3. As mentioned before, layer 1 and 2 consist of intact and/or kakiritic gneisses, slates and phyllites. The highest value for Young's moduli for these sections was chosen to be the modulus of intact phyllite, which is about 10 GPa.

With these results the distances to the tunnel perimeter for the different Young's moduli were calculated using an analytical approach according to Sulem et al. (1987) (Figure 3a). The implementation in the model and the range of each Young's modulus can be seen in Figure 3b.

Table 3. Young's moduli in dependence of the depth for the different layers

Confining Stress Layer	0 [MPa]	2 [MPa]	5 [MPa]	9 [MPa]	15 [MPa]	20 [MPa]
1	1.6	3.0	5.4	7.8	10	10
2	0.8	1.5	2.7	3.9	5.8	7.5
3	4.4	8.3	14.9	21.5	31.9	41.3

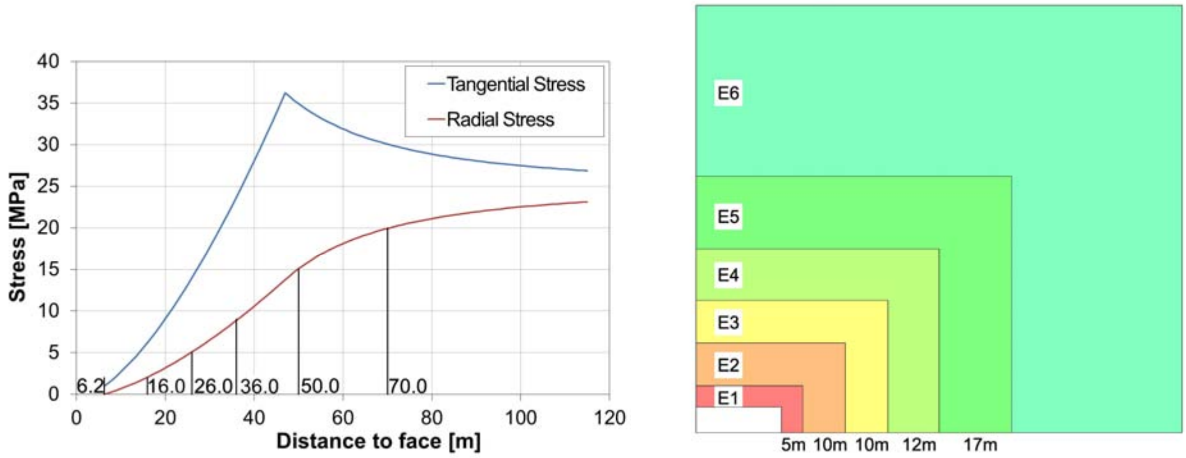


Figure 3. a) Determination of the distance to the tunnel perimeter; b) Distribution of the different Young's moduli in the numerical model

During each excavation round, Young's moduli in front of the face and in radial direction were adjusted (Figure 3b).

3.4 Implementation of support elements

The amount of the implemented support was gathered from the monthly project reports and from the design drawings.

Face bolts and rock bolts in radial direction were implemented with a passive cohesion increase as suggested by Egger (1973) and Wullschläger (1988). The installation of the steel arches was simulated with beam elements in the numerical model. The beam elements were activated at a distance of 15-20 m behind the face, simulating closing the friction loops on the construction site. The time of closing the friction loop and installing the steel arches in the numerical simulation was determined by investigating the time displacement lines of the measurement results for the different cross sections. The shotcrete was installed at a distance of about 30 m to the face where radial deformations already declined.

3.5 Time-dependent material behaviour

Time-dependent calculations were performed with the parameters from the approximation cone calculated for the Drucker-Prager criterion (Table 2). A creep law was used with a rate-dependent power law model. To obtain reasonable results, a static excavation step was always followed by a "Visco step" to see the influence of the time-dependent material behaviour. Parameter studies were performed to obtain comparable results with measurements. Table 4 shows the used power law parameters. For the three ground layers the same creep parameters were used.

