Estimation of rock mass properties of heavily sheared flysch using data from tunnelling construction

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Abstract: Estimation of rock mass properties can be achieved by laboratory testing, in situ testing, back analysis or the use of rock mass classifications (GSI, RMR, Q, etc.). Although reasonable geotechnical parameters, from the use of the rock mass classification schemes, are needed for design before construction, the best way to estimate geotechnical parameters is by back analysis, which can only be done during construction.

The Egnatia motorway in Greece will be 680km long when complete and will have a total of 74 tunnels (mainly twin), of a combined length of 100km. Two of these tunnels are partly driven in heavily sheared flysch and have suffered major deformation by ground squeezing. The flysch is clayey in nature and often exhibits a chaotic structure.

The tunnels have been driven under a cover of 90m to 180m and have been excavated by conventional methods with a top heading and a bench. Although GSI and RMR values used for the design have been proved similar with those estimated from back analysis, the value of the uniaxial compressive strength of the intact rock was found lower than the one assumed for the design. Sensitivity analysis showed that the differences in unconfined strength value for such tectonically disturbed weak geomaterial are most critical in the evaluation of the deformations of the tunnel.

Resume: L' estimation des propriétés des massifs rocheux peut être faite par des essais au laboratoire ou in situ, par des analyses en retour ou par le moyen des classifications des massifs rocheux (GSI, RMR, Q etc.). Bien que des paramètres géotechniques nécessaires pour le projet peuvent être raisonnablement sélectionner avec l' application des classifications, la meilleure façon est par des analyses en retour qui cependant ne peuvent être faites que quand la construction est en cours pas les mesures des convergences.

L'autoroute de Egnatia au nord de le Grèce, longue de 680km aura, quand elle sera finie, 74 tunnels avec une longueur de 100km. Deux de ces tunnels ont souffert des déformations importantes par convergence du materiel, qui est un flysch pélitique souvent d' une structure chaotique.

Les deux tunnels out une couverture de 90m et 180m et ont été construit en phases («top heading» et «bench»). Bien que les valeurs de GSI on de RMR utilisées pour le projet ont été concordantes avec les résultants des analyses en retour, les valeurs de la résistance à la compression simple, σ_{ci}, ont été trouvées inférieures à celles prises a l' etude du projet. L'analyse de sensibilité motre que la variation de σ_{ci} dans ces formations tectoniquement perturbées, est très importante pour l'évaluation des de formations d' un tunnel.

Keywords: weak rocks, 3D models, tunnels, mechanical properties, deformation, data analysis

INTRODUCTION

Tunnelling through tectonic shear zones is always a challenge due to the geometrical complexity and the presence of poor quality rock masses with high variability in their occurrence. The design of tunnels in weak rock masses such as sheared flysch presents a major challenge to geologists and engineers. The complex structure of these materials, resulting from their depositional and tectonic history, means that they cannot easily be classified in terms of the widely used rock mass classification systems.

The two tunnels, from which the data have been analyzed in the paper, are excavated along the Egnatia Motorway, today under construction. Egnatia Motorway comprises a very important and modern infrastructure for the communication of Greece with Europe, the Balkans and the East. It starts from Igoumenitsa at the west, runs across the northern part of the country, ending in the east at the Greek-Turkish borders. The Motorway will be 680 km long, providing to the European Union access to the east with no border crossing and belongs to the 14 projects of a Trans-European network. Until now 534 km have already been constructed.

When complete, the Egnatia Motorway will have a total of 74 road tunnels of an overall combined length of 99km. The excavation diameter of the conventionally excavated tunnels is about 12m and among those already finished, most were opened by the top heading and bench method. The Egnatia Motorway runs across the entire width of Greece traversing almost perpendicularly the main geotectonic units of the country crossing perpendicularly major tectonic contacts and thrusts, where weak rock masses of flysch are present (Marinos, Aggistalis & Kazilis, 2004).

Estimation of the mechanical parameters of such sheared siltstone or shale is a difficult task since the strength of the intact parts can hardly be measured in the laboratory. Representative strength values calculated with back analysis are presented in this paper.

Two tunnels of a similar quality geomaterial but different tectonic evolution, both in a thrust environment, are presented. The rock mass in the first tunnel, (case study 1), is derived from the huge overthrust of the Pindos unit formation over the Ionian flysch in Metsovo area, in Western Greece, and in the second (case study 2) is derived from the shearing of a large anticline in siltstones. Large parts of the tunnels had been driven through heavily sheared flysch and exhibit squeezing behavior even under medium overburden (tens of metres). Sections with large and greater than expected deformations were chosen to estimate and re-evaluate the geotechnical parameters for this complex and weak geomaterial. An example of the sheared flysch, which was excavated in case study 1, is shown in Figure 1.

Figure 1. Heavily sheared grey and reddish siltstones produced by a huge overthrust, in case study 1

GEOLOGICAL CONDITIONS

Geological model

The geological model is the basic initial step on which the design of the tunnel is based. As a result, the geological conditions to be encountered in terms of quality of the ground are defined and the type, location and size of potential hazards can be identified. In the case of major tectonic thrusts such as in the area where the two tunnels are located, the alignment, even crossing them perpendicularly, deals with rock masses, mainly flysch, disturbed in a very wide zone often extended through satellite shears.

The two tunnels, part of them consisting of similar quality geomaterial, have different tectonic histories. The first is situated in an area, which is derived from the huge overthrust of the Pindos unit formation over the Ionian, whereas the second is a result of shearing by anticlines and synclines in siltstones. Thus, significant part of these tunnels had been driven through heavily sheared clayey flysch (siltstones-clayshales with spare sandstone layers), which exhibited squeezing behaviour even under only tens of meters of overburden. The corresponding rock masses can be classed as a tectonic mélange since they consist of chaotic, heterogeneous geological mixtures of blocks of different types and sizes, surrounded by weaker, sheared finer-grained rocks. In the present study, this particular geological formation, assuming that it has a chaotic structure, is considered as a «pseudo-isotropic mixture» (Marinos & Hoek, 2000). This assumption, when reasonable, can lead to a straight forward estimation of the geotechnical parameters for the design of the support categories. The presence of better quality blocks along the sheared mass may improve the stability of the surrounded rock mass, depending on their location and size. A tunnel drive through this geomaterial requires continuous geological and geotechnical characterization, as well as state of the art monitoring, to comprehend the interaction of the melange's internal block/matrix structure and their impact on the excavation and can only be conducted during tunnel construction. Such an effort is described in a recent paper (Button *et al*. 2004).

