

ASSESSMENT OF ROCK MASS PARAMETERS BASED ON THE MONITORING DATA FROM THE KORALM INVESTIGATION TUNNELS

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Equilibrar a Poesia com a Engenharia

Anselmo Mendes

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ABSTRACT

The Koralm project is a new 130 km long railway link in the South of Austria connecting the cities of Graz and Klagenfurt. The core of the new section is the 32.9 km Koralm base tunnel.

Since 2002 two shafts and three tunnel lots have been under progress from the portals of the base tunnel. The activities shall investigate the in-situ conditions of the sedimentary sections and parts of the crystalline section of the Koralm ridge. The project also passes a large regional fault zone, the Lavanttal fault zone. The investigation measures shall provide improved data for the tender of the tunnel drives at both sides of the Koralm.

The research deals with the findings from the two Western tunnels, the three km long tunnel Mitterpichling and the five km long tunnel Paierdorf. These investigation tunnels provided extremely valuable data from the observation of the ground behaviour during tunnelling as well as the required methods for stabilization of the face and tunnel sections. The Author has made a systematic back-analysis of the tunnel at typical geological formations. The back-analysis is based on detailed monitoring data included in a spreadsheet based on the convergence confinement theory. Relevant bibliographic references allow the definition of a narrow range from the values arising from the spreadsheet.

From this work more accurate geotechnical parameters can be derived. Also, by means of numerical analysis with the finite difference program FLAC the back calculated values could be verified.

Generally it appears that real parameters of stiffness and shear strength of the rock mass can be assumed to be considerably higher than the prognosis derived from borehole data.

KEYWORDS: back-analysis, rock mass strength, tunnel monitoring data, fault zone, soft rock.

RESUMO

O projecto Koralm pretende criar uma nova ligação ferroviária de cerca de 130km entre as cidades de Graz e Klagenfurt no sul da Áustria. Este projecto levará à construção de um túnel com aproximadamente 32.9km que atravessará a montanha Koralpe.

Desde 2002 que dois poços e três frentes de escavação estão a ser construídos nas extremidades do túnel. Estes trabalhos têm como principal objectivo investigar in-situ as secções sedimentares e ainda parte das secções cristalinas da cordilheira. O traçado desenvolve-se igualmente numa zona de falha regional designada por falha de Lavanttal. As investigações visam fornecer dados para que um melhor dimensionamento da escavação do túnel principal, possa ter lugar.

A presente dissertação tem por base os dois túneis piloto situados na entrada Oeste do túnel principal. Mitterpichling com 3km de extensão e Paierdorf com cerca de 5km. A partir da construção destes túneis inúmeros e valiosos dados foram recolhidos a partir da monitorização do comportamento do maciço rochoso, bem como dos métodos empregues para a estabilização da frente de escavação e hastes. Estes dados serviram de base para o Autor realizar uma retroanálise sistemática com vista a determinar os parâmetros resistentes do maciço (i.e. ângulo de atrito, coesão e módulo de deformabilidade). Esta retroanálise foi realizada com recurso a uma folha de cálculo que incorpora a teoria da convergência confinamento aplicada a escavações circulares, bem como as referências bibliográficas mais relevantes.

A metodologia proposta foi aplicada não só à zona de falha, mas também às diferentes condições geológicas mais relevantes encontradas durante os trabalhos exploratórios, nomeadamente: areia, cascalho e siltito. Desta análise, valores mais precisos para os parâmetros resistentes do maciço foram encontrados sendo que, a verificação dos resultados obtidos com o modelo proposto, foi realizada com recurso ao programa de diferenças finitas FLAC.

De uma maneira geral pode-se concluir que o maciço apresenta na realidade maior resistência do que a estimada nos trabalhos de prospecção geotécnica.

PALAVRAS-CHAVE: retroanálise, parâmetros maciço rochoso, monitorização de túneis, falhas, rochas brandas.

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SYMBOLS AND ABBREVIATIONS

BIM-rock – block-in-matrix rock

c – cohesion [MPa]

E – Deformation modulus [MPa]

$E_{\min}$ – minimum deformation modulus calculated from the convergence confinement method [MPa]

FLAC – Fast Lagrangian Analysis of Continua, numerical analysis software

GA – Gebirgsart (from German: rock mass type)

GRC – ground reaction curve

H – overburden [m]

LDP – longitudinal displacement profile

p_0 – in situ stress [MPa]

p_i – internal support pressure [MPa]

p_i^{cr} – critical internal support pressure [MPa]

p_s – support pressure [MPa]

p_s^{max} – maximum support pressure [MPa]

r_0 – initial tunnel radius [m]

RESS – required excavation and support sheet

r_p – plastic zone radius [m]

SCC – support characteristic curve

s – influence area

u_0 – displacement at the face [mm]

u_0/u_{mes} – ratio of pre displacement to measured displacement

u_{ie} – elastic displacement [mm]

u_{ip} – plastic displacement [mm]

u_{max} – maximum short term radial displacement calculated from plain strain analysis [mm]

u_r – radial displacement of the wall at a given point from the tunnel face [mm]

γ – rock mass specific weight [kN/m³]

σ_1 – major principal stress [MPa]

σ_3 – minor principal stress [MPa]

σ_{at} – atmospheric pressure [MPa]

σ_{cm} – uniaxial compressive strength [MPa]

ϕ – friction angle of the rock mass [°]

INTRODUCTION

1.1. INTRODUCTION

Efficient tunnel construction starts long before the mechanical process is set into place. In fact, depending on the design and construction methods used, it may take longer to plan tunnel operations than to construct the physical work. Huge effort is made to predict the behaviour of the rock mass to be excavated in the most realistic way. This first step is generally guided by a geotechnical team. Detailed geological investigation of the alignment to be built enables defining a model consisting of the most important rock mass types, faults, discontinuities etc. From this study it is possible to define homogeneous regions which will be further detailed by field investigations. Boreholes, laboratory tests and other features are used together with the generated model to predict a range of values for the rock mass parameters.

However, despite the intensive attempts to find the most realistic behaviour of the masses, many uncertainties remain until the construction takes place. Therefore, only during construction it is possible to have the full assessment of the geotechnical conditions. To assist construction and understand how the rock mass reacts to the advancing tunnel large quantities of instrumentation are used in the current tunnelling practice. The most common and ordinary measurements are tunnel wall displacements determined through topographic surveying which help the definition of adequate supporting measures. However, after construction and stabilization of the underground work all this valuable data is stored in big grey boxes and forgotten in humid basements. An important resource is therefore wasted while it could be used to compare the geotechnical predictions and the actual constructed rock mass behaviour using back analysis.

At the new Koralm railway tunnel between Graz and Klagenfurt, the construction of several kilometres of investigation tunnels through the main different geological conditions is foreseen. With all the gathered data available from the construction site and the need to enhance the design for main tubes, this seemed to be the right opportunity to make an extensive research in order to assess, in a more reliable way, the rock mass parameters.

This estimation of plausible rock mass strength values is developed using a back analysis methodology mainly based on the convergence confinement method and the surveyed displacements of the tunnels. A review of the geotechnical tunnel design methods used in the current practice is given. Also, several proposals for estimation of rock mass strength parameters (i.e. friction angle, cohesion and deformation modulus) given by different authors and references are discussed and used as a guideline. This allows the definition of plausible ranges of values arising from a spreadsheet developed to find the possible elasto-plastic valid solutions.

The main research comprehends the study of a major cataclastic rock fault zone with high overburden and moderately squeezing tunnelling conditions. Also, in order to cover the most representative rock mass types found during the excavation works, characteristic sections with predominantly sand, gravel and siltstone are analysed with the proposed methodology.

The most plausible combinations of parameters are plotted and the acceptability of the findings is discussed regarding the relevant theory in each case. Also, numerical analyses are performed to verify the analytical methodology used and the results found.

1.2. KORALM INVESTIGATION TUNNELS

The following discussion deals with the analysis of the investigation tunnels of the Koralm tunnel. The Koralm tunnel is the core piece of the new railway connection between Graz and Klagenfurt in the south of Austria. With a length of 32.9km it will go into operation as a double-tube tunnel in 2016. A sum of approximately eleven km of exploratory tunnels will be constructed prior to the main tunnel drive. The two pilot tunnels used for this study are located in the western subsection, and were built to better understand the build-up and behaviour of the rock mass types, allowing a more accurate design of the main tubes. Mitterpichling is a shallow tunnel mainly driven in Neogene formations consisting of siltstone, sand and gravel. Mitterpichling tunnel connects to Paierdorf tunnel which has increasing overburden extending from the Neogene formations through a large fault zone into the crystalline rocks. The rock mass types range from soft and highly sheared cataclastic rock to hard gneiss. A scheme of the Koralm investigation tunnels is shown on the following figure.

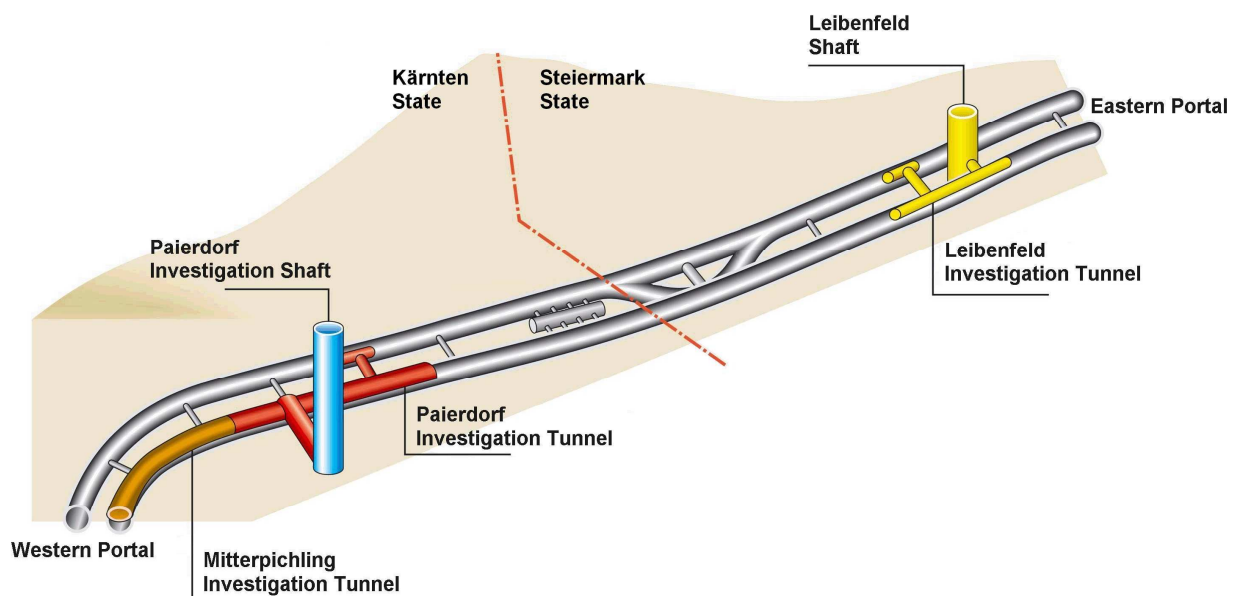


Fig.1.1 – Koralm investigation tunnels scheme

2

GEOTECHNICAL TUNNEL DESIGN

2.1. CONVERGENCE CONFINEMENT METHOD

Usually a tunnel passes through variable geological conditions. A tunnel support design is required to cover each predicted scenario. This is usually approached by a number of support categories which shall all together cover the full range of expected rock mass behaviour.

The design of a tunnel support is a three-dimensional problem with two main interactive elements: the rock mass and the tunnel support. This interaction is both dependent on the location, i.e. position relative to the tunnel face and sometimes dependent on time, i.e. in case of rheological rock mass behaviour.

A relatively old method to approach tunnel design is the so-called Convergence Confinement Method. This is basically a two-dimensional plane strain model of a hole in an infinite pre-stressed slab. There are closed mathematical solutions for elastic and elasto-plastic material and isotropic stress conditions. The effect of the third dimension (distance to tunnel face) is introduced by successive release of an internal pressure, which corresponds to the primary stress at the starting point. The displacement response of the tunnel perimeter versus the internal stress produces the so-called Ground Reaction Curve (GRC). In the case of a supported tunnel the load release on the rock mass is partially absorbed by yielding of the rock and partially transferred to the support. The load transfer from the rock mass into the supported is represented by the Support Characteristic Curve (SCC). The tunnel deformation ahead and behind of the face is of importance being this behaviour represented by the longitudinal displacement profile (LDP).

With these three components defined, the convergence confinement method can be applied to study tunnel behaviour along the different stages of excavation. The study of tunnel behaviour allows the definition of different solutions to the support and excavation types needed to a safe tunnelling construction.

To define how the rock mass behaves due to different load states, a failure criterion must be selected. The most commonly used in tunnel design is the Hoek-Brown failure criterion which was developed along and that it is explained in its last version by Hoek and Marinos (2007). However, for the sake of simplicity the Mohr-Coulomb failure criterion is used because all gathered data available from the construction site is referred to this criterion. Its review is given as well as the most relevant formulation applicable.

2.1.1. MOHR-COULOMB FAILURE CRITERION

The onset of plastic failure is defined as the point where the rock mass can no longer carry additional loads without suffering irreversible displacements. The Mohr-Coulomb failure criterion defines that for different values of effective confining stress σ_3 this starting point is:

$$\sigma_1 = \sigma_{cm} + k\sigma_3 \quad (2.1)$$

The uniaxial compressive strength of the rock mass σ_{cm} is given by:

$$\sigma_{cm} = \frac{2c \cos \varphi}{1 - \sin \varphi} \quad (2.2)$$

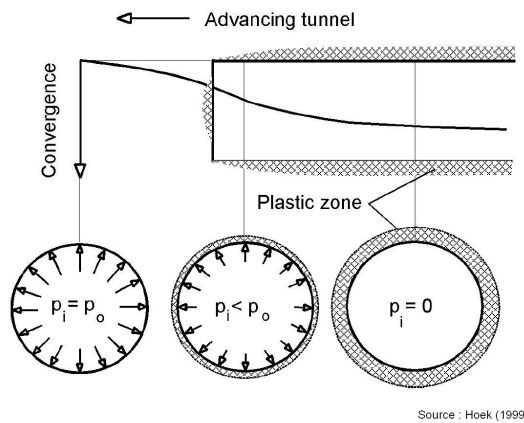
The slope k of σ_1 versus σ_3 is:

$$k = \frac{1 + \sin \varphi}{1 - \sin \varphi} \quad (2.3)$$

Where σ_1 is the axial stress at which failure occurs, σ_3 the confining stress, c the cohesion and φ the friction angle of the rock mass as referred by Hoek (1999).

2.1.2. GROUND REACTION CURVE

As Carranza-Torres and Fairhurst (2000) describe, the GRC is the relationship between the decreasing internal pressure p_i and the increasing radial displacement of the wall u_r and plastic zone surrounding the tunnel. The internal pressure is the idealized way of modelling the support and the relief due to the tunnel advance. It is defined as the pressure acting around the perimeter of the tunnel in a certain cross section. Fig.2.1 shows the decrease of the p_i value for an unsupported tunnel.



Source : Hoek (1999)

Fig.2.1 – Assumed support pressure p_i at different positions

The critical internal pressure p_i^{cr} is the value at which plastic deformation begins to develop. This means that if the internal pressure falls below this value, a failed area surrounding the tunnel develops. The critical internal pressure is defined by:

$$p_i^{cr} = \frac{2p_0 - \sigma_{cm}}{1+k} \quad (2.4)$$

If the internal support pressure is greater than the critical value the behaviour of the rock mass will be elastic; meaning that no failure occurs. In this condition the elastic displacement of the tunnel wall u_{ie} is given by:

$$u_{ie} = \frac{r_0(1+\nu)}{E}(p_0 - p_i) \quad (2.5)$$

Where ν is the Poisson's ratio, E is the deformation modulus and r_0 is the initial tunnel radius. When the internal pressure p_i is less than the critical internal pressure p_i^{cr} , failure occurs and a region of extension r_p develops around the tunnel. In this condition the plastic radius zone r_p is:

$$r_p = r_0 \left[\frac{2(p_0(k-1) + \sigma_{cm})}{(1+k)((k-1)p_i + \sigma_{cm})} \right]^{\frac{1}{k-1}} \quad (2.6)$$

The matching displacement of the tunnel walls under plastic behaviour u_{ip} is given by the equation:

$$u_{ip} = \frac{r_0(1+\nu)}{E} \left[2(1-\nu)(p_0 - p_{cr}) \left(\frac{r_p}{r_0} \right)^2 - (1-2\nu)(p_0 - p_i) \right] \quad (2.7)$$

It is therefore possible to create a spreadsheet containing the above formulae in order to predict the ground reaction either under elastic or plastic conditions. An example of the spreadsheet is given on appendix 3.

2.1.3. LONGITUDINAL DEFORMATION PROFILE

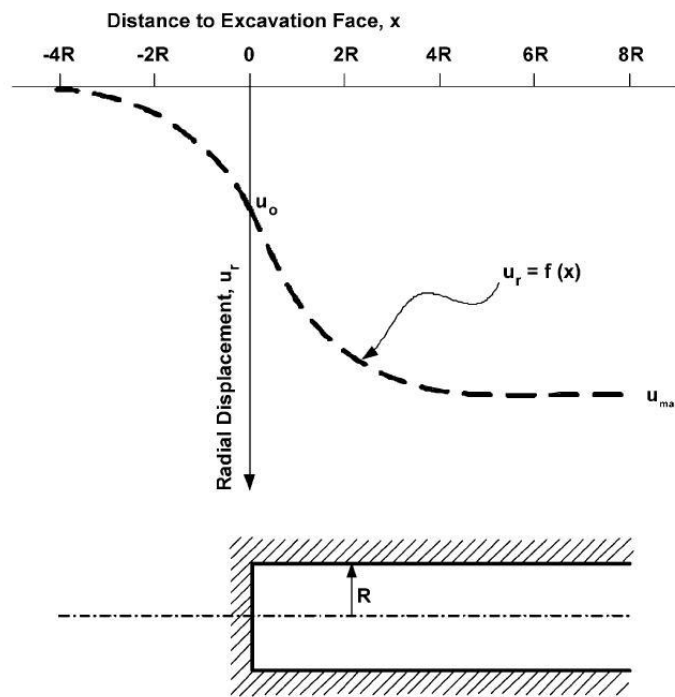
In a tunnel the LDP is defined as the deformation of the tunnel wall along the longitudinal axis, ahead of and behind the face. As shown in Fig.2.2 deformation starts about two to four tunnel diameters in front of the face, i.e. into the rock mass. However, this value differs from author to author depending also on the rock mass behaviour. Therefore, care must be taken when performing calculations. The total closure at the face u_0 under elastic conditions is about one-third of the maximum short term radial displacement u_{max} for an unsupported tunnel calculated from plain strain analysis. The radial

displacement ends about two to four tunnel diameters behind the face for linear elastic rock, but it can reach several tunnel diameters if elasto-plastic behaviour is observed.

The estimation of the normalized LDP, as explained by Unlu and Gercek (2003), was developed by several researchers who defined equations for the elastic behaviour of the tunnel. If plastic analysis is performed, an empirical best fit curve based on measured closure data available from Chern et al (1998) can be used. This best fit curve is used to calculate the tunnel face displacement and is given by:

$$\frac{u_r}{u_{\max}} = \left(1 + e^{\left(\frac{-d_t}{1.10} \right)^{-1.7}} \right) \quad (2.8)$$

Where $d_t = X/r_0$, u_r is the average radial displacement at a specified longitudinal position X , and r_0 is the tunnel radius. X is the distance measured from the tunnel face, being positive towards the excavated zone ($X > 0$) and negative into the unexcavated rock ($X < 0$).



Source : Unlu and Gercek (2003)

Fig.2.2 – Longitudinal displacement profile

On the other hand, according to Hoek et al (2008), a large bullet shaped plastic yielding zone develops in three dimensions on the advancing front. To take into account the influence of this large plastic zone a normalized plastic zone radius r_p/r_0 must be considered and a new equation is therefore defined as follows:

$$\frac{u_0}{u_{\max}} = \frac{1}{3} e^{-0.15 P_r} \quad (2.9)$$

Where $u_0/u_{\max}$ is the correlation at the face ($X/r_0=0$), $P_r=r_p/r_0$ is the maximum plastic radius and r_p is the plastic zone radius (2.6).

To determine the LDP, software based analyses were performed with RocSupport from Rocscience (2005). The calculation performed by the software is based on equation (2.8) from Chern et al (1998).

Once equation (2.9) is a best fit curve, more realistic values should be found for estimating displacements. For this reason a comparative analysis between the two formulae is performed. The formula (2.8) used by RocSupport is proved to give higher displacement values for the same strength input data as shown in the following table.

Table 2.1 – Tunnel face displacements according to different proposed theories

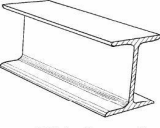
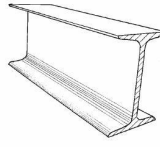
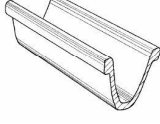
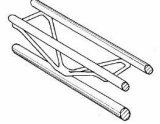
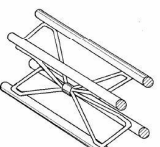
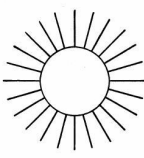
E [MPa]	c [MPa]	ϕ [°]	U ₀ [mm]	
			Equation(2.8) RocSupport	Equation(2.9)
200	0,55	40	87,38	76,40
	1,00	30	85,76	74,20
	1,50	20	86,63	74,00
	2,00	10	91,77	76,70
800	0,55	40	21,85	19,10
	1,00	30	21,44	18,60
	1,50	20	21,66	18,50
	2,00	10	22,94	19,20

From the gathered information it was decided to use the best fit equation (2.9) defined by Hoek et al (2008) for the calculations on this study.

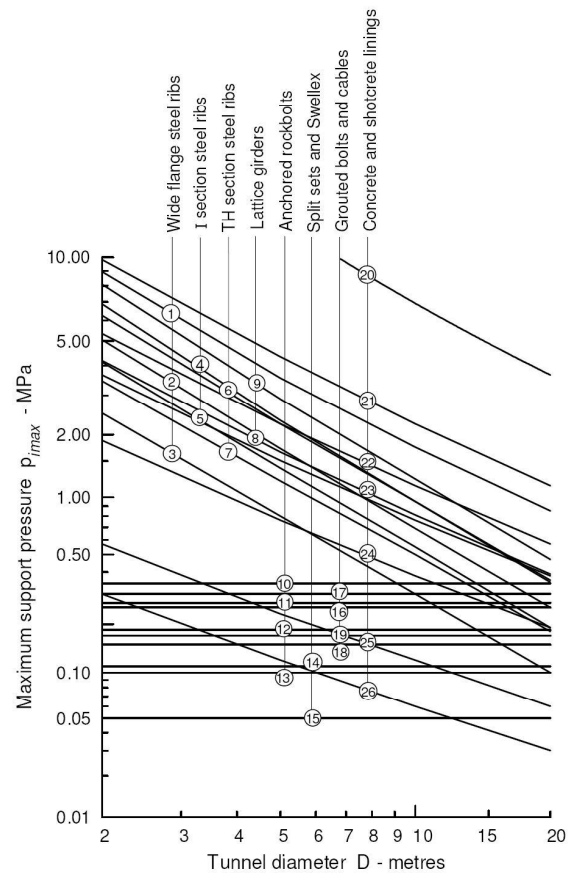
2.1.4. SUPPORT CHARACTERISTIC CURVE

The SCC is the relation between the increasing pressure on the support, p_s , with increasing radial displacement. When support is installed the initial support pressure is considered to be zero; becoming greater with the increasing load induced by convergence. The pressure carried by the support depends on the stiffness, the distance from the face when it was installed and the maximum loading bearing capacity $p_s^{\max}$ of the support chosen. The support reacts like a system formed by springs, meaning that induced loads from the rock mass are gradually transferred until equilibrium or the $p_s^{\max}$ is reached.

Equations are published for the calculation of the stiffness and capacity of different support systems; however, they often refer to an ideal state given by homogeneous conditions around a perfectly circular tunnel. Therefore equations must not be used to define precise support characteristics but to have a general idea how the different support types will work. An example of these equations is given by Hoek (1999) who shows different support types and their maximum bearing capacity as it is seen in the following figure.

Support type	Flange width - mm	Section depth - mm	Weight - kg/m	Curve number	Maximum support pressure $p_{i\max}$ (MPa) for a tunnel of diameter D (metres) and a set spacing of s (metres)
 Wide flange rib	305	305	97	1	$p_{i\max} = 19.9D^{-1.23}/s$
	203	203	67	2	$p_{i\max} = 13.2D^{-1.3}/s$
	150	150	32	3	$p_{i\max} = 7.0D^{-1.4}/s$
 I section rib	203	254	82	4	$p_{i\max} = 17.6D^{-1.29}/s$
	152	203	52	5	$p_{i\max} = 11.1D^{-1.33}/s$
 TH section rib	171	138	38	6	$p_{i\max} = 15.5D^{-1.24}/s$
	124	108	21	7	$p_{i\max} = 8.8D^{-1.27}/s$
 3 bar lattice girder	220	190	19	8	$p_{i\max} = 8.6D^{-1.03}/s$
	140	130	18		
 4 bar lattice girder	220	280	29	9	$p_{i\max} = 18.3D^{-1.02}/s$
	140	200	26		
 Rockbolts or cables spaced on a grid of $s \times s$ metres	34 mm rockbolt			10	$p_{i\max} = 0.354/s^2$
	25 mm rockbolt			11	$p_{i\max} = 0.267/s^2$
	19 mm rockbolt			12	$p_{i\max} = 0.184/s^2$
	17 mm rockbolt			13	$p_{i\max} = 0.10/s^2$
	SS39 Split set			14	$p_{i\max} = 0.05/s^2$
	EXX Swellex			15	$p_{i\max} = 0.11/s^2$
	20mm rebar			16	$p_{i\max} = 0.17/s^2$
	22mm fibreglass			17	$p_{i\max} = 0.26/s^2$
	Plain cable			18	$p_{i\max} = 0.15/s^2$
	Birdcage cable			19	$p_{i\max} = 0.30/s^2$

Support type	Thickness - mm	Age - days	UCS - MPa	Curve number	Maximum support pressure $p_{i\max}$ (MPa) for a tunnel of diameter D (metres)
 Concrete or shotcrete lining	1m	28	35	20	$p_{i\max} = 57.8D^{-0.92}$
	300	28	35	21	$p_{i\max} = 19.1D^{-0.92}$
	150	28	35	22	$p_{i\max} = 10.6D^{-0.97}$
	100	28	35	23	$p_{i\max} = 7.3D^{-0.98}$
	50	28	35	24	$p_{i\max} = 3.8D^{-0.99}$
	50	3	11	25	$p_{i\max} = 1.1D^{-0.97}$
	50	0.5	6	26	$p_{i\max} = 0.6D^{-1.0}$



Source : Hoek (1999)

Fig.2.3 – Equations for estimated maximum support capacities of different systems

2.1.5. INTERACTION BETWEEN THE COMPONENTS OF THE CONVERGENCE CONFINEMENT METHOD

Each one of the convergence confinement components is valid by itself but they only have a real meaning when the interaction between them is studied. Therefore studying and understanding the relation between the three basic components is of main importance. These interactions are shown in

two dimensional approaches. On one side the relation between the GRC, the SCC and its influence on the face and cross-section stability; on the other side in the LDP as its evolution within the advance of the tunnel excavation.

The following figure shows a typical construction and interaction between the ground reaction curve and the support characteristic curve.

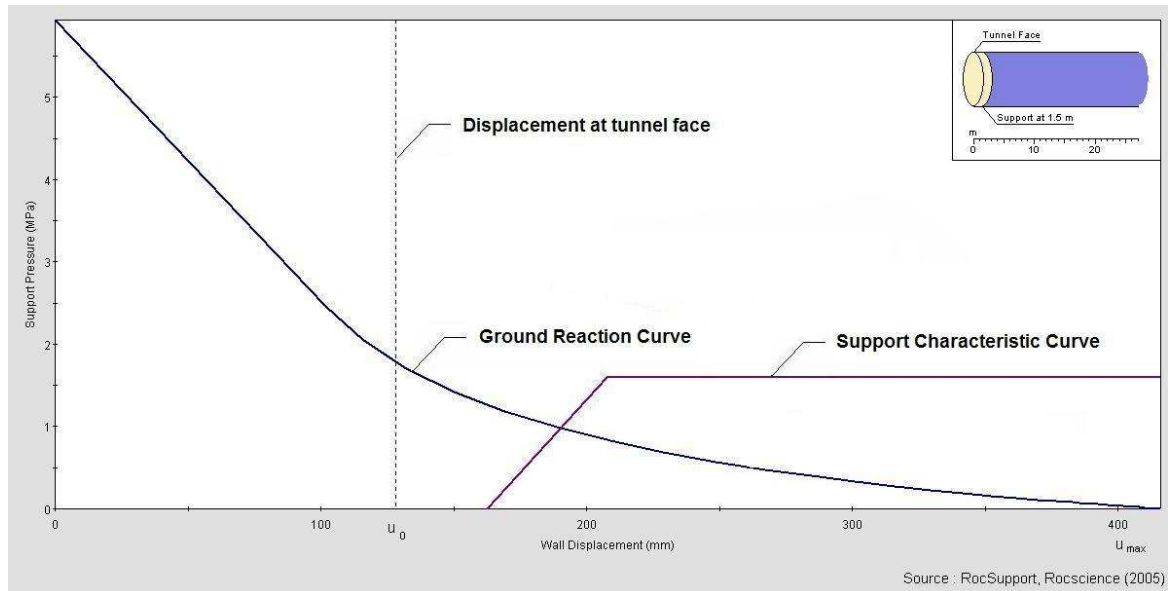


Fig.2.4 – Representation of the Ground Reaction Curve and Support Characteristic Curve interaction

From the Fig.2.4 the decreasing internal pressure corresponding to the GRC construction is noticed. One may see the inward displacement of the tunnel beginning before excavation takes place as discussed in 2.1.3. Therefore at the tunnel face (dashed line) the displacement is already u_0 . The example shows an installation of the support 1.5m behind the tunnel face. After setting the support into place it gradually starts to receive load from the continuously deforming wall of the tunnel. This will be happening up to the point where, either equilibrium is reached and the tunnel ceases its short term radial displacement, or the support maximum pressure is reached. If this last condition is met, lining materials yield. To avoid excessive yield of the support, either maximum support pressure can be increased, or more displacement allowed to take place. This situation is only possible because this study considers an ideal model.

The support installation time and position plays an important two-dimensional role. An old theory, stating that if installed too early or too close to the face, it may not be able to handle all the inward displacement, is out of date as referred by Schubert (2009). Therefore currently support should always be placed immediately, once rock mass tends to loosen otherwise.

The other two-dimensional problem is the evolution of the longitudinal profile along its axis. This study allows understanding the face stability and the total inward displacement expected at each tunnel cross-section. With the combination of these two two-dimensional analyses a three-dimensional approach is defined and the behaviour of the rock mass around the tunnel advance is comprehended.

2.2. OBSERVATIONAL METHOD

When dealing with the design of geotechnical structures one has to face a great amount of uncertainties derived from the inability to accurately predict the ground's physical properties and their space and time variability (ground behaviour during construction), as well as from the necessary simplifications and assumptions of the geotechnical design models. The observational method, formerly developed by Terzaghi, Peck (1967) and Peck (1969), is currently considered as the most effective approach to overcome these uncertainties.

The traditional design method has always been the conventional approach where the design is finalized before the construction begins and the design parameters are based on a conservative interpretation of the available investigation ground data. Monitoring data during construction are used only to verify assumed ground conditions and to ensure acceptable limits of structural behaviour. On the other hand, the observational method is based on the adoption of more optimistic design parameters, the design can be subjected to modifications during construction and monitoring and instrumentation play a major role in its efficient implementation.

The continuous observation during construction is of great importance in dealing with the geotechnical uncertainties. The observational approach is based on the continuous acquisition and evaluation of monitoring data, the review of the design during construction and its adjustment to actual conditions. After every excavation stage, monitoring data are checked against trigger values and necessary adjustments are made before the next stage. The unexpected conditions, such as unacceptable displacements can be detected on time and be repaired without significant consequences in the advance of the construction. Therefore a ductile mode of failure and a design type, which, in light of the monitoring data, can be modified in a timely way, are required.

There have been different definitions and approaches for the observational method. The one described in CIRIA 185 (1999) defines it as “a continuous, managed, integrated process of design, construction control, monitoring and review that enables previously defined modifications to be incorporated during or after construction as appropriate. The objective is to achieve greater overall economy without compromising safety”. The basic steps of the observational method for the design and construction of an underground structure, such as for a tunnel, could be described as follows:

- extensive soil investigations, realistic ground characterization;
- assessment of the most probable ground conditions, ground behaviour and ground support interaction during excavation;
- establishment of the design based on the most probable conditions;
- assessment of the most unfavourable deviations from these conditions;
- selection of the quantities to be observed during construction and calculation of their values under most probable and most unfavourable conditions, specification of acceptable limits and setting of trigger values;
- selection in advance of contingency plans or modification of the design for every significant deviation of the observational data from those predicted on the initial design;
- commencement of construction;
- acquisition and evaluation of extensive monitoring data of performance (e.g. displacements) in every excavation stage;
- comparison of monitoring data with trigger values ;
- review of the design, decision of adjustments to actual conditions or implementation of contingencies.

The acceptable limits of behaviour are calculated based on the “most probable” and “characteristic” parameters. The use of a “traffic light” system can facilitate the application of the observational method, providing a rapid way of decision making and implementation of the planned modifications. Generally, the Green/Amber response zone initiates mobilization and preparation of contingencies and the Amber/Red zone initiates their implementation. The use of this system is illustrated in Fig.2.5 for a tunnel excavation.

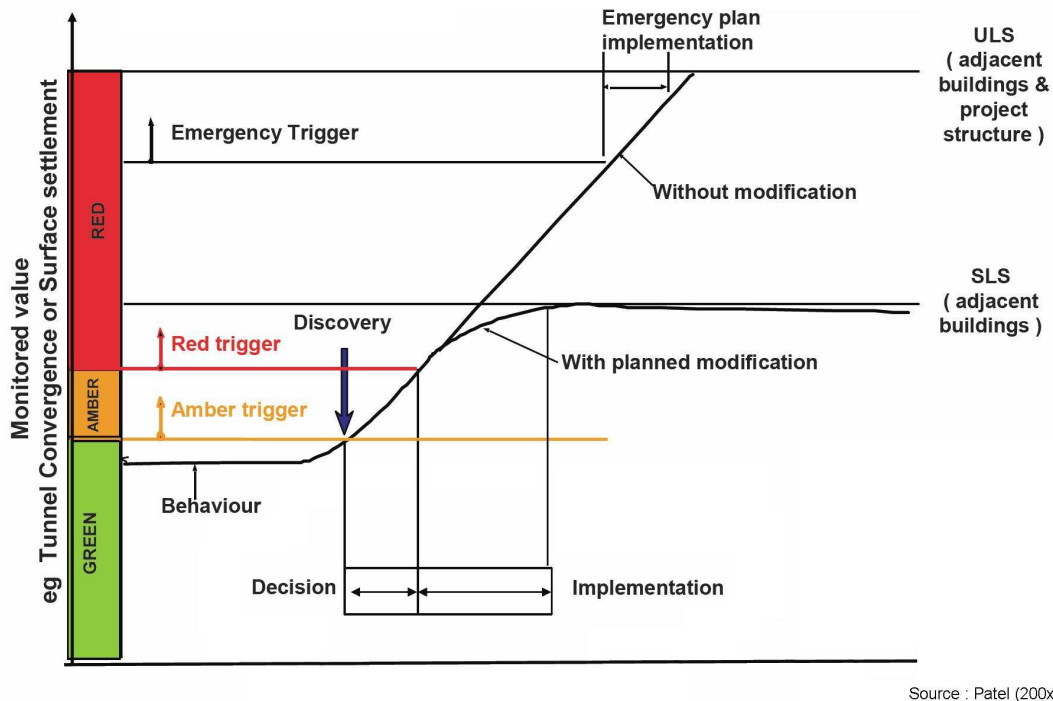


Fig.2.5 – Traffic light system for implementation of planned modifications

Moreover, according to CIRIA 185 (1999) the response zones are linked to the “most probable” (green/amber limit) and the “characteristic values” (amber-red limit). The most probable design parameters represent the probabilistic mean of all possible sets of conditions or, in general terms, the design condition most likely to occur. The “characteristic value” or moderately conservative parameter is a cautious assessment of the value of a parameter, worse than the probabilistic mean but not as severe as the worst credible value (used by the Eurocode 7 for the serviceability limit states checks and by the conventional design methods) (Fig.2.6).

The observational method can result into major programme and cost savings, as it allows the continuous review of the construction method and the adjustment to the actual conditions. In this way, alternative solutions (e.g. in the support measures or the excavation sequence) can be applied if the actual conditions prove to be more favourable, resulting in substantial net cost savings, outweighing the cost of a conforming design.