Table 4. Creep parameters for simulating time-dependent material behaviour

Creep Parameter	A $[N^{-n}m^{2n}K^{-m-1}]$	n [-]	m [-]
	2E-11	1.0	-0.6

4 Results and discussion

4.1 Use of a depth-dependent Young's modulus

Figure 4 shows the measured deformations of the tunnel at the crown and the sidewalls. Results at the crown from simulation with and without the use of a depth-dependent Young's modulus are also shown. The values for simulation with a depth-dependent Young's modulus are mentioned in Table 3.

The values used in the simulation without a depth-dependent Young's modulus are 4, 2 and 11 GPa for the ground layers 1 to 3 (compare with Table 3).

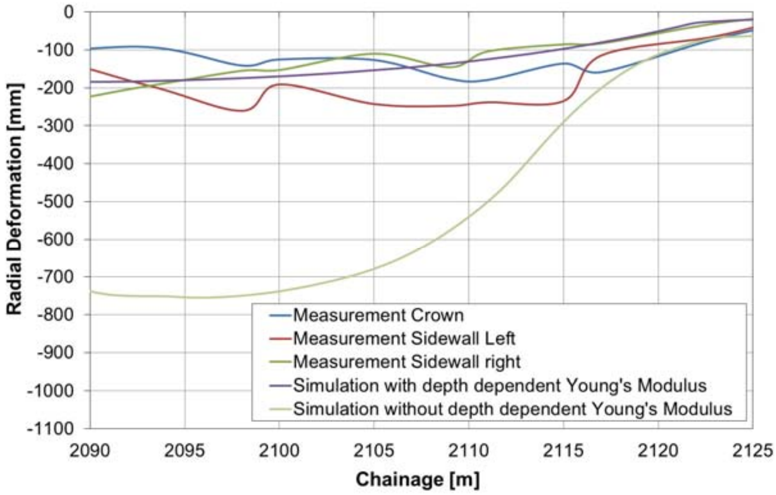


Figure 4. Measurement results and results from simulation

The measurement results for the investigated section show that the deformations vary from 100 to 250 mm at the beginning and the middle of the section and to an amount of about 50 mm at the transition to the more competent Aare Massif. The measurable deformations from the numerical simulations without a depth-dependent Young's modulus vary from about 750 mm at the beginning to about 70 mm at the end of the investigated section. With the implementation of the depth-dependent Young's modulus these deformations decrease to 180 mm at the beginning and 30 mm at the end of the investigated section. Figure 4 shows that deformations are over-predicted by far when using only a homogenous Young's modulus. With the use of a depth-dependent stiffness approach the simulation results meet the measurements from the construction site.

4.2 Different constitutive laws

A Mohr Coulomb and a Drucker Prager failure criterion were used. For the simulations using the Drucker Prager failure criterion two parameter sets were used as mentioned in chapter 3.2.