Case study 1

In this tunnel the motorway runs through a contact zone of the huge overthrust of the Pindos Unit formation over the Ionian Flysch in Metsovo area. The base of the overthrusted-sheared material is mainly in the Pindos flysch. This thrust is a result of a big compression event during the alpine orogenesis with an ENE to WSW direction. This contact is dipping to ENE and is located very close to the tunnel area. This flysch is characterized by different alternations of siltstones and sandstones. Within the area of the tunnel, the flysch is of a more clayey nature and often exhibits a chaotic structure. Only small sections of the tunnel drive met sheared but not structureless parts of the initial geological stratigraphy.

The main thrust movement is associated with satellite shears within the thrusted body. These shears are generally marked by a reddish siltstone sequence of the Pindos flysch, which acted as a 'soap layer' and when vegetation is absent these shears can be mapped. The flysch has thus suffered from large compression and very weak rocks masses have been produced. The overall rock mass is highly heterogeneous and anisotropic and additionally affected by extensional faulting having produced mylonites due to this severe tectonic stressing. The original structure is no longer recognizable and blockiness is lost. Inside the body of the sheared mass it is reasonable to expect less deformed sandstone blocks (1.5-2 m^3) with more rock-like behavior. However, this material is likely to be cut by small shears and in the scale of the tunnel, these boulders do not contribute significantly to the overall strength of the rock mass. Additionally, due to the heterogeneity in both horizontal and vertical direction these better conditions do not occur in the whole tunnel section (e.g. foundation area of the elephant foot of the top heading).

A conceptual model has been drawn to picture the geological structure of the tunnel area in Figure 2.

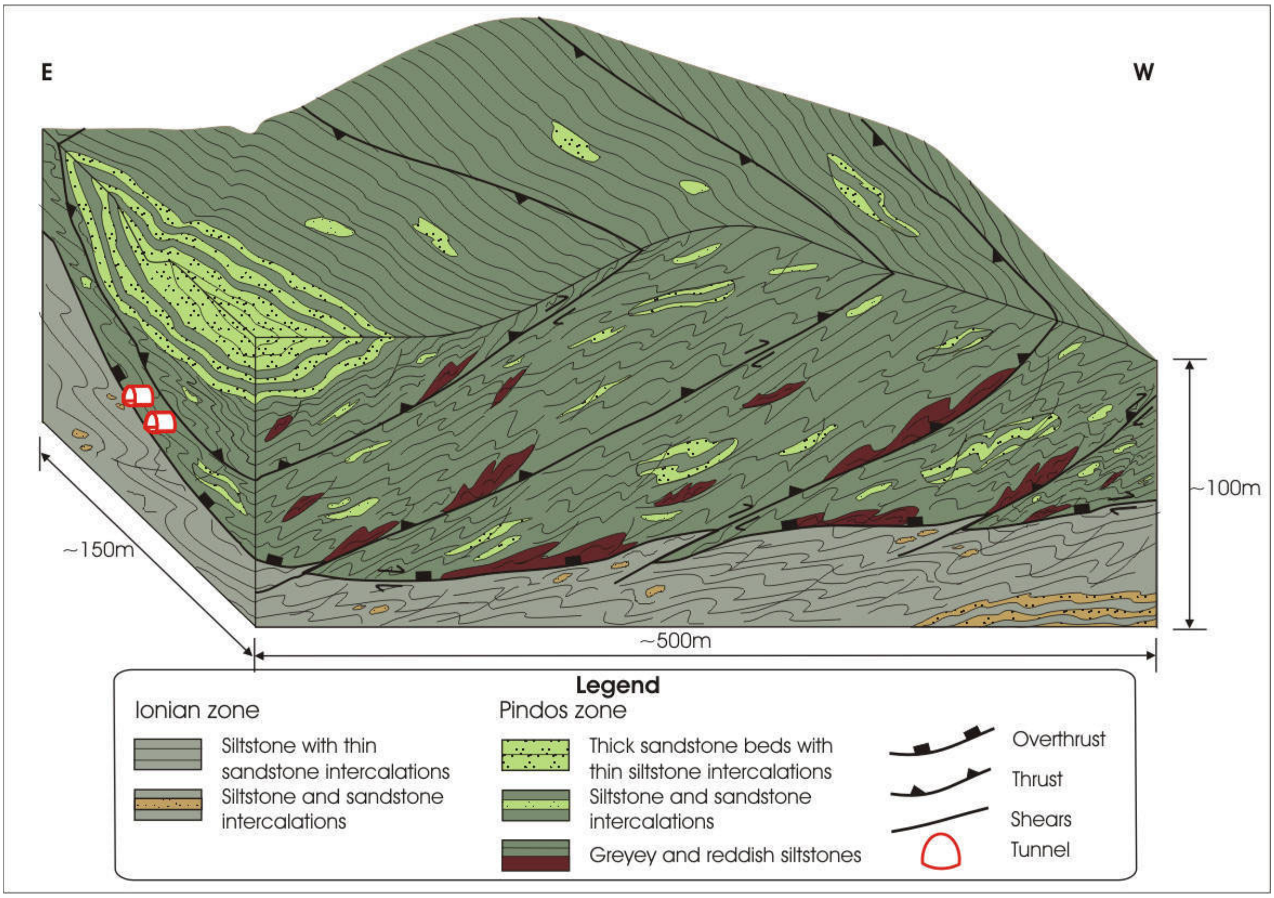

Figure 2. Conceptual engineering geological model of the tunnel in case study 1 tunnel

Case study 2

The tunnel was driven through the Ionian flysch formation. Lithologically it comprises alternations of sandstones and siltstones with a predominance of either siltstone or sandstone. The tunnel has a perpendicular direction (N010^o) to the axis of large synclines and anticlines (N080^o-N260^o). Where flysch, composed of siltstones with thin layers of sandstone, is further more folded it, has produced very poor rock masses. Folding and faulting often entail a clearly visible chaotic structure of isolated lensed blocks of hard rock ‘floating’ within a soft clayey-silty matrix. Well-defined shears, frequently oriented parallel to the foliation planes, which constitute the widespread structural feature of the rock mass, were almost vertically crossed. These weak zones were effectively predicted during the design. The highest cover of the tunnel is situated in such geomaterial and part of it suffered severe squeezing.

An engineering geological model for a section of this tunnel is shown in Figure 3. It is noted that both 3-D models were generated after the completion of the tunnels.

