1. Introduction

Three different tunnels have been driven in the Beskydy Mountains (PL, CZ, SK) in the geological environment of the Outer Carpathians Flysch: the Polana tunnel exploratory gallery (Slovakia, 28sqm, length 400m, depth 10-40m), the Laliki motorway tunnel (Poland, 115sqm, length 700m, depth 7-35m), and the Jablunkov railway tunnel (Czech Rep., 110sqm, length 600m, depth 6-25m). In spite of the fact that each tunnel is situated in a different country, their distance from each other never exceeds more than 30km. This makes it possible to give some general recommendations for tunnelling in the Outer Flysch. This article presents the geotechnical risks met throughout the project phases: geotechnical survey, data evaluation, design, and construction.

2. Geological Conditions

The wider environment is in the Outer Western Carpathians, which consist mainly of sediments of the Flysch character (rotation of claystone, siltstone, sandstone and conglomerates), represented by the Silesian and Raca Units. Both units form individual nappes shifted over each other, with the Magura Nappe. The complex structure is accompanied by tectonic faults.

From the engineering geology point of view, the Flysch complex is a typical landslide area. The routes of the tunnels are situated in the upper part of the Silesian unit from the Palaeogene period (30-45 million years), which consists mostly of claystones with flint and sandstone layers. The tunnels are driven in unfavourable geological conditions in the strata of a cyclic Flysch with a dominance of calcified claystones with very low or extremely low strength (R5-R6). The claystones are strongly weathered; they change to stiff clays towards the overburden (F8 after ASTM classification). The rock material falls apart into a fine soil after excavation. Water ingress was not detected; the tunnel face was dry to moist. For the amount of RMR points (< 25 points) the rock mass is valued as poor rock to very poor rock (after Bieniawski – RMR).

3. Polana Exploratory Gallery, SK

The first tunnel built in the region of the Outer Carpathian Flysch was the Horelica tunnel, finished in 2003. The extremely difficult geology both in the portal and tunnel vicinity caused the National Motorway Society of Slovakia to explore the geological and geotechnical conditions in the vicinity of the next intended motorway, the Polana, by excavating an exploratory gallery.

The Polana exploratory gallery was designed in the profile and route of the future emergency gallery for the motorway tunnel. The length was intended to be 600m of the entire 800m of the
future tunnel, with a cross section of 28sqm, and the depth below the ground surface reaching 10 to 40m.

3.1. Polana Exploratory Gallery – Excavation, Monitoring, Testing

The excavation technology was done according to NATM, where the lining and support strictly corresponded with the geology and monitoring results. The lining consisted of 230-300mm shotcrete dependent on deformational response, a double layer of wire mesh, rock bolts – 3m, spilling optional. Excavations proceeded as gently as possible – with a tunnel excavator using a rock hammer. Unfortunately, limited blasting had to be used in the vicinity of solid sandstone layers. This was later identified as critical. Solid sandstone layers were often surrounded by laminated claystones. If the hard sandstone layer was located only in the tunnel face, not above the tunnel roof, the blasting disturbed the claystone above, which fell, creating large overbreaks. This happened under a low overburden in weathered rock – to 15m below the ground surface. Furthermore, blasting significantly changed the natural stress status in the surrounding rock mass and disabled intended exploratory work. Fortunately, the blasting was only used for very limited chainage of the excavation.

Higher and progressive deformation of the lining was observed along fault zones in the area of higher overburden >30m below ground surface. Neither the shotcrete thickness nor the number or length of rock bolts helped to control the progressive convergence. Only the invert closure stopped the deformation and stabilized the lining.

An enlarged cross section of 72sqm – a top heading profile of the future motorway tunnel – was excavated in a 30m-long area, at a depth of 30m below the surface. The combined monitoring profile was situated there and in-situ testing was done.