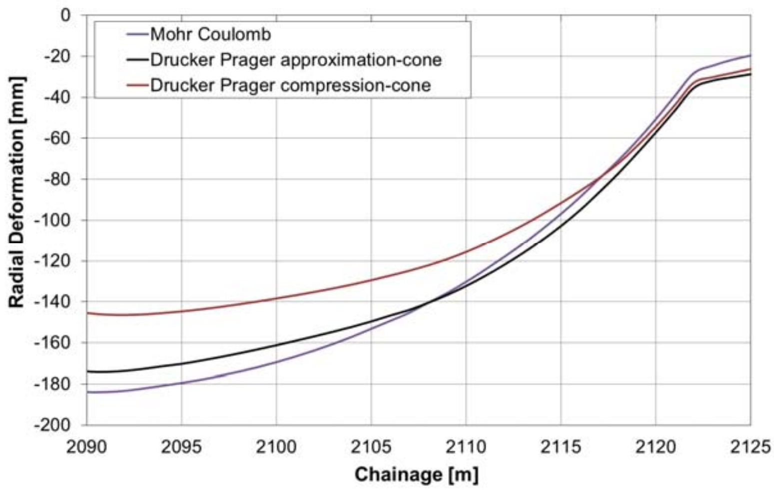
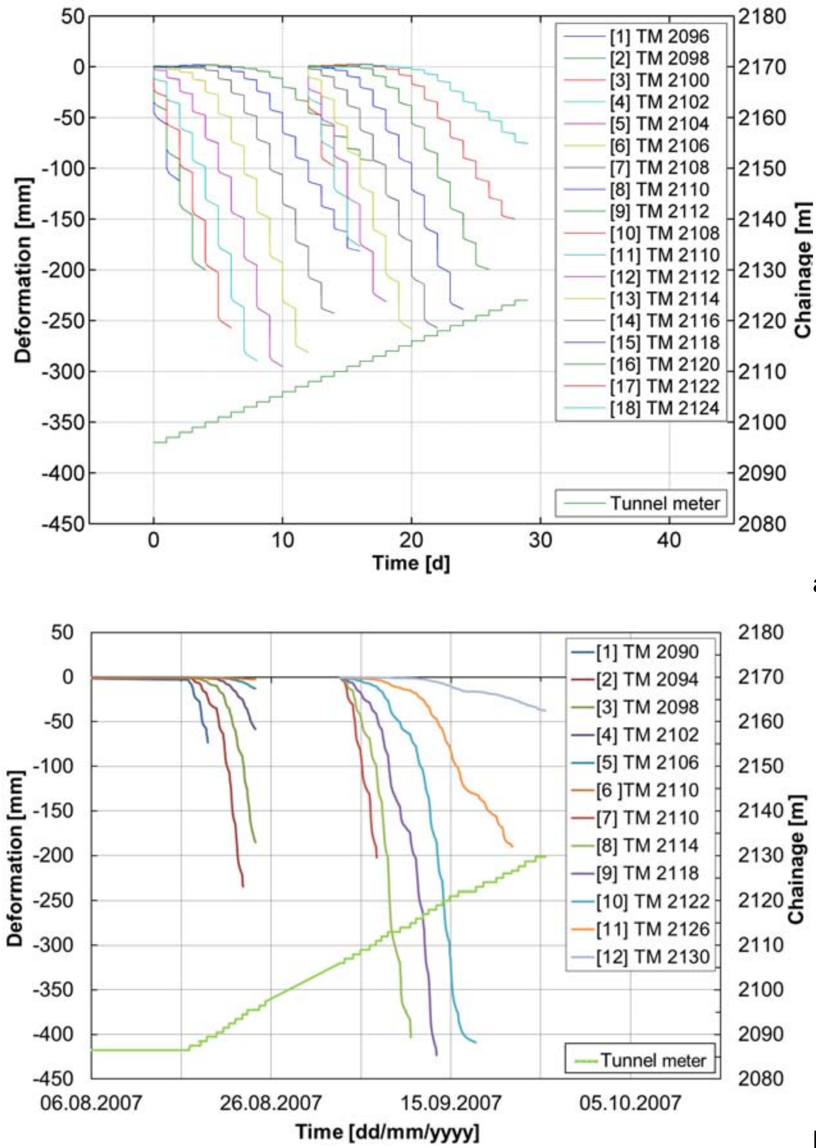


Figure 5. Results for simulations with a Mohr Coulomb and a Drucker Prager failure criterion

With the parameter set for the approximation-cone the simulation results are similar to the ones using the Mohr Coulomb failure criterion. By using the parameters for the compression cone the deformations are smaller.