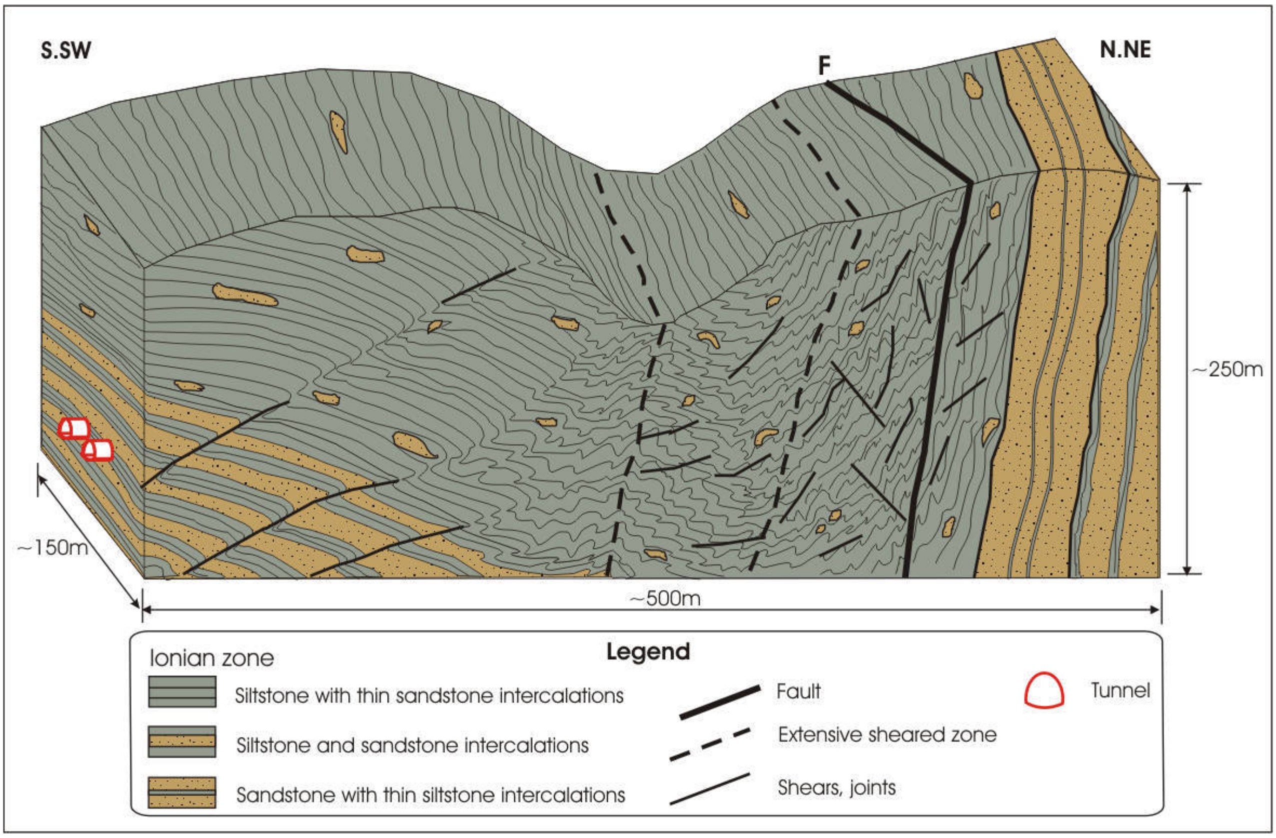

Figure 3. Engineering geological model of a section of the tunnel in case study 2 tunnel

GEOTECHNICAL PARAMETERS

One of the major problems in designing underground openings is that of estimating the strength and deformation properties of the in-situ rock mass. Understanding the behavior of rock mass requires a study of the intact rock material and of individual discontinuity surfaces which go together to make up the system.

Estimation of rock mass properties can be achieved by one of the following methods: a) laboratory testing, b) in situ testing, c) use of rock mass classifications (GSI, RMR, Q, etc.) and d) back analysis. However, samples are not representative of the rock mass due to the disturbance, jointing and the heterogeneity of most formations. Additionally, it is often not realistic or always feasible to carry out in situ tests. To estimate reasonable geotechnical parameters for the design of the tunnel support categories before its construction, where back analysis is not possible, there is no option but to rely upon the use of a rock mass classification scheme that is correlated with the basic parameters needed for the design. Back analysis is the best way to estimate the geotechnical parameters when construction has started by evaluation of the deformation measurements, and it can be used to validate or modify the parameters used.

Reliable estimates of the strength and deformation characteristics of rock masses are required for almost any form of analysis used for the design of underground excavations. Hoek and Brown proposed a method for obtaining estimates of the strength of jointed rock masses, based upon an assessment of the interlocking of rock blocks and the condition of the surfaces between these blocks. This method has been modified over the years in order to meet the needs of users and cases that were not initially considered. The “Geological Strength Index”, GSI, as a tool for such an assessment, was introduced by Hoek (1994) and developed by Marinos & Hoek (2000) and Hoek, Marinos & Marinos (2004).

Basic inputs of the Hoek and Brown failure criterion are estimates or measurements of the uniaxial compressive strength (σ_{ci}), the material constant (m_i) that is related to the frictional properties of the rock and of the GSI. The Hoek-Brown constant m_i can only be determined by triaxial testing on core samples or estimated from a qualitative description of the rock material. This parameter depends upon the frictional characteristics of the component minerals in the intact rock sample and it has a significant influence on the strength characteristics of rock. When such tests are not feasible, representative values for each intact rock may be chosen from the literature, (Hoek & Marinos, 2000). The Geological Strength Index can be estimated directly from the relevant charts. In the case of flysch formations, GSI value can be derived from the GSI chart for heterogeneous rock masses that is defined in Figure 4.