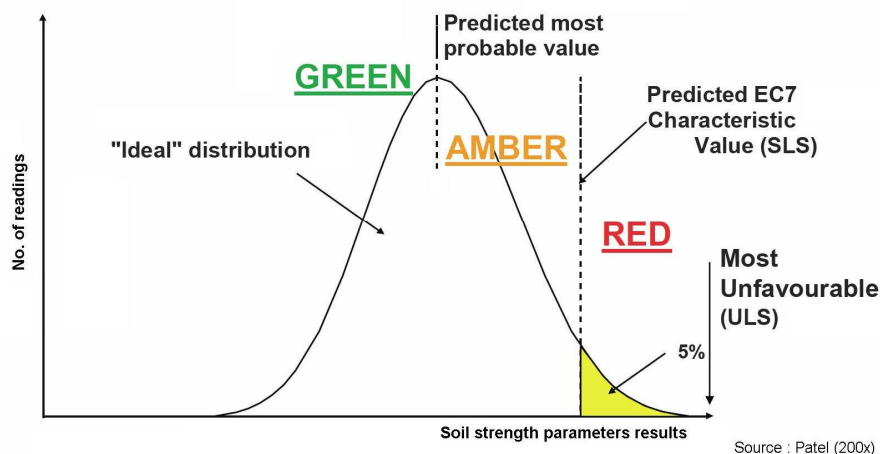


Fig.2.6 – Ideal EC7 predicted versus measured performance

Nowadays, monitoring and data evaluation methods have dramatically improved and new methods are being developed. The measured data can be rapidly processed and interpreted, also by means of numerical analysis models. In that way a more accurate prediction of the ground conditions and an optimization of the design during construction is possible.

Finally, the observational method seems as the best approach to minimize the uncertainties of the complex geological conditions. Nevertheless, for its successful implementation an appropriate preparation is necessary which includes rigorous monitoring and observation strategy, qualified personnel, construction control and efficient coordination between the working teams of the project. The operational framework should be defined, such as the responsibilities of the different parties involved (clients, contractors, designers). Moreover, because of the adoption of more optimistic design parameters and thus the reduction of safety factors, a lot of concerns need to be clarified about the limitations and the proper use of this method.

2.3. BIM-ROCK THEORY

Medley (1999) stated “The geotechnical literature suggests that most earth materials are relatively straight-forward to characterize by a few well-understood descriptors such as “sand”, “clay”, “sandy clay” and “weathered granite”. But geological uniformity is rare, much to our technical discomfort.” This observation denotes the importance of the study of complex formations classified as block-in-matrix (BIM-rock) materials.

BIM-rock is mainly defined as a chaotic mixture formed by hard blocks of rock, of different lithologies surrounded by a weaker matrix. The general geotechnical over conservative practice of designing using the strength of the weaker matrix, led to extensive research made by several authors. Researchers focused on the estimation and understand of the importance of block volumetric percentage, orientation and sampling in the strength of BIM-rock masses.

2.3.1. BLOCK VOLUMETRIC PROPORTION ESTIMATION

Assessing block proportion is of main importance because BIM-rocks overall strength is directly related to this value. Studies by Lindquist (1994), Lindquist and Goodman (1994), Medley and

Goodman (1994) and Medley (1994) give the guidance to adequate measuring and sampling of such materials, since they have been based in field and laboratory tests.

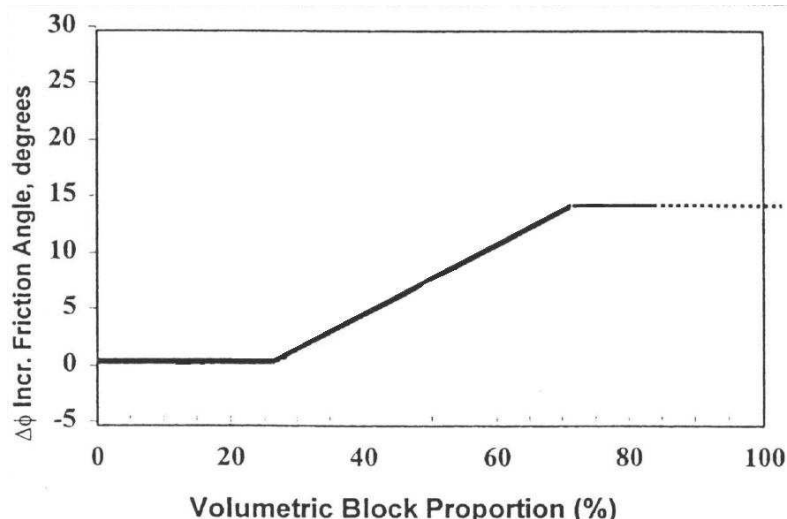
Medley and Lindquist (1995) showed by means of experimental tests that volumetric block proportion is scale-independent. From this assumption the authors conclude that geotechnical characterization of such formations should not be avoided because of its “complexity”, once the normalized block-size distribution allows laboratory tests to be a good approach to real rock masses. Therefore reliable results are found from the estimation of block volumetric proportion no matter the scale of the sample. This allows the use of geotechnical information from the investigation tunnels construction as it will be explained later in chapter 4.

2.3.2. EFFECT OF BLOCK PROPORTION ON STRENGTH PARAMETERS

Lindquist and Goodman (1994) proved from laboratory testing that the overall strength of BIM-rocks is given from the strength of the matrix plus a numerical increase depending on the volumetric block proportion. This influence results in an increase of the internal friction angle and a decrease of cohesion with increasing block proportion. An explanation for this fact was found by the authors after observation of the testing specimens. As failures tend to form along the block-matrix contacts, once these are the weakness surfaces; with the increase of block proportion cohesion will decrease because higher density of these surfaces is found. On the other hand friction angle increases because failure surface becomes more tortuous (it needs to fail around more blocks).

Another finding, made by the researchers, is that block strength parameters do not influence the overall strength of the BIM-rock because failure always occurs on the contact matrix-block. Therefore, the strength of the blocks is not relevant so long as the blocks are stronger than the matrix.

The increase in friction angle of the matrix with the block proportion ranges from 0° to 15° respectively for 25% and 75%. Higher values than 75% of block proportion were found not to influence the value of friction angle as well as values under 25%. A resume of these values is given by Medley (1999) and is shown in Fig.2.7.



SOURCE: After Medley (1999)

Fig.2.7 – Friction angle increase with block proportion increase in BIM-rock material

The deformation modulus of the BIM-rock was also found to be influenced by the volumetric block proportion of the sample by Lindquist & Goodman (1994). The term deformation modulus is used by the researchers because during virgin loading the model specimens underwent both elastic and plastic deformations. An expected increase is observed as the volume of stiffer material is higher resulting in an overall stiffer model. This increase in stiffness is, however, not quantified by the authors.

3

BACK ANALYSIS METHODOLOGY

3.1. BACK ANALYSIS CONCEPT

After excavation and placement of the initial support of the Koralm investigation tunnels a large amount of surveying data was available. These data consists of absolute and relative displacements on several points of the tunnel and derived stresses acting in the lining. All these data, in the Austrian tradition and expertise, is available for the longitudinal alignment of the tunnel in closely spaced cross-sections which allow a rather good understanding of how the rock mass develops on a given tunnel. Oreste (2004) observed that often this data is not used and a precious resource is this way wasted. Therefore, the main goal of this study is to use the gathered data from the tunnels to back-calculate plausible rock mass parameters – namely cohesion, friction angle and deformation modulus – in several characteristic tunnel sections. In other words, the researcher wants to back analyse the measured displacements obtained from the construction of the referred tunnels and predict the most realistic rock mass properties of the cross-sections analysed.

The procedure carried out in the back analysis consists of fixing certain key parameters and combining them in different combinations with the other parameters. To determine the most realistic parameters combinations the convergence confinement method is applied. Numerical analyses with FLAC are used to verify the proposed values. Results from the calculations must fit the available measurements and with these matching values an envelope of possible data is gathered. To settle on the plausible range of values theoretical formulation is revised and some considerations on the applicability of these formulae given.

3.2. DISPLACEMENTS

3.2.1. MEASURED DATA

Measured displacements arise from the survey targets installed behind the excavated face of the tunnel. The data is evaluated in a software tool – Tunnel:Monitor – which allows the visualization of the monitoring data in different charts and graphics, relating the total displacements on several points and their evolution over time and space. With the graphical information displayed, it is possible to analyse the maximum displacements along the longitudinal profile on the different measurement dates. Evaluating the shape of the connection lines between the different points, it is possible to acknowledge in a qualitative way different stiffness and rock mass properties.

Fig.3.1 shows the graphic data printed from Tunnel:Monitor in which each line represents the displacement versus tunnel chainage at the specific date. Analysing the diagram it is possible to see

that excavation took place initially (tunnel face advancing from left to right) in stiffer rock mass. At chainage 208 the excavation reached a softer rock mass zone. It is possible to see therefore, higher vertical displacement values starting to develop at chainage 199 and stabilizing again at chainage 213.

Measured data available from the tunnel monitoring is the base for the back analysis evaluation because analytical and numerical data, expressed as vertical and total final displacements, must fit the observations gathered. Hence special care is given interpreting the diagrams, mainly the final displacements and their development along the longitudinal profile.

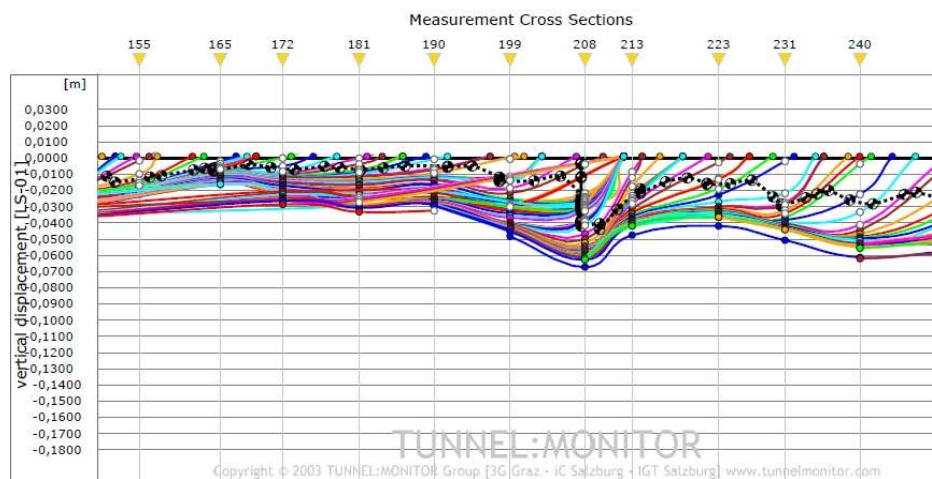


Fig.3.1 – Displacements along the longitudinal profile

3.2.2. ASSESSMENT OF PRE-DISPLACEMENTS

Prediction of pre-displacements is another important factor to be met for understanding the global behaviour of the tunnel. An extensive research was done in this field by Sellner (2000). From it the development of GeoFit, an artificial intelligence software code, was possible. It allows the estimation of pre-displacements at the tunnel face based on a best-fit of displacement data available from tunnelling construction. Time-dependent behaviour of the rock mass and support as well as the tunnel face advance are incorporated into the analytical model allowing the interaction between these features to be studied. This information is very important once it gives the possibility of defining a range of values to be met when performing calculations with the convergence confinement method.

An interesting finding is the ratio between the monitored displacement and the pre-displacement ahead of the face. Based on several analyses from different monitored cross-sections of the investigation tunnels, the relation between the pre-displacement and the measured displacement was found to be typically about 1:1. This means that the amount of pre-displacement occurring at the face is approximately the same as the measured inward displacement. This behaviour allows the definition of a narrow range upon the possible values estimated from the ratio of pre displacement to measured displacement (u_0/u_{mes}). The definition of this range can be seen on the example spreadsheet given in appendix 3 where the ratio of pre displacement to measured displacement is displayed.

3.3. KEY PARAMETERS VARIATION – CONVERGENCE CONFINEMENT METHOD

The key parameters analysed in this work are the friction angle, the cohesion and the deformation modulus (Mohr-Coulomb material model). These parameters are combined in several different possibilities and suitable values selected according to the matching displacements observed. The

combinations are in a spreadsheet where friction angle and deformation modulus are fixed and the cohesion is found. This way it is possible to combine all the parameters to find the potential values for the rock mass properties. The analysis is done to different deformation modulus values covering the whole plausible stiffness range. These combinations are then plotted in graphics of cohesion against friction angle for each deformation modulus allowing the simple visualization of all data.

With the convergence confinement method a wide range of possible values is easily analysed; therefore a selection criterion which allows the values to fall in a narrow practical range needs to be found. Several theoretical approaches were adopted to meet this requirement and a brief discussion is presented in the following sections.

3.4. PREDICTED VALUES FOR THE ROCK MASS PARAMETERS

During initial design of the investigation tunnels, geological field investigations were carried out to determine the most probable distribution and behaviour of the different rock mass types. Also laboratory tests were performed in order to predict mechanical properties for the different strata. From these investigation works several reports were produced; classification of the different rock mass types, geological profiles drawings, identification of fault zones, etc.

The most relevant report for this study was done by 3G and BGG (2003) and refers to the rock mass classification and parameterization. This report defines for each rock mass type (GA – from German “Gebirgsart”) the range of values for the friction angle, cohesion and deformation modulus among other technical information. The most relevant GA’s used on this study are given in appendix 2

3.5. DEFORMATION MODULUS ESTIMATION FROM OTHER SOURCES

Estimating a narrow range for the deformation modulus of the rock mass is an important factor since deformability significantly influences its behaviour. This estimation is, however, normally obtained from expensive and time-consuming field and laboratory tests, whose results can have a large scatter and only represent a narrow investigation area. Therefore references to this estimation are found to different conditions.

3.5.1. HABIMANA’S PROPOSED THEORY FOR TECTONISED ROCK MASSES

Habimana et al (2002) refer to Jambu’s suggested expression for quartzitic sandstones and phyllitic schists with different tectonisation degree. This reference is given by the following equation.

$$E = K \times \sigma_{at} \left(\frac{\sigma_3}{\sigma_{at}} \right)^n \quad (3.1)$$

Where E is the elastic tangent modulus, σ_3 the minor principal stress (overburden $H \times$ rock mass specific weight γ), K a modulus number, n an exponent and σ_{at} the atmospheric pressure ($\sigma_{at}=100\text{kPa}$). K and n are determined by plotting E against σ_3 on log-log scales and fitting a straight line to the data. Values for K and n were also plotted by Habimana. For a more user friendly approach the author proposes an extension of the generalized Hoek Brown failure criterion to take into account cataclastic

rocks. A summarized table with different values for different tectonisation degrees and GSI values is proposed as follows.

Table 3.1 – Deformability parameters for different rock mass types with different tectonisation degrees, after Habimana et al (2002)

Deformability parameters for phyllitic shist			
K	n	GSI	tectonisation degree
522	0.49	25	highly tectonised
1285	0.288	15	extremely tectonised
140	0.583	5	extremely tectonised
Deformability parameters for quartzitic sandstone			
K	n	GSI	tectonisation degree
294	0.61	15	extremely tectonised
1581	0.355	25	highly tectonised

3.5.2. GRUNDBAU-TASCHENBUCH ESTIMATION

In the technical guide Grundbau-Taschenbuch from Witt (2008), the estimation of the deformation modulus is given according to equation (3.2).

$$E = v_e \sigma_{at} \left(\frac{\sigma_3}{\sigma_{at}} \right)^{w_e} \quad (3.2)$$

The values of v_e and w_e are listed in the resume table 3.2. The original table with other data is found in Fig.A.1.1 in appendix 1. σ_{at} and σ_3 are as described previously.

Table 3.2– Description of soil types and values of v_e and w_e

Nr	Description	German reference	English reference	v_e	w_e
1	Poorly graded gravels or gravel-sand mixtures, little or no fines	GE	GP	400	0.6
				900	0.4
2	Well-graded gravels, gravel-sand mixtures, little or no fines	GW, GI	GW	400	0.7
				1100	0.5
3	Silty gravels, gravel-sand-silt mixtures	GÜ, GŤ	GM	400	0.7
				1200	0.5
4	Clayey gravels, gravel-sand-clay mixtures	GU, GT	GC	150	0.9
				400	0.7
5	Poorly graded sands or gravelly sands, little or no fines	SE	SP	150	0.75
				700	0.55
6	Well-graded sands, gravelly sands, little or no fines	SW, SI	SW	200	0.7
				600	0.55
7	Silty sands, sand-silt mixtures	SU, ST	SM	150	0.8
				500	0.65
8	Clayey sands, sand-clay mixtures	SÜ, SŤ	SC	50	0.9
				250	0.75
9	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	UL	ML	40	0.8
				110	0.6
10	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	UM, UA	MH	30	0.9
				70	0.7
11, 12	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	TL, TM	CL	10	1
				50	0.9
13	Inorganic clays of high plasticity, fat clays	TA	CH	6	1
				20	1
14	Organic silts and organic clays	OU, OT	OL, OH	5	1
				20	0.9
15	Peat and other highly organic soils	HN, HZ	Pt	3	1
				8	1
16	Mud	F		4	1
				10	0.9

3.5.3. MINIMUM DEFORMATION MODULUS DEFINITION

Following the convergence confinement method and knowing the total displacement at the point of equilibrium it is possible to conclude that equation (2.5) can be rearranged in order to find the lower value of the deformation modulus. This lower value is explained from the ground reaction curve analysis. If elastic behaviour is observed the reaction curve is linear becoming gradually parabolic shaped when it enters the plastic level. The deformation modulus estimation is given by the slope of this curve. On the following figure an example ground reaction curve is displayed. The curve shows the initial elastic behaviour of the rock mass as a linear function. As plastification starts to occur the slope of the curve becomes smaller corresponding to a higher deformation modulus. From this simple observation it is possible to define the lowest deformation modulus.

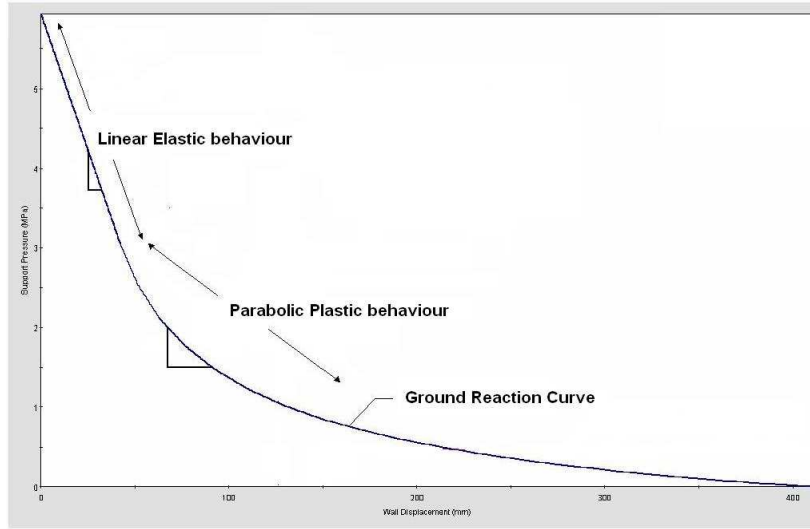


Fig.3.2 – Ground reaction curve

Knowing the elastic displacement, from the measured data, occurring on the tunnel for a given cross section, we have all the input data needed for the equation defined as follows.

$$E_{\min} = \frac{r_o(1+\nu)}{u}(p_0 - p_i) \quad (3.3)$$

Where $E_{\min}$ is the minimum deformation modulus calculated from the convergence confinement method, r_o the radius of the tunnel, u the total displacement at point of equilibrium, p_0 the in-situ pressure and p_i the internal support pressure.

3.6. FRICTION ANGLE ESTIMATION FROM OTHER SOURCES

Lang et al (2007) propose an equation to calculate the internal friction angle according to the grain size distribution curve. The equation is given as follows.

$$\varphi = A + B + C + D \quad (3.4)$$

$$\begin{aligned} A &= \frac{1}{7} [< 0,002mm] \\ B &= \frac{1}{5} [0,002 - 0,01mm] \\ C &= \frac{1}{3} [0,01 - 0,2mm] \\ D &= \frac{1}{2,5} [> 0,2mm] \end{aligned} \quad (3.5)$$

Where A, B, C and D correspond to the percentage of particles found in each of the considered ranges. Percentage values are read from the grain size distribution curve for each type of rock mass studied. The grain size distribution curves are found in the geological reports (Fig.A.2.8). These curves show the average distribution for a given type of rock mass assessed from the preliminary geological studies.

Two correcting factors are given to take into account different conditions of the samples used in formula (3.4); the grain size distribution ϕ_1 , and the degree of compaction ϕ_2 .

The first refers to the grading of the grain size distribution curve. If the sample is well graded the friction angle should be increased and if poor graded decreased. A normal grading produces no change in the friction angle value.

The second correction factor is the degree of compaction of the sample measured by the density of the proctor test. If dense composition is observed than an increase to the friction angle is proposed. A decrease of the value found with (3.4) is suggested if loose composition is found. Again if normal density is observed no change should be made.

These correcting factors, however, are not taken into account in this work. These estimates are made for additional reference and limit the range of friction for each rock mass.

3.7. INTERNAL SUPPORT PRESURE ESTIMATION

For the calculation of the mobilized axial force that can be expected to be the internal support pressure described in section 2.1.2, the equation for thin-walled pressure vessels is used. The equation gives the stress in a thin-walled pressure vessel in the shape of a cylinder and is defined by (3.6).

$$N = p_i \times r \quad (3.6)$$

Where N is the mobilized axial force, p_i the internal support pressure and r the radius of the tunnel. Equation (3.7) gives the stress in a point

$$\sigma_t = \frac{N}{A} \quad (3.7)$$

Considering a unitary length, the area A is equal to the thickness of the wall. σ_t is the stress on the lining calculated using a rheological shotcrete model using the deformation history of the lining derived from the measured displacements of the tunnel. This analysis is performed with the help of Tunnel:Monitor which gives the shotcrete stress over the time. Therefore, to be sure that we are out of tunnel face influence an average stress value of 27 MPa is chosen as it can be seen in section A2.4 in appendix 2.

From the two previous equations we can define the internal support pressure due to the shotcrete lining as follows.

$$p_i = \frac{\sigma_t \times t}{r} \quad (3.8)$$

Each advance in the tunnel characteristics of the support installed must be reported. These reports are the so-called required excavation and support sheets (RESS) and are produced each time there is a change in the support characteristics. From these sheets it is possible to read the number of bolts installed as well as the profile perimeter and round length excavated. This information enables the definition of an influence area, s , for each bolt. From it, the internal support pressure arising from the rockbolts can be estimated using the equations given in figure 2.3 by Hoek (1999) for calculation of the maximum support capacity of different systems.

Total internal support pressure is therefore given by the sum of the shotcrete and bolting forces.

4

KORALM INVESTIGATION TUNNELS BACK ANALYSIS

4.1. BACK ANALYSIS OF PAIERDORF LAVANTTAL FAULT ZONE

4.1.1. GEOLOGICAL MODEL

Paierdorf east drive chainages 1+160 to 1+340 were found to be a major fault zone, which is the core of the regional Lavanttal fault zone. This area consists of cataclastic rock showing very low rock mass strength properties at an average overburden of 220m. The rock is described as the product of dislocation metamorphism occurring on faults or intense plastic deformation zones. It is also described by Blyth and Freitas (1984) and Goodman (1993) as mylonite if fined-grained and as phyllonite if larger sizes of the particles are found by microscope. The geological report written by Fasching (2008) reveals a complicated system of highly sheared material arising from fine grained cataclastic rock to big intact blocks classified as GA37 and GA38. This system is supposed to be formed by a block-in-matrix zone and a fault as shown in Fig.4.1.

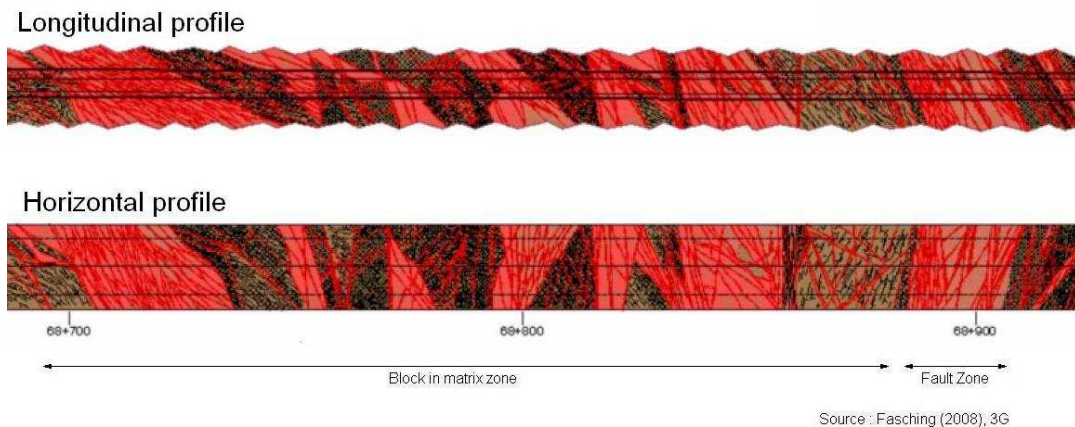


Fig.4.1 – Lavanttal fault zone

Fig.4.2 shows the crown settlement in the core of the fault zone of about 150mm. This is the reference displacement value to match with the back calculations.

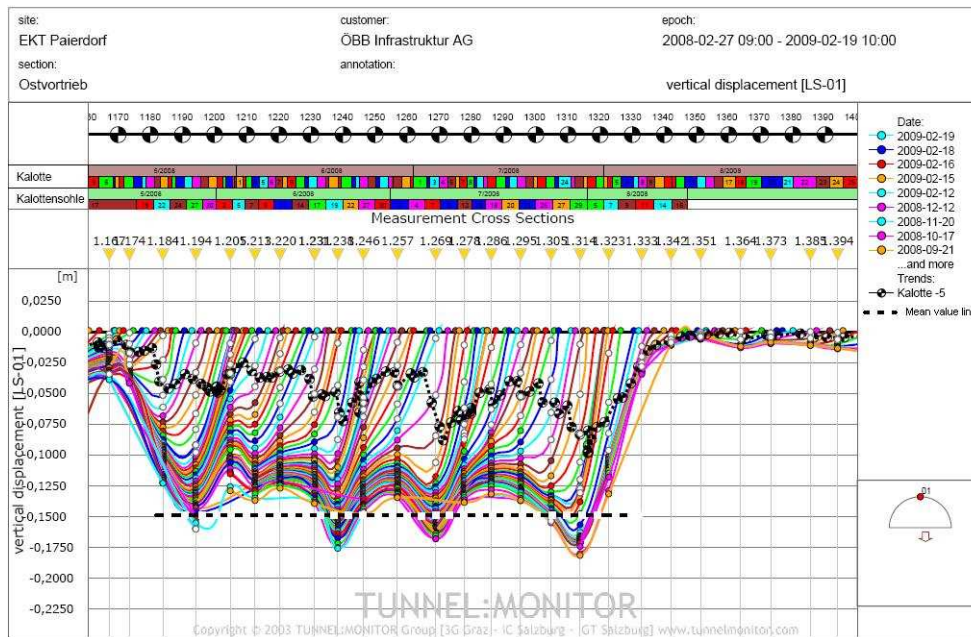


Fig.4.2 – Mean vertical displacement of Lavanttal fault zone

4.1.2. DEFORMATION ELEMENTS

When the excavation reached the Lavanttal core fault zone, large displacements started to be surveyed due to the material characteristics and high overburden. This led to the implementation of a stress controlling mechanism. The mechanism is composed by compressible sets of steel tubes, designated as deformation elements, embedded in the tunnel lining allowing deformation of the lining and avoiding failure of the shotcrete due to excessive yield. This is particularly advantageous at young ages while the shotcrete cannot be loaded with high stresses. Fig.4.3 shows the installation in the investigation tunnel of these deformation elements.

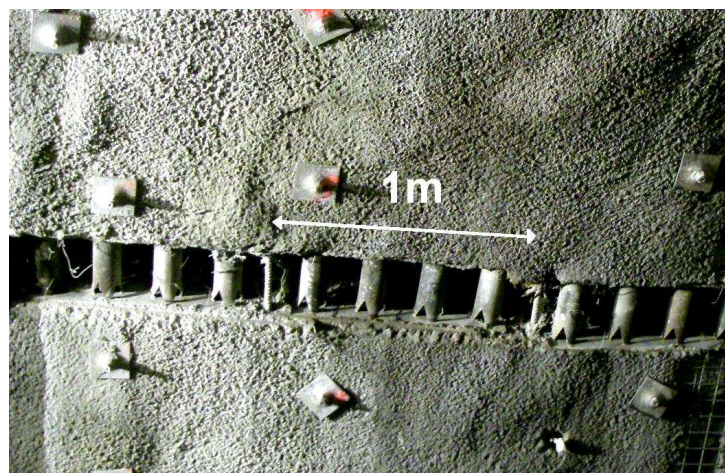
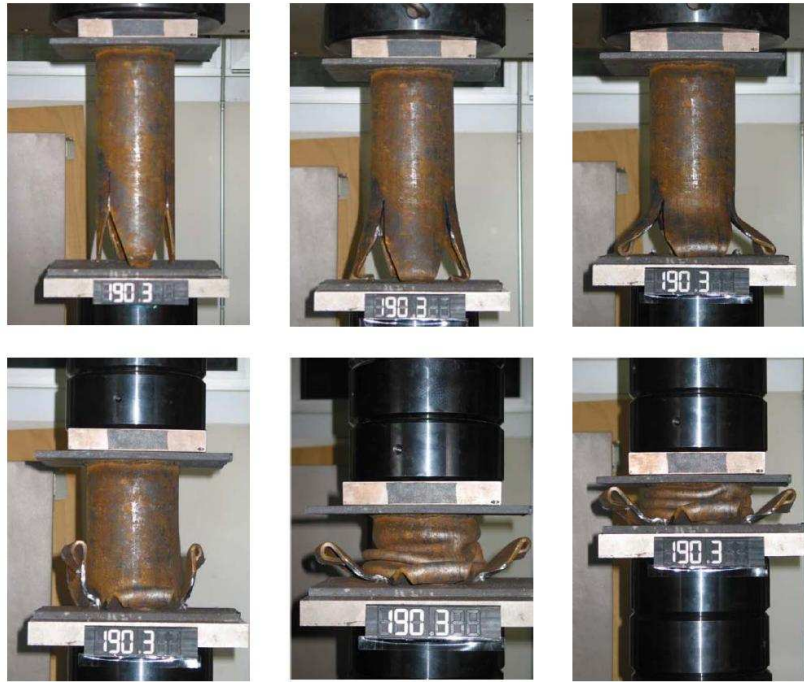


Fig.4.3 – Deformation elements installed in the tunnel lining

Several types are currently in use depending on the maximum load capacity needed and deformations expected to occur. Definition of the loading capacity and therefore the maximum support pressure given by the tubes is done through laboratory tests. Fig.4.4 shows the squeezing behaviour observed

during one of the laboratory tests performed to assess the maximum bearing capacity of the tubes used in the segments on the work site.



Source : TU (2008)

Fig.4.4 – Stress controllers bearing capacity test

In Fig.4.5 it is possible to see the diagram of one of the loading tests. The axial force is displayed against the radial displacement for the whole length of possible deformation. From the diagram an average value of 250kN in the initial 70mm of deformation (corresponding to the real deformation observed in the tunnel) is expected.



Source : TU (2008)

Fig.4.5 – Stress controllers test diagram

Knowing that each segment installed is 1m long and has four deformation elements (Fig.4.3), the average axial force supported by the stress controllers is 1000kN/m.

4.1.3. ROCK MASS MODEL AND SUPPORT

A simple analytical model is used to give the general overview of the rock mass behaviour. Mohr-Coulomb failure criterion is used for the rock mass yield around the tunnel.

The model considers a circular tunnel subjected to a hydrostatic stress field with an overburden of 220m. Considering the unit weight of the rock mass to be 22kN/m^3 the primary in situ stress p_0 is 4.84MPa. The cross-section of the tunnel is 4.5m radius and the internal support pressure, p_i , determined, is 0.46MPa. This internal support pressure value is derived from the deformation elements and the rockbolts installed along the cross section of the tunnel.

Due to the deformation elements installed along the cross section of the tunnel, an internal support pressure of 220kN/m^2 is estimated. This value arises from the application of the thin-walled pressure vessels given in equation (3.6). The axial force used on the equation is 1000kN/m due to existence of four elements in each meter as shown before. From the RESS (Fig. A.2.17) it is possible to define a rockbolting spacing of 1.2m which, applying Hoek (1999) table (Fig.2.3), gives an expected rockbolt contribution for the internal support pressure of 240kN/m^2 .

As described in section 3.2 pre-displacements at the tunnel face are estimated using the code GeoFit. Using the empirical displacement prediction method proposed by Sellner (2000) a pre-displacement of 150mm is expected in this section as shown in Fig.4.6. Using this information only the values following a pre-displacement to measured displacement ratio (u_0/u_{mes}) near to 1:1 are acceptable. However, it is not always possible to achieve a ratio of 1:1 (u_0/u_{mes}). Therefore, it is considered that a deviation of $\pm 5\%$ of the displacements is acceptable which, in this case, means that measured displacements can be $150\text{mm} \pm 5\%$ (acceptable range from 142mm to 158mm).

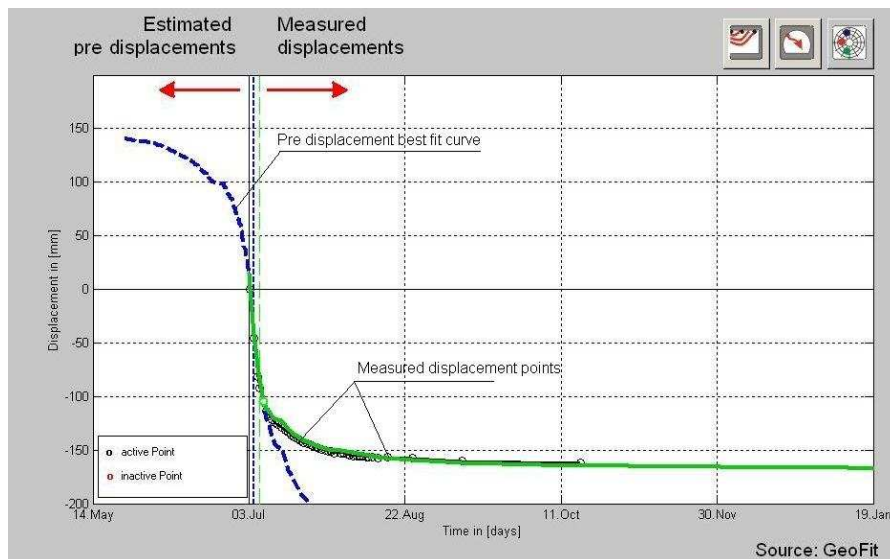


Fig.4.6 – GeoFit pre displacement calculation for Lavanttal fault zone

In Fig.4.6 the estimation of pre-displacements is given by GeoFit applied to chainage 1+268. Represented by a continuous line are the measured displacements (u_{mes}) while the dashed line

represents the “S” shape fitting curve which allows the assessment of the face pre-displacements (u_0). This analysis is done for different cross-sections on the Lavanttal fault zone showing a similar behaviour in all of them. Table 4.1 summarizes the input data used for the convergence confinement back calculations.

Table 4.1 – Convergence confinement input data for Lavanttal fault zone (average values)

Convergence confinement input data		Paierdorf East
Parameter		0+960 to 1+340
		GA37 + GA38
measured displacement - u [mm]		150
unit weight [kN/m ³]		22
Overburden [m]		220
Pre displacement at the face [mm]		150
number of rockbolts [n]		16
rockbolts internal support pressure [MPa]		0.24
pi deformation elements bearing capacity [kN/m]		1000
deformation elements estimated internal support pressure [MPa]		0.22
total internal support pressure [MPa]		0.46

4.1.4. CONVERGENCE CONFINEMENT ESTIMATION OF VALUES

A spreadsheet containing the convergence confinement method is developed to calculate the plausible values for the rock mass parameters. This spreadsheet, as explained in appendix 3, has the following basic procedure: an elastic deformation modulus, E , is assumed and a set of corresponding shear strength parameters c and ϕ determined. By variation of the deformation modulus from the minimum value (i.e. elastic behaviour) to higher values, the whole field of possible parameters is determined. The formulation used, is described in sections 2.1.1. to 2.1.4. Diagram of Fig.4.7 gives an example of the possible values of friction angle and cohesion for a deformation modulus of 200MPa.