The geotechnical monitoring included convergence, loads on lining, lining stresses, extensometers, and levelling on the terrain. The combined monitoring profile consisted of these measurements. In addition, monitoring was done using inclinometers and deformation of portal walls was checked in tunnel entrance area. The combined monitoring profile was situated in the enlarged section. Boreholes for extensometers were drilled in a fan pattern. Before extensometers were installed into the boreholes, pressure meter in-situ testing had been executed there to check the initial stress state in the surrounding ground. Pressure cells for stress and strain loads in the lining, convergence and levelling were also measured in the same chainage. It was identified that a natural vault (unloading zone) was generated at the distance of 4-6m beyond the tunnel lining, full development came after 3 excavation steps (4m = 0.7 x height of the section).

\[ E_{\text{def,per}} = 0.3-0.5 \times E_{\text{def,par}} \]

where \( E_{\text{def,per}} \) is the modulus of deformation in the direction perpendicular to bedding, \( E_{\text{def,par}} \) is the modulus of deformation in the direction parallel to bedding [2].

The sets of specimens were picked from the gallery face to check the geotechnical parameters. The homogenous units were determined on the basis of face mapping, monitoring results, lab and in-situ tests. The geotechnical parameters of the “homogenized” rock mass units were based on the above-mentioned tests and the Hoek-Brown theory [1].

The ground water represents its own phenomenon in the Flysch environment. The mass of Polana Hill is the water source for a neighbouring village. Hence, detailed hydro monitoring was carried out throughout the project. Despite the fact that relatively high water inflows were monitored in the gallery face from time to time, the water sources remained uninfluenced. The water inflows were closely associated with tectonic faults or saturated sandstones surrounded by watertight claystones. Obviously, excavation of the tunnel can drain water sources but only to limited extent. Several saturated zones are isolated by the layers of claystones. Global drainage of the overburden is not the case for the Flysch.
The gallery excavation was ended after 400m in fairly good rock mass with typical rhythmic alternation of more solid tables of sandstones (R2) and tabular to laminated claystones (R5-R6). The next stage of the geological survey focused on horizontal boreholes drilled both into the gallery face and the gallery side towards the future motorway tunnel. The directions of the boreholes were preferred to be as perpendicular to the bedding as possible to predict the geological situation in as large area as possible. Another set of pressiometric tests was done in the boreholes. Additional boreholes were drilled in the opposite portal. To set the global dip and strike situation in Polana Hill, the set of simple ditches was excavated along the entire tunnel route. This very simple tool provided an extremely useful and satisfactory overview of the global bedding and the main fault direction phenomena.

4. Jablunkov Tunnel, CZ

The reconstruction of the Jablunkov tunnel should be understood more as driving a new tunnel. The new tunnel follows the track of the original northern tunnel tube due to lack of space in the mountain pass and to decrease the total costs of modernization for the entire railway. The original stone tunnel lining is partly turned down and partly a component of the new primary lining. The original southern tunnel tube is to be an escape gallery. The new 612m-long tunnel is built using the NATM method. The southern historical tunnel (from 1871) will partly serve as an escape gallery and will partly be back-filled.

4.1. Jablunkov Tunnel – Geological Aspects

During a recent excavation, laminated dark-grey claystones were found, slightly folded, partly schistose, or even totally crushed. Claystones contain fine irregular fillers of siltstones or sandstones, with a thickness up to 5cm (Fig. 4). Stratifications with medium declination towards south-east predominate; the schistosity has a steep declination with a predominant east-west direction. The excavation headed towards the north. The rock mass was heavily broken by several tectonic faults represented by “tectonic mirrors” – a completely slicken slide surface of foliation up to a glass structure. The dip and strike of main discontinuities changed rapidly in synclinals and anticlines.

4.2. Jablunkov Tunnel – Stability Aspect

The top heading excavation (58sqm) started at the strongest NATM class of 5a, which consisted of 350mm shotcrete, double wire mesh, 6m long rock bolts, spilling optional and enlarged footing.
(elephant foot). The deformation values of the primary lining were different than expected at the start of the tunnel top heading excavation in April 2008. The mechanism of the deformation itself was similar, as expected. The left footing of the tunnel was pressed into the rock mass and the minimal value of deformation occurred in the right part, where the new lining was footed into the historical rock-block lining (Fig. 2), but convergence profiles at approx. 30m of excavation had unexpected steep settlement curves and a reduction did not occur even after more than one week, i.e. the tunnel face was 15-20m away from the profile. In the case of these profiles, the settlement limit of 50mm given by the designer was many times exceeded (Fig. 3). All monitoring results were the base for the following extra measures (widening of the footing – “elephant foot”, side micropiles 6-9m, rock bolts, modification of the top heading invert geometry) [4]. These had only one aim – to stop the extreme settlement of the left footing of the tunnel top heading. Unfortunately, all the extra measures used had only a limited effect. The aim of fixing the settlement was not exceeded. The temporary invert closure through the entire top heading seemed to be the only measure that finally worked well and stopped deformations. The shape of the top heading and its invert closure was modified and a new set of NATM classes was established.