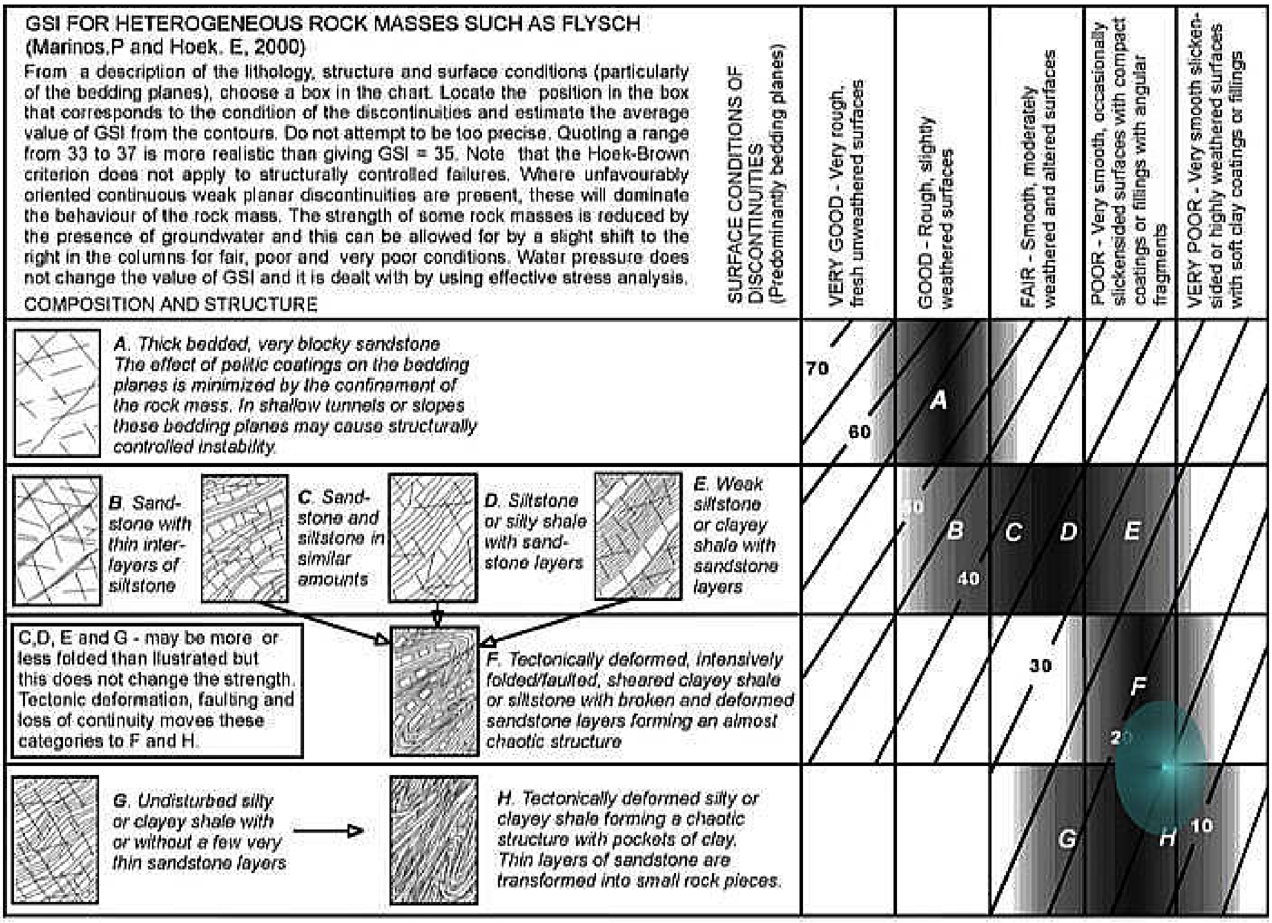

Figure 4. GSI for heterogeneous rock masses such as flysch (Marinos & Hoek, 2000). The area defined by the ellipse indicates the characterization limits of the described geomaterial

In addition to the GSI values presented in Figure 4, it is necessary to consider the selection of the "intact" rock properties σ_{ci} and m_i for heterogeneous rock masses such as flysch. Because the sandstone layers are usually separated from each other by weaker layers of siltstone or shales, rock-to-rock contact between blocks of sandstone may be limited. As a result, the outcropping geological materials do not permit high quality sampling and consequently performance of laboratory tests to derive the design geotechnical parameters is difficult. Therefore, it is proposed that a 'weighted average' of the intact strength properties of the strong and weak layers should be used (Marinos & Hoek, 2000).

One of the few courses of action that can be taken to resolve this dilemma is to use the Point Load Test on samples in which the load can be applied normal to bedding or schistosity. The uniaxial compressive strength of the intact rock samples can be estimated, with a reasonable level of accuracy, by multiplying the point load index $I_{s(50)}$ by 13, (Tsiambaos & Sabatakakis 2004), for $I_{s(50)}$ values up to 2 MPa, where $I_{s(50)}$ is determined according to ISRM (1985) suggestions. The triaxial cell, known as the Hoek cell, was developed specifically for testing under triaxial compression rocks such as siltstone, shale strata that are sensitive to desiccation. Nevertheless, when laboratory testing is not possible, point load tests, should be carried out as soon after core recovery as possible in order to ensure that the moisture content of the sample is close to the in situ conditions.

Indeed, there are certain problems in measuring the uniaxial compressive strength obtained from laboratory testing. Firstly, it is extremely difficult to recover core of "intact" and representative rock elements from the heterogeneous and tectonically disturbed rock mass and to prepare specimens for laboratory testing. Secondly, even if specimens are prepared and tested successfully, the nature of the materials means that there will be a large scatter of the uniaxial strength values determined from these tests. The first problem is quite difficult to solve since an examination of the core from any drill hole in a disturbed flysch will reveal a significant heterogeneity and the presence of closely spaced bedding planes and possibly other discontinuity surfaces. Hence, almost any uniaxial test on a typical laboratory specimen (50 to 100 mm diameter core) will contain elements of the "rock mass" and will not be representative of the uniaxial compressive strength of the intact rock components. Consequently, any laboratory tests carried out on core samples will be more representative of the rock mass than of the intact rock components. Using the results of such tests in the Hoek-Brown criterion will impose a double penalty on the strength (in addition to that imposed by GSI) and will give unrealistically low values for the rock mass strength (Hoek & Marinos, 2000). On the other hand, when samples can be prepared these are generally of high strength rock and may not be representative of the weak heavily sheared product that embraces them.

The heterogeneity and the complexity of a sheared flysch are shown in Figure 5. It is comprised of either siltstones and clayey shales on the one side or alternations of siltstones and thin sandstone beds on the other side. It is particularly difficult to recover and prepare competent samples to measure the ¨intact¨ properties of these geomaterials.

Figure 5. Heavily folded alternations of grey and reddish siltstones with thin sandstone beds showing the heterogeneity, the complexity of the formation and the difficulty of estimating the ¨intact¨ rock properties

Impact of groundwater on the mechanical properties

The influence of groundwater on the behaviour of the rock mass surrounding a tunnel is very important and has to be taken into account in the estimation of potential tunnelling problems.