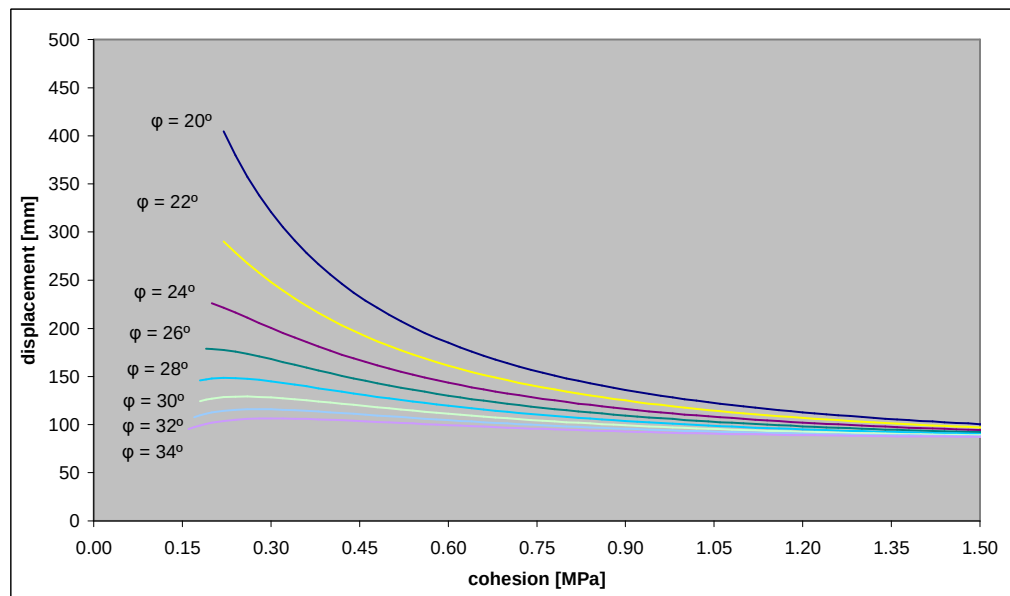
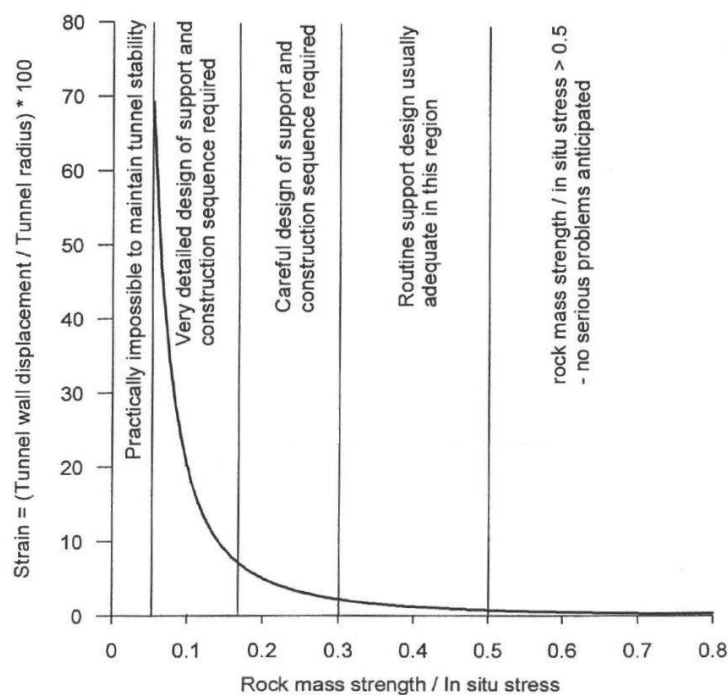


Fig.4.7 – Calculated displacements for 200MPa deformation modulus

Repeating the same procedure for different deformation moduli it is possible to have the range of possible friction angle and cohesion defined in table 4.2. The input data for these back calculations is given in table 4.1.

The acceptability of these possible values is directly related to the ratio of rock mass strength to in situ stress as described by Hoek (1999a). The author defines that with a ratio below 0.05 it is practically impossible to maintain tunnel stability. This is shown in Fig.4.8 where the different categories of tunnel support design problems are defined.

This approach is also valid in this study and values are deemed as unacceptable if the ratio of rock mass strength to in situ stress is under 0.10. For this reason curves showing the possible values in Fig.4.7 have a starting point higher than 0MPa of cohesion.



Source : Hoek (1999a)

Fig.4.8 – Approximate categories of tunnel support design problems by Hoek (1999a)

Table 4.2 summarizes the back calculated data from the convergence confinement method for deformation modulus varying from 150MPa to 500MPa and friction angle from 11° to 34°.

The table is organized in order to enable the reading of the expected cohesion value for a certain friction angle and deformation modulus in each column. The symbol (-) in certain cells means that it is not possible to achieve 150mm of vertical displacement i.e., all the values found are under 150mm. This is explained either by the high value of friction angle, the high deformation modulus value or a combination of both. In Fig.4.7 it is possible to observe this behaviour for the 30°, 32° and 34° of friction angle curves.

Also, for each combination the ratio between pre-displacement and measured displacement (u_0/u_{mes}) is given. In this way it is possible to define which values follow the observation described in section 3.2.2 and 4.2.2 (ratio u_0/u_{mes} approximately 1:1).

Table 4.2 – Convergence confinement back calculated values

E	150 MPa		200 MPa		250 MPa		300 MPa		350 MPa		400 MPa		450 MPa		500 MPa	
φ [°]	c	u_0/u_{mes}	c	u_0/u_{mes}	c	u_0/u_{mes}	c	u_0/u_{mes}	c	u_0/u_{mes}	c	u_0/u_{mes}	c	u_0/u_{mes}	c	u_0/u_{mes}
11	1.78	0.50	1.32	0.50	1.10	0.50	0.96	0.50	0.88	0.48	0.80	0.48	0.76	0.46	0.70	0.47
12	1.70	0.51	1.26	0.51	1.04	0.51	0.90	0.51	0.82	0.50	0.76	0.48	0.70	0.48	0.66	0.47
13	1.64	0.51	1.20	0.52	0.98	0.52	0.86	0.51	0.76	0.52	0.70	0.51	0.64	0.51	0.60	0.50
14	1.58	0.51	1.14	0.53	0.92	0.54	0.80	0.53	0.72	0.52	0.64	0.53	0.60	0.51	0.56	0.50
15	1.52	0.51	1.08	0.53	0.88	0.54	0.74	0.55	0.66	0.54	0.60	0.54	0.54	0.55	0.50	0.54
16	1.46	0.52	1.02	0.54	0.82	0.55	0.70	0.55	0.60	0.57	0.54	0.57	0.50	0.56	0.46	0.55
17	1.40	0.52	0.96	0.55	0.76	0.57	0.64	0.58	0.56	0.58	0.50	0.58	0.44	0.60	0.40	0.60
18	1.34	0.53	0.90	0.57	0.70	0.59	0.58	0.61	0.50	0.62	0.44	0.63	0.40	0.62	0.36	0.63
19	1.28	0.53	0.86	0.57	0.66	0.60	0.54	0.62	0.46	0.63	0.40	0.65	0.34	0.69	0.30	0.71
20	1.22	0.54	0.80	0.58	0.60	0.62	0.48	0.66	0.40	0.69	0.34	0.71	0.30	0.72	0.26	0.75
21	1.16	0.54	0.74	0.60	0.54	0.66	0.42	0.71	0.34	0.76	0.28	0.80	0.24	0.83	0.19	0.93
22	1.10	0.55	0.68	0.62	0.48	0.69	0.36	0.77	0.28	0.85	0.22	0.94	0.16	1.10	0.12	1.23
23	1.04	0.56	0.62	0.64	0.42	0.74	0.30	0.86	0.20	1.07	0.09	1.72	-	-	-	-
24	0.98	0.56	0.56	0.67	0.36	0.81	0.20	0.87	-	-	-	-	-	-	-	-
25	0.92	0.57	0.50	0.70	0.28	0.95	-	-	-	-	-	-	-	-	-	-
26	0.86	0.58	0.44	0.74	0.24	1.07	-	-	-	-	-	-	-	-	-	-
27	0.78	0.61	0.36	0.83	0.22	1.16	-	-	-	-	-	-	-	-	-	-
28	0.72	0.62	0.26	1.04	-	-	-	-	-	-	-	-	-	-	-	-
29	0.66	0.64	0.24	1.10	-	-	-	-	-	-	-	-	-	-	-	-
30	0.58	0.67	-	-	-	-	-	-	-	-	-	-	-	-	-	-
31	0.50	0.71	-	-	-	-	-	-	-	-	-	-	-	-	-	-
32	0.42	0.77	-	-	-	-	-	-	-	-	-	-	-	-	-	-
33	0.24	1.01	-	-	-	-	-	-	-	-	-	-	-	-	-	-
34	0.24	1.06	-	-	-	-	-	-	-	-	-	-	-	-	-	-

4.1.5. DEFORMATION MODULUS ESTIMATED VALUES

Several references are given in section 3.5 for the estimation of the deformation modulus of different rock mass types under different conditions. Due to the highly sheared material found, it is reasonable to consider the rock mass with an extreme degree of tectonisation. Therefore revising Habimana et al (2002) approach it is possible to calculate the deformation modulus for quartzitic sandstone and phyllitic schist considering that the materials behave like cataclastic rocks.

From the grain size distribution curves available it is possible to define GA38 as sand with fines matching material 8 of the Fig.A1.1 in appendix 1 according to Witt (2008). The rock mass GA37 is defined as Light plastic silt – material 9 – according to the same classification system.

Finally, and considering the convergence confinement method the minimum deformation modulus expected to be found as explained in 3.5.3 is defined.

With these different theories it is possible to calculate the expected values and narrow them to a reasonable range. This concept is shown in the next figure where the deformation moduli are plotted in relation to an overburden ranging from the surface to 750m depth.

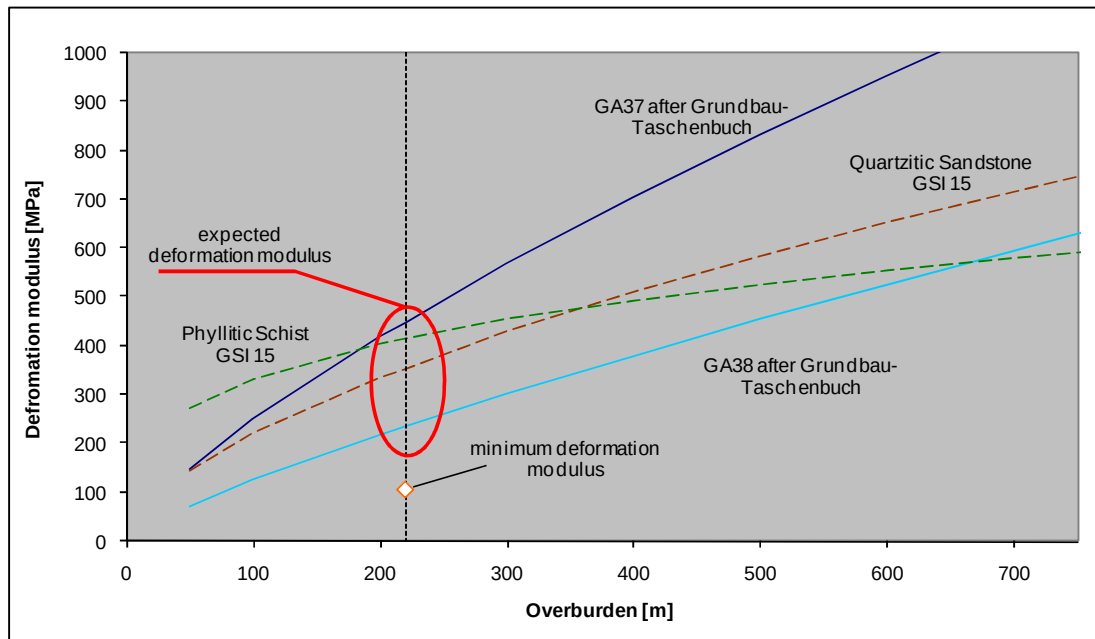


Fig.4.9 – Expected deformation modulus range for cataclastic rock

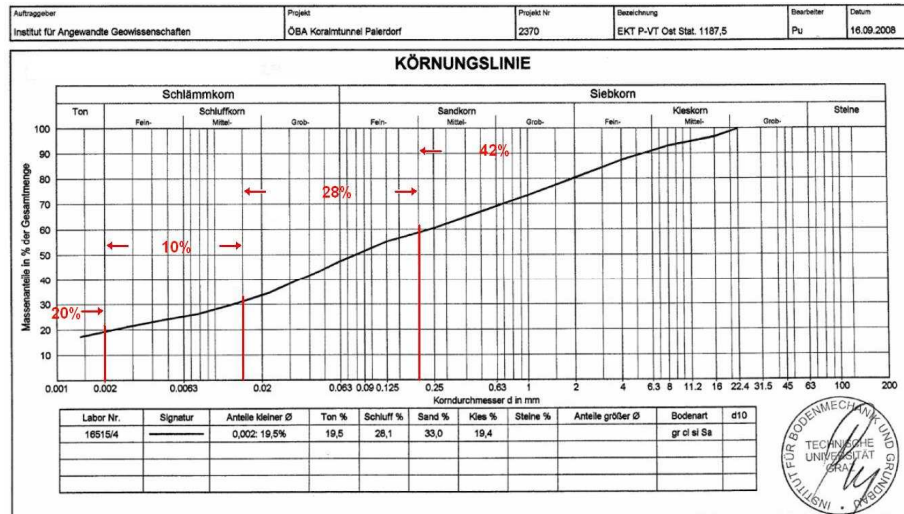
From Fig.4.9 it is possible to see that a reasonably narrow range can be defined for the deformation modulus. Green and brown dashed lines represent Habimana's approach for Phyllitic Schists and Quartzitic Sandstone with an extreme degree of tectonization. Grundbau-Taschenbuch guide from Witt (2008) gives the estimation for the deformation modulus represented by the plain lines for the GA37 and GA38 respectively. Minimum deformation modulus arising from the convergence confinement theory as explained in section 3.5.3 is also plotted in the diagram.

Considering the different approaches and reviewing Fig.4.9 it is evident that the expected range of values for the deformation modulus should be greater than 100MPa (minimum deformation modulus value). Both equations proposed by Habimana et al (2002) and Witt (2008) agree on a relatively narrow range of values as shown in the figure. With this information the expected values range from [125,450] MPa.

4.1.6. FRICTION ANGLE ESTIMATED VALUES

The calculated value of friction angle arises from the grain size distribution curves as described in section 3.6.

Drawing on the grain size distribution curves the borders for each of the ranges described by Lang et al (2007) and reading the percentages of material within each set, it is possible to apply equation (3.4). With this it is possible to find an expected friction angle corresponding to the grain size distribution curve. Fig.4.10 shows an example of the constructed diagram. Also in table 4.3 it is possible to read the percentages and consequent friction angle assessment.



Source: After Fasching (2008), 3G

Fig.4.10 – Calculation of friction angle from grain size distribution curve

From the grain size distribution curve in Fig.4.10 it is possible to read the percentages of material under 0.002mm as 20%, between 0.002mm and 0.01mm as 10%, between 0.01mm and 0.2mm as 28% and above 0.2mm as 42% of the total material. These percentages are summarized in table 4.3 as well as the expected friction angle.

Table 4.3 – Estimation of friction angle from Lang et al (2007) proposed formula

particle size range [mm]		Chainage 1187.5
A	< 0,002	20 %
B	0,002 - 0,01	10 %
C	0,01 - 0,2	28 %
D	> 0,2	42 %
		$\frac{1}{7}A + \frac{1}{5}B + \frac{1}{3}C + \frac{1}{2,5}D$
estimated ϕ [°]		31

Table 4.4 summarizes all values arising from the grain size distribution curves available from the core of the Lavanttal fault zone. The procedure used for the calculation is the same as described above. Associated to the friction angle estimation is the corresponding GA. This is done because each chainage has a predominant GA; therefore there is a need to distinguish between them.

Table 4.4 – Summary of Lavanttal fault zone friction angle estimation

particle size [mm]		grain size distribution [%]							
		Chainage							
		1086.5	1070.9	1140.8	1187.5	1210.5	1297.4	1302.6	1329.9
A	< 0,002	6	4	16	20	18	27	20	10
B	0,002 - 0,01	20	13	25	10	15	16	11	12
C	0,01 - 0,2	26	33	38	28	41	22	37	28
D	> 0,2	48	50	21	42	26	35	32	50
		--- GA37 ---			----- GA38 -----				
estimated φ [°]		33	34	28	31	30	28	30	33

4.1.7. BIM-ROCK APPROACH

To get away from the underestimation of rock mass parameters when dealing with cataclastic rocks, one of the possible approaches is to consider the rock mass as BIM-rock, as described by Medley (1994). This theory states that if clear difference in stiffness of the materials is found, it is possible to divide them into softer matrix and stiffer blocks. These two components, when measured, give the block-in-matrix proportion. If block proportion is between 25% and 75%, it was proved by Lindquist (1994) that the internal friction angle of the weaker matrix can be increased from 0° to 15°.

Geological reports from the construction by Fasching (2008) allow the knowledge of the rock mass types and their distribution at the tunnel face for each chainage. Fig.4.11 represents a diagram of Fasching's (2008) report. From this it is possible to establish that block proportion is given by the "Fels" (from German: rock), "Zerrütung" (from German: crushed rock) and "Block" (from German: block); and the matrix proportion is given by the sum of "grobkörniger Kataklastit" (from German: coarse grained cataclastic rock) and "feinkörniger Kataklastit" (from German: fine grained cataclastic rock). Therefore, the average proportion for the chainages in study is 50% block and 50% matrix (Fig.4.11). According to the BIM-rock literature this may cause an increase of about 7.5° in the friction angle of the cataclastic rock matrix (Fig.2.7).

Considering that prognosis values are assessed from the analysis of the weaker matrix, the most realistic friction angle corresponds to the sum of the prognosis value with the increase (7.5°) expected due to the BIM-rock theory. Also, one of the findings by Lindquist (1994) is that deformation modulus is increased due to the raise of stiffer material percentage. Even though there is no reference to the magnitude of this increase the deformation modulus is expected to be above 100MPa for this analysis, as explained in section 4.2.4. Also, since laboratory tests have been performed, grain size distribution curves are available; therefore, applying Habimana's theory, the deformation modulus can be defined in a range between 125MPa and 450MPa (section 4.2.4).

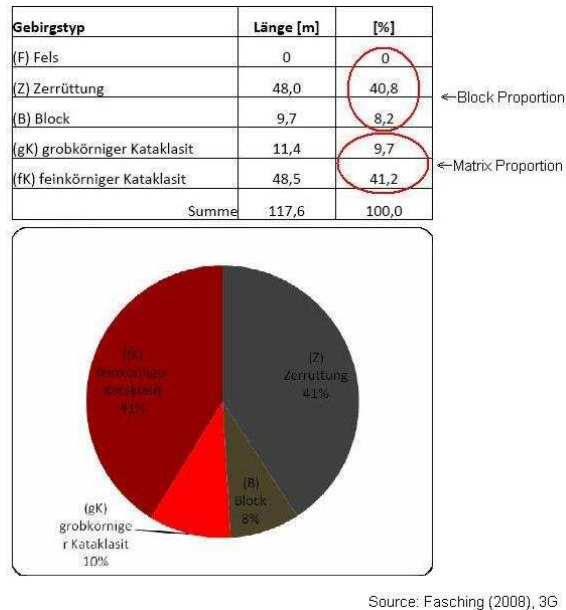


Fig.4.11 – Block-in-matrix proportion estimated from the geological report

4.1.8. COMPARISON OF CALCULATED VALUES WITH PROGNOSIS

The geological report made during the initial investigation works refers to several rock mass types. As stated before, the Lavanttal fault zone was found to be composed mainly of fine grained and coarse grained cataclastic rock, GA37 and GA38 respectively. A major issue in this study is to find out whether the predicted strength parameters match the real behaviour of the excavated investigation tunnels or not. Thus, a comparison between the predicted values and the calculated parameters from back analysis is made.

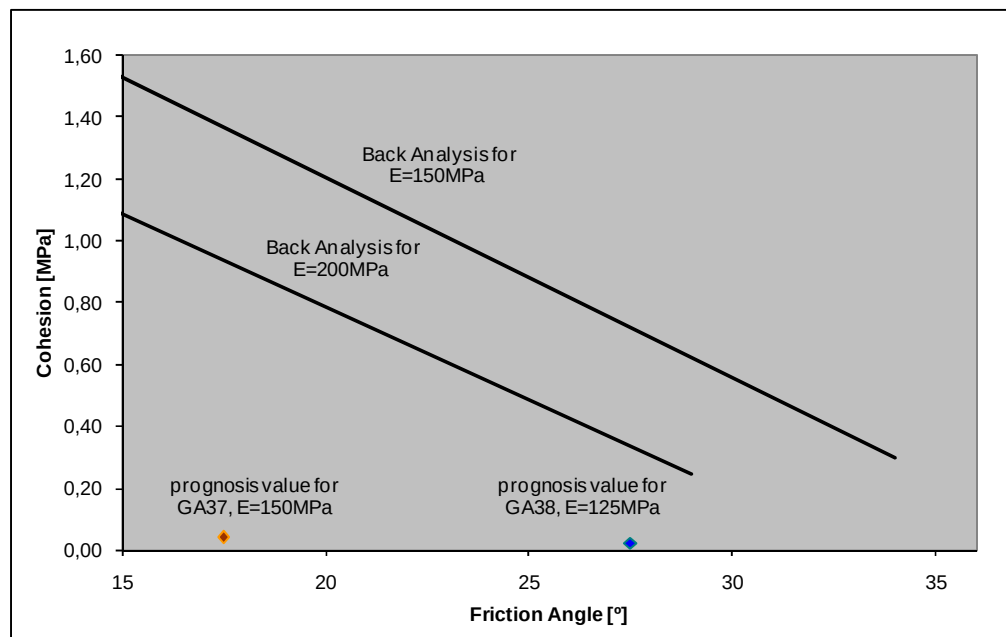


Fig.4.12 – Prognosis parameters vs. back calculated values from convergence confinement method

Diagram of Fig.4.12 plots the estimated values of GA37 and GA38 given by the geological report and the back calculated values considering the convergence confinement method. Predicted deformation modulus range from 100MPa to 200MPa however, results from the convergence confinement theory are only plausible for 150MPa or more (table 4.2). Therefore it is possible to compare GA37 and GA38 prognosis with the back calculated 150MPa and 200MPa curves.

From the diagram it is clear that the calculated values are not in accordance with the predicted results, which can mean an overconservative approach upon the estimation of the rock mass parameters. Such underestimation of parameters may be caused by the uncertainties when dealing with field and laboratory tests.

Plotting the back calculated convergence confinement curves for 200MPa, 250MPa and 350MPa (deformation modulus estimation) and the friction angle increased due to the BIM-rock theory in the same diagram (Fig.4.13) closer values are found. The reader could be persuaded to extend the lines representing possible back analysed values in order to reach cohesion values close to zero matching the improved BIM-rock GA's. However, this cannot be done if the imposition defined for the ratio of pre-displacement to measured displacement (dashed line 1:1) is to be achieved.

Bearing this in mind we can say that the back analysis process carried out at this zone, met several ideas proposed when dealing with cataclastic rock masses. Hence BIM-rock theory is proved to give more realistic values when dealing with the friction angle increase. On the other hand the decrease in cohesion expected to occur is not observed. In fact, the opposite effect (cohesion increase) is more plausible being the possible explanation the high overburden and high confining pressures. This certainly leads to an increase of the overall stiffness as well as a more favourable arrangement of the grains (particles), therefore also increasing cohesion.

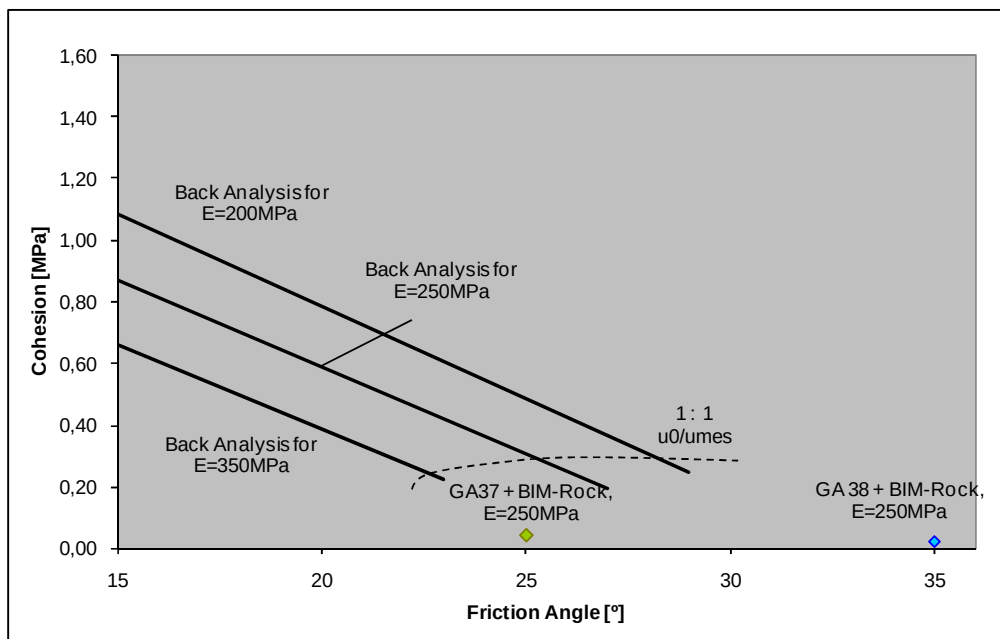


Fig.4.13 – BIM-rock improved strength material vs. back analysis calculations

4.1.9. ESTIMATED RANGE OF VALUES FROM THE BACK ANALYSIS (SUMMARY)

In the previous sections several ways of estimating the rock mass strength parameters were presented. From these calculations and assumptions a summary diagram showing the most “plausible values” that are expected to characterize this rock mass can be plotted as follows:

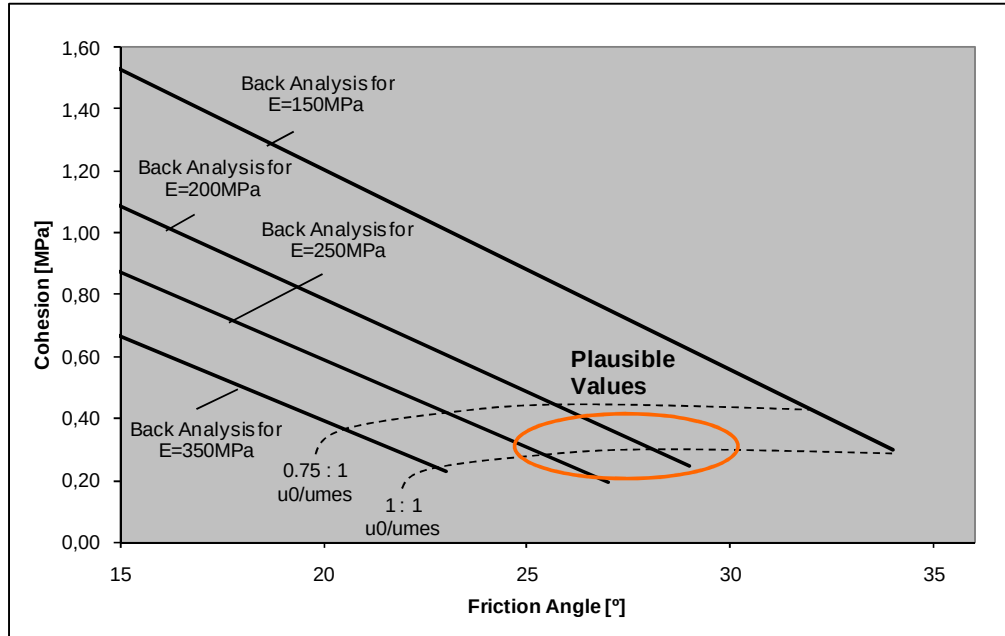


Fig.4.14 – Summary diagram of expected values for the Lavanttal fault zone

The plausible values are:

- Friction angle [26,30] (°)
- Cohesion [0.20,0.40] (MPa)
- Deformation modulus [175,275] (MPa)

The values, however, cannot be combined in all ranges. This means that when combining values, a low cohesion must have high friction angle and deformation modulus etc. Fig.4.14 should be taken into account when combining values so as to make sure that one combination does not correspond to different points in the diagram.

4.1.10. NUMERICAL ANALYSIS SOLUTION

After assessing the rock mass strength parameters through the back analysis methodology proposed, it is expected that values correspond to the reality observed. In addition, numerical analyses with the finite difference element program FLAC are performed to validate the values obtained with the proposed model. The input data used for the calculations arise from one of the combination of possible values shown in Fig.4.14 and summarized in table 4.5.

Table 4.5 – Numerical analysis input data for the Lavanttal fault zone

Numerical analysis input data	
Lavanttal fault zone	
Deformation modulus [MPa]	250
Friction angle [°]	26
Cohesion [MPa]	0,35
Overburden [m]	220
Unit weight [kN/m ³]	22

The definition of the numerical model is done according to the construction drawings and support installed. Fig.4.15 represents a comparison between the cross section design and the numerical profile modelled. It can be seen that two different temporary invert profiles and six deformation elements were initially designed. Upon construction the shallow temporary invert was excavated and only the two upper deformation controllers were cast in place. However, they were not installed in any of the positions designed but rather in an intermediate position as it can be seen in the FLAC model (Fig.4.15).

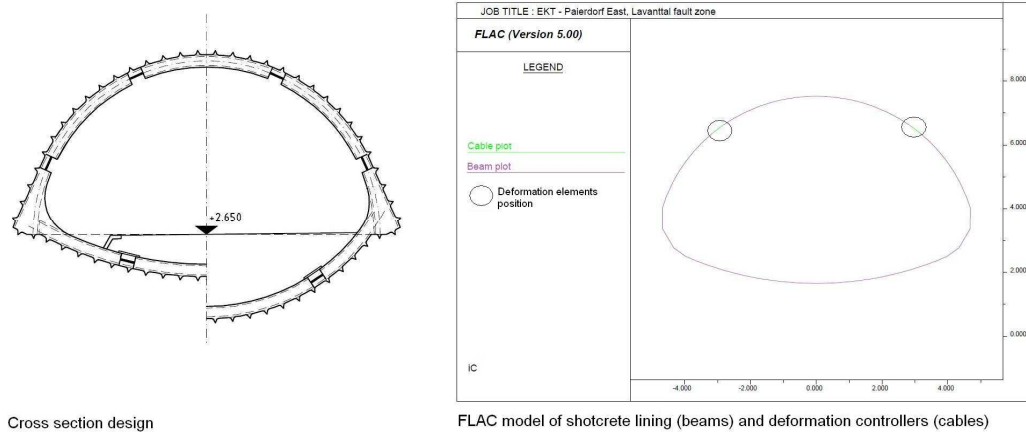


Fig.4.15 – Comparison between numerical model and design draw of Lavanttal cross section

Stress controllers are simulated by cable elements while the shotcrete lining is represented by beam structural elements. Cable elements only support axial forces corresponding to the idealized behaviour of a deformation controller. After initial deformation the system is expected to stabilize what is not possible with cable elements once forces from the rock mass yield require moments to be taken. Therefore in the last stage of the calculation, the cable elements are replaced by beam elements capable of absorbing the moments. Fig.4.16 represents the final displacements calculated with the numerical model. Fig.4.17 shows the history plot of point number 4. This point corresponds to the central crown position and is equivalent to the surveyed position.

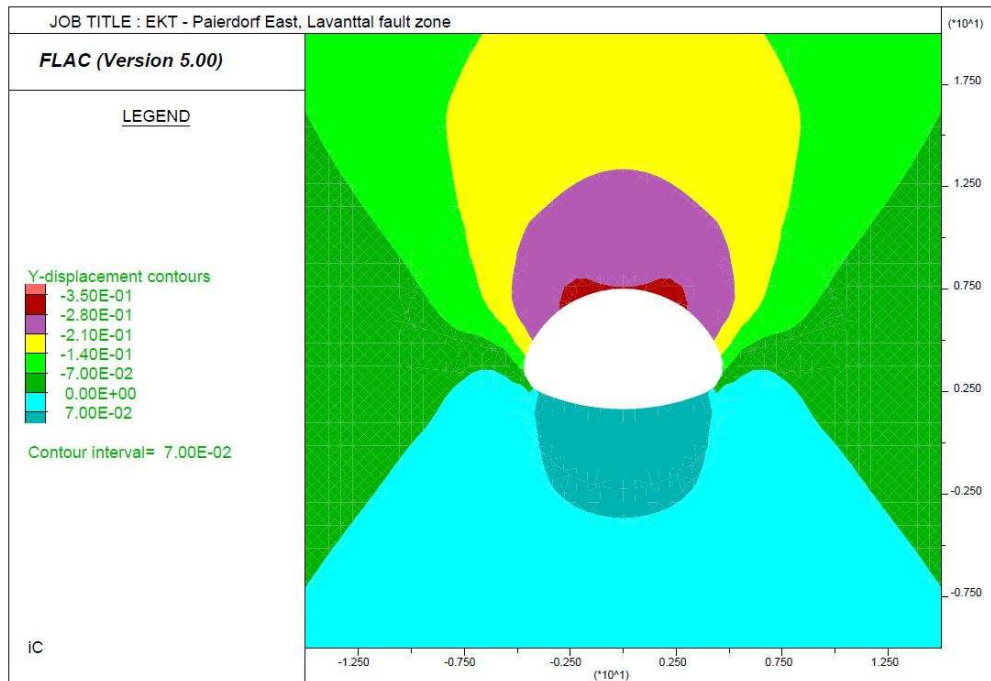


Fig.4.16 – Numerical calculation of Lavanttal cross section – Final displacements

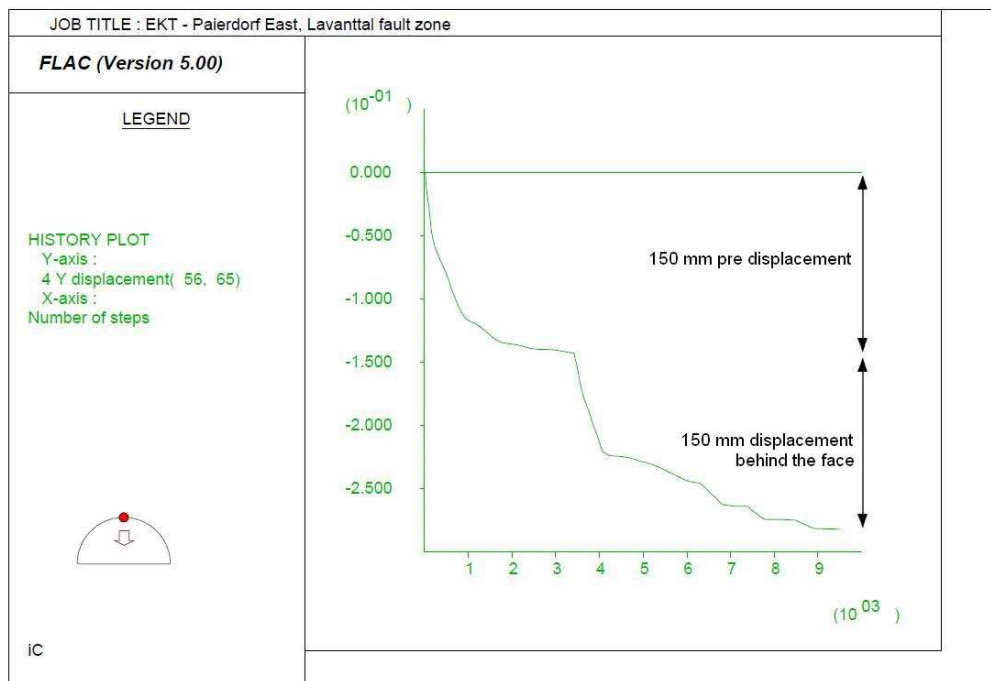


Fig.4.17 – Numerical calculation of Lavanttal cross section – Evolution of displacements

From Fig.4.16 and Fig.4.17 it is evident that maximum displacement expected with the back calculated strength parameters is around 300mm, i.e. 150mm of pre-displacement and 150mm of displacement behind the face. When looking to the displacement evolution diagram (Fig.4.17) one can

observe that the ratio of pre-displacement to the measured displacement is approximately 1:1 as it was predicted. With this information it is possible to confirm that the most probable strength values calculated with the convergence confinement methodology achieve a good relation with the numerical analysis modelling.

In order to assess reasonable parameters for typical rock mass types, a set of cross sections composed of different materials types is analysed. The selected cross sections intend to cover some of the most representative materials found during the excavation of the investigation tunnels. For this reason each one of the chainages chosen is composed of only one main rock mass type namely sand, siltstone and gravel. The study goes along the different conditions found on Mitterpichling and Paierdorf tunnels such as the different overburdens and different support types.

4.2. SAND ZONE

The east drive of Mitterpichling investigation tunnel crosses, at chainage 1+957, a zone composed mainly of sandy sedimentary material with an overburden of about 68m. Geotechnical preliminary investigations defined the rock mass type mainly composed of GA4 and GA7, sand and siltstone respectively. However, upon construction a mixture mostly of sand and sandstone classified as GA4 and GA8 was found at this cross-section as it can be seen from Fig.4.18. In the prognosis document, the cohesion of the materials was defined close to zero while the friction angle and deformation modulus from [25,32] (°) and [100,300] (MPa) respectively. This can be read in Figs.A2.1 and A2.5 in appendix 2.