4.3 Time-dependent material behaviour

The results for the extrusion in front of the tunnel face in axial direction for a strain-dependent creep law are shown in Figure 6a, the measured deformations are shown in Figure 6b.



a)

b)

Figure 6. a) Axial extrusion in front of the face (simulation results); b) measurement results from RH-Extensometers

The magnitude of the measured deformations in front of the face is about 400 mm for three RH-Extensometers. The others are significantly smaller and vary from 10 to 230 mm. The difference in this dimension most likely results from changing geological/geotechnical conditions. The results from the simulations show axial extrusions in front of the face ranging from 50 to 300 mm, which lies between the minimum and the maximum extrusion that was measured.

In addition, it is interesting to mention Cantieni et al. (2011), who investigated the connection between radial displacements and the core extrusion in front of the face for the Gotthard Base Tunnel. It was found that it is only possible to a certain extent to predict radial deformations based on results from RH-Extensometers.

5 Conclusion

It is shown that it is possible to obtain comparable results between numerical simulations and measurement results of such a challenging construction site like the Gotthard Base Tunnel. This is valid when looking into radial displacements and the extrusion of the face.

A prerequisite for these simulations is to have very good geological and geotechnical documentation during the design and the construction stage as was the case during the construction of the Gotthard Base Tunnel. Especially the determination of Young's moduli with different lateral pressures has great relevance for predicting deformations in tunnels with squeezing conditions.

6 Acknowledgements

We want to thank Dipl.-Ing. ETH Heinz Ehrbar und Dr. Rupert Lieb from the AlpTransit Gotthard Ltd for providing the high quality data from the construction lot Sedrun, especially the Tavetsch Intermediate formation as well as the Clavaniev Zone and the permission to use these data for this specific research project.

Thanks are also due to Dr. Max John who helped the project team with his great experience on several occasions.

The project team furthermore wants to thank the Austrian Research Promotion Agency (FFG) for funding this project in the course of the programme line "ModSim Computational Mathematics".

7 References

Anagnostou, G., Pimentel, E., Cantieni, L. (2008) Report on behalf of AlpTransit Gotthard Ltd.: Felsmechanische Laborversuche Los 378 — Schlussbericht—AlpTransit Gotthard Basistunnel, Teilabschnitt Sedrun. ETH Zurich, Institute for Geotechnical Engineering. Chair of Underground Construction. Zurich, Switzerland.

Asef, M., Reddish, D.J. 2002. The impact of onfining stress on the rock mass deformation modulus, Géotechnique 52, 4, 235-241.

Cantieni, L., Anagnostou, G., Hug, R. 2011. Interpretation of Core Extrusion Measurements When Tunnelling Through Squeezing Ground. Rock Mech. and Rock Eng. 44, 641-670.

Egger, P. 1973. Einfluss des Post-Failure Verhaltens von Fels auf den Tunnelausbau unter besonderer Berücksichtigung des Ankerausbaus, Universität Karlsruhe, Dissertation.

Kovari, K. 2009. Design Methods with Yielding Support in Squeezing and Swelling Rocks. World Tunnel Congress. Budapest. Hungary.

Sulem, J., Panet, M., Guenot, A. 1987. An Analytical Solution for Time-dependent Displacements in Circular Tunnel. International Journal on Rock Mechanics and Mining Sciences & Geomechanics Abstract 24. 155-164.

Thut, A., Naterop, D., Steiner, P., Stolz, M. 2006. Tunnelling in squeezing rock – yielding elements and face control. 8th International Conference on Tunnel Construction and Underground Structures. Ljubljana.

Verman, M., Singh, B., Viladkar, N., Jethwa, J.L. 1997. Effect of Tunnel Depth on Modulus of Deformation of Rock Mass. Rock Mech. Rock Eng. 30, 3, 121 – 127.

Vogelhuber, M. (2007) Der Einfluss des Porenwasserdrucks auf das mechanische Verhalten kakiritisierter Gesteine. PhD Thesis. ETH Zurich. Zurich, Switzerland.

Wullschläger, D. 1988. Ein Verbundwerkstoffmodell für die Systemankerung im Tunnelbau. Dissertation. Universität Karlsruhe.