When the water is not drained it reduces the effective stresses and thus the shear strength along discontinuities and finally, in all cases, the strength of the rock mass. In addition, particularly important when dealing with shales, siltstones and similar rocks is that they are susceptible to changes in moisture content, which directly affect their strength. Furthermore, many of these materials will disintegrate very quickly if they are allowed to dry out after removal from the core barrel. Hence, testing of the "intact" rock to determine the uniaxial compressive strength σ_{ci} and the constant m_i must be carried out under conditions that are as close to the in situ moisture conditions as possible.

This deterioration, in such geomaterials, may also take place during the underground excavation as long as the surrounded rock mass has deformed and loosened. During excavation, the surrounding groundwater conditions change due to the drainage or even with the time-dependent change of moisture content. Thus, the rock mass deteriorates since it has been "relaxed" and the strength parameters are reduced.

All the above phenomena are particularly true in the case of a sheared flysch.

Case study 1-2

During the geotechnical site investigation uniaxial compression and point load tests across the core diameter and perpendicular to bedding or any discontinuity were performed to determine the uniaxial compressive strength σ_{ci} of the intact rock. In Figure 6 the statistical distribution of σ_{ci} for both case studies is presented. The results of case study 1 refer to weak siltstone, whereas the ones of case study 2 refer to weak siltstones of varying quality. The uniaxial compressive strength of these geomaterials from point load tests was calculated by multiplying $I_{s(50)}$ by 13 since the tests gave index values lower than 2MPa.

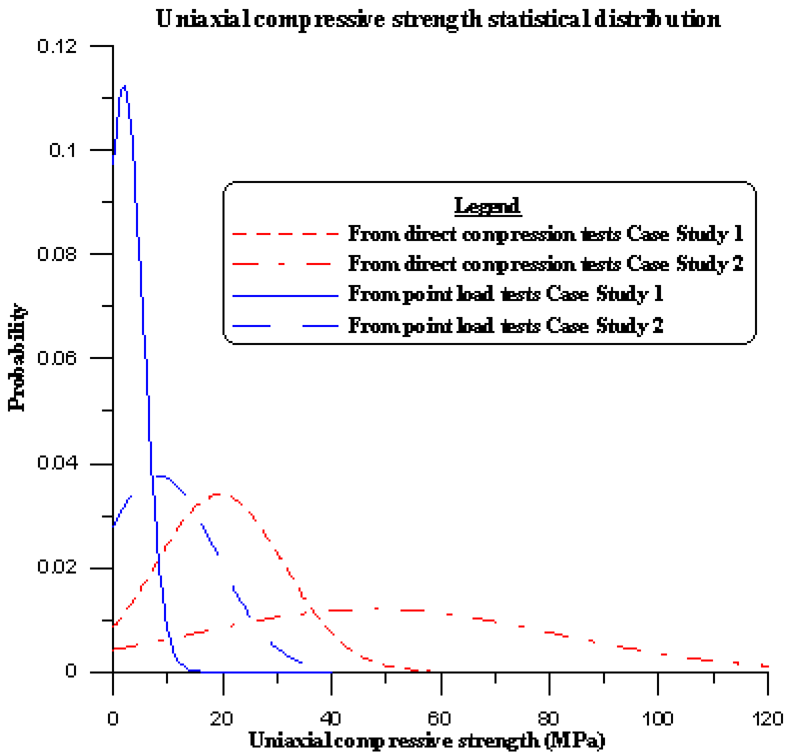

Figure 6. Statistical distribution of uniaxial compressive strength results derived either by direct laboratory testing or by point load tests

In case study 1, the statistical distribution of the point load test results was determined taking into account 25 specimens. The mean value of uniaxial compressive strength derived by point load test ($Is_{(50)}$ x 13) of this sample is 1.9MPa with a standard deviation of 3.55MPa. On the other hand, the sample for uniaxial compression test is considerably smaller, since only 5 specimens of the specific material were tested measuring the mean value of the sample to be 19.5 MPa with a standard deviation of 11.7 MPa. In the second case study, the statistical distribution of the point load test results was determined taking 44 specimens into analysis. The mean value of uniaxial compressive strength derived by point load test ($Is_{(50)}$ x 13) of this sample is 8.2MPa with a standard deviation of 10.6MPa. The sample of the uniaxial compression test was considerably smaller, since 8 specimens of the specific material could be tested. The mean value of the sample is 47.8 MPa with a standard deviation of 33.1 MPa.

As it is shown in figure 6 there is high uncertainty in choosing a σ_{ci} value for the design, since we understand that due to the very weak character of these materials it was not possible to test a large amount number of specimens since samples cannot be prepared, typical for the poor quality, due to infirmity to form adequate cores. Because of this difficulty it can be assumed that specimens were chosen from sections where stronger siltstones dominated, especially as far as the uniaxial compression tests are concerned. This is obvious from the samples tested for the uniaxial compressive strength determination that give much higher values than the ones expected for a heavily sheared siltstone. Additionally, a very important statement is that the scatter of laboratory testing results for this material is very wide, because of its chaotic structure of the mass from where the specimens were taken, as well as their desiccation which takes place when they are allowed to dry out after removal from the core barrel.

According to the former, uniaxial compression and point load testing results should only be taken into consideration only when special care and attention is given during the sampling, using certain types of samplers, and the transportation of the samples to the laboratory so that the hazard of a change in the material's moisture content is drastically limited. In other cases, the results are not reliable and their evaluation proves to be problematic.

Design geotechnical parameters

The range of the engineering geological parameters to calculate the geotechnical strength values that were used in the design for sheared siltstone in both case studies was 15-25 for GSI; 8-9 MPa for σ_{ci}; 6-8 for constant m_i and γ=0.027MN/m^3 for unit weight. The overburden height, for which the support categories were designed to adequately support this specific geomaterial, was 90 m for case study 1 and 150m for case study 2.

BACK ANALYSIS

Method of analysis

As discussed previously, the lack of reliable laboratory test results for intact properties in such a material makes the back analysis as the most reliable method to determine these properties and also the rock mass characterization values (e.g. GSI) also. However, since there is no closed-form analytical solution that relates these parameters with the displacements of a supported tunnel shell as the construction advances, this type of analysis is carried out using the finite element method. In this paper this type of analysis is used to determine the parameters of heavily sheared flysch, utilizing the convergence measurements along the two tunnels of Egnatia Motorway mentioned above in selected stations during the excavation.