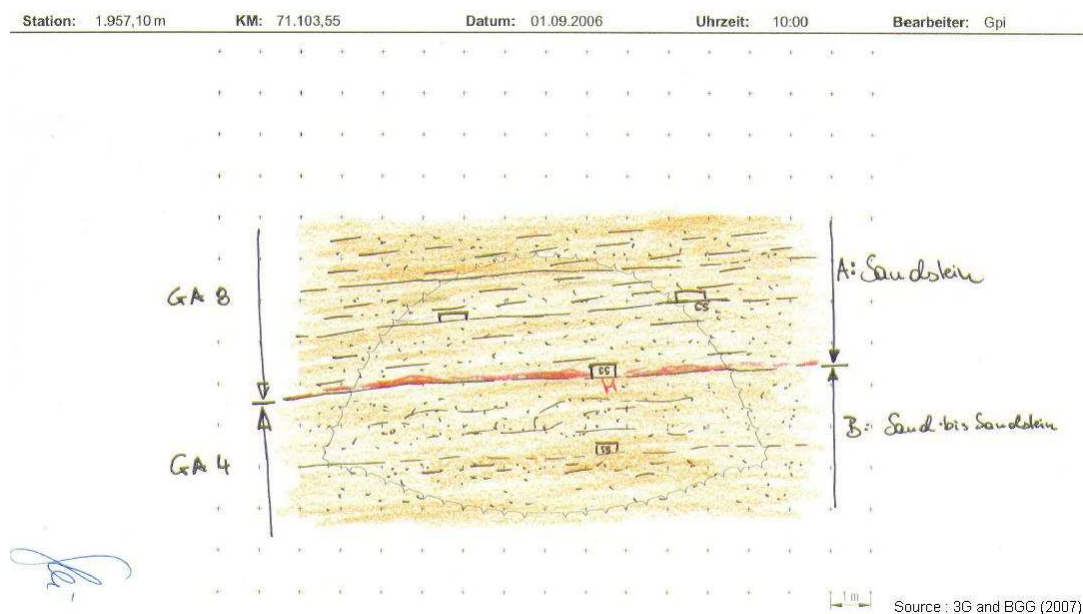


Fig.4.18 – Tunnel face draw at chainage 1+957 of Mitterpichling East investigation tunnel

The unit weight of the material was estimated according to the reference values given in the Grundbau-Taschenbuch book. The sand in this cross section corresponds to the material 7 being therefore possible to read in Fig.A.1.1 in appendix 1 a unit weight of 22kN/m^3 .

Monitoring data registered a maximum inward displacement at the crown of about 60mm. From the shotcrete stresses and bolting analysis the internal support pressure can be set as 0.15MPa.

From shotcrete stress analysis it is possible to determine an average stress value of 1.49MPa. Applying equation (3.8) and knowing that the shotcrete thickness is 0.25m, it is possible to define the contribution for the internal support pressure as 0.08MPa. According to the Required Excavation and Support Sheet (RESS) four bolts are installed. Considering Hoek (1999) equation for the estimation of the maximum support pressure of a 34mm rockbolt a value of 0.07MPa is found. The sum of these two values gives the estimated internal support pressure to be used in the calculations.

Table 4.6 – Convergence confinement input data for sand cross section

Convergence confinement input data		Mitterpichling East
Parameter		1+953
		GA4 + GA8
measured displacement - u [mm]		60
unit weight [kN/m ³]		22
Overburden [m]		68
Pre displacement at the face [mm]		40
number of rockbolts [n]		4
rockbolts internal support pressure [MPa]		0.07
pi	average shotcrete stress [MPa]	-1.49
	shotcrete thickness [m]	0.25
	shotcrete internal support pressure [MPa]	0.08
	total internal support pressure [MPa]	0.15

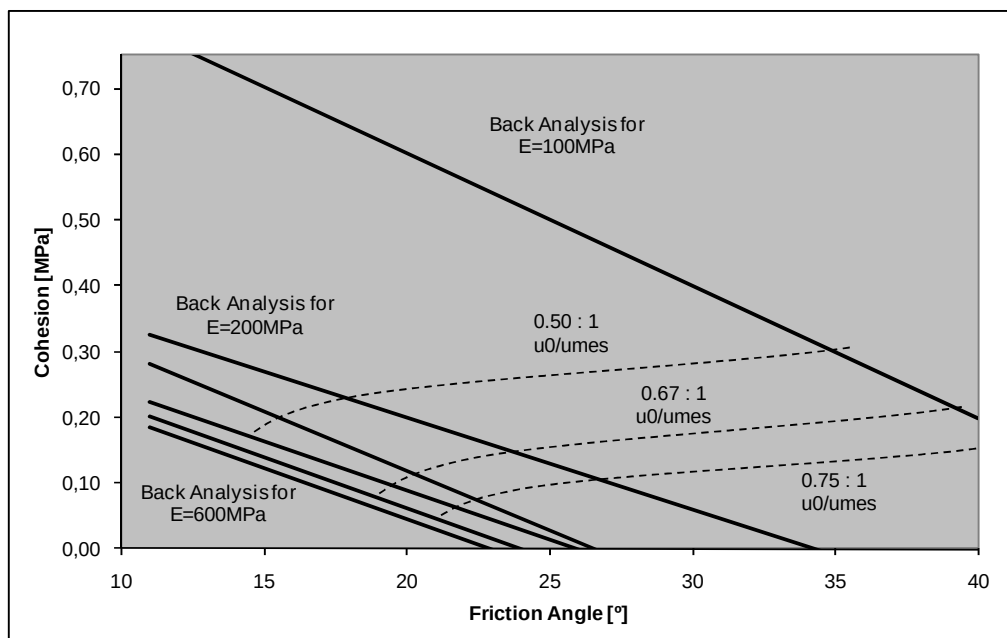


Fig.4.19 – Range of possible rock mass parameters values for Sand

From these considerations and applying the convergence confinement proposed method it is possible to plot the diagram of Fig.4.19. From it one can read the expected shear strength parameters (i.e.

friction angle and cohesion) for deformation moduli ranging from 100MPa to 600MPa. Each line shows an increase of 100MPa in deformation reading from the top to the bottom. Dashed lines represent the u_0/u_{mes} found for the possible combinations. Input data used for the back calculation is resumed in table 4.6.

4.2.1. KEY PARAMETERS ESTIMATION

Assessment of the deformation modulus follows the guidance of the Grundbau-Taschenbuch. Values of deformation modulus for this cross section are expected to be between 100MPa to 300MPa. The data calculated are displayed in Fig.4.20 and gives an overview of the possible values. This range is defined by the higher and lower calculated values from the technical guide.

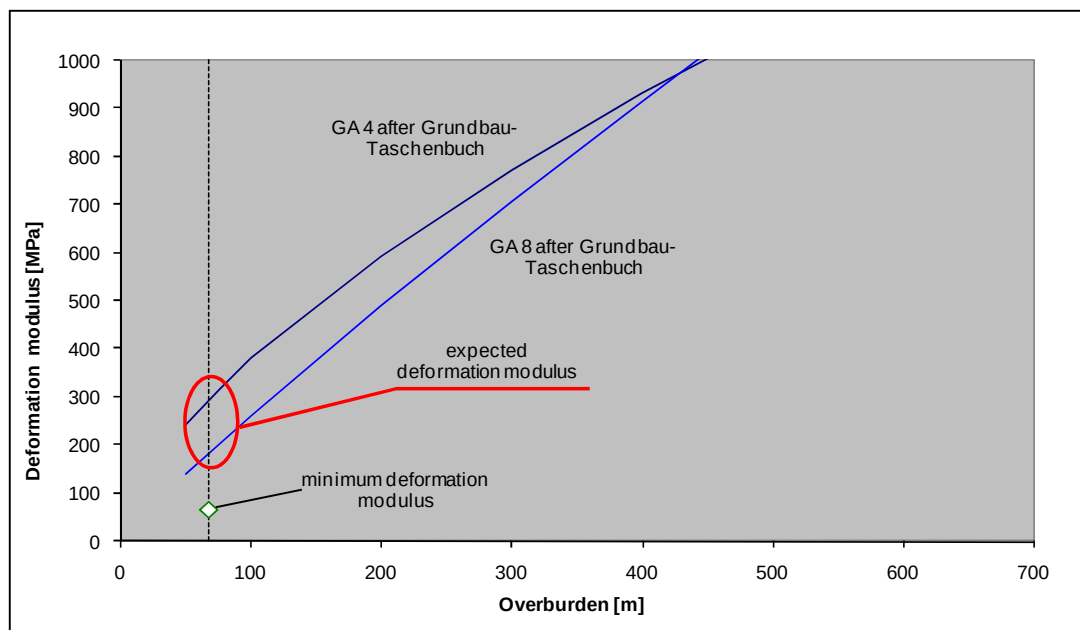


Fig.4.20 – Estimation of deformation modulus for Sand

The friction angle can be calculated, as described previously in section 3.6, from the analysis of the grain size distribution curve. The corresponding curve is taken from the geological report as a range of possible values assessed at the preliminary stage of investigation. Therefore, three values were calculated from this curve corresponding to the upper and lower boundaries and the mean value between them. The grain size distribution curve can be found in Fig.A.2.10 in appendix 2. Table 4.7 summarizes the percentage values and the corresponding calculated friction angles.

Table 4.7 – Summary of Sand zone friction angle

		grain size distribution [%]		
particle size [mm]		Sample		
		1	2	3
A	< 0,002	1	3	6
B	0,002 - 0,01	2	9	13
C	0,01 - 0,2	25	43	62
D	> 0,2	72	45	19
estimated ϕ [°]		38	35	32

As discussed earlier face pre-displacements are related to the measured crown settlements in a factor of approximately 1:1. Analysis with GeoFit for this cross section showed a somewhat different value, however, the estimated pre-displacement being around 70% of the total displacement measured (ratio 0.67:1).

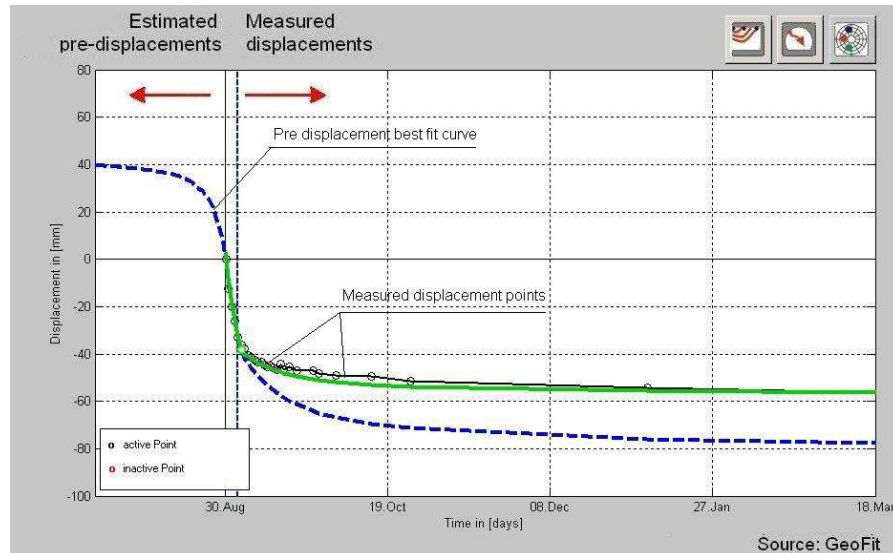


Fig.4.21 – GeoFit pre displacement calculation for the Sand cross-section at Mitterpichling east chainage 1+957

4.2.2. COMPARISON BETWEEN CALCULATED VALUES AND PROGNOSIS

With the convergence confinement spreadsheet values a diagram, comparing the back analysed and the predicted values from the geological study, is plotted. Since the predicted range of deformation modulus ranges from 100MPa to 300MPa these are the values to be compared.

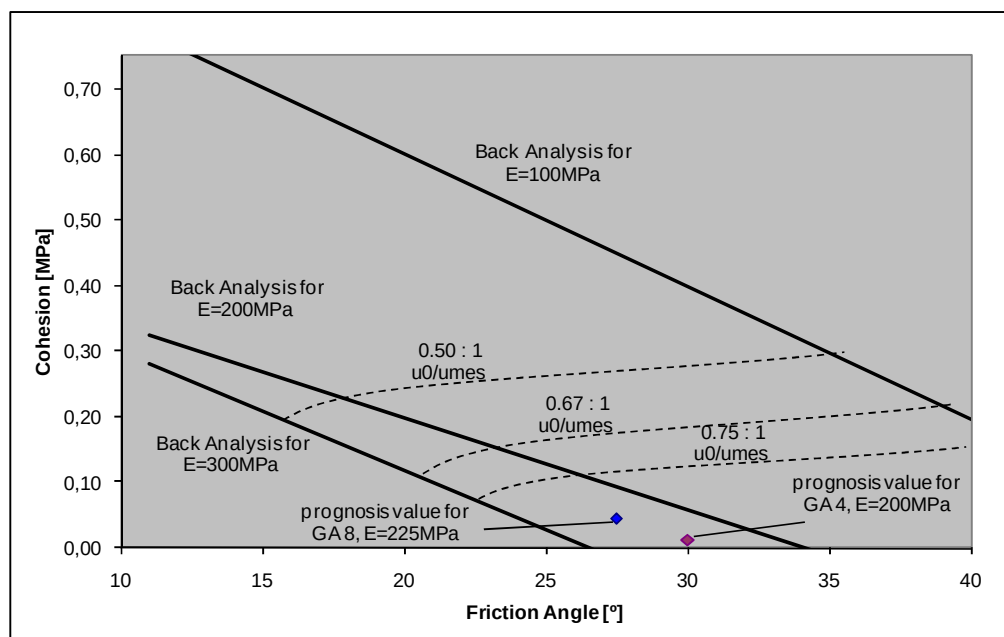


Fig.4.22 – Comparison between prognosis and calculated values

This cross section was found to be the one where the best relation between the geological assessment and the back analysis values occurred as it can be seen from Fig.4.22 where geological predicted values lie between the convergence confinement calculations. However, cohesion should be relatively higher in order to match the ratio u_0/u_{mes} calculated with GeoFit (Fig. 4.21).

4.2.3. ESTIMATED RANGE OF VALUES FROM THE BACK ANALYSIS OF THE SAND CROSS SECTION

A diagram resuming the estimation of the strength values for the sand cross section in study is presented in Fig.4.23. The expected values are:

- Friction angle [27,33] (°)
- Cohesion [0.15,0.25] (MPa)
- Deformation modulus [150,200] (MPa)

The diagram in Fig.4.23 should be comprehended as follows: considering the most probable value of cohesion to be 0.20MPa and the deformation modulus as 150MPa, friction angle must be 29°. This way of reading must always be followed when finding values from the ranges defined using Fig.4.23 as guidance for a correct matching.

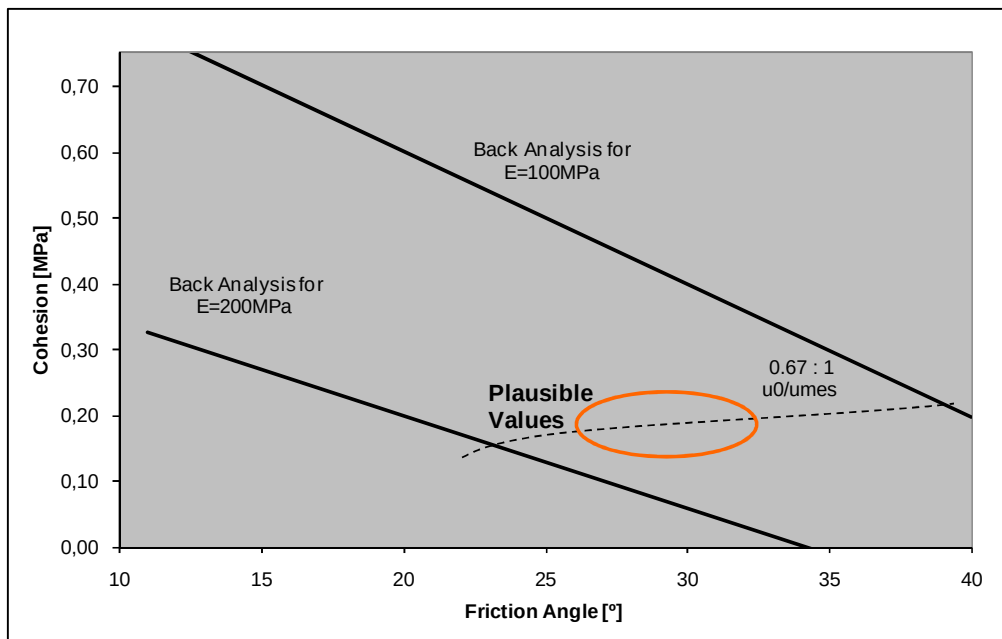


Fig.4.23 – Summary diagram of expected values for Sand cross section

4.2.4. NUMERICAL ANALYSIS SOLUTION

Back analysed strength parameters are compared with the numerical finite differences software FLAC. The model used in FLAC is based on the construction site information reported by 3G & BGG (2007) (rockbolts, shotcrete, cross section profile, etc). The rock mass strength properties arise from one of the possible combinations of Fig.4.23. The combination used is given in table 4.8.

Table 4.8 – Numerical analysis input data for sand

Numerical analysis input data	
Sand cross section	
Deformation modulus [MPa]	150
Friction angle [°]	31
Cohesion [MPa]	0,22
Overburden [m]	68
Unit weight [kN/m ³]	22

For the assessment of the pre-displacement in FLAC a relaxation factor is applied to the rock mass model reducing its overall stiffness thus simulating the tunnel face advance. Total displacements are found after simulation of the excavation stages by consequent changing of the relaxation factor; top heading and bench excavation, application of young shotcrete lining, temporary invert excavation with application of young shotcrete and hardening of the shotcrete properties in the top heading and bench and hardening of the invert shotcrete layer properties. The initial relaxation factor corresponding to the tunnel face advance is 0.50. This initial relaxation factor is increased in each of the excavation stages to 0.75, 0.85, 0.90 and 1.00. In the numerical tool the relaxation factor corresponds to the amount of rock mass strength, i.e. the unbalanced forces in the system after calculation of the stage steps will act in the rock mass in the relaxation factor magnitude. If a relaxation factor of 0.75 is applied it means that 75% of the rock mass strength will be used.

Figs.4.24 and 4.25 show the numerical analysis calculated displacements as well as the history evolution plot at the crown position. A pre-displacement value of 40mm and a displacement behind the face of approximately 56mm is foreseen in Fig.4.25. This makes the total displacement 96mm showing that good agreement between the convergence confinement back calculated values and the numerical calculations is achieved.

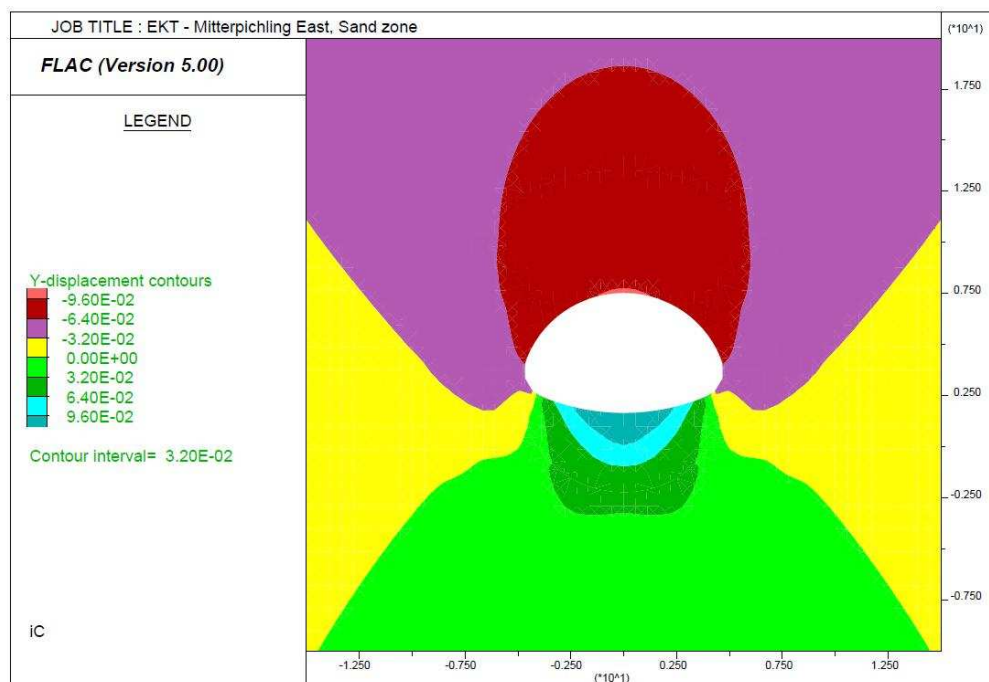


Fig.4.24 – Numerical calculation of Sand cross section – Final displacements

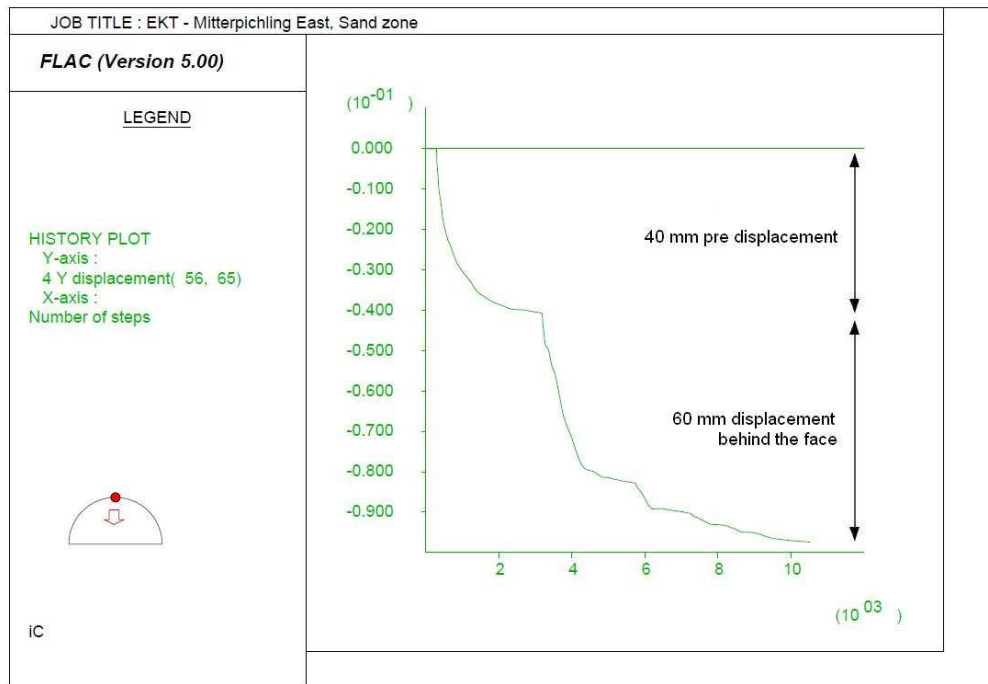


Fig.4.25 – Numerical calculation of Sand cross section – Evolution of displacements

4.3. GRAVEL ZONE

Paierdorf west drive crosses, from chainage 0+600 to 0+240, a major gravel zone. At chainage 0+536 it is composed only of GA5, classified as gravel by the geological report and which parameters are [30,35] (°) of friction angle, [0.01,0.025] (MPa) of cohesion and [250,350] (MPa) of deformation modulus (Fig.A2.2). The tunnel face mapping registered by the geological team 3G and BGG (2007) is given in Fig.4.26.

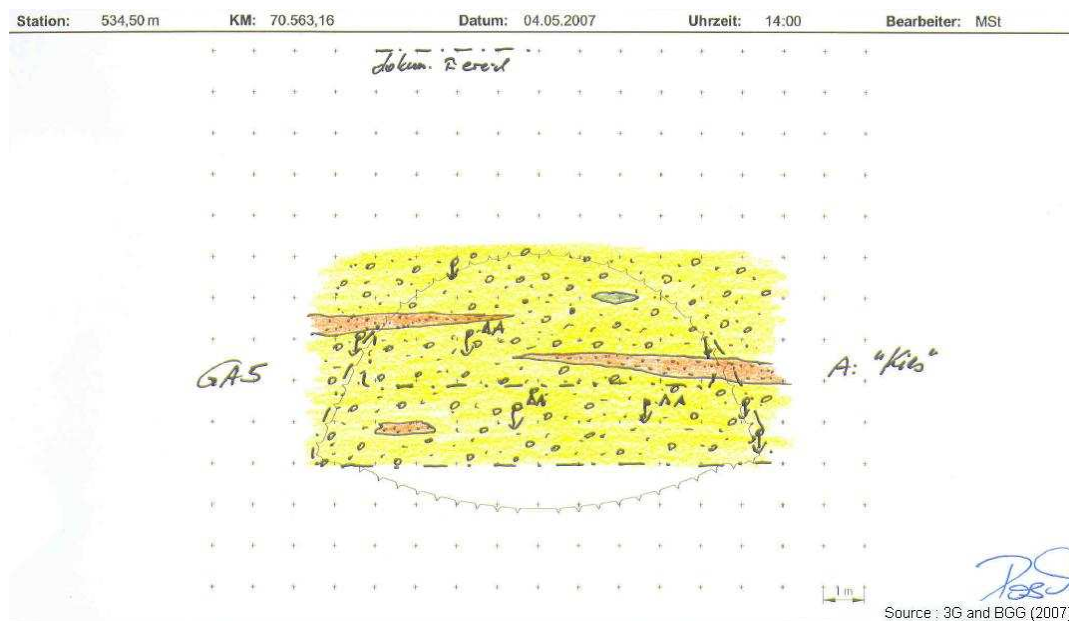


Fig.4.26 – Tunnel face mapping at chainage 0+536 of Paierdorf West investigation tunnel

The monitoring operation carried on the investigation tunnel showed a displacement of 22mm occurring after the excavation and the stress analysis revealed the inner lining to be in tension. This means that no internal support pressure is taken into account from the shotcrete. On the other hand a mesh of 8/6 rockbolts was placed during construction (eight in one round length, six in the following and so on). These rockbolts have therefore a spacing of 1.41m which, considering a 34mm rockbolt, gives a maximum p_i value of 0.17MPa according to Hoek (1999) proposed formulae (Fig.2.3). The rock mass unit weight of 21kN/m³ was considered to be the minimum value for material 3 found in Grundbau-Taschenbuch. The reference table where it is possible to read this value is given in Fig.A.1.1 in appendix 1.

Table 4.9 – Convergence confinement input data for gravel cross section

Convergence confinement input data		Paierdorf West
Parameter		0+536
		GA5
measured displacement - u [mm]		25
unit weight [kN/m ³]		21
Overburden [m]		90
Pre displacement at the face [mm]		22
number of rockbolts [n]		8 / 6
rockbolts internal support pressure [MPa]		0.17
pi	average shotcrete stress [MPa]	+ 1.43
	shotcrete thickness [m]	0.30
	shotcrete internal support pressure [MPa]	0.00
	total internal support pressure [MPa]	0.17

From this information summarized in table 4.9 it is possible to back calculate the range of possible values for the rock mass parameters as it can be seen in diagram of Fig.4.27. Dashed lines mark the ratio u_0/u_{mes} explained in section 3.2.2.

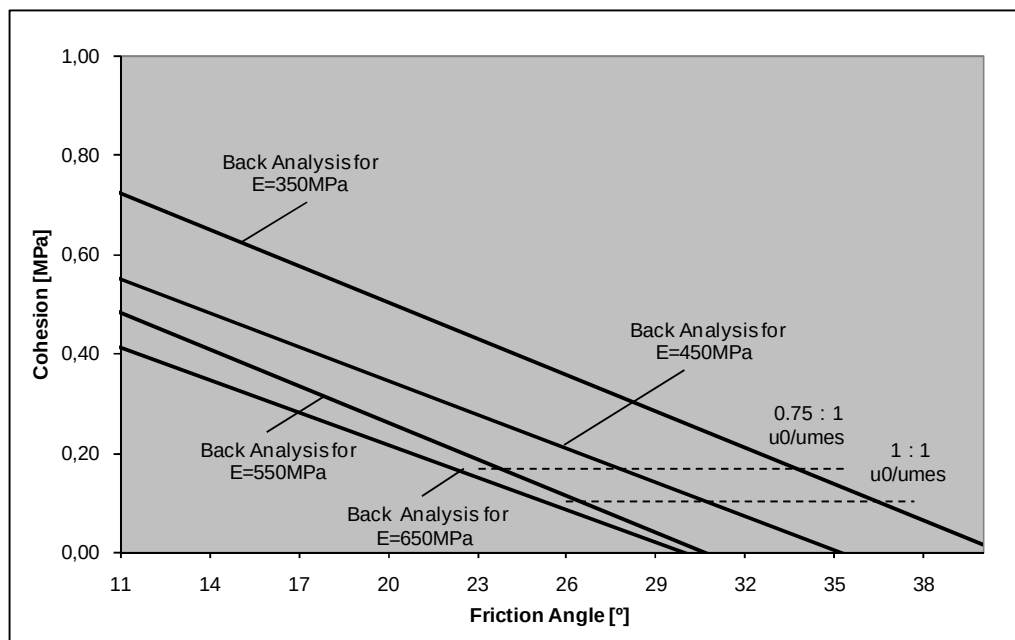


Fig.4.27 – Range of possible rock mass parameters values for Gravel

4.3.1. KEY PARAMETERS ESTIMATION

The deformation modulus calculations according to Witt's (2008) Grundbau-Taschenbuch revealed a range of values arising from 300MPa to 550MPa assuming the higher and lower values of material 3 found in Fig.A.1.1 in appendix 1. Also, calculating the minimum deformation modulus defined from the convergence confinement method, as explained in section 3.5.3, a value of 224MPa is obtained. This relatively high minimum value has a plausible explanation in the low displacement measured once it has a proportional influence in the arising equation. The way how to calculate the minimum value for the deformation modulus is given by equation (4.1)

$$E_{\min} = \frac{r_o(1+\nu)}{u}(p_0 - p_i) \Leftrightarrow$$

$$E_{\min} = \frac{4,5(1+0,3)}{0,045}(1,89 - 0,17) = 224MPa \quad (4.1)$$

As shown in Fig.4.28 the plausible range for the deformation modulus in this cross section can be considered from 350MPa to 550MPa.

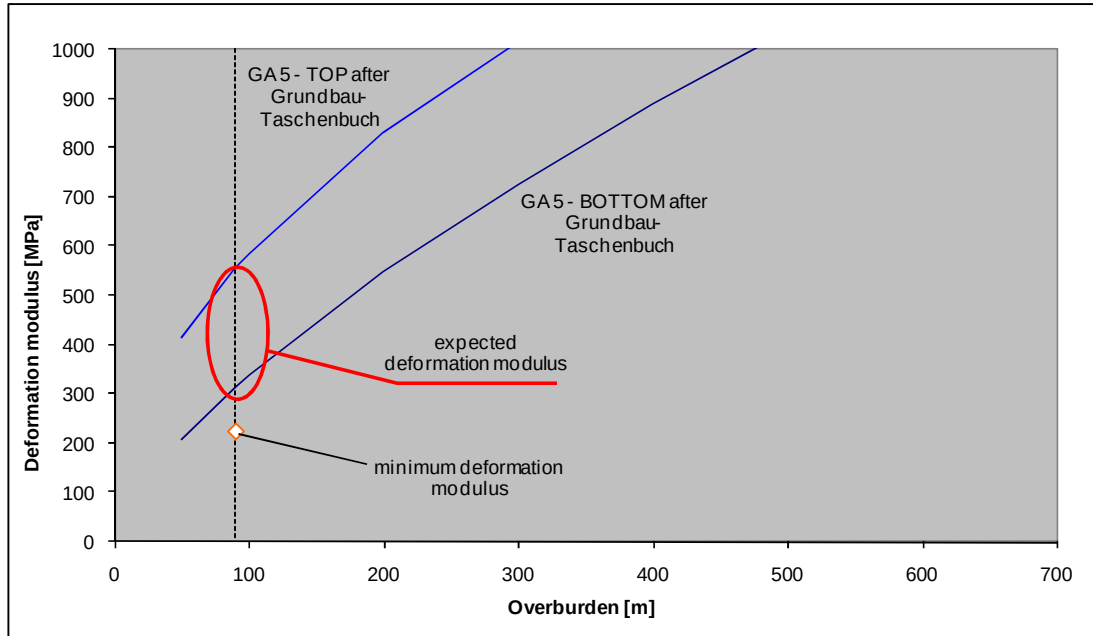


Fig.4.28 – Estimation of deformation modulus for Gravel

Going through Lang et al (2007) proposed method for estimation of the friction angle a relatively small range is found from the grain size distribution curve given in Fig.A.2.11 of appendix 2. Values are expected to be within 36° and 38°. This is a common value of friction angle for silty gravels and gravel-sand-silt mixtures as it is the case here. Moreover, considering a certain degree of cementation expected due to the presence of silt and sand particles, cohesion can be considered ranging from 0.05MPa to 0.20MPa.

For low inward displacements values, as observed in this cross section (25mm), Hölzl (2009) suggests that pre-displacements at the face have the same magnitude as the measured ones. This suggestion corroborates the fitting curve method used by GeoFit proposed by Sellner (2000).

4.3.2. COMPARISON BETWEEN CALCULATED VALUES AND PROGNOSIS

Geotechnical prognosis for the gravel rock mass type reported low cohesion values together with relatively high friction angles. Also, the deformation modulus corresponding to this geotechnical prognosis is lower than the one back calculated with the convergence confinement method. Plotting in the same diagram the prognosis values ($c=0.01\text{MPa}$; $\phi=31^\circ$; $E=350\text{MPa}$) and the back calculated values significant differences between them (Fig.4.29) are noticed. From the diagram it is possible to get the information that with 31° of friction angle (prognosis) cohesion should be 0.10MPa and deformation modulus 450MPa .

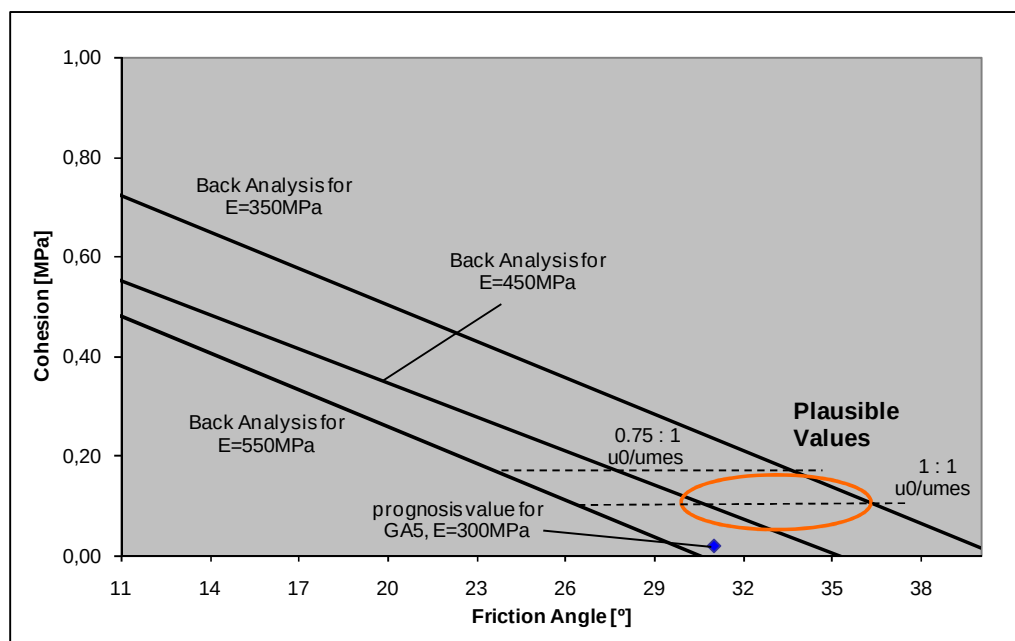


Fig.4.29 – Comparison between geological prediction and calculated values for Gravel

4.3.3. ESTIMATED RANGE OF VALUES FROM THE BACK ANALYSIS OF THE GRAVEL CROSS SECTION

Resuming the information discussed before, the back analysis of the gravel cross section allowed the definition of the following possible values:

- Friction angle $[31,36]$ ($^\circ$)
- Cohesion $[0.05,0.20]$ (MPa)
- Deformation modulus $[350,450]$ (MPa)

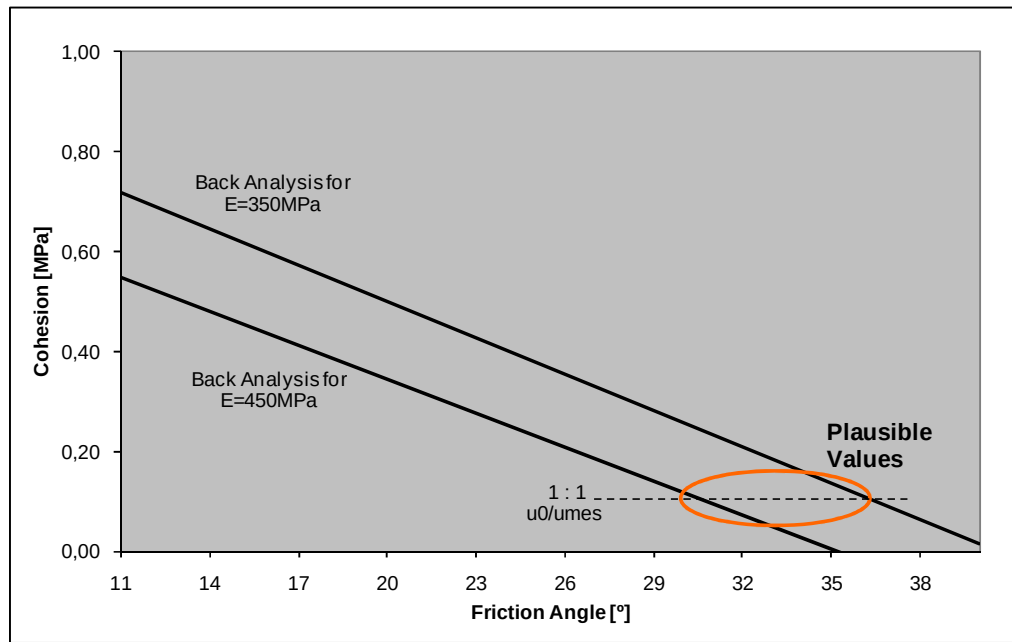


Fig.4.30 – Summary diagram of expected values for Gravel cross section

Again the range of values assessed cannot be combined in all possibilities. In order to have reliable values one must choose only one point from the area of “plausible values” in the diagram and read the values. In this way it is possible to have plausible and mathematically correct combinations of strength parameters according to the convergence confinement theory.

4.3.4. NUMERICAL ANALYSIS SOLUTION

So as to assess the reliability of the model used, numerical analysis using FLAC were performed. The input data, used for generating the model of the gravel cross section, arise from one of the possible combinations from Fig.4.30 and a resume of the characteristics is shown on table 4.10. The support characteristics (number of rockbolts, shotcrete thickness etc.) and cross sectional profile are given by the Required Excavation and Support Sheets shown in appendix 2.

Table 4.10 – Numerical analysis input data for Gravel

Numerical analysis input data	
Gravel cross section	
Deformation modulus [MPa]	450
Friction angle [°]	32
Cohesion [MPa]	0,10
Overburden [m]	70
Unit weight [kN/m3]	21

In Figs.4.31 and 4.32 the final inward displacements and the evolution of the total displacements are shown. Pre-displacements are calculated by applying a relaxation factor to the rock mass which simulates the arch effect in front of the face due to the tunnel advance. Final displacement is reached

modelling the top heading and bench excavation, cast in place of shotcrete and rockbolts, invert excavation and shotcrete application on the invert.

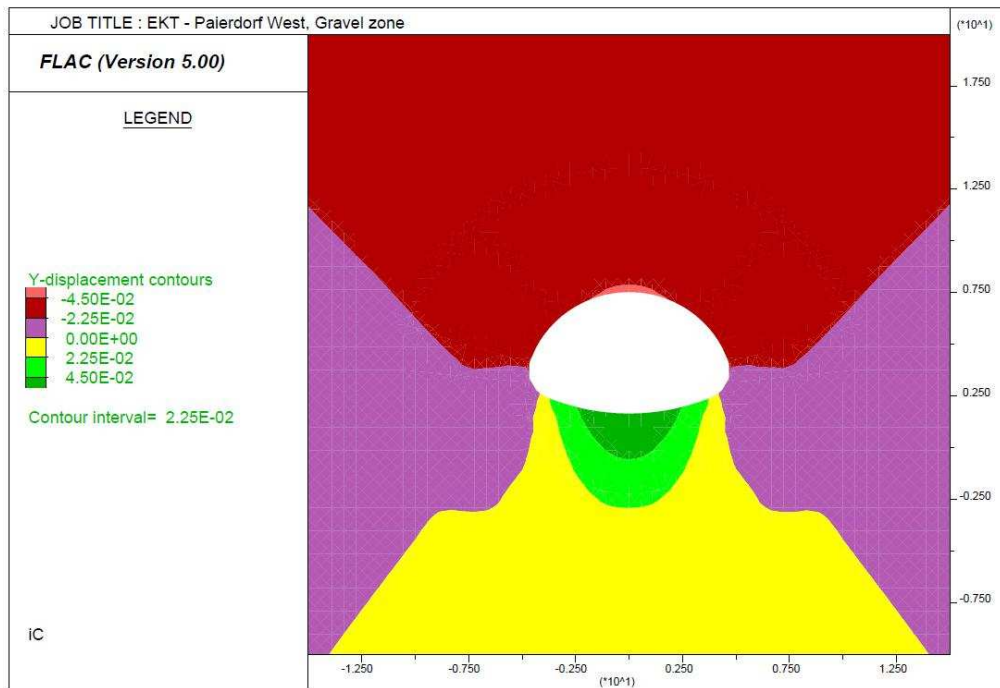


Fig.4.31 – Numerical calculation of Gravel cross section – Final displacements

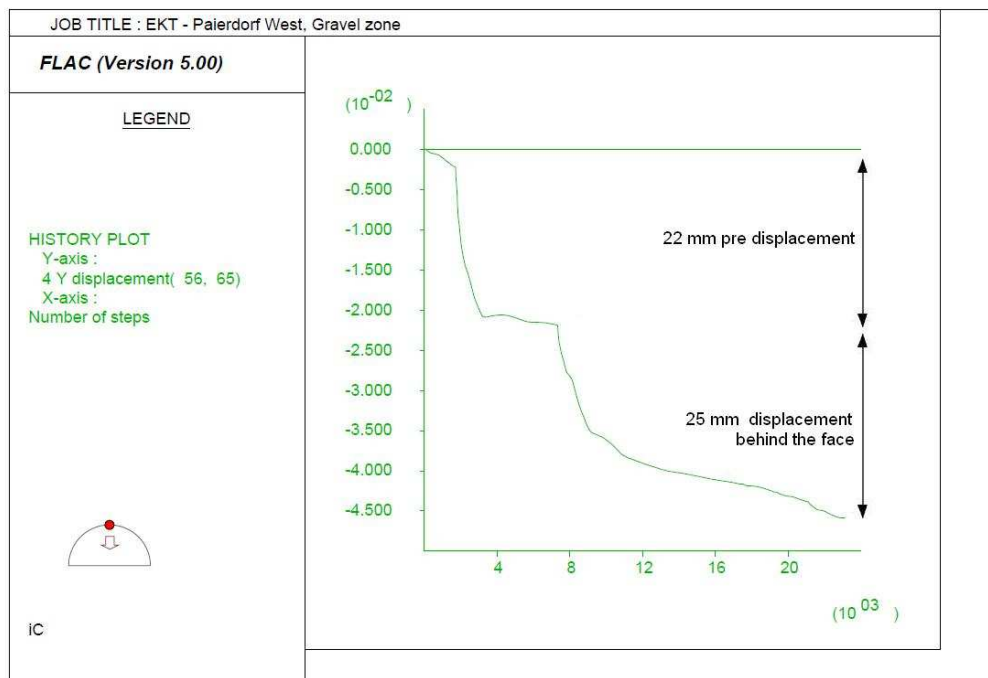


Fig.4.32 – Numerical calculation of Gravel cross section – Evolution of displacements

From the figures it is possible to read a pre-displacement of about 22mm as well as a displacement behind the face of about 25mm (Fig.4.32). The final displacement corresponding to the sum of pre-displacement and displacement behind the face is approximately 47mm. These results confirm that both models are in accordance and also the foreseen ratio of 1:1 (pre-displacement to displacement) is observed.

4.4. SILTSTONE ZONE

Siltstone material covers most of Mitterpichling and a part of Paierdorf investigation tunnels. For this reason many different conditions, such as weatherization, overburden or ground water are reported leading to the need of a detailed study of some of the most representative conditions. The selected cross sections include a fault zone at chainage 2+120 and a homogeneous undisturbed material at chainage 2+062 of Mitterpichling East drive. Both have an overburden of 70m. Paierdorf East drive chainage 0+250 is characterized by a higher overburden, about 140m, which showed slight squeezing problems. The three cross sections studied have their input characteristics to the proposed model summarized in table 4.11. The geological report given by 3G & BGG (2007) allows the graphical understanding of the conditions found. Fig.4.33 shows the fault zone and the undisturbed material in Mitterpichling investigation tunnel and Fig.4.34 shows the cross section found in Paierdorf East which is located in a homogeneous rock mass with high overburden chainage.

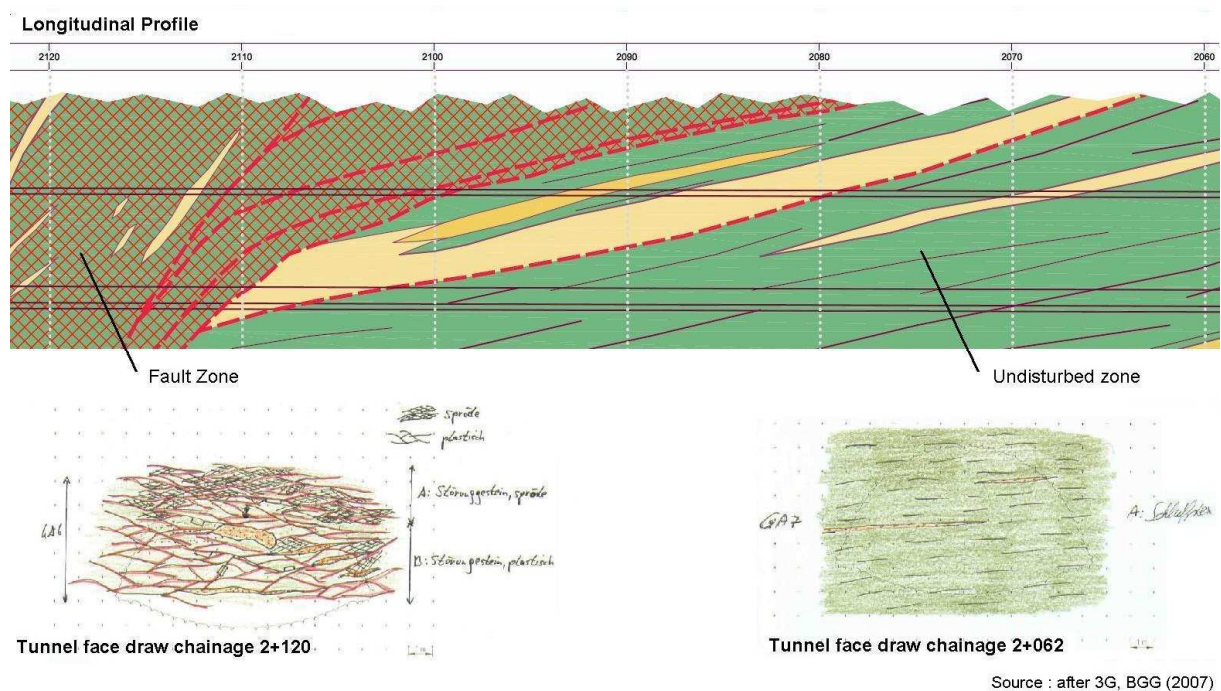


Fig.4.33 – Longitudinal profile and tunnel face draws of Mitterpichling East drive siltstone zone

Rock mass types found GA6 and GA7 have different predicted characteristics given by the geological report as follows:

- GA6
Cohesion [0.03,0.05] (MPa)
Friction angle [18,20] (°)
Deformation modulus [70,150] (MPa)

- GA7
Cohesion [0.07,0.12] (MPa),
Friction angle [22,25] (°)
Deformation modulus [200,350] (MPa)

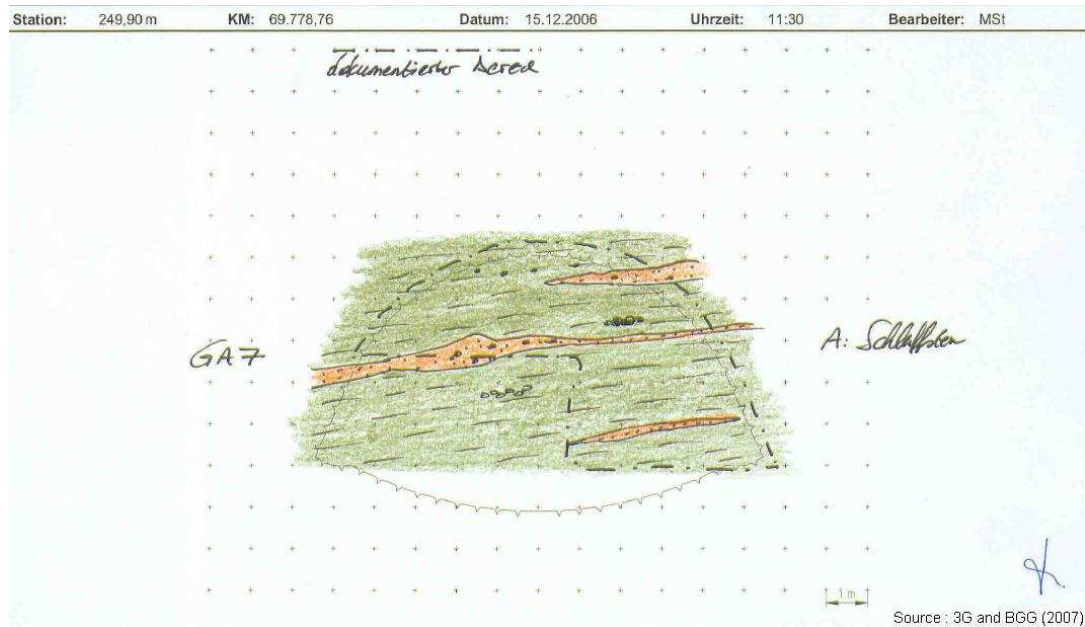


Fig.4.34 – Tunnel face mapping at chainage 0+250 of Paierdorf East investigation tunnel

Table 4.11 – Convergence confinement input data for siltstone cross sections

Convergence confinement input data		Mitterpichling East		Paierdorf East
Parameter		2+062	2+120	0+250
		GA7 - undisturbed	GA6 - fault	GA7 - squeezing
measured displacement - u [mm]		25	70	60
unit weight [kN/m ³]		21	21	21
Overburden [m]		70	70	140
Pre displacement at the face [mm]		20	70	55
number of rockbolts [n]		4	6	6
rockbolts internal support pressure [MPa]		0.07	0.10	0.10
pi	average shotcrete stress [MPa]	-1.72	-2.44	-6.41
	shotcrete thickness [m]	0.25	0.30	0.30
	shotcrete internal support pressure [MPa]	0.10	0.16	0.42
	total internal support pressure [MPa]	0.17	0.26	0.52

The unit weight found for the cross sections is derived from the geological description of the conditions found during construction given by 3G & BGG (2007). This description, cross checked and compared with typical values for siltstone materials given by Witt (2008) and Hunt (1984), allows the consideration of 21kN/m³ as a reasonable value. It is important to notice that this value is only an assumption from the literature and that laboratory tests (not available) are needed for more reliable predictions.

Information about the internal support pressure given by the shotcrete and the rockbolting is summarized in table 4.11. It is possible to read the shotcrete stress analysis average value as well as the shotcrete thickness and the number of bolts installed. With the information it is possible to estimate the contribution of each of the support systems and the total internal support pressure. The number of rockbolts and shotcrete thickness is found on the different RESS given in section A2.3 of appendix 2. Also a resume of the stress analysis of the shotcrete can be found in appendix 2 section A2.4. The information presented before allow the back calculation of the possible values with the convergence confinement methodology and the plotting of the following diagrams.

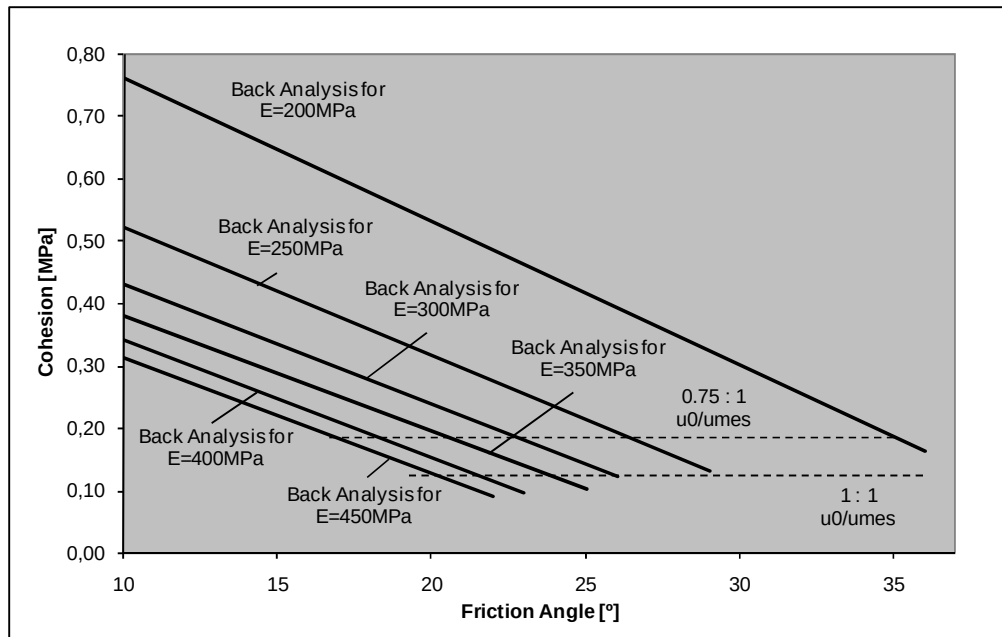


Fig.4.35 – Range of possible rock mass parameters values for undisturbed siltstone chainage 2+062

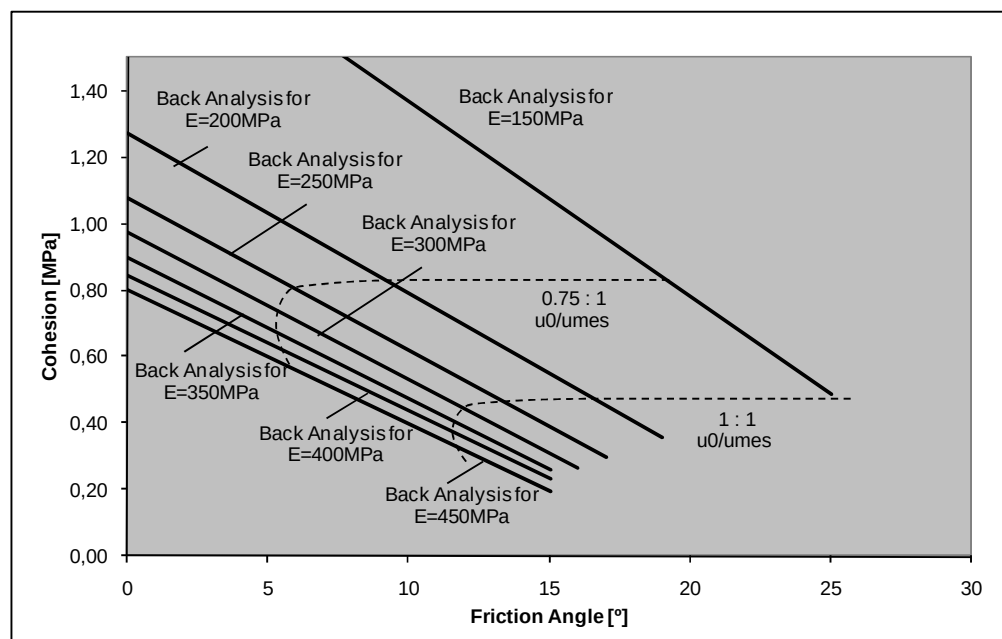


Fig.4.36 – Range of possible rock mass parameters values for the squeezing siltstone chainage 0+250

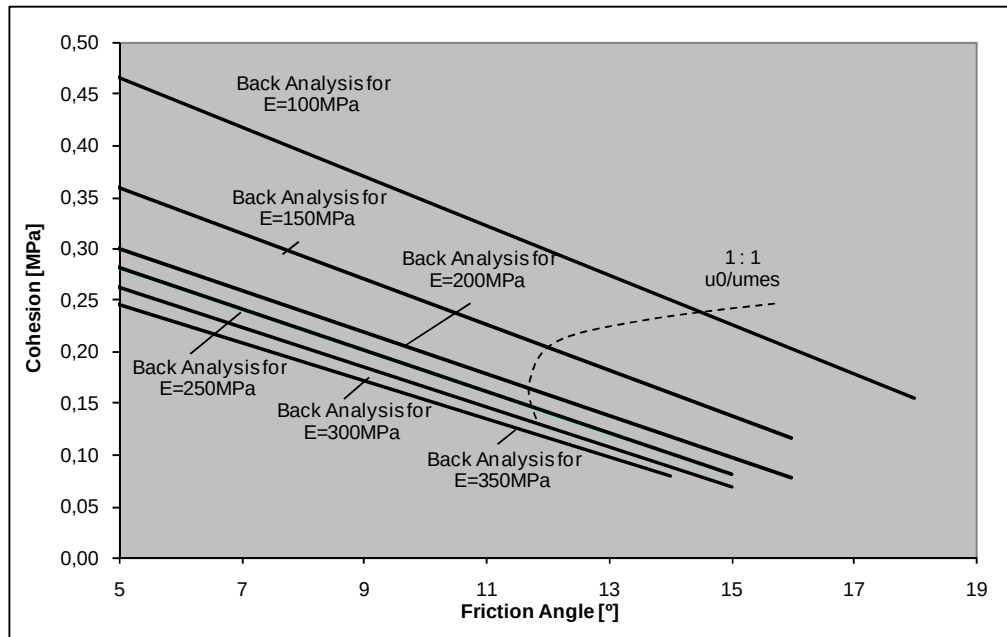


Fig.4.37 – Range of possible rock mass parameters values for the fault siltstone chainage 2+120

Diagrams of Figs.4.35, 4.36 and 4.37 show the back calculated values (E , c and ϕ) for each chainage as well as the different ratios of pre-displacement to measured displacement (u_0/u_{mes}). Depending on the situation found in each cross section (undisturbed, slight squeezed or fault) back calculated values are considerably different even in chainages with the same rock mass (Fig.4.35 and Fig.4.36).

4.4.1. KEY PARAMETERS ESTIMATION

Estimating deformation modulus for the siltstone material from Grundbau-Taschenbuch guide book revealed low values. This reason led to the consideration of the values proposed by this approach not relevant since applying the minimum deformation modulus estimation discussed earlier in section 3.5.3, stiffer and more plausible values have been found. The findings are displayed in Fig.4.38 where the difference between values calculated by the different ways can be noticed.

The low values plotted in the figure by the continuous lines for GA7 and GA6 reflect the estimation of deformation modulus according to the guide book. Higher points represented by triangles are found to be the minimum acceptable values of deformation modulus.

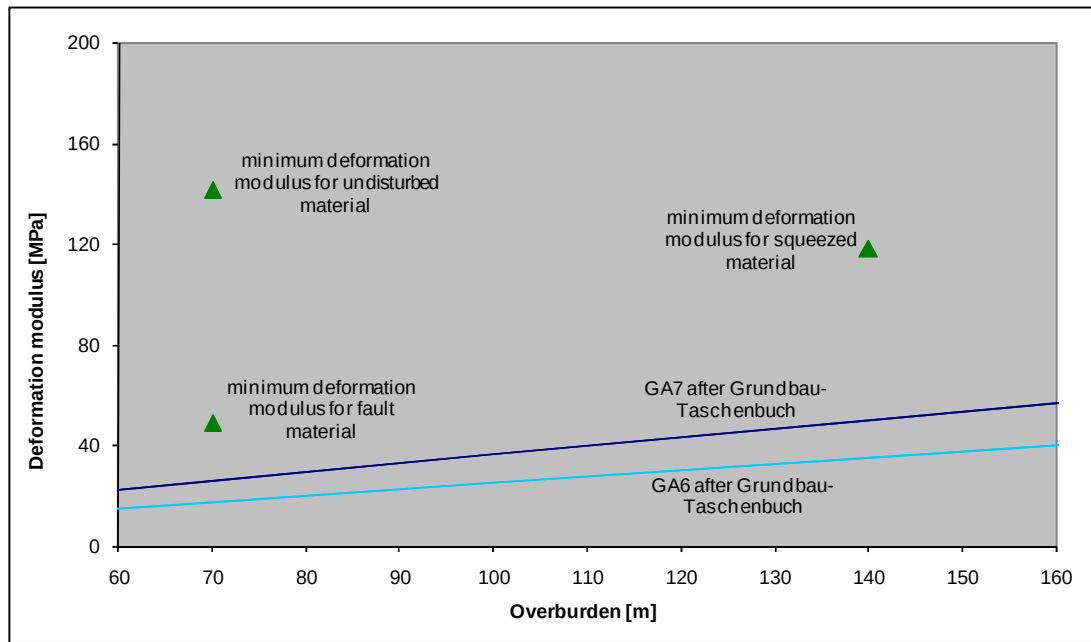


Fig.4.38 – Deformation modulus estimation for siltstone material

Friction angle estimation arising from the exposed methodology and according to the grain size distribution curve obtained from the laboratory tests in the prognosis phase is given as follows.

Table 4.12 – Summary of Siltstone zone friction angle estimation

		grain size distribution [%]		
particle size [mm]		Sample		
		1	2	3
A	< 0,002	5	20	32
B	0,002 - 0,01	50	45	44
C	0,01 - 0,2	40	30	20
D	> 0,2	5	5	4
estimated ϕ [°]		26	24	22

4.4.2. COMPARISON BETWEEN CALCULATED VALUES AND PROGNOSIS

Back calculated values of the siltstone material in the different cross sections studied shown that higher cohesion values are expected when compared with the predicted values. The following figures show the different comparisons made for the different siltstone cross sections.

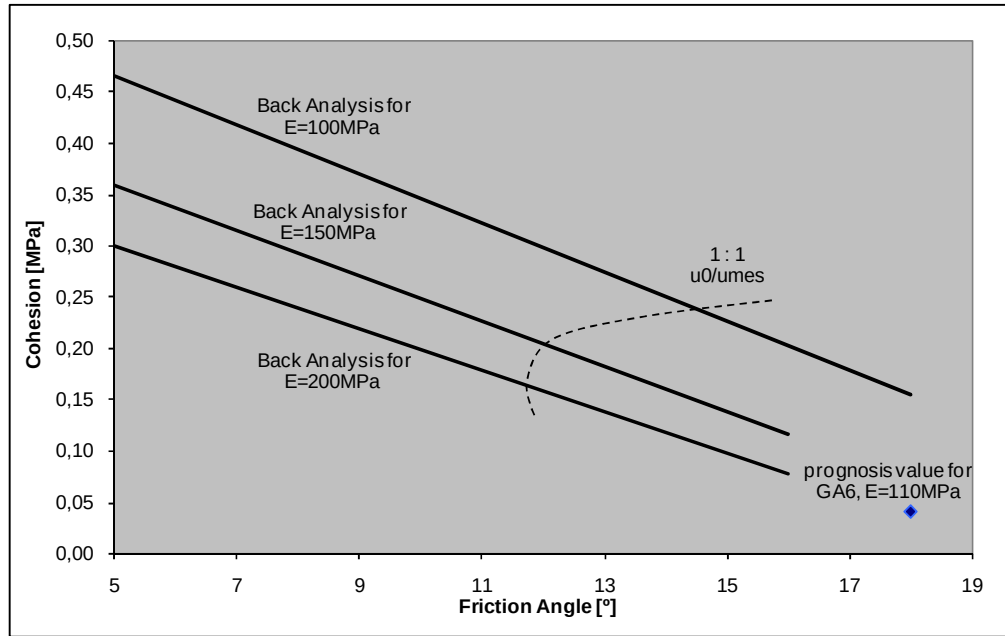


Fig.4.39 – Comparison between prognosis and calculated values for the fault siltstone chainage 2+120

Chainage 2+120, represented in Fig.4.39 where fault siltstone material was found reveals back calculated values with higher cohesion but lower friction angle. The same behaviour is observed in the slightly squeezed siltstone chainage 0+250 shown in Fig.4.40.

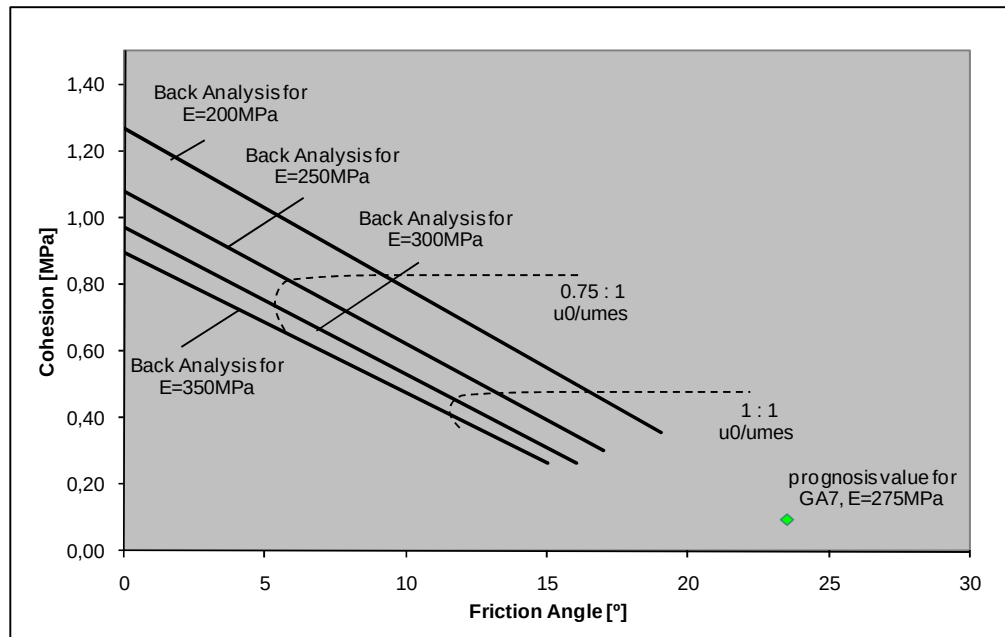


Fig.4.40 – Comparison between prognosis and calculated values for the squeezing siltstone chainage 0+250

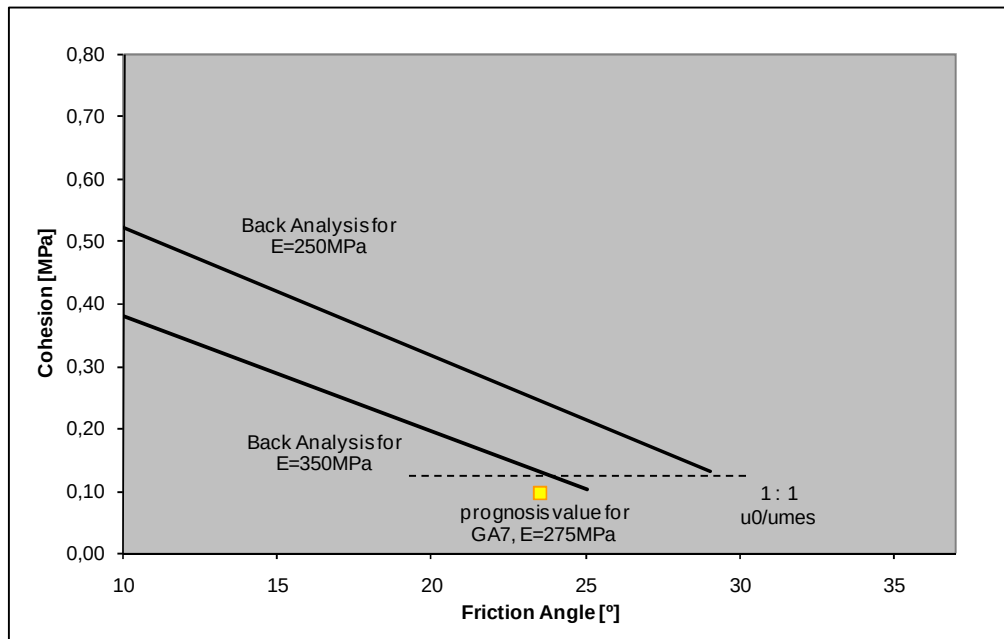


Fig.4.41 – Comparison between prognosis and calculated values for the undisturbed siltstone chainage 2+062

Chainage 2+062, as it can be seen in Fig.4.41, shows good association between the geotechnical predicted values and the back analysis making the exception of the siltstone cross sections.

The comparison between the values show that back calculated values reveal lower friction angles but higher cohesion values while deformation modulus are approximately in the same range. This behaviour is observed mainly in the disturbed siltstone cross sections (fault and slight squeezing) where if high stiffness is observed then low friction angle values are to be expected and vice-versa (Figs. 4.39 and 4.40).

It is worth to notice that none of the other cross sections analysed (sand, gravel and cataclastic rock) shown lower back calculated values of friction upon back calculation with the convergence confinement when comparing to the geotechnical prediction. This behaviour makes the siltstone material an exception in the general pattern followed.

4.4.3. ESTIMATED RANGE OF VALUES FROM THE BACK ANALYSIS OF THE SILTSTONE CROSS SECTIONS

According to the reference discussed, friction angle values are plausible between 22° and 26° (table 4.12). However, according to the convergence confinement method discussed before, these reference values are not achievable when dealing with the fault zone (Mitterpichling chainage 2+120). This fact leads to the definition of different ranges of possible values for the siltstone according to the in-situ conditions found.

Mitterpichling 2+120 fault chainage plausible values are given by the following ranges and the diagram of Fig.4.42.

- Friction angle [12,15] (°);
- Cohesion [0.16,0.25] (MPa);
- Deformation modulus [100,200] (MPa).

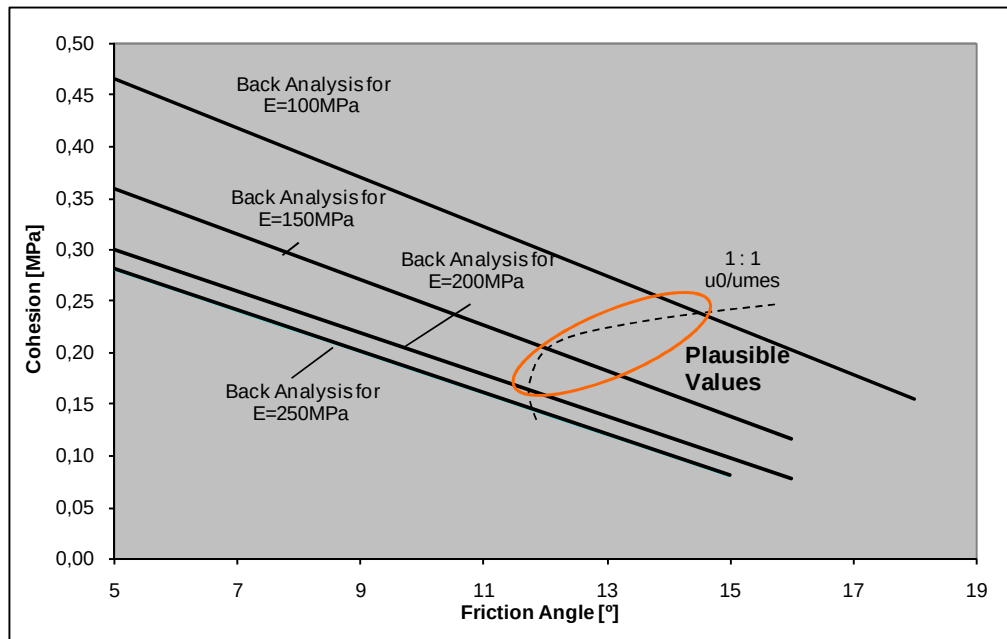


Fig.4.42 – Summary diagram of expected values for fault siltstone cross section 2+120

The undisturbed siltstone chainage 2+062 is expected to have values of strength parameters as shown in the following ranges and Fig.4.43.

- Friction angle [24,29] (°);
- Cohesion [0.10,0.20] (MPa);
- Deformation modulus [250,350] (MPa).

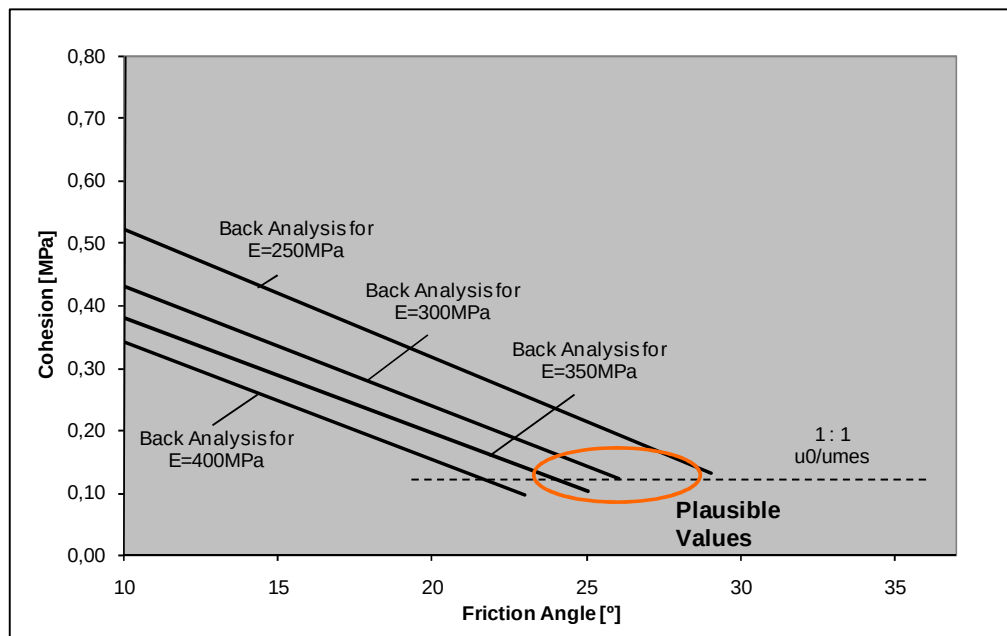


Fig.4.43 – Summary diagram of expected values for undisturbed siltstone cross section 2+062

The slightly squeezed siltstone chainage 0+250 despite the same rock mass composition (GA7) as chainage 2+062, is expected to have lower values of friction angle and deformation modulus but higher values of cohesion as shown in Fig.4.44.