The interpretation of data of the tunnel deformation is a laborious task and contains a lot of uncertainties and difficulties obtaining the "appropriate" deformation measurements for the back analysis. These uncertainties depend mainly on the construction sequence, the rate of advance, the geometry of the tunnel closure and the in-situ behavior of the rock mass. Analytically, monitoring stations and time were chosen in respect to a logical low rate of advance and little or no disturbance by the advance of the twin bore that followed. Furthermore, the uniformity of the closure was carefully examined and sections with large differential movements, such as settlements, were avoided. Finally, considerable effort has been made to exclude possible time dependent effects of the rock mass. This was accomplished by excluding measurements from targets that were placed in distances from the excavated face, which appeared to be longer than the ones theoretically required for the rock mass to reach full confinement.

Back analyses were held, aiming more to give a close range of values of for the parameters GSI, σ_{ci} and m_i, rather than to determine them with unique values. Previous experience with this weak material suggested a range of values for these parameters. Thus, GSI values range from 10 to 20, σ_{ci} 3 to 7 MPa and m_i 4 to 7. Three different models were generated, to approach three groups of monitoring targets in specific positions, two in case study 1 and one in case study 2. The models consisted of a certain number of stages, in order accurately to simulate with significant accuracy the construction sequence in each position before, at the moment, and after the placement of the targets, until the full confinement of the modeled bore tunnel was achieved.

As far as the in situ stresses are concerned, it was decided to perform an analysis of the ratio of horizontal to vertical stress K_o value of 0.6 and 0.7 for a more sensible realistic analysis of the deformation measurements.

Numerical Analysis

During the excavation of both tunnels, significant problems of overstressing of the primary support in some stretches of the tunnel occurred. To determine the parameters of the heavily sheared flysch through the back analysis, three monitoring stations were chosen. An example of vertical displacements that were included in this analysis is presented in figure 7.

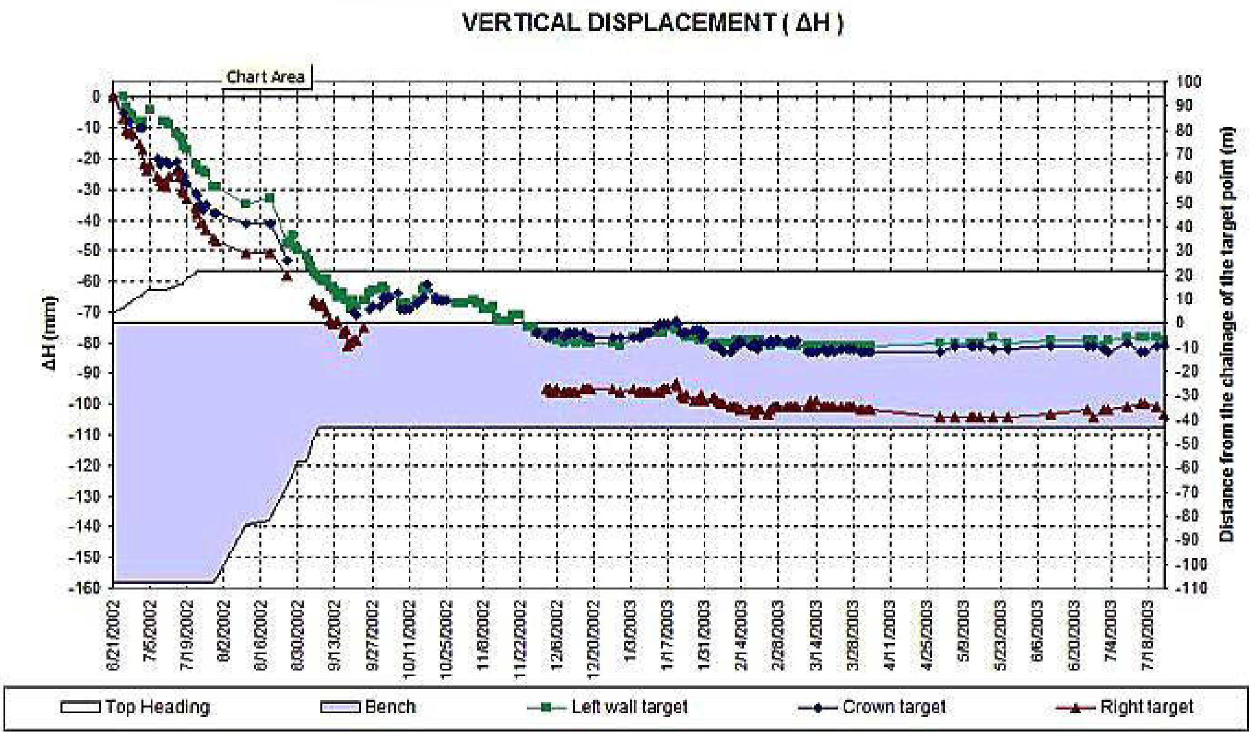

Figure 7. Example of vertical displacements on a monitoring station that was modelled in back analysis (data from the construction records of the tunnel)

Support measures

For case study 1 two typical support categories were modeled. The first one consists of a heavy forepole umbrella and a significant number of fiberglass nails to improve the face stability, a very dense grid of fully bonded 6m long anchors embedded in a HEB180 steel set with embedded 250 mm thick shotcrete layer. The excavation of the top heading and bench comes with temporary and permanent invert closure respectively. The second support category consists of a light forepole umbrella and a significant number of fiberglass nails. Self drilling, fully bonded anchors are used, either 12 m or 9 m long, while the 300 mm thick shotcrete layer is reinforced with 4-bar lattice girders in the foundation area of which micropiles are installed. The excavation of the top heading and bench includes a temporary and permanent invert closure respectively.

For case study 2 one typical support category was modeled. Face stability is accorded using fiberglass nails. The shell is supported with a dense grid of fully bonded 6m long anchors and HEB 160 steel sets embedded in a 25cm thick shotcrete layer. Temporary and permanent invert closure is formed as well.

The two tunnels have been successfully completed and these problematic sections were dealt with by localized re-evaluation of the support measures or the addition of some support elements.

Back Analysis description

The numerical analysis was held with the finite element program Phase2 Version6.0. The aim of the analysis was to determine the values of the geotechnical parameters of the specific rock masses so that the vertical and horizontal displacements calculated at the crown and walls of the bore tunnel are similar to the ones measured. The progress of the construction was simulated using equivalent reduced deformation modulus inside the excavation area, depending on the confinement ratio at each specific distance ahead of and behind the tunnel face, which was calculated according to the results presented by Chern theory (Chern, Shiao & Yu, 1998). The properties of the shell, as a combination of shotcrete and steel sets, were increased from stage to stage.

In Figure 8 the results given by a finite element analysis for one of the models are shown. The figure shows the vertical displacements in specific points on the shell of the monitored bore tunnel, as well as the yielded rock mass and support elements for specific parameters that are shown at the top of it.

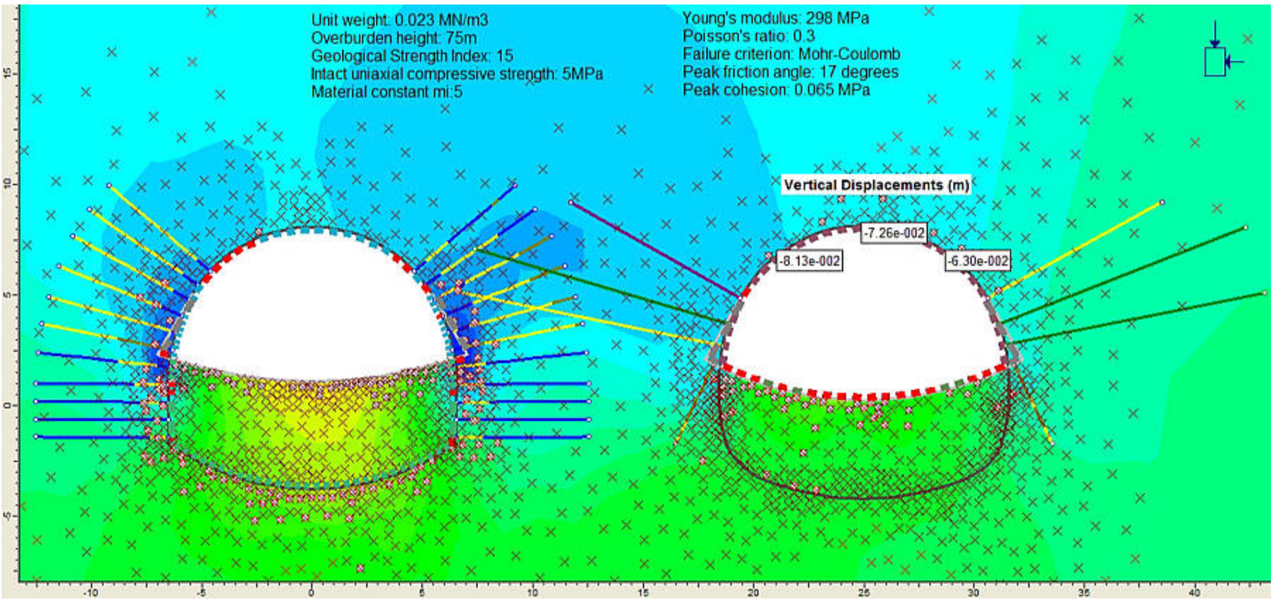

Figure 8. Indicative numerical analysis results of the case study 1 model. It is noted that in this station the support measures are different for each bore

Analysis results

During the process and because of the low values of the ratio of uniaxial compressive strength of the rock mass to the in situ stresses (σ_{cm}/p_0), the sensitivity of the deformation results given by the finite element analysis proved to be quite significant. A slight increase of the intact strength σ_{ci} or even the material constant m_i, especially when the strength as well as the GSI was given low values, could increase or decrease the deformations even by an order of magnitude. This influence resulted in relatively few and quite specific groups of design parameters and balanced the inevitable uncertainties during the assumption of the models. This sensitivity meant that the ranges of parameters required to satisfy the field observations were relatively narrow. This is an interesting and satisfying result in view of the wide range of uncertainties involved in the construction of the model.

Additionally, the empirical relationships by Marinos & Hoek (2000) and Hoek, Carranza Torres & Corkum (2002) were used to determine the sensitivity of the radial strain during the excavation due to the variation of the GSI, σ_{ci} and m_i values. For each one of the three parameters, a number of curves are generated, to show the influence of each parameter, when the other two remain constant. This is to say that for each parameter, each curve referred to a different set of the other two constant parameters. For all the possible couples of values for the constant parameters, the curves showed an area between an upper curve for the lowest values of the constant parameters, and a lower curve for the highest and lowest values of the constant parameters respectively. The mean curves, representative for each one of the areas described above, of the strain variation with a weighted value of GSI, σ_{ci} and m_i are presented in Figure 9. It is obvious that the variation of the uniaxial compressive strength, σ_{ci}, value is more significant than the variation of the other two geotechnical parameters. It can be also observed that the rate of the radial strain is increased significantly when the values of GSI, σ_{ci} and m_i become very low (for $\omega < 0.2$).

The relationships used for calculating the radial strain are the following:

$$\sigma_{cm}=\sigma_{ci}((m_b+4s-a(m_b-8s)(m_b/4+s)^{(a-1)})/(2(1+a)(2+a)))$$

$$m_b=m_i\exp(GSI-100)/(28-14D))$$

$$s=\exp((GSI-100)/(9-3D))$$

$$a=1/2+1/6(\exp(-GSI/15)-\exp(-20/3))$$

$$\varepsilon(\%)=100(0.002-0.0025(p_i/p_0))(\sigma_{cm}/p_0))^{(2.4(pi/p0)-2)}$$

where σ_{cm} is the global strength of the rock mass, m_i is the material constant of the intact rock, σ_{ci} is the intact rock uniaxial compressive strength in MPa, GSI is the geological strength index, D is the disturbance factor, ε is the radial strain of the tunnel, p_0 is the in situ stress in MPa and p_i is the internal tunnel support pressure in MPa.