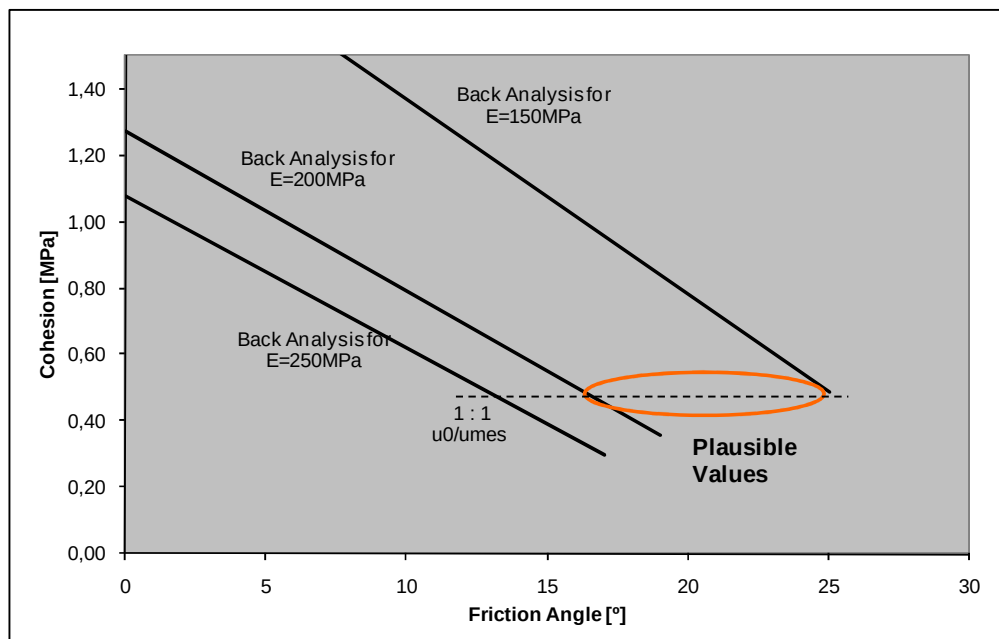


Fig.4.44 – Summary diagram of expected values for slight squeezed siltstone cross section 0+250

Ranges of values arising from the previous figure are:

- Friction angle [17,25] (°);
- Cohesion [0.40,0.60] (MPa);
- Deformation modulus [150,200] (MPa).

4.4.4. NUMERICAL ANALYSIS SOLUTION

For the different siltstone cross sections analysed numerical calculations with the FLAC v5.0 are run. The numerical model used for assessment of the reliability of the back analysed results is the same in its basic form (i.e. the same model as for sand and gravel) with the exception of the strength parameters of the rock mass and the rockbolt simulation.

The model simulates all the excavation stages as well as the pre-displacements occurring due to the face advance of the tunnel. The first step corresponds to the relaxation of the rock mass strength parameters which allows deformation to take place before excavation giving the expected pre displacement. The next stage corresponds to the excavation of the top heading and bench and lining installation on these segments. The shotcrete is simulated by structural beam elements with strength properties corresponding to those used on the construction. Since contribution for the internal support pressure given by the rockbolts it is not considerable high in the undisturbed chainage 2+062 (table 4.11), rockbolting have been only simulated in the two other cross sections. This can be defined as a stiffer area where the confining pressure given by the rockbolts act as a cohesion increase as explained by Wullschläger (1988).

Finally excavation of the temporary invert and shotcrete application takes place. After simulation it is possible to plot displacements at the crown position along the time allowing this way the understanding of settlements evolution.

Rock mass strength parameters used in the calculation steps are given by the possible combinations of Figs.4.42, 4.43 and 4.44 and summarized in table 4.13.

Table 4.13 – Numerical analysis input data for Siltstone

Numerical analysis input data for siltstone cross sections			
	CS 2+062	CS 2+120	CS 0+250
Deformation modulus [MPa]	300	150	175
Friction angle [°]	26	12	23
Cohesion [MPa]	0,15	0,20	0,46
Overburden [m]	70	70	140
Unit weight [kN/m ³]	21	21	21

4.4.4.1. Mitterpichling undisturbed cross section 2+062

The final displacement calculated numerically reached approximately 47mm corresponding to 22mm of pre-displacement and 25mm of displacement behind the face. These values show good agreement with the surveyed data revealing the predicted ratio of 1:1 to be a reasonable estimation. These conclusions arise from the final displacements shown in Fig.4.45 and the evolution of crown displacements plotted in Fig.4.46.

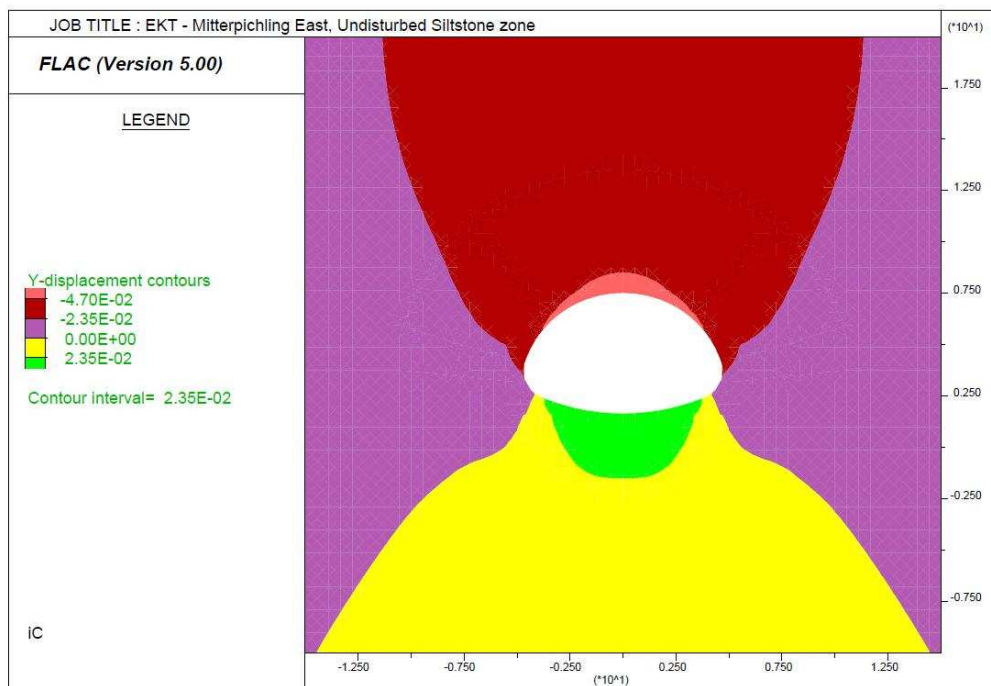


Fig.4.45 – Numerical calculation of undisturbed Siltstone cross section – Final displacements

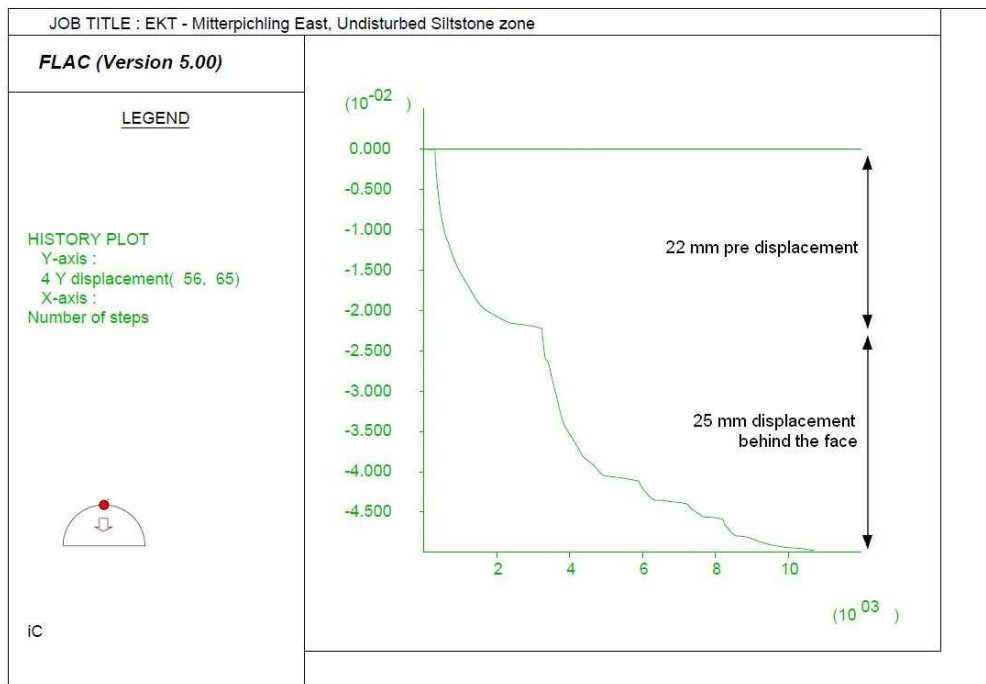


Fig.4.46 – Numerical calculation of undisturbed Siltstone cross section – Evolution of displacements

4.4.4.2. Mitterpichling fault cross section 2+120

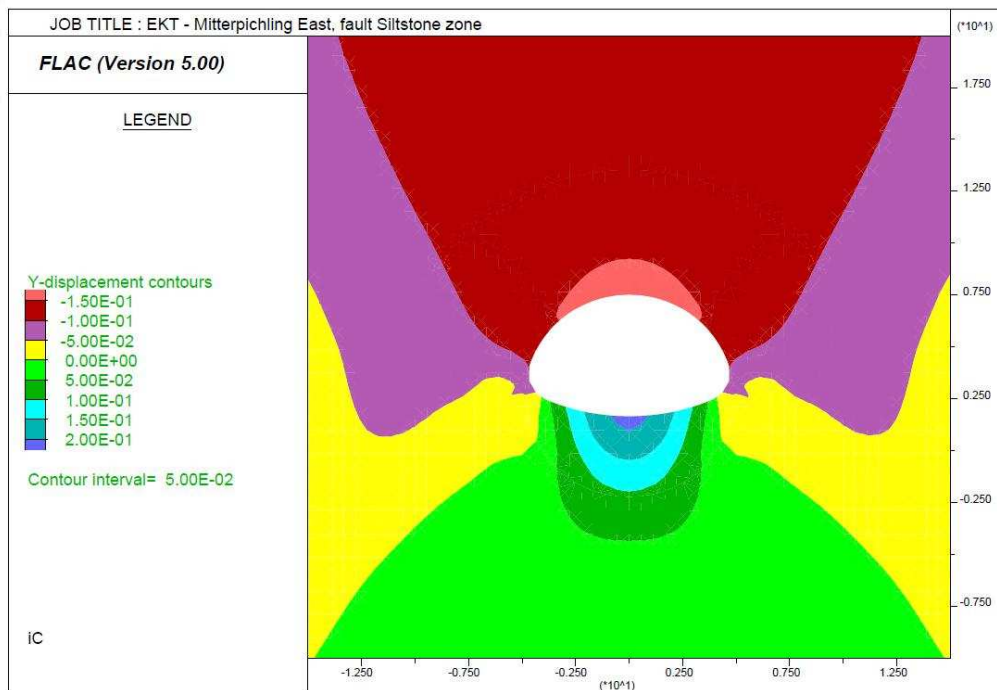


Fig.4.47 – Numerical calculation of fault Siltstone cross section – Final displacements

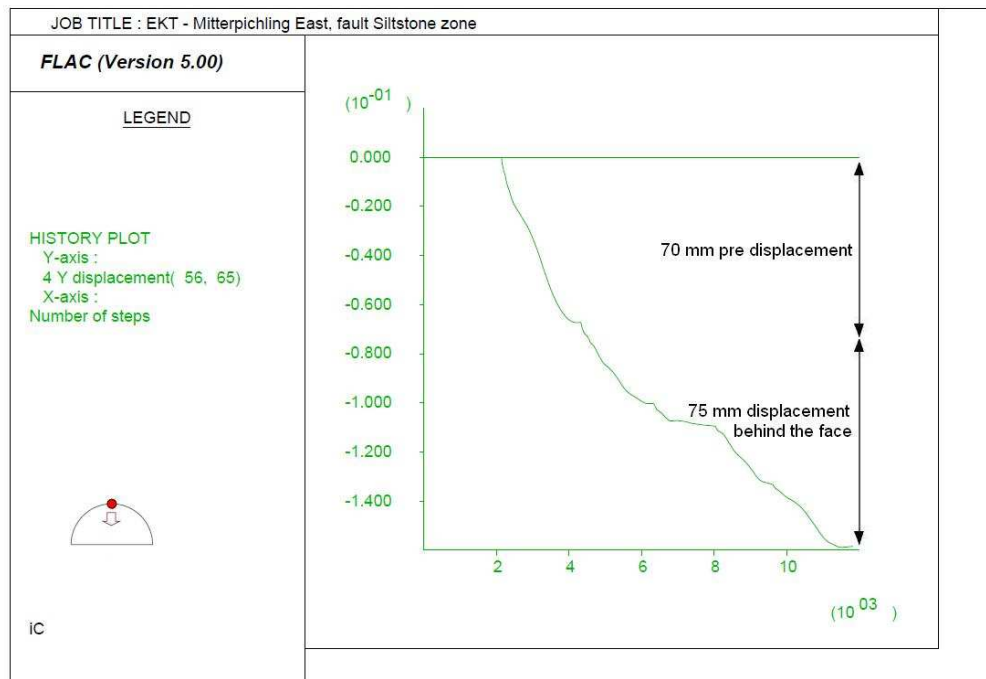


Fig.4.48 – Numerical calculation of fault Siltstone cross section – Evolution of displacements

From Figs.4.47 and 4.48 it is possible to observe the expected pre-displacement and the displacement behind the face to be approximately 70mm and 75mm respectively, giving a total displacement of 145mm. These values show that the numerical model verifies the back calculated values and the expected behaviour of pre-displacement to measured displacement (u_0/u_{mes}).

4.4.4.3. Paierdorf slightly squeezed cross section 0+250

Using the numerical model described previously (section 4.6.4) and applying the strength parameters specified in table 4.13 it is possible to reach the results displayed in Figs.4.49 and 4.50 for the slightly squeezed Paierdorf cross section. Again both pre-displacement and displacement behind the face are confirmed with the numerical tool.

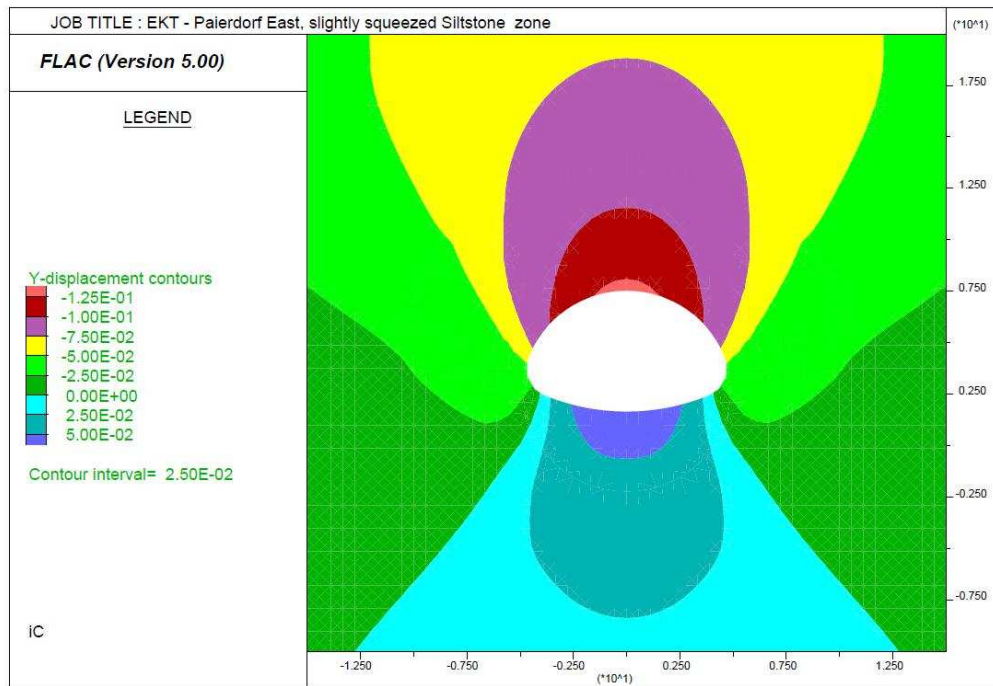


Fig.4.49 – Numerical calculation of slight squeezed Siltstone cross section – Final displacements

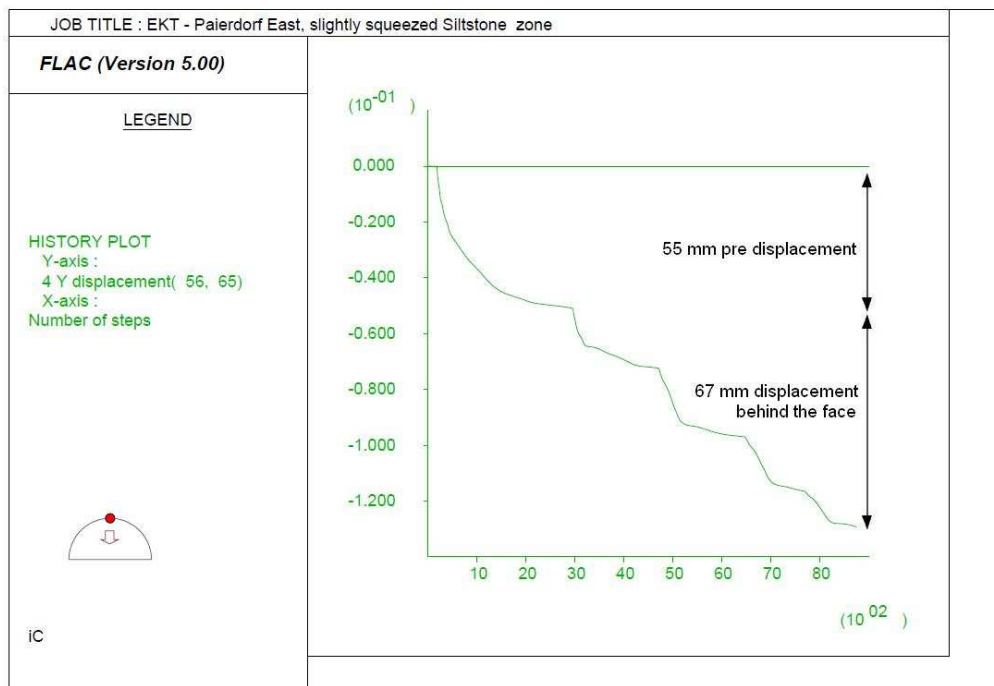


Fig.4.50 – Numerical calculation of slight squeezed Siltstone cross section – Evolution of displacements

Generally it is observable that numerical analyses displacements are in accordance with the back calculation method proposed. This means that the back analysis proposed method can be verified and the results will be similar to those arising from more complex and time consuming models.

CONCLUSIONS

5.1. GENERAL IDEA

Construction of the Koralm investigation tunnels allowed the development of a back analysis method to assess the rock mass strength parameters based on the convergence confinement theory and the measured data gathered from the work site. The most relevant geological conditions found during excavation (i.e. cataclastic rock, sand, gravel and siltstone) were analysed and engineering values for the rock mass parameters presented according to the back calculation methodology developed.

From the gathered results a main general idea is kept. Prognosis values of deformation modulus and cohesion are low when compared to the back calculated results. This is clearly seen in the comparison diagram shown in Fig.5.1 which reveals lower prognosis values of average stiffness in most of the studied cases. Minimum purely elastic deformation moduli calculated give the lower value achievable with the convergence confinement theory. For this reason it is plausible to accept higher values since rock masses have been submitted to yield phenomena and consequently plastic deformations.

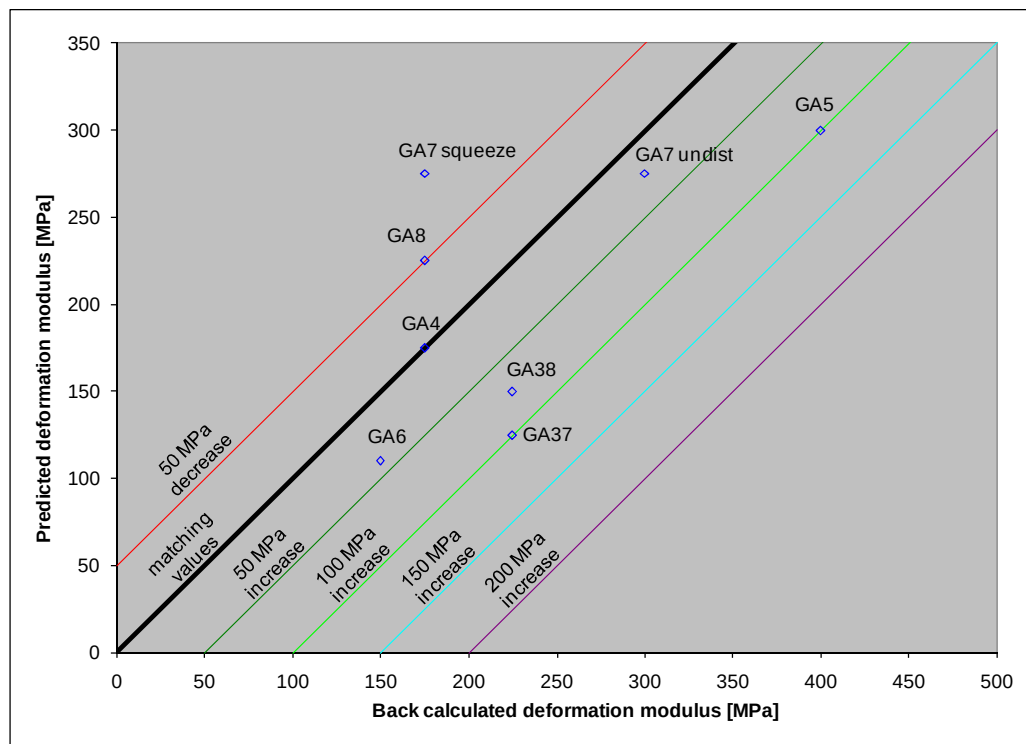


Fig.5.1 – Comparison between prognosis and back calculated deformation modulus values

Estimation of the friction angle based on the grain size distribution curves proposed by Lang et al (2007) gives a realistic approximation. However, beside the Lavanttal fault zone where laboratory tests were performed during the construction of the tunnel, the grain size distribution curves were not provided from the tunnelling works. This means that grain size distribution curves from the preliminary geological reports have been used, giving a characterization of the material in a large array of possible values. With it some inaccuracy was added to the results. Despite this, results found for each of the rock masses studied were a reliable source for narrowing the range of possible values. It is therefore possible to state that, when available, grain size distribution curves can be a good source to estimate the friction angle. Comparison of prognosis and back calculated values shown that generally higher friction angles are to be expected. The exception to this trend is revealed by the siltstone materials found in the disturbed chainages of Paierdorf 0+250 (GA7 squeeze) and Mitterpichling 2+120 (GA6) as can be seen in Fig.5.2.

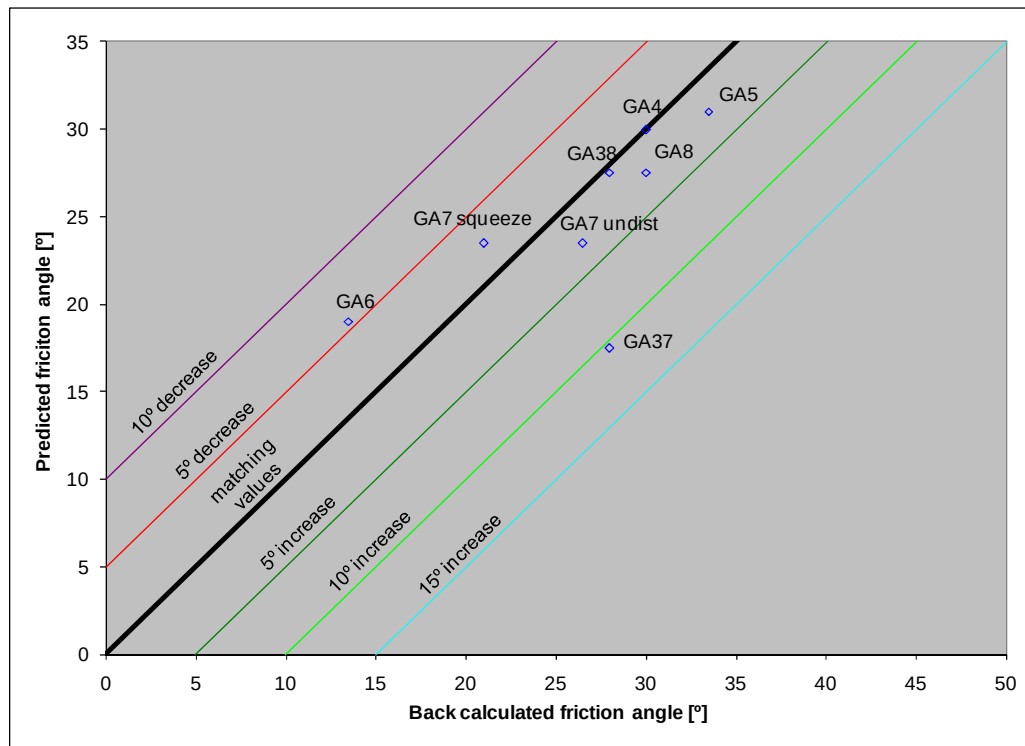


Fig.5.2 – Comparison between prognosis and back calculated friction angle values

Cohesion values revealed the higher differences between prognosis and back calculations. This can be seen in Fig.5.3 where back calculated values are in average 6 times greater than prognosis.

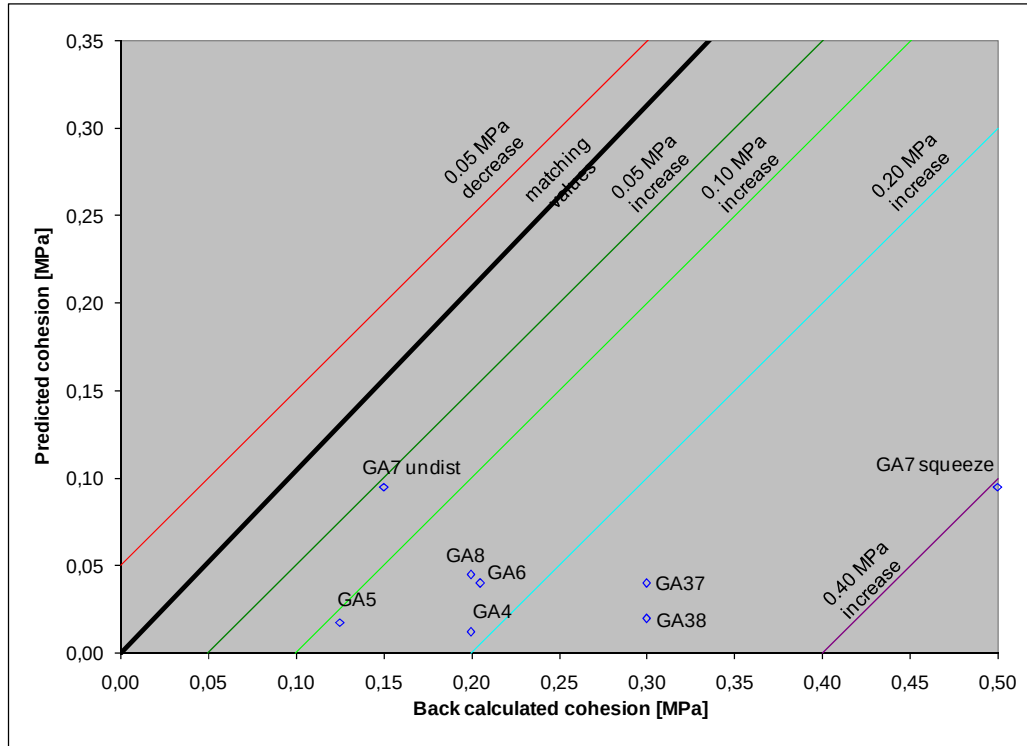


Fig.5.3 – Comparison between prognosis and back calculated cohesion values

5.2. MAJOR LIMITATIONS

The methodology relies on a model that can only be an idealization of the real interaction between the rock mass and the support. Therefore many simplifications influence the final results obtained. Reference to the most significant assumptions is presented.

The unit weight of the different materials is an important input factor because it is directly linked with the in-situ pressure which influences the Ground Reaction Curve. This value should be accessed from laboratory tests of the material of the studied cross section which, in this case, were not available. For this reason the unit weight was estimated from tables, charts and geological reports from the work site leading to some uncertainties and certainly inaccurate results.

The internal support pressure p_i , used in the back calculations with the convergence confinement method, was estimated from the shotcrete stress analysis and rockbolts installed. Once the rockbolts are not equally spaced and the shotcrete is not structured in a close ring on reality this assumption was made to enable the definition of an equal pressure around the tunnel perimeter. This hypothetic way of defining the internal support pressure brings errors to the close mathematical formulation used.

Also, the convergence confinement model is based on a circular shape tunnel which in this case where top heading and invert excavation were installed is not real. These geometric simplifications made for suiting the reality into the model have major impact in the support characteristic curve and again in the final results gathered.

5.3. SPECIFIC CROSS SECTIONS ANALYSIS

Regarding the core of the Lavanttal fault zone studied it was possible to demonstrate that starting from the prognosis parameters more accurate design parameters can be assessed. The rock mass found is mainly a mixture of coarse and fine grained cataclastic rock which, considering Medley's (1994) approach, can be classified as a BIM-rock. For this reason an increase in the friction angle of the matrix is proposed regarding the volumetric proportion found. Together with the deformation modulus estimation proposed by Habimana et al (2002), for highly tectonised rock masses, reliable values were found. These values are an update to the prognosis which compared with the back calculations shown good agreement. For this reason it was possible to observe that the BIM-rock theory is a reliable source when dealing with this type of material.

Gravel and Sand cross sections studied showed that high friction angles as well as high deformation modulus are to be expected as plausible. As a general trend, the stiffness of the rock mass in tunnel depth is high.

Siltstone material is found to be one of the main components of the investigation tunnels. Therefore, the study tried to cover a wide range of situations with faulted, undisturbed and slightly squeezing siltstone cross sections being investigated. Friction angle assessment revealed that similar values were given by the prognosis and the back calculations in Mitterpichling undisturbed cross section (2+062) and slightly squeezing Paierdorf East (0+250). At Mitterpichling fault section (2+120) the convergence confinement back analysis suggests lower values of friction angle to be plausible results. These values are relatively lower than the predicted effective values from the prognosis. However, back calculations showed also that cohesion is expected to be higher than prognosis. This fact leads to a certain "balance", being the uniaxial compressive strength kept in the same level either by higher friction and lower cohesion or vice-versa. Back calculated ranges of deformation modulus showed that a slightly stiffer model should be considered in Mitterpichling's undisturbed (2+062) and fault (2+120) cross sections. At Paierdorf cross section (0+250) good agreement with the prognosis was achieved.

5.4. CONCLUSIONS

The back-analysis tool developed in this research provides a fast field analysis to assess a plausible range of rock mass parameters based on high-level monitoring data. An estimate of pre-displacements and level of lining loads is required as additional input.

The simple analytical model is completely confirmed by the numerical FLAC studies in all the cases. Therefore, as a general approach the analytical model is sufficient and the limitations of the model are small compared to the uncertainties in the choice of parameters.

REFERENCES

- 3G, BGG (2003). *Koralmbahn Graz – Klagenfurt, abschnitt deutschladsberg – St. Andrä, Erkundungstunnel Paierdorf; Geologie, Geotechnik und Hydrogeologie*. 3G Gruppe Geotechnik Graz; BGG Baugrunderkung Geomechanik Geohydrologie. Personal communication.
- 3G, BGG (2007). *Koralmbahn Graz – Klagenfurt, abschnitt deutschladsberg – St. Andrä, Erkundungstunnel Paierdorf Vortrieb West; Baugeologische dokumentation*. 3G Gruppe Geotechnik Graz; BGG Baugrunderkung Geomechanik Geohydrologie. Personal communication.
- Blyth, F.G.H., Freitas, M.H. (1984). Metamorphic Rocks. In *A Geology for Engineers*, 133-143, The Bath Press, Avon.
- Carranza-Torres, C., Fairhurst, C., (2000). *Application of the convergence confinement method of tunnel design to rock masses that satisfy the Hoek–Brown failure criterion*. Tunnelling Underground Space Technology 15 (2), 187–213.
- Fasching, F. (2008). *Geological report Störungscharakteristik EKT-Paierdorf wsc*. 3G Gruppe Geotechnik Graz. Personal communication.
- Goodman, Richard E. (1993). Metamorphic Rocks. In *Engineering Geology*, 302-324, John Wiley & Sons, Inc., Singapore.
- Habimana, J., Labiouse, V., Descoeurdes, F. (2002). *Geomechanical characterization of cataclastic rocks: experience from the Cleuson – Dixence project*. International Journal of Rock Mechanics & Mining Sciences, 3 April 2002, 677-693, Elsevier Ltd.
- Hoek, E. (1999). *Support for very weak rock associated with faults and shear zones*. International Symposium on Rock Support and Reinforcement Practice in Mining, Kalgoorlie, Australia, 14-19 March, 1999.
- Hoek, E. (1999a). *A discussion on acceptability criteria for temporary support and final linings of large span transportation tunnels in poor rock*. 22 March 1999. Vancouver
- Hoek, E., Carranza-Torres, C., Diederichs, M., Corkum, B. (2008). *Kersten Lecture: Integration of geotechnical and structural design in tunnelling*. Proceedings University of Minnesota 56th Annual Geotechnical Engineering Conference. Minneapolis, 29 February 2008, 1-53
- Hoek, E., Marinos, P. (2007). *A brief history of the development of the Hoek-Brown failure criterion*. Soils and Rocks, No. 2. November 2007, 1-13
- Hölzl, H. (2009). Geoconsult Consulting Engineers. Personal communication.
- Hunt, R.E. (1984). Measurements of properties. In *Geotechnical engineering investigation manual*, 195, McGraw-Hill, Inc., USA.

Lang, H.J., Huder, J., Amann, P., Puzrin, A.M. (2007). Festigkeitseigenschaften der Böden. In *Bodenmechanik und Grundbau*, 86, Springer, Leipzig.

Lindquist, E.S. (1994). *The mechanical properties of a physical model melange*. Proceedings of the 7th International Congress of the International Association of Engineering Geology, Lisboa, Portugal, 5-9 September, 1994, 819-826, Oliveira, R. et al. (eds), Balkema, Rotterdam.

Lindquist, E.S., Goodman, R.E. (1994). *Strength and deformation properties of a physical model melange*. Proceedings of the 1st North American Rock Mechanics Symposium, University of Texas, Austin, 1-3 June, 1994, 843-850, Nelson & Laubach (eds), Balkema, Rotterdam.

Medley, E. (1994). *Using stereological methods to estimate the volumetric proportions of blocks in melanges and similar block-in-matrix rocks (bimrocks)*. Proceedings of the 7th International Congress of the International Association of Engineering Geology, Lisboa, Portugal, 5-9 September, 1994, 1031-1039, Oliveira, R. et al. (eds), Balkema, Rotterdam.

Medley, E. (1997). *Uncertainty in estimates of block volumetric proportions in melange bimrocks*. Proceedings of the International Symposium on Engineering Geology and the Environment, Athens, Greece, 23-27 June, 1997, 267-272, Marinos P.G. et al. (eds), Balkema, Rotterdam.

Medley, E. (1998). *Confronting geological complexity: Working with melange bimrocks*. Proceedings of the 8th International Congress of the International Association for Engineering Geology and the Environment, Vancouver, Canada, 21-25 September, 1998, 227-233, Moore, D. et al. (eds), Balkema, Rotterdam.

Medley, E. (1999). *Systematic characterization of melange bimrocks and other chaotic soil/rock mixtures*. In *Rock and Soil Engineering*, May 2009, 152-162, Verlag Glückauf GmbH.

Medley, E., Goodman, R.E. (1994). *Estimating the block volumetric proportions of melanges and similar block-in-matrix rocks (bimrocks)*. Proceedings of the 1st North American Rock Mechanics Symposium, University of Texas, Austin, 1-3 June, 1994, 851-858, Nelson & Laubach (eds), Balkema, Rotterdam.

Moritz, B., Goldberger, H., Schubert, P. (2006). *Application of the observational method in heterogeneous rock mass with low overburden*. Felsbau 24-1, 62-72, Austrian Society for Geomechanics, Verlag Glückauf GmbH, Essen.

Nicholson, D., Tse, C.M., Penny, C. (1999). *The Observational Method in ground engineering: principles and applications*, Report 185, CIRIA, London.

Nossan, A.S. (2006). *Observations on the Observational Method*. Active Geotechnical Design in Infrastructure Development, Slovenian Geotechnical Society, 29-31 May 2006, 171-178, Logar, J., Gaberc, A., Majes, B. (eds.), Ljubljana, Slovenia.

Oreste, P. (2004). *Back-analysis techniques for the improvement of the understanding of rock in underground constructions*. Tunnelling and Underground Space Technology 20, 7-21, Elsevier Ltd.

- Patel, D., Nicholson, D., Huybrechts, N., Maertens, J. (200x). *The Observational Method in Geotechnics*. (www.arup.com/assets/download/41A6ACB0-DD6E-06AC-7C03C57BF321BA5.pdf) 14.04.2009.
- Peck, R.B. (1969). *Advantages and limitations of the observational method in applied soil mechanics*. *Geotechnique*, 19 (2), 171-187.
- Rocscience, Inc. (2005). *RocSupport – Rock support interaction and deformation analysis for tunnels in weak rock – Tutorial Manual*, 1- 73
- Schubert, P. (2009). iC consulenten. Personal communication.
- Sellner P.J. (2000). *Prediction of displacements in tunnelling*. Ph.D. dissertation, Department of Civil Engineering, Graz University of Technology.
- Terzaghi, K., Peck, R.B. (1967). *Soil mechanics in Engineering Practice*, John Wiley, New York.
- TU, G. (2008). Technical University Graz. *Prüfprotokoll – Überprüfung Stauchelemente 2.Teil*. Personal communication.
- Unlu, T., Gercek, H. (2003). *Effect of Poisson's ratio on the normalized radial displacements occurring around the face of a circular tunnel*. *Tunnelling and Underground Space Technology* 18, 547–553
- Verman, M., Singh, B., Viladkar, M.N., Jethwa, J.L. (1997). *Effect of tunnel depth on modulus of deformation of rock mass*. *Rock Mechanics and Rock Engineering*, 30 (3), 121-127, Springer-Verlag.
- Witt, K.J. (2008). *Eigenschaften von Boden und Fels-ihre Ermittlung im Labor*. In *Grundbau-Taschenbuch, Geotechnische Grundlagen*, 124-125, Ernst & Sohn, Berlin.
- Wullschläger, D. (1988). *Ein verbundwerkstoffmodell für die systemankerung im tunnelbau*. Dissertation. Fakultät für Bauingenieur- und Vermessungswesen der Universität Fridericiana zu Karlsruhe.

A1

APPENDIX 1 TECHNICAL INFORMATION

A1.1. CHARACTERISTIC SOIL PARAMETERS

Fig.A.1.1 – Grundbau-Taschenbuch table

Spalte	a	b	c			d		e		f	g	h		i	
Zeile Nr.	Bodenart	Boden- Gruppe nach DIN 18 196	Korngrößen- verteilung	Ungleich- förmig- keitszahl	Plastizitätsgrenzen des Kornanteils < 0,4 mm			Wichte		Zusammendrück- barkeit erster- dichteter Böden $E_s = v_e \cdot \sigma_{at} \left(\frac{\sigma}{\sigma_{at}} \right)^{w_e}$	Δu	Scherparameter		Durch- lässig- keits- koeffi- zient k m/s	
					w _L %	w _P %	I _P %	γ kN/m ³	γ' kN/m ³			w	Q _{Pr}		w _{Pr}
1	Kies, gleichkörnig	GE	< 5	< 60	—	—	—	16,0 19,0	9,5 10,5	4 1	1,70 1,90	8 5	—	32 35	2,10 ⁻¹ 1,10 ⁻²
2	Kies, sandig, mit wenig Feinkorn	GW, GI	< 5	< 60	—	—	—	21,0 23,0	11,5 13,5	6 3	2,00 2,25	7 4	—	32 35	1,10 ⁻² 1,10 ⁻⁶
3	Kies, sandig, mit Schluff- oder Tonbeimengungen, die das Korn- gerüst nicht sprengen	GÜ, GT	8 15	< 60	20 45	16 25	4 25	21,0 24,0	11,5 14,5	9 3	2,10 2,35	7 4	0,7 0,5	32 35	1,10 ⁻⁵ 1,10 ⁻⁸
4	Kies-Sand-Feinkornmisch; das Feinkorn sprengt das Korngerüst	GU, GT	20 40	< 60	20 50	16 25	4 30	20,0 22,5	10,5 13,0	13 6	1,90 2,20	10 5	0,9 0,7	22 30	1,10 ⁻⁷ 1,10 ⁻¹¹
5	Sand, a) Feinsand gleichkörnig b) Grobsand	SE	< 5	100	—	—	—	16,0 19,0	9,5 11,0	22 8	1,60 1,75	15 10	0,75 0,60	30 32	1,10 ⁻⁴ 2,10 ⁻⁵
		SE	< 5	100	—	—	—	16,0 19,0	9,5 11,0	16 6	1,60 1,75	13 81	0,70 0,55	30 34	1,10 ⁻³ 5,10 ⁻⁴
6	Sand, gut abgestuft und Sand, kiesig	SW, SI	< 5	> 60	—	—	—	18,0 21,0	10,0 12,0	12 5	1,90 2,15	10 6	0,70 0,55	33 41	5,10 ⁻⁴ 2,10 ⁻⁵
7	Sand mit Feinkorn, das das Korngerüst nicht sprengt	SU, ST	8 15	> 60	20 45	16 25	4 25	19,0 22,5	10,5 13,0	15 4	2,00 2,20	11 7	0,80 0,65	30 32	2,10 ⁻⁵ 5,10 ⁻⁷
8	Sand mit Feinkorn, das das Korngerüst sprengt	SÜ, ST	20 40	> 60	30 50	20 50	16 30	18,0 21,5	9,0 11,0	20 8	1,70 2,00	19 12	0,90 0,75	25 32	2,10 ⁻⁶ 1,10 ⁻⁹
9	Schluff, leicht plastisch	UL	> 50	> 80	5 50	25 35	21 28	17,5 21,0	9,5 11,0	28 15	1,60 1,80	22 15	0,80 0,60	25 30	1,10 ⁻⁵ 1,10 ⁻⁷
10	Schluff, mittel- und ausgeprägt plastisch	UM, UA	> 80	100	5 50	35 60	22 25	17,0 20,0	8,5 10,5	35 20	1,55 1,75	24 18	0,90 0,70	22 29	2,10 ⁻⁶ 1,10 ⁻⁹
11	Ton, leicht plastisch	TL	> 80	100	6 20	25 35	15 22	19,0 22,0	9,5 12,0	28 14	1,65 1,85	20 15	1,00 0,90	24 32	1,10 ⁻⁷ 2,10 ⁻⁹
12	Ton, mittelpastisch	TM	> 90	100	5 40	40 50	18 25	18,0 21,0	8,5 11,0	38 18	1,55 1,75	23 17	1,00 0,95	10 20	5,10 ⁻⁸ 1,10 ⁻¹⁰
13	Ton, ausgeprägt plastisch	TA	100	100	60 85	20 35	25 55	16,5 20,0	7,0 10,0	55 20	1,45 1,65	27 20	1,00 0,90	6 15	1,10 ⁻⁹ 1,10 ⁻¹²
14	Schluff oder Ton, organisch	OU, OT	> 80	100	5 30	45 70	30 45	15,5 18,5	5,5 8,5	60 26	1,45 1,70	27 18	1,00 0,90	15 22	1,10 ⁻⁹ 2,10 ⁻¹¹
15	Torf	HN, HZ	—	—	—	—	—	10,4 12,5	0,4 2,5	800 80	—	—	1,00 1,00	24 30	1,10 ⁻⁵ 1,10 ⁻⁸
16	Mudde	F	—	—	100 250	30 80	50 170	12,5 16,0	2,5 6,0	160 50	—	—	1,00 0,90	18 26	1,10 ⁻⁷ 1,10 ⁻⁹

Source: Witt (2008), Grundbau-Taschenbuch

A2

APPENDIX 2 TUNNEL GEOLOGICAL AND GEOTECHNICAL DATA

A2.1. ROCK MASS TYPES DESCRIPTION USED ON THE STUDY (GEBIRGSART)

Fig.A.2.1 – Rock mass type GA4

HL-AG: Koralmbahn Graz - Klagenfurt,
Abschnitt Deutschlandsberg - St. Andrä,
Erkundungstunnel Palterdorf

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Ausschreibungsplanung
September 2003

(b) Gebirgsart 4

GA 4	
Lithologie (Hauptgesteinsart)	"Sand"
Korngröße	sanddominiert, z.T. schluffig und untergeordnet kiesig und blockig
Verhältnis Matrix / Komponenten	nicht anwendbar
Zementation	keine
Quellfähige Tonminerale	nicht signifikant
Quelleigenschaften	nicht signifikant
Festigkeitseigenschaften	keine signifikanten tiefenabhängigen Verände- rungen; wesentliche Beeinflussung durch Wasserzutritte
Beschreibung	Mittelkörniges Lockergestein mit lokalen Übergän- gen zu gering zementiertem Sandstein (s. GA 8). Im Kontaktbereich zum kristallinen Grundgebirge können verstärkt Komponenten von Kies- bis Blockgröße angetroffen werden; dichte bis sehr dichte Lagerung; vielfach hohe Glimmergehalte

3G, Graz
BGG, Wolfsberg

KB.VB16.BGG/3G.X.218.0

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Gebirgskennwerte bzw. Parameter		Einheit	Größe / Einstufung	Bemerkung
Reibungswinkel	φ	Grad	28 bis 32	-
Kohäsion	c	kN/m ²	5 bis 20	-
Elastizitätsmodul	E	MN/m ²	100 bis 300	-
Gebirgs- durchlässigkeit	-	-	sehr schwach durchläs- sig bis durchlässig	-
Quellpotential	k	m/s	5,0·10 ⁻⁹ bis 5,0·10 ⁻⁸	-
Injizierbarkeit	-	-	nicht quellfähig	-
Wasserempfindlichkeit	-	-	gering wirksam	Festbinde- mittel, Geste
Erweichbarkeit	-	-	stark veränderlich	-
Lösbarkeit	-	-	stark erweichbar	-
			Tunnelbagger	-

3G, Graz
BGG, Wolfsberg

KB.VB16.BGG/3G.X.218.0

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Fig.A.2.3 – Rock mass type GA6

HL-AG: Koralmbahn Graz - Klagenfurt,
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(d) Gebirgsart 6

GA 6	
Lithologie (Hauptgesteinsart)	Kataklastit (Tertiär)
Korngröße	in Abhängigkeit vom Ausgangsgestein fein- bis grob- oder gemischtkörnig
Verhältnis Matrix / Komponenten	korn- und matrixgestützte Typen mit z.T. fließenden Übergängen
Zementation	überwiegend fehlend, in feinkörnigen Festgesteinen geringe, überwiegend tonige Zementation
Quellfähige Tonminerale	z.T. hohe Gehalte möglich, jedoch konzentriert auf Zonen beschränkter Mächtigkeit
Quelleigenschaften	z.T. signifikantes Quellpotential vorhanden
Festigkeitseigenschaften	keine signifikanten tiefenabhängigen Veränderungen; wesentliche Beeinflussung durch Wasserzutritte
Beschreibung	variable Korngrößenverteilungen, z.T. mit Komponenten des Ausgangsgesteins; in feinkörnigen Festgesteinen hohe Trennflächendichte möglich; reduzierte Lagerungsdichte in Lockergesteinen

3G, Graz
BGG, Wollsegg

KB.VB16.BGG/3G.X.218.0

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3G, Graz
BGG, Wollsegg

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Gebirgskennwerte bzw. Parameter		Einheit	Größe / Einstufung	Bemerkung
Reibungswinkel	φ	Grad	18 bis 20	–
Kohäsion	c	kN/m ²	30 bis 50	–
Elastizitätsmodul	E	MN/m ²	70 bis 150	–
Gebirgsdurchlässigkeit	–	–	sehr schwach durchlässig	–
	k	m/s	< 10 ⁻⁶	–
Quellpotential	–	–	gering	z.T. hoch
Injizierbarkeit	–	–	nicht wirksam	–
Wasserempfindlichkeit	–	–	stark veränderlich	–
Erweichbarkeit	–	–	stark erweichbar	–
Lösbarkeit	–	–	Tunneltagger	–

3G, Graz
BGG, Wollsegg

KB.VB16.BGG/3G.X.218.0

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Geigekennwerte bzw. Parameter	Einheit	GröÙe / Einstufung	Bemerkung
Druckfestigkeit	N/mm²	0,3 bis 0,5	
Reibungswinkel	Grad	22 bis 25	-
Kohäsion	kN/m²	70 bis 120	-
Elastizitätsmodul	N/mm²	200 bis 350	-
Gebigs- durchlässigkeit	-	sehr schwach bis schwach durchlässig	-
k	m/s	10 ⁻² bis 10 ⁻⁶	-
Quellpotential	-	mittel	untergeordnet hoch
Injizierbarkeit	-	gering wirksam	Feinst- bindemittel, Gele
Wasserempfindlichkeit	-	veränderlich bis stark veränderlich	-
Erweichbarkeit	-	stark erweichbar	-
Lösbarkeit	-	Tunnelbagger	bereichsweise Bagger + LSHM oder TSM

GA 7	
Lithologie (Hauptgesteinsart)	Schluff- / Tonstein, gering zementiert
Korngröße	schluffdominiert, z.T. tonig, sandig, lokal Kieskomponenten
Verhältnis Matrix / Komponenten	nicht anwendbar
Zementation	geringe, überwiegend tonige, untergeordnet karbonatische Zementation; bei karbonatischer Zementation lokal erhöhter Zementationsgrad möglich
Quellfähige Tonminerale	im Allgemeinen geringe Smektitgehalte vorhanden, lokal höhere Smektitgehalte möglich
Quelleigenschaften	im Allgemeinen geringes bis mäßig hohes Quellpotential vorhanden, hohe Quellpotentiale lokal möglich
Festigkeitseigenschaften	keine signifikanten tiefenabhängigen Veränderungen; wesentliche Beeinflussung durch Wasserzutritte; lokal variable Festigkeitseigenschaften aufgrund unterschiedlicher Zementationsmittel (tonig bzw. karbonatisch) sowie unterschiedlicher Zementationsgrade
Beschreibung	massiges bis geschichtetes feinkörniges Festgestein, sehr mürbe bis mürbe, örtlich mäßig hart bis hart; mechanisch relevante Trennflächen nur vereinzelt in Form von Schicht-, Kluff- und Störungslächen ausgebildet; z.T. hoher Glimmergehalt

Fig.A.2.5 – Rock mass type GA8

HL-AG: Koralmbahn Graz - Klagenfurt,
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September 2003

Gebirgskennwerte bzw. Parameter		Einheit	Größe / Einstufung	Bemerkung
Druckfestigkeit	σ	MN/m ²	0,2 bis 0,4	
Reibungswinkel	φ	Grad	25 bis 30	–
Kohäsion	c	kN/m ²	30 bis 60	–
Elastizitätsmodul	E	MN/m ²	150 bis 300	–
Gebirgs- durchlässigkeit	–	–	schwach durchlässig	–
	k	m/s	10 ⁻⁷ bis 10 ⁻⁶	örtlich bis 5,0·10 ⁻⁶ m/s
Quellpotential	–	–	nicht quellfähig	–
injizierbarkeit	–	–	gering wirksam	Feinst- bindemittel, Gele
Wasserempfindlichkeit	–	–	stark veränderlich	–
Erweichbarkeit	–	–	stark erweichbar	–
Losbarkeit	–	–	Tunnelbagger	bereichsweise Bagger + LS/HM oder TSM

3G, Graz
BOG, Wolfsberg

KS_VB16_SGG/SG X 216.0

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Fig.A.2.6 – Rock mass type GA37

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September 2003

Parameter	Einheit	Größe / Einstufung	Bemerkung
Gebirgsdurchlässigkeit	-	sehr schwach durchlässig	-
Queltpotential	-	nicht bis gering quellfähig	-
Injizierbarkeit	-	nicht bis gering wirksam	-
Wasserempfindlichkeit	-	nicht bis mäßig veränderlich	-
Erweichbarkeit	-	gering	-
Lösbarkeit	-	Tunnelbagger	-
Abrasivität	-	-	-

HL-AG, Koralmbahn Graz - Klagenfurt,
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(v) Gebirgsart 37

GA 37			
Lithologie (Hauptgesteinsart)	Kataklasit		
Tropie	isotrop bis anisotrop		
Zerlegung	0,006 bis 6 cm		
Verwitterung			
Verkarstung	nicht anwendbar		
Gesteinskennwerte			
UCS [MPa]			
m _i [-]			
c [MPa]			
φ [°]			
E _i [MPa]			
ν [-]			
CAI [-]			
Trennflächenkennwerte	(Trennflächenkennwerte aus Labortests)		
Reibungswinkel [°]			
Restreibungswinkel [°]			
Gebirgskennwerte	Minimalwert	Maximalwert	
GSI [-]			
σ [MPa]			
c [MPa]	0,01	0,03	0,25
φ [°]	25	30	30
E [MPa]	100	200	300

Fig.A.2.7 – Rock mass type GA38

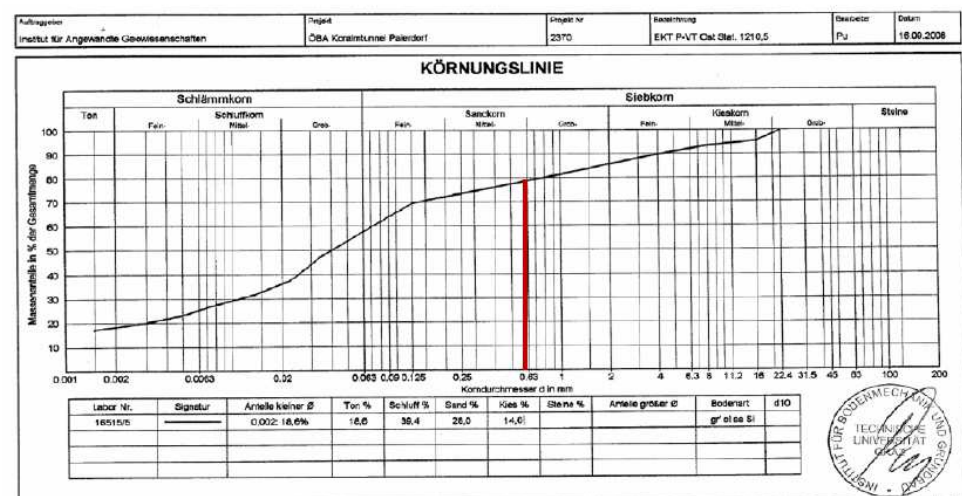
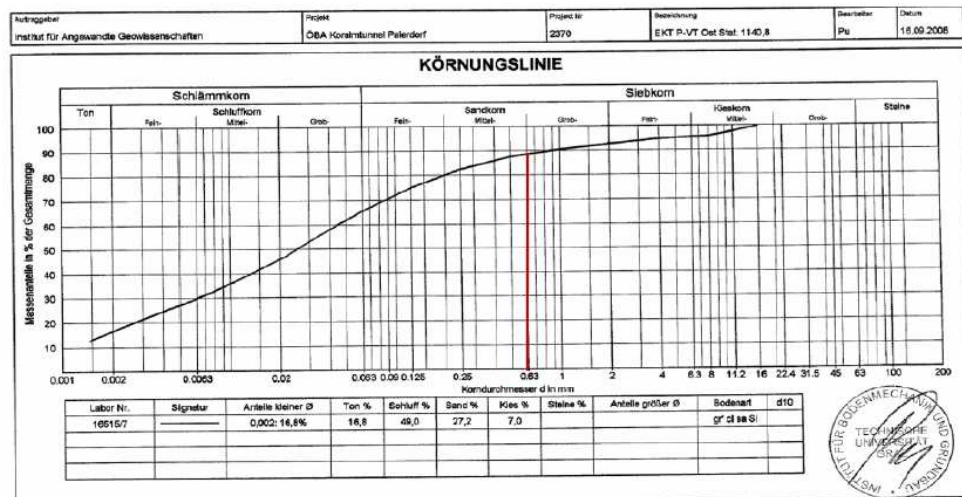
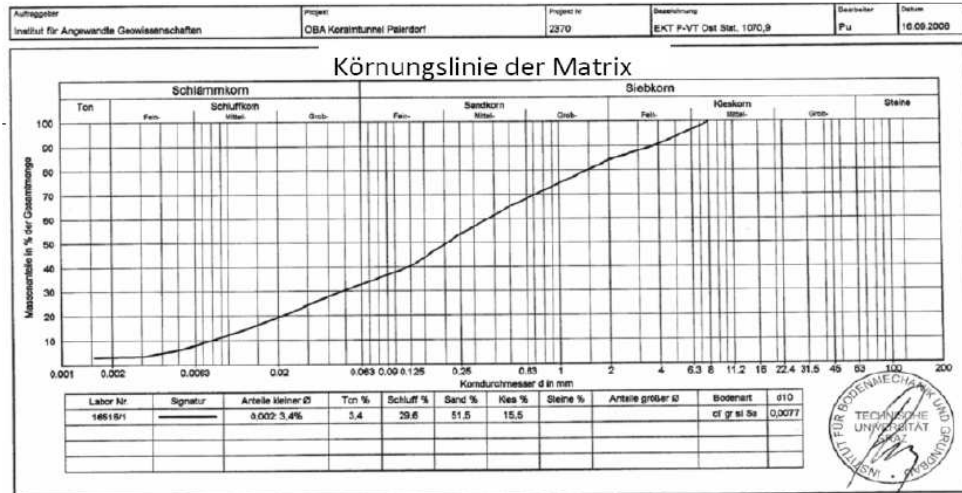
(δ) Gebirgsart 38

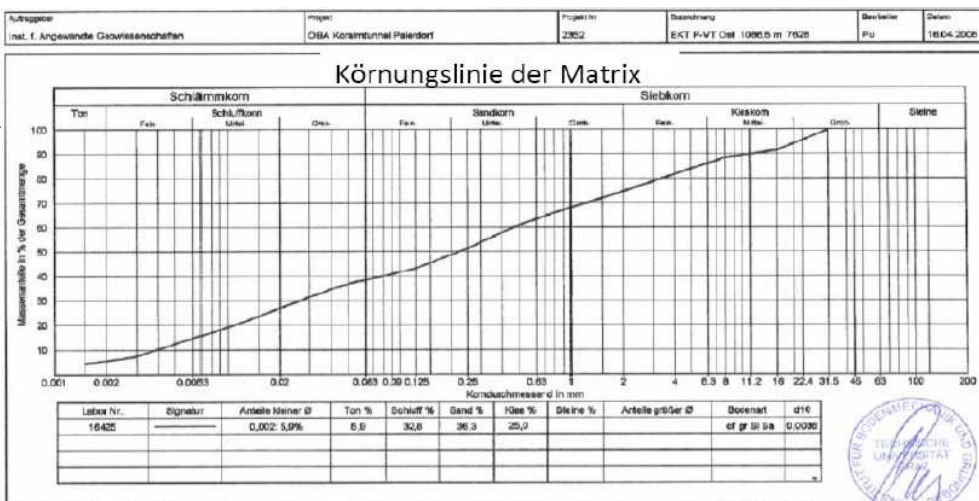
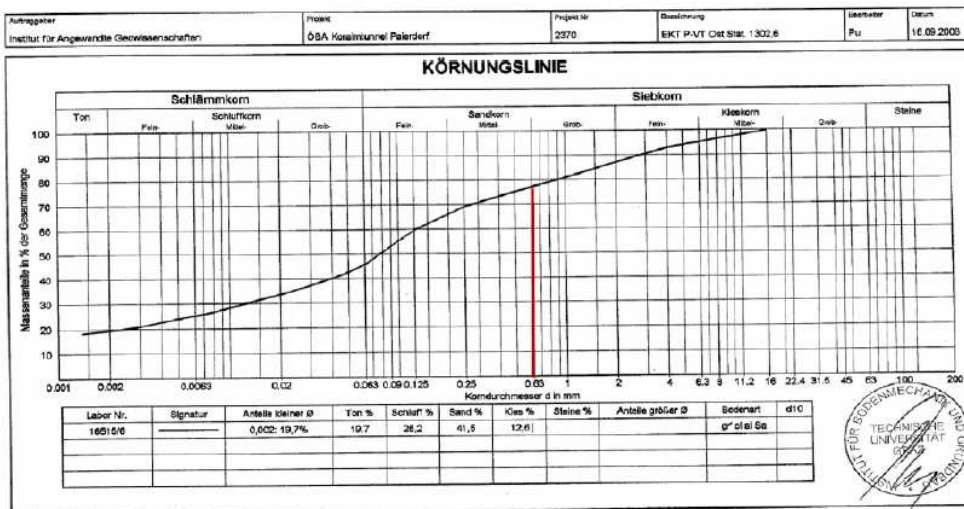
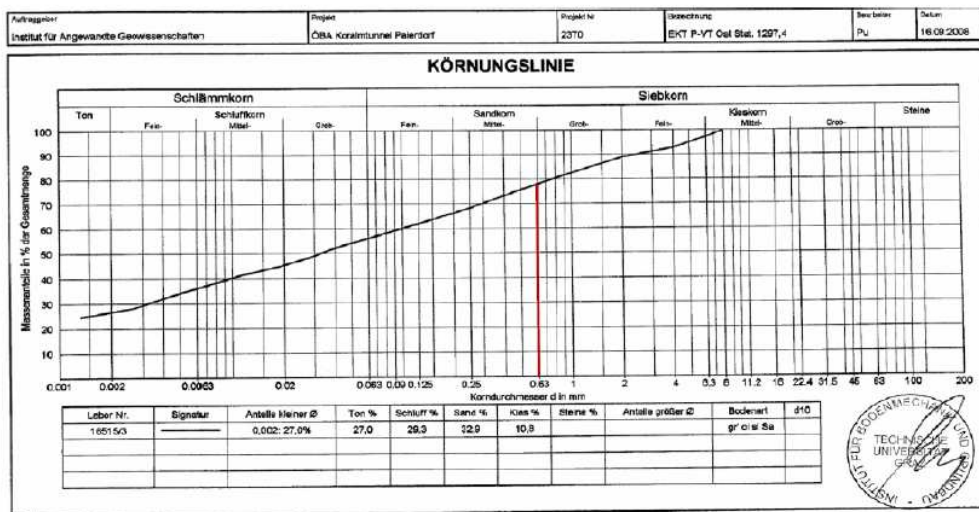
GA 38			
Lithologie (Hauptgesteinsart)	Kalkasit		
Tropie	Isotrop bis anisotrop		
Zerlegung	< 0,006 cm		
Vervitterung			
Verkarstung	nicht anwendbar		
Gesteinskennwerte			
UCS [MPa]			
m_1 [-]			
c [MPa]			
φ_1 [°]			
E_1 [MPa]			
ν_1 [-]			
CAI [-]			
Trennfächenskennwerte	(Trennfächenskennwerte aus Laborversuchen)		
Reibungswinkel [°]			
Restreibungswinkel [°]			
Gebirgskennwerte	Minimalwert	Maximalwert	
GS1 [-]			
σ [MPa]			
c [MPa]	0,03	0,05	
φ [°]	15	20	
Γ [MPa]	80	170	

Parameter	Einheit	Größe / Einstufung	Bemerkung
Gebirgsdurchlässigkeit	–	sehr schwach durchlässig	–
Quellpotential	–	nicht bis hoch quellfähig	–
injizierbarkeit	–	nicht wirksam	–
Wasserempfindlichkeit	–	stark veränderlich	–
Erweichbarkeit	–	stark erweichbar	–
Lösbarkeit	–	Tunnelbagger	–
Abrasivität	–	–	–

A2.2. GRAIN SIZE DISTRIBUTION CURVES

Fig.A.2.8 – Lavanttal fault zone grain size distribution curves





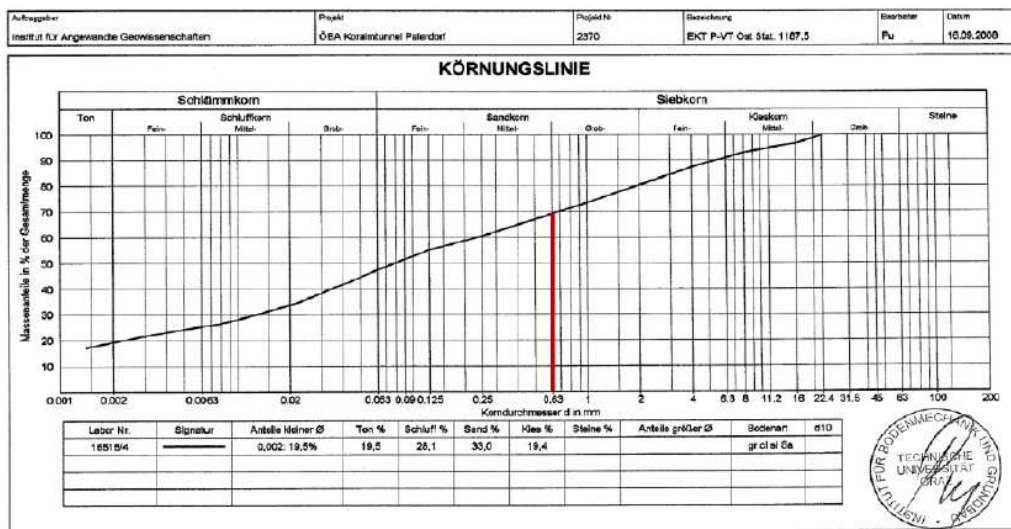
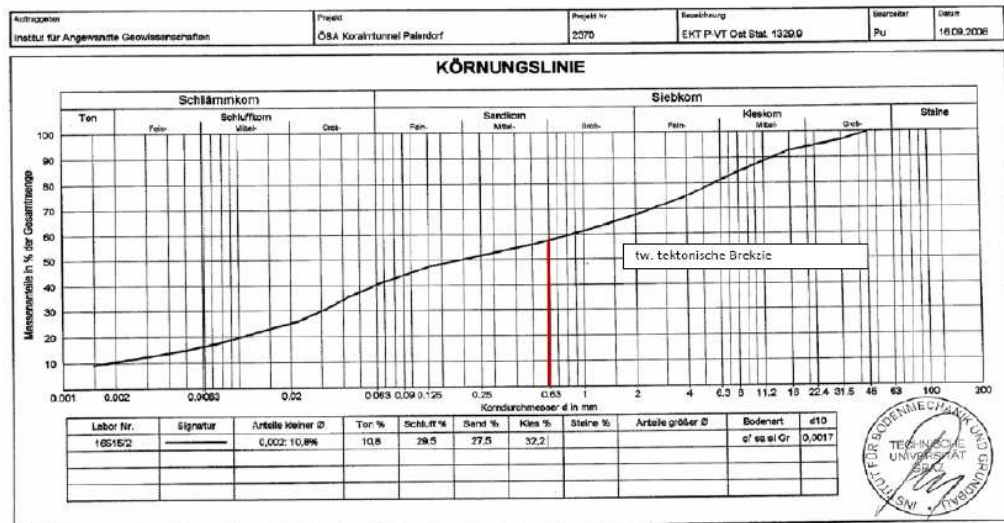


Fig.A.2.9 – Siltstone grain size distribution curve

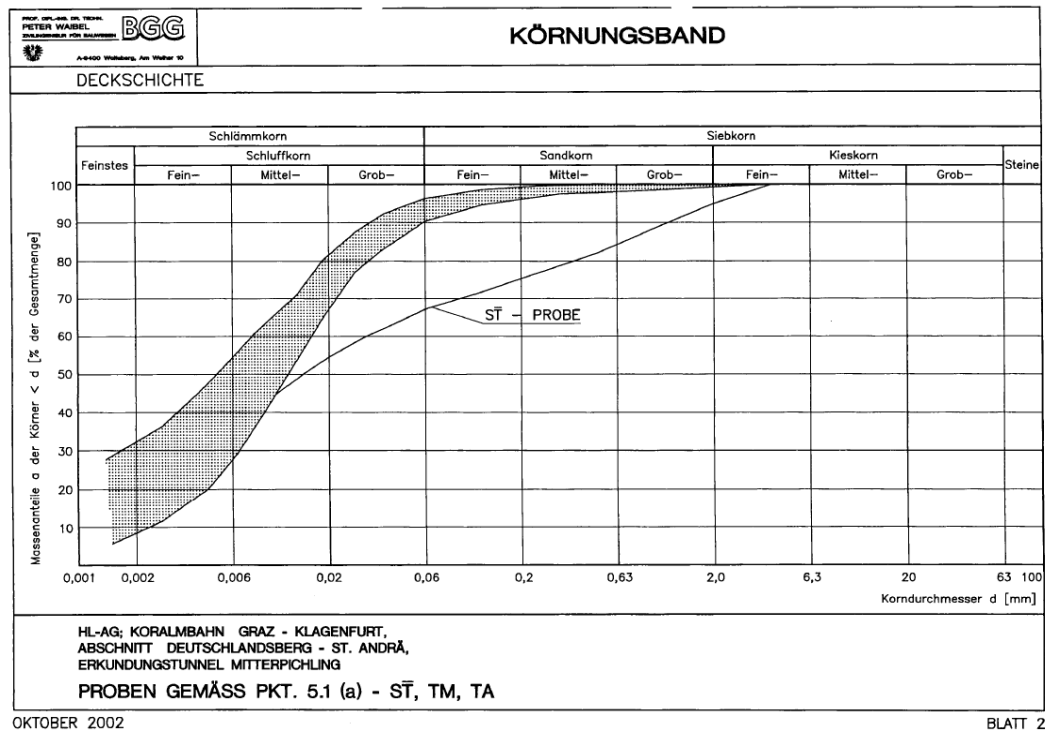


Fig.A.2.10 – Sand grain size distribution curve

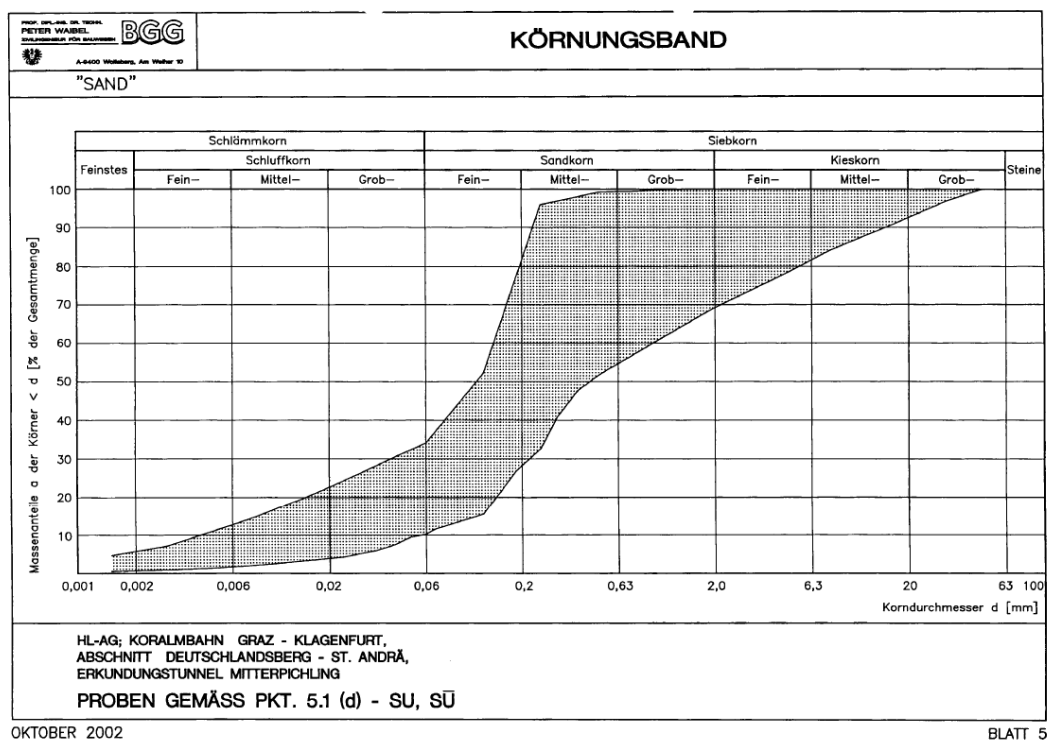
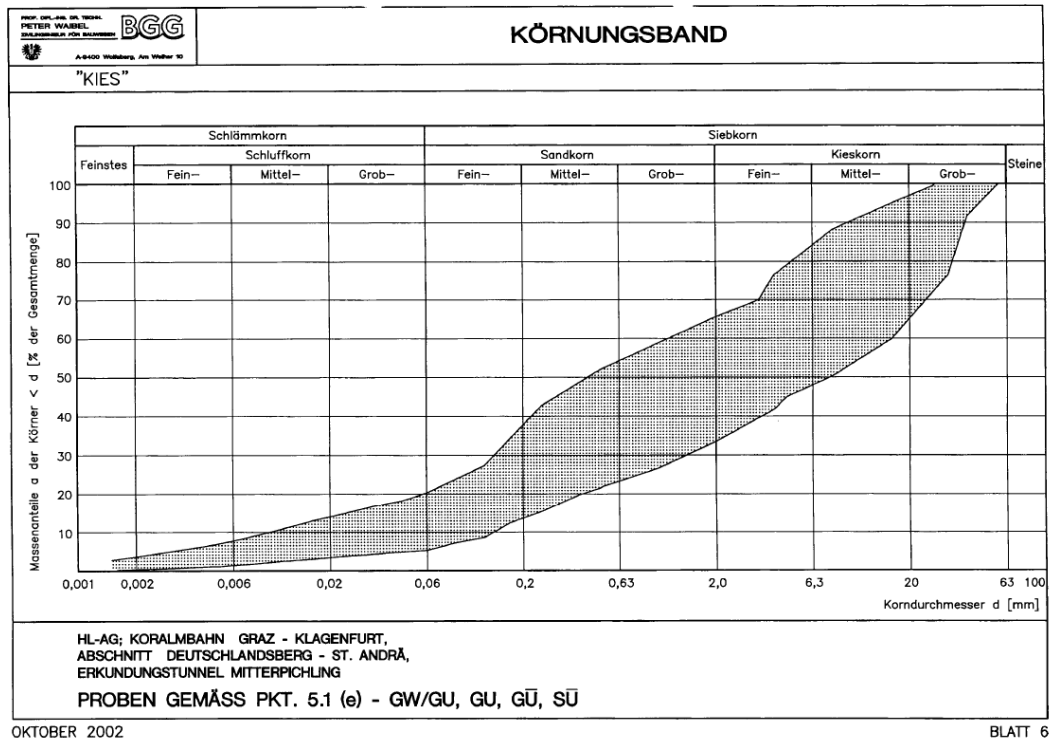