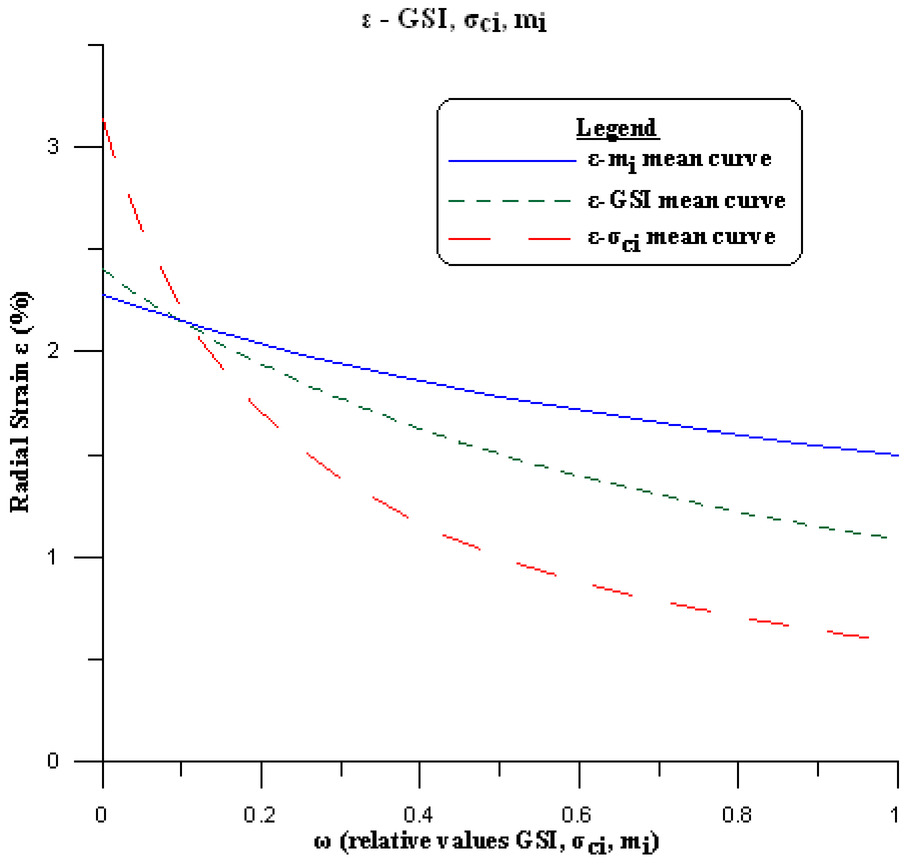

Figure 9. Sensitivity curves showing the influence of the geotechnical parameters GSI, σ_{ci}, and m_i on the radial strain, calculated for internal support pressure of 1MPa

On the vertical axis of Figure 9 the tunnel radial strain is displayed. On the horizontal axis of Figure 9 a weighted parameter ω is shown. This was created, so that all the parameters of the rock mass can be displayed on the same axis. The value of ω is calculated for every geotechnical parameter according to the relationship:

$$\omega = (X_i-X_{i,min}) / (X_{i,max}-X_{i,min}), \quad X= GSI, UCS, m_i.$$

where ω is the weighed measure, X_i is each one of the parameters separately, m_i is the material constant of the intact rock, σ_{ci} is the intact rock uniaxial compressive strength in MPa, GSI is the geological strength index. The minimum and maximum values of the geotechnical parameters used for this sensitivity analysis procedure, based on the knowledge of the nature of the geomaterial are: GSI_{min}=10, GSI_{max}=30, $\sigma_{ci,min}$=2MPa, $\sigma_{ci,max}$=10MPa, $m_{i,min}$=4, $m_{i,max}$=8.

In addition, considering the results of the above analysis, it can be seen that as the parameter, namely the values of the three engineering geological parameters, decreases, the influence of a potential change becomes more important.

To sum up, as far as back analyses are concerned, the results for both case studies were similar. The ranges of the geotechnical parameters that come up best satisfy the field observations were 13-17 for the Geological Strength Index (GSI), 4-5MPa for the intact uniaxial compressive strength (σ_{ci}) and 5-6 for the material constant m_i.

Table 1. Back analysis and concluded results of the GSI, σ_{ci} and m_i and rock mass properties derived from Roclab

	GSI	σ_{ci} (MPa)	m_i	c (kPA)	φ(°)	E(MPa)
case study 1*	12-15	3-5	5	50-70	13.5-17	325-425
case study 2†	15-17	5-6	5-6	90-120	14.5-17	425-510
Sheared Flysch‡	13-17	4-5	5-6	60-85	15-18	355-510

*c and φ estimated for an overburden height of 90m
† c and φ estimated for an overburden height of 150m
‡ c and φ estimated for an overburden height of 100m

It should be noted that the resulted parameters from the back analysis refer to the 'intact undisturbed" rock properties of the heterogeneous rock mass including both the sheared siltstone and the sandstone floaters. These properties should be finally used for the design of the tunnel support measures. These results are in agreement with another case study, in similarly sheared flysch, described by Tsatsanifos *et al.* (2000).

CONCLUSIONS

Tunnelling in weak geomaterial, derived by thrusting of flysch formations, is always a challenge for engineering geologists and tunnel engineers. For geologists, this challenge is induced by the difficulty of mapping shear zones, mainly when they are spread over hundreds of meters, but also on testing them either in the field or in a laboratory.

Such rock masses are composed of inherently weak materials that have been heavily sheared to the point where the original structure of the rock mass is no longer recognisable. These masses exhibit squeezing behaviour even under a cover of only tens of meters and require numerical analysis for the design of the support measures. The rock mass strength parameters needed for such an analysis can be sufficiently estimated by the Hoek and Brown failure criterion as long as the rock mass reacts isotropically to the underground excavation. Basic inputs of this criterion, except the GSI value for this 'pseudo-isotropic mixture', consist of the GSI value, the uniaxial compressive strength of the intact material (σ_{ci}) and a material constant (m_i).

Regarding the values of the intact strength σ_{ci}, the chaotic structure and the sensitivity to the environmental factors that characterize the heavily sheared flysch demand high quality laboratory testing standards in order to acquire representative values, as a wide scatter of values can easily be present.

Back analysis, whenever possible, is the most reliable tool for the determination of the rock mass properties, especially when such very weak and sheared geomaterials are encountered and problems like the ones described above occur. It is of great importance though, to be aware of the uncertainties and the limitations of this procedure.

The two tunnel case studies presented have provided valuable data for the geological nature of the sheared flysch and its behaviour during tunnelling. Though shear zones were generally well predicted during the design, a better understanding of the tectonic evolution is always displayed with 3D conceptual models constructed after completion of the tunnels.

Deformation measurements were used for different sections of the two tunnels respecting the uncertainties and difficulties such as the construction sequence, the rate of advance, the geometry of the tunnel closure and the in-situ behavior of the rock mass. The results from the back analysis, using the numerical program Phase^2 v.6.0, showed that the ranges of engineering geological parameters for the described geomaterial range for GSI: 13 – 17, for σ_{ci}: 4 - 5MPa and whereas for the material constant m_i: the values 5-6 appear to be the most appropriate. It should be noted that the strength of the intact rock, in siltstones or clayey shales, can be further reduced during excavation, due to desiccation and deterioration of the surrounding rock, and can endorse further deformations.

A sensitivity analysis performed to display how these parameters affect the radial strain of a tunnel, showed clearly that the variation of the uniaxial compressive strength is more significant than the variation of the other two parameters. Furthermore a slight increase of the intact strength σ_{ci} or the material constant m_i, as well as the GSI (if there is a value change of over 5), especially when the values are low, could increase or decrease the deformations even by order of magnitude.

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