Fig.A.2.11 – Gravel grain size distribution curve



A2.3. REQUIRED EXCAVATION AND SUPPORT SHEET (RESS)

Fig.A.2.12 – RESS for chainage 2+024 (undisturbed Siltstone) of Mitterpichling investigation tunnel

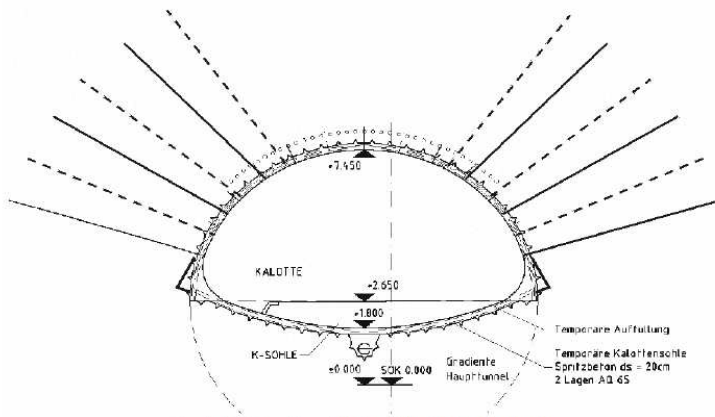
 KORALMTUNNEL weiterführende Erkundungen B1260 EKT-Mi: Erkundungstunnel Mitterpichling AUSBAUFESTLEGUNG OSTVORTRIEB		ARBEITSGEMEINSCHAFT IGT 3G ÖRTLICHE BAUAUFSICHT											
von Station:	2.024,10	bis Station:		Basisvortrieb:	K6-Cs	Blatt-Nr.:	Ost 101						
Übermaß (üm) in [cm]: <table border="1"> <tr> <td>linker Ulm:</td> <td>10 cm</td> </tr> <tr> <td>Firste:</td> <td>10 cm</td> </tr> <tr> <td>rechter Ulm:</td> <td>10 cm</td> </tr> </table>		linker Ulm:	10 cm	Firste:	10 cm	rechter Ulm:	10 cm						
linker Ulm:	10 cm												
Firste:	10 cm												
rechter Ulm:	10 cm												
MQ: Messbolzen lt. Messanweisung GT-U unterster Anker: +4,05 m abwechselnd unterhalb bzw. oberhalb der SpB-Aussparung ("Schlitz"). RQ: keine RS-Kaverne		(Symbolbild: Anzahl und Art der Stützmittel siehe unten!)											
Abschlagl.:	[m]	>1,00 - 1,30 m		Brustkeil:	-								
Bogen:	Typ	70/20/30		öffnen in TF:	n. Erf.								
BstG:	Typ/Lagen	AQ 65, 2 Lagen		als TF gilt Öffnen+Vorspr.+Brustsich.									
SpBStärke:	ds	30 cm											
Brustvers.:	Stärke/Fläche	10 cm bzw. 5 cm n.Erf. **)		BstG:	AQ 50 n.Erf. **)								
Brustanker:	Typ/BL/Länge/Stk.	12m IBO 250 kN; min. 4 Stk; Überlappung mind. 6 m;											
Fußverbr.:	Fläche [m²]	0,3 m²											
Fußplatte:	J/N	ja											
Kal. Sohle:	ds	30 cm	BstG (Typ):	AQ 65, 2 Lagen									
	Nachlauf	max. 8 Abschl.	Einbaulänge:	max. 4 Abschl.									
vorausell. Entwässerung: min. 2 Stk. mit ET 33m (Ausführung lt. Plan GCS. D.168) ...*)													
Anker: Ankertyp: IBO (Ulm) Länge: 6 m Anzahl: 4 Stk. Radialabst.: 1,5 m Verpressgut: CEM II Bruchlast: 250 kN Bohren: naß				Vorausellende Sicherung: Typ: PowerSet Länge: 4m Anzahl: n.Erf. Durchm.: DN51 Abstand [e]: max. 40cm W/Z Wert: - Verp.Druck: -									
Anmerkungen: *) Anzahl, Station und Bohransatzpunkt werden durch den Geologen vor Ort festgelegt. ***) zumindest 1,5 m² BstG und 10 cm SpB 10 pro Brustanker!													
AN		Datum		ÖBA-L									
Hagenhofer		29.09.2006		Wakolbinger									
P:\common\BM-ÖBA für Richi\B1260\B1260-ABFL-OST_001-050.xls\ABFL (50)													

Fig.A.2.13 – RESS for chainage 1+918 (Sand) of Mitterpichling investigation tunnel

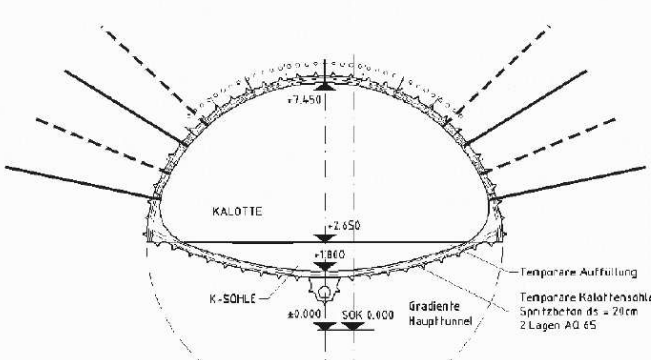
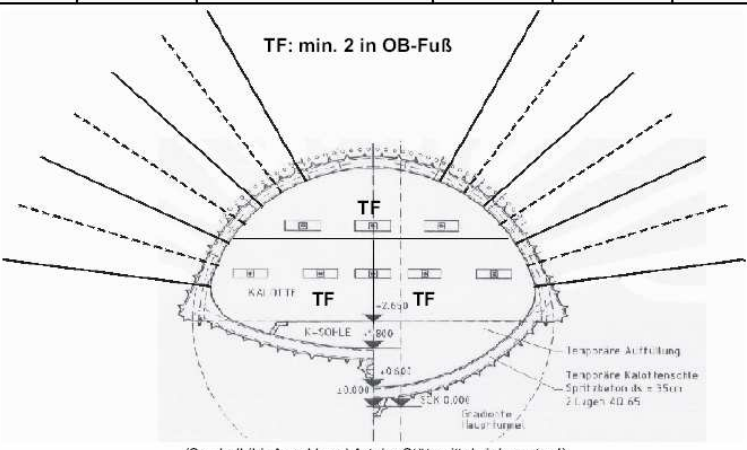
 KORALMTUNNEL weiterführende Erkundungen B1260 EKT-Mi: Erkundungstunnel Mitterpichling AUSBAUFESTLEGUNG OSTVORTRIEB		ARBEITSGEMEINSCHAFT IGT 3G ÖRTLICHE BAUAUFSICHT											
von Station:	1.918,10	bis Station:		Basisvortrieb:	K6-Cs	Blatt-Nr.:	Ost 96						
Übermaß (üm) in [cm]: <table border="1"> <tr> <td>linker Ulm:</td> <td>10 cm</td> </tr> <tr> <td>Firste:</td> <td>10 cm</td> </tr> <tr> <td>rechter Ulm:</td> <td>10 cm</td> </tr> </table> MQ: Messbolzen lt. Messanweisung GT-U unterster Anker: +4,05 m abwechselnd unterhalb bzw. oberhalb der SpB-Aussparung ("Schlitz"). RQ: keine RS-Kaverne								linker Ulm:	10 cm	Firste:	10 cm	rechter Ulm:	10 cm
linker Ulm:	10 cm												
Firste:	10 cm												
rechter Ulm:	10 cm												
													
Abschlagl.:	[m]	>1,00 - 1,30 m		Brustkeil:	-								
Bogen:	Typ	70/20/30		öffnen in TF:	n. Erf.								
BstG:	Typ/Lagen	AQ 65, 2 Lagen		als TF gilt Öffnen+Vorspr.+Brustsich.									
SpBStärke:	ds	25 cm											
Brustvers.:	Stärke/Fläche	10 cm bzw. 5 cm n.Erf. **)		BstG:	AQ 50 n.Erf. **)								
Brustanker:	Typ/BL/Länge/Stk.	12m IBO 250 kN; min. 4 Stk; Überlappung mind. 6 m;											
Fußverbr.:	Fläche [m²]	0,3 m²											
Fußplatte:	J/N	ja											
Kal.Sohle:	ds	20 cm	BstG (Typ):	AQ 65, 2 Lagen									
	Nachlauf	max. 8 Abschl. ***)	Einbaulänge:	max. 4 Abschl. ***)									
vorausel. Entwässerung: min. 2 Stk. mit ET 33m (Ausführung lt. Plan GCS. D.168) ...*)													
Vakuumspieße 4m, Anzahl n.Erf.													
Anker:				Vorauselende Sicherung:									
Ankertyp:	IBO (Ulm)			Typ	PowerSet								
Länge:	4 m			Länge:	4m								
Anzahl:	4 Stk.			Anzahl:	n.Erf.								
Radialabst.:	1,5 m			Durchm.:	DN51								
Verpressgut:	CEM II			Abstand [e]:	max. 40cm								
Bruchlast:	250 kN			W/Z Wert:	-								
Bohren:	naß			Verp.Druck:	-								
Anmerkungen: *) Anzahl, Station und Bohransatzpunkt werden durch den Geologen vor Ort festgelegt. **) zumindest 1,5m² BstG und 10cm SpB pro Brustanker! ***) Sohle gilt ab Station 1912,9													
AN		Datum		ÖBA-L									
Hagenhofer		20.08.2006		Steindorfer									
P:\common\BM-ÖBA\für Richi\B1260\B1260-ABFL-OST_001-050.xls\ABFL (50)													

Fig.A.2.14 – RESS for chainage 2+111 (fault Siltstone) of Mitterpichling investigation tunnel

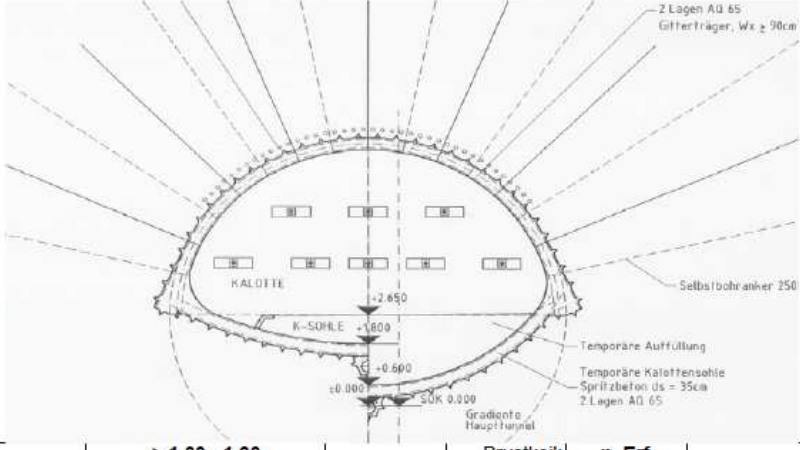
OBB Infrastruktur Bau		KORALMTUNNEL weiterführende Erkundungen B1260 EKT-Mi: Erkundungstunnel Mitterpichling		ARBEITSGEMEINSCHAFT IGT 3G ÖRTLICHE BAUAUFSICHT							
AUSBAUFESTLEGUNG OSTVORTRIEB											
von Station:	2.111,20	bis Station:		Basisvortrieb:	K7-Cs						
				Blatt-Nr.:	Ost 102						
Übermaß (üm) in [cm]: <table border="1"> <tr> <td>linker Ulm:</td> <td>10 cm</td> </tr> <tr> <td>Firste:</td> <td>10 cm</td> </tr> <tr> <td>rechter Ulm:</td> <td>10 cm</td> </tr> </table>		linker Ulm:	10 cm	Firste:	10 cm	rechter Ulm:	10 cm				
linker Ulm:	10 cm										
Firste:	10 cm										
rechter Ulm:	10 cm										
MQ: Messbolzen lt. Messanweisung GT-U unterster Anker: +4,05 m abwechselnd unterhalb bzw. oberhalb der SpB-Aussparung ("Schlitz"). RQ: keine RS-Kaverne		(Symbolbild; Anzahl und Art der Stützmittel siehe unten!)									
Abschlagl.: [m]		>0,80 - 1,00 m		Brustkeil: -							
Bogen: Typ		70/20/30		öffnen in TF: mind. 6 TF							
BstG: Typ/Lagen		AQ 65, 2 Lagen		als TF gilt Öffnen+Vorspr.+Brustsich.							
SpBStärke: ds		30 cm									
Brustvers.: Stärke/Fläche		10 cm vollflächig		BstG: AQ 50							
Brustanker: Typ/BL/Länge/Stk.		12m IBO 250 kN; min. 6 Stk; Überlappung mind. 6 m;									
Fußverbr.: Fläche [m²]		0,3 m²									
Fußplatte: J/N		ja									
Kal.Sohle: ds		30 cm		BstG (Typ): AQ 65, 2 Lagen							
Nachlauf		max. 8 Abschl.		Einbaulänge: max. 4 Abschl.							
vorausell. Entwässerung: min. 2 Stk. mit ET 33m (Ausführung lt. Plan GCS. D.168) ...*)											
Anker:		Vorausellende Sicherung:									
Ankertyp:	IBO (Ulm)	Typ	PowerSet								
Länge:	6 m	Länge:	4m								
Anzahl:	6 Stk.	Anzahl:	n.Erf.								
Radialabst.:	1,5 m	Durchm.:	DN51								
Verpressgut:	CEM II	Abstand [e]:	max. 40cm								
Bruchlast:	250 kN	W/Z Wert:	-								
Bohren:	naß	Verp.Druck:	-								
Anmerkungen:											
*) Anzahl, Station und Bohransatzpunkt werden durch den Geologen vor Ort festgelegt.											
AN		Datum		ÖBA-L							
Hagenhofer		24.10.2006		Wakolbinger							
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Fig.A.2.15 – RESS for chainage 0+534 (Gravel) of Paierdorf investigation tunnel

 KORALMTUNNEL weiterführende Erkundungen B1258 EKT-PD: Erkundungstunnel Paierdorf AUSBAUFESTLEGUNG EKT-West		ARBEITSGEMEINSCHAFT IGT 3G ÖRTLICHE BAUAUFSICHT							
von Station:	534,50	bis Station:							
Basisvortrieb:		T-K6-Df	Blatt-Nr.: West 24						
Übermaß (üm) in [cm]: <table border="1"> <tr> <td>linker Ulm:</td> <td>10,0 cm</td> </tr> <tr> <td>Firste:</td> <td>10,0 cm</td> </tr> <tr> <td>rechter Ulm:</td> <td>10,0 cm</td> </tr> </table>		linker Ulm:	10,0 cm	Firste:	10,0 cm	rechter Ulm:	10,0 cm		
linker Ulm:	10,0 cm								
Firste:	10,0 cm								
rechter Ulm:	10,0 cm								
MQ: Messbolzen lt. Messanweisung GT-U unterster Anker: auf + 3,75 m (knapp unterhalb des "Schlitzes") RQ: "Typ 2" **)		(Symbolbild; Anzahl und Art der Stützmittel siehe unten!)							
Abschlagl.:	[m]	> 1,00 - 1,30	Brustkeil: n. Erf.						
Bogen:	Typ	95/26/34	öffnen in TF: min. 3 TF.						
BstG:	Typ/Lagen	AQ 65 / 2 Lg.	als TF gilt Öffnen+Vorspr.+Brustsich.						
SpBStärke:	ds	35 cm							
Brustvers.:	Stärke/Fläche	10 cm vollflächig	BstG: AQ 50 vollfl.						
Brustanker:	Typ/BL/Länge/Stk.	IBO 250 kN 12 m; min. 8 Stk.; Überlappung mind. 6 m							
Fußverbr.:	Fläche [m²]	0,3							
Fußplatte:	J/N	ja							
flache	ds	35 cm ***)	BstG (Typ): AQ 65, 2 Lg. ***)						
Kal.Sohle:	Nachlauf	max. 8 Abschl.	Einbaulänge: max. 4 Abschl.						
vorausel. Entwässerung: **) min. 2 Stk. im linken u. rechten Ulm ...*)									
Entwässerungslanzen, Vakuumlanzen, 4m (2m+2m) nach geotechn. Erf.									
Anker:		Vorauselende Sicherung:							
Ankertyp	IBO	Typ	PowerSet						
Länge:	6,0 m	Länge:	4,0 m						
Anzahl:	8 / 6 Stk.	Anzahl:	n. Erf.						
Radialabst.:	1,50 m	Durchm.:	DN 51						
Verpressgut:	CEM II	Abstand [e]:	max. 20 cm in Firste						
Bruchlast:	250 kN		max. 40 cm in unterer Ulme						
Bohren:	naß	Verp. Druck							
Anmerkungen: *) VE: Typ, genaue Anzahl und Bohransatzpunkt werden durch den AG (Geologen) festgelegt; bei Bedarf werden aus dem fertigen Sohlgewölbe ergänzende VE-Rohre in Vortriebsrichtung gebohrt **) VE-Kavernen: Profil über 3 Abschlänge auf üm + 40 cm vergrößern. Startbogen am Ende der VE-Kavernen innerhalb der Vorausentwässerungsröhre (RQ Typ 2 + üm!) ***) Kalottensohle: Vorspritzen 5cm / 1.Lg.BstG + Abstandhalter 15cm + 2.Lg.BstG / SpB 30cm auf eine saubere Überlappung ("Kasterl auf Kasterl") ist zu achten! Spritzen von der Mitte zum Ulm; Rückprall ausblasen! Nachweis der Geometrie (Ausbruch + Innenkante) mittels Vermessung, Schablone oder Abstichmaße (Protokoll AN!)									
AN		Datum							
Dobernigg		04.05.2007							
		ÖBA-L							
		Posch							

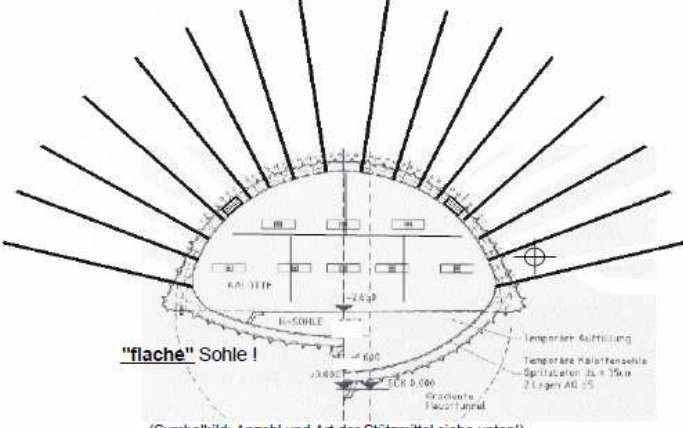
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Fig.A.2.16 – RESS for chainage 0+250 (Squeezed Siltstone) of Paierdorf investigation tunnel

 KORALMTUNNEL weiterführende Erkundungen B1258 EKT-PD: Erkundungstunnel Paierdorf Infrastruktur Bau		ARBEITSGEMEINSCHAFT IGT 3G ÖRTLICHE BAUAUFSTICHT							
AUSBAUFESTLEGUNG EKT-Ost									
von Station:	238,20	bis Station:							
Basisvortrieb:		T-K6-Ds	Blatt-Nr.: Ost 18						
Übermaß (üm) in [cm]: <table border="1"> <tr> <td>linker Ulm:</td> <td>10,0 cm</td> </tr> <tr> <td>Firste:</td> <td>10,0 cm</td> </tr> <tr> <td>rechter Ulm:</td> <td>10,0 cm</td> </tr> </table>		linker Ulm:	10,0 cm	Firste:	10,0 cm	rechter Ulm:	10,0 cm		
linker Ulm:	10,0 cm								
Firste:	10,0 cm								
rechter Ulm:	10,0 cm								
MQ: Messbolzen lt. Messanweisung GT-U unterster Anker: auf + 3,75 m (knapp unterhalb des "Schlitzes") RQ: keine									
Abschlagl.:	[m]	> 1,00 - 1,30	Brustkeil: n. Erf.						
Bogen:	Typ	95/26/34	öffnen in TF: min. 4 TF.						
BstG:	Typ/Lagen	AQ 65 / 2 Lg.	als TF gilt Öffnen+Vorspr.+Brustsich.						
SpBStärke:	ds	35 cm							
Brustvers.:	Stärke/Fläche	vollflächig 10cm	BstG: vollflächig AQ 50						
Brustanker:	Typ/BL/Länge/Stk.	min. 8 Stk. IBO 250 kN 12m; Überlappung mind. 6 m *)							
Fußverbr.:	Fläche [m²]	0,3							
Fußplatte:	J/N	ja							
tiefe	ds	35 cm	BstG (Typ): AQ 65, 2 Lg.						
Kal. Sohle:	Nachlauf	min. 4 - max. 10 Abschl.	Einbaulänge: max. 4 Abschl.						
vorausell. Entwässerung: min. 4 Stk. im linken u. rechten Ulm **)									
Vakuumanlagen, 4m (2m+2m) nach geotechn. Erf.									
Anker:		Vorausellende Sicherung:							
Ankertyp	IBO	Typ	PowerSet						
Länge:	9 m	Länge:	4,0m						
Anzahl:	6 Stk.	Anzahl:	n. Erf.						
Radialabst.:	1,50 m	Durchm.:	DN51						
Verpressgut:	CEM II	Abstand [e]:	max. 40 cm ***)						
Bruchlast:	250 kN	W/Z Wert:							
Bohren:		Verp. Druck							
Anmerkungen:									
*) Um zu erkennen, welche Brustanker neu oder alt sind, sind diese unbedingt zu kennzeichnen									
**) Bohransatzpunkt, Neigung und Station wird durch den Geologen vor Ort festgelegt									
***) Im Bereich von Sanden Spiessabstand max. 25cm									
Kalottensohle: Vorspritzen 5 cm / 1.Lg.BstG + Abstandhalter 15cm + 2.Lg.BstG / SpB 30cm auf eine saubere Überlappung ("Kasterl auf Kasterl") ist zu achten! Spritzen von der Mitte zum Ulm; Rückprall ausblasen! Nachweis der Geometrie (Ausbruch + Innenkante) mittels Vermessung, Schablone oder Abstichmaße (Protokoll AN)!									
AN	Datum	ÖBA-L							
	10.12.2006								
Dobernigg		Wakolbinger							

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Fig.A.2.17 – RESS for Lavanttal fault zone of Paierdorf investigation tunnel

 KORALMTUNNEL weiterführende Erkundungen		ARBEITSGEMEINSCHAFT IGT 3G ÖRTLICHE BAUAUFSTICHT							
B1258 EKT-PD: Erkundungstunnel Paierdorf AUSBAUFESTLEGUNG EKT-Ost									
von Station:	1.213,40	bis Station:							
Basisvortrieb:		K-K6-Vs	Blatt-Nr.: Ost 65						
Übermaß (üm) in [cm]: <table border="1"> <tr> <td>linker Ulm:</td> <td>20,0 cm</td> </tr> <tr> <td>Firste:</td> <td>20,0 cm</td> </tr> <tr> <td>rechter Ulm:</td> <td>20,0 cm</td> </tr> </table>				linker Ulm:	20,0 cm	Firste:	20,0 cm	rechter Ulm:	20,0 cm
linker Ulm:	20,0 cm								
Firste:	20,0 cm								
rechter Ulm:	20,0 cm								
MQ: Messbolzen lt. Messanweisung GT-U unterster Anker: auf + 3,65 m (knapp unterhalb des "Schlitzes") (Achtung: Vollbohrung im rechten Ulm!) RQ: Typ 2									
									
(Symbolbild; Anzahl und Art der Stützmittel siehe unten!)									
Abschlagl:	[m]	> 1,00 - 1,30	Brustkeil: n. Erf.						
Bogen:	Typ	95/22/32	öffnen in TF: min. 5 TF						
BstG:	Typ/Lagen	AQ 65 / 2 Lg.	als TF gilt Öffnen+Vorspr.+Brustsich.						
SpBStärke:	ds	30 cm							
Brustvers.:	Stärke/Fläche	vollflächig, 10cm; *)	BstG: vollflächig AQ 50 *)						
Brustanker:	Typ/BL/Länge/Stk.	min. 8 Stk. 12m IBO 250 kN; >6m Überlap. *)							
Fußverbr.:	Fläche [m²]	0,3 m	Verformungsschlitz: 2 Stk						
Fußplatte:	J/N	ja	Stachelemente: Typ A/4Stk. je Element						
flache	ds	30 cm	BstG (Typ): AQ 65, 2 Lg.						
Kal.Sohle:	Nachlauf	max. 8 Abschl.	Einbaulänge: max. 4 Abschl.						
vorausell. Entwässerung: EKB gem. Anordnung ÖBA / Geologe									
Anker:		Vorausellende Sicherung:							
Ankertyp	Selbstbohr	Typ	RohrSp. PowerSet **)						
Länge:	6,0 m	Länge:	4,0 m 4,0 m						
Anzahl:	16 Stk.	Anzahl:	n. Erf. n. Erf.						
Radialabst.:	1,0 m	Durchm.:	DN 38 DN50						
Verpressgut:	CEM II	Abstand [e]:	25 cm 25 cm						
Bruchlast:	250 kN	W/Z Wert:	- -						
Bohren:		Verp. Druck:	- -						
Anmerkungen: Stachelemente: Laibung vorspritzen, Elemente ca 1m unter Bogenschloß tangential einbauen gem. Plan ...@D.284.0! zu den Stachelementen benachbarte Anker mit 50 cm Abstand einbauen! *) im gestörten Festgestein (u.a. "mixed-face Bedingungen") OB-Sicherung (SpB, BstG u. OB-Anker) sowie Teilflächen in Absprache mit ÖBA anpassen. **) je nach Standfestigkeit der Bohrlöcher Kalottensohle: Vorspritzen 5 cm / 1.Lg.BstG + Abstandhalter 15cm + 2.Lg.BstG / SpB 25 cm auf eine saubere Überlappung ("Kasterl auf Kasterl") ist zu achten! Spritzen von der Mitte zum Ulm; Rückprall ausblasen!									
AN	Datum	ÖBA-L							
Fischer	04.06.2008	Posch							

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A2.4. SHOTCRETE STRESS ANALYSIS RESUME

Fig.A.2.18 – Shotcrete stresses on the analyzed cross sections

Description of columns in the result file	
CS	cross section
Time (h)	Age of shotcrete in hours
N	number of grid point
L	number of layer (at grid point position)
fcu [Pa]	ultimate concrete strength at with age (see time above)
sig(1) [Pa]	stress in circumferential direction

SILTSTONE CS 0+250						
CS	Time (h)	n	L	fcu[Pa]	sig(1) [Pa]	sig(1) [MPa]
CS00250	624.0	10	1	2.78E+07	-6.00E+06	
CS00250	624.0	10	2	2.78E+07	-6.11E+06	
CS00250	624.0	10	3	2.78E+07	-6.22E+06	
CS00250	624.0	10	4	2.78E+07	-6.36E+06	
CS00250	624.0	10	5	2.77E+07	-6.72E+06	
CS00250	624.0	10	6	2.76E+07	-7.06E+06	
Average value						-6.41

compression

SILTSTONE CS 2+062						
CS	Time (h)	n	L	fcu [Pa]	sig(1) [Pa]	sig(1) [MPa]
CS02062	624.0	11	1	2.78E+07	-1.56E+06	
CS02062	624.0	11	2	2.78E+07	-1.67E+06	
CS02062	624.0	11	3	2.78E+07	-1.67E+06	
CS02062	624.0	11	4	2.78E+07	-1.76E+06	
CS02062	624.0	11	5	2.77E+07	-1.82E+06	
CS02062	624.0	11	6	2.76E+07	-1.83E+06	
Average value						-1.72

compression

SILTSTONE CS 2+120						
CS	Time (h)	n	L	fcu [Pa]	sig(1) [Pa]	sig(1) [MPa]
CS02120	624.0	8	1	2.78E+07	-3.97E+06	
CS02120	624.0	8	2	2.78E+07	-3.32E+06	
CS02120	624.0	8	3	2.78E+07	-2.69E+06	
CS02120	624.0	8	4	2.78E+07	-2.10E+06	
CS02120	624.0	8	5	2.77E+07	-1.54E+06	
CS02120	624.0	8	6	2.76E+07	-1.03E+06	
Average value						-2.44

compression

SAND CS 1+953						
CS	Time (h)	n	L	fcu [Pa]	sig(1) [Pa]	sig(1) [MPa]
CS01953	624.0	10	1	2.78E+07	-2.41E+06	
CS01953	624.0	10	2	2.78E+07	-2.00E+06	
CS01953	624.0	10	3	2.78E+07	-1.62E+06	
CS01953	624.0	10	4	2.78E+07	-1.27E+06	
CS01953	624.0	10	5	2.77E+07	-9.64E+05	
CS01953	624.0	10	6	2.76E+07	-6.95E+05	
Average value						-1.49
compression						
GRAVEL CS 0+536						
CS	Time (h)	n	L	fcu [Pa]	sig(1) [Pa]	sig(1) [MPa]
CS00536	623.0	10	1	2.78E+07	1.42E+06	
CS00536	623.0	10	2	2.78E+07	1.47E+06	
CS00536	623.0	10	3	2.78E+07	1.49E+06	
CS00536	623.0	10	4	2.78E+07	1.48E+06	
CS00536	623.0	10	5	2.77E+07	1.42E+06	
CS00536	623.0	10	6	2.76E+07	1.33E+06	
Average value						1.43
tension						

A3

APPENDIX 3 SPREADSHEET CALCULATIONS

A3.1. USER'S INTERFACE

Fig.A.3.1 – User interface sheet

INPUT DATA		OUTPUT DATA		
250	E [MPa]	for E= 250 MPa		
75	H [m] overburden	ϕ [°]	c [MPa]	u0 / umeasured
22,5	γ [kN/m ³]	0	0,54	0,48
0,20	pi [MPa] supported	1	0,52	0,48
0,30	v	2	0,50	0,48
4,50	r0 [m]	3	0,48	0,49
60	maximum observed displacement [mm]	4	0,46	0,49
0+000	Cross Section	5	0,44	0,50
		6	0,42	0,51
		7	0,40	0,51
		8	0,38	0,52
		9	0,36	0,53
		10	0,34	0,54
		11	0,32	0,56
		12	0,30	0,57
		13	0,28	0,59
		14	0,26	0,61
		15	0,24	0,63
		16	0,22	0,66
		17	0,19	0,73
		18	0,17	0,78
		19	0,15	0,83
		20	0,12	0,98
		21	0,09	1,22
		22	0,06	1,63
		23	0,00	1,49
		24	0,00	1,37
		25	0,00	1,60
		26	0,00	1,46
		27	0,00	1,34
		28	0,00	1,23
		29	0,00	1,13
		30	0,00	1,05
		31	0,00	0,97
		32	0,00	0,89
		33	0,00	0,83
		34	0,00	0,77
		35	0,00	0,72
		36	0,00	0,67
		37	0,00	0,63
		38	0,00	0,59
		39	0,00	0,56
		40	0,00	0,23

Colour signs :

1,00	ideal value
0,65	possible value
0,50	value not to consider

The basic procedure is given as follows. A deformation modulus E is assumed and a set of corresponding shear strength parameters c and ϕ determined. By variation of the deformation modulus from the minimum value (i.e. elastic behaviour) to higher values the whole field of possible parameters is determined.

For the use of the spreadsheet developed the user is asked for the input parameters to be used in the cross section in study. Requested parameters are deformation modulus “ E ”, tunnel overburden “ H ”, unit weight of the rock mass “ γ ”, internal support pressure “ p_i ”, Poisson ratio “ ν ”, original tunnel radius “ r_0 ”, and the “maximum observed displacement” from the surveyed data. The data is inserted in the “INPUT DATA” and the linking formulae return in the “OUTPUT DATA” table values for cohesion “ c ”, friction angle “ ϕ ”, and ratio of pre-displacement to measured displacement “ u_0/u_{measured} ” (Fig.A.3.1). These values are given for a friction angle variation from 0° to 40° , which is considered to be representative of the rock masses studied.

The colour signs used enable a fast first reading of the values to be considered or disregarded.

In the “ u_0/u_{measured} ” column three colours are used. The ideal values, represented in green, highlight the range between 0.70 and 1.30, corresponding to ratios of pre-displacement to displacement close to 1:1 as it was described before and referenced by Sellner (2000).

If the colour of the cell is red it means that ratios above 1.30 and under 0.70 occur in the cross section for the combination of values inputted. These values are not considered as valid combinations of cohesion and friction angle because pre-displacement occurring at the face is either higher or lower than the defined range (1:1). All the other values between the “green range” and the “red range” are displayed in yellow meaning that it is not the ideal value but it is also not a value to disregard.

The colour cells are only a user’s friendly tool and they should not be considered correct always. Therefore care must be taken when reading the values and a critical engineer view is requested all the time.

A3.2. CALCULATION APPROACH

The input data given by the user is used in different sheets (one for each friction angle value) for the calculation of the estimated values. In each of the sheets the calculation method is the same and explained in the following paragraphs.

Each friction angle will have INPUT FIXED DATA (table A.3.1) and INPUT VARIABLE DATA (table A.3.2). The fixed data is exemplified in the following table.

Table A.3.1 – Input fixed data

INPUT FIXED DATA	
250	E [MPa]
1.69	p_0 [MPa]
0.00	p_i [MPa] unsupported
0.20	p_i [MPa] supported
0.35	<-- rad --- ϕ [°] --> 20.00
0.30	ν
2.04	k
4.50	r_0 [m]

Where the in-situ stress p_0 is the product between the overburden, H , and the unit weight, γ . The value of the slope k is given by equation (2.3) and the unsupported internal pressure p_i , is always considered to be 0. This fixed data is always the same for a given friction angle.

Variable input data, unchangeable by the user, is the simple way of varying the other convergence confinement input value, i.e. the cohesion. This is done changing the values of cohesion in each line of the spreadsheet by 0.02MPa. The total range of values for cohesion goes from 2.00MPa to 0.00MPa. Together with the 0.02MPa variation it is considered to be a plausible range of cohesion values for this study. For each cohesion value a corresponding uniaxial compressive strength of the rock mass σ_{cm} given by equation (2.2) is automatically calculated. These two parameters are the variable input data and an example is given as follows.

Table A.3.2 – Input variable data

INPUT VARIABLE DATA	
c [MPa]	σ_{cm} [MPa]
2.00	5.71
1.98	5.66
...	...
0.14	0.40
0.13	0.37
0.12	0.34
0.11	0.31
0.10	0.29
0.09	0.26
...	...

With the input data the required parameters used in the convergence confinement back-analysis can be calculated. An example of the calculated output values is shown in table A.3.3.

Table A.3.3 – Output data

OUTPUT VALUES							
rp unsupported [m]	rp supported [m]	pcr	u supported [mm]	umax [mm]	u0 [mm]	u0/umax	u "measured" [mm]
3.89	3.76	-0.77	42.29	44.42	13.00	0.29	29.28
3.90	3.77	-0.75	42.08	44.23	12.95	0.29	29.13
...	...	...	...	...	...	...	...
15.20	10.16	0.98	104.46	249.11	50.03	0.20	54.43
16.11	10.51	0.99	110.97	278.00	54.16	0.19	56.82
17.18	10.89	1.00	118.39	313.62	58.96	0.19	59.43
18.43	11.31	1.01	126.91	358.34	64.61	0.18	62.30
19.94	11.78	1.02	136.75	415.73	71.30	0.17	65.46
21.77	12.31	1.03	148.24	491.35	79.28	0.16	68.96
...	...	...	...	...	...	...	...

Where the “rp unsupported” represents the radius of the plastic zone with an internal support pressure of 0, the “rp supported” the plastic radius given by the internal support pressure defined by the user both given by equation (2.6). The critical internal pressure “pcr” is defined by equation (2.4), the “u supported” is the plastic displacement with a radius of “rp supported” defined by equation (2.7) and the “umax” is the unsupported maximum plastic displacement due to the “rp unsupported” also given by equation (2.7). The ratio “u0/umax” is defined, as explained before, by Hoek et al (2008) given by equation (2.9). The product between “u0/umax” and “umax” gives the tunnel face displacement “u0”. The “umeasured” is calculated from the difference between “umax” and “u0”.

Finally the “u measured” (the value to be reached) is compared with the input data “maximum observed displacement” (Fig. A.3.1) and if the same value is found the corresponding cohesion is plotted in the output table seen by the user. The same procedure is carried to find the corresponding “u0”. This value is divided by the “maximum observed displacement” and the user is able to read in the output data (Fig.A.3.1) the $u_0/u_{measured}$.

The methodology followed in the study consists in finding for each value of elasticity modulus the most plausible values of cohesion and friction angle with the help of the spreadsheet described above.