Class 5a stands for excavation of the top heading in full profile with immediate invert closure, i.e. creation of a closed ring as fast as possible. Class 5b stands for a vertically divided face of the top heading into the left and right partial parts with gradual invert closure. The 5c class stems from 5b, when the pipe umbrella is added. This class was designed for excavation in the most difficult geological conditions, such as when strongly weathered rock mass is affected by inflows of water. The lining thickness and reinforcement was the same for all classes, 5 to 5c.
It is clear now that this type of support was able to eliminate the negative deformation behaviour of the primary lining. Significantly, convergence results were eliminated. Settlement and lateral deformation were kept at acceptable values.

![Jablunkov Tunnel: typical geology](image)

**Fig.4:** Jablunkov Tunnel: typical geology

### 5. Laliki Tunnel, PL

The new Laliki Tunnel was designed as a double-lane motorway tunnel, with a cross section of 114sqm and depth of 7-35m below the surface. Only one tunnel tube was excavated; the emergency gallery (cross section of 28sqm) was excavated along the entire length of the intended future second tunnel tube. The distance between the tunnel and gallery is 30m in axis.

#### 5.1. Laliki Tunnel – Geological Aspect

The excavation started from the southern portal. The tunnel profile was divided horizontally into the top heading (72sqm), bench (21sqm) and invert (22sqm). Typical Flysch geology was met during the first 70m of excavation – periodical alternation of foliaceous to laminated claystones (R5) and foliaceous sandstones (R2), and the strike and dip of the main discontinuity system was 150°/85° (Fig. 5). Systematic forepoling of 4m-long spiles were used to keep the roof stable. The face itself was also sufficiently stable. The excavation continues safely with good progress using the measures described in the rock mass valued RMR>20 with no or limited tectonic faults. Local face and roof instability were determined by water which accumulated in the contact zone of permeable sandstones and impermeable claystones. Over breaks of max 2cubm appear there.

The tunnel face became unstable when laminated claystones were weakened by tectonic faults (Fig. 6). Systematic spilling of 6m-long spiles every 2nd step was designed originally but this measure was not sufficient. The face was divided into 6 parts, excavated separately and solid face-supporting wedges were used. Despite the fact that this procedure enabled safe excavation the progress decreased to minimum. Micropiles forepoling (12m long, 4m overlapping) the tunnel roof were used after difficult negotiations with the client. This only extra measure provided such a roof and face support that excavation of the top heading could progress in the full face and sufficiently in advance. The appearance of various tectonic faults in laminated claystones with very limited natural strength shaped most of the tunnel face and roof stability problems. This broken rock mass was valued at RMR<20.
The fact that the real geological and geotechnical behaviour of the rock mass was not as expected one was determined soon after excavation started. The determining difference was in the relative appearance of claystones and sandstones, when claystones were underestimated, and laminated claystones were missing completely. Last but not least, the difference represents an entire lack of a description of the tectonic faults, namely their quality, appearance and the dip and strike. The faults determined face and tunnel stability. Hence, a set of rock specimens were taken from the tunnel faces for lab testing. A large-scale box-shear test was also applied to recomposed rock material in the lab. Furthermore, a set of in-situ testing was implemented on the rock mass, mainly a pressiometric test of rock mass in bores drilled in the tunnel face. The large-scale shear test results correspond best with the in-situ testing of the rock mass [3]. Lab and in-situ testing together with tunnel face mapping was analyzed in detail using the Hoek-Brown theory [1]. Interpretations of all results led to correction of parameters so that the new calculation could be done. The new calculation clarified the primary lining design, estimated the rock mass response for excavation and set the optimal excavation procedure and progression.

5.2. Laliki Tunnel – Stability Aspect

The primary lining consisted of 200-300mm thick shotcrete, double layer wire mesh, 4-6m long rock bolts, spilling optional. Initially, the top heading should have been excavated with no invert closure; only enlarged footing (elephant foot) was recommended in low quality rock mass. If the overburden did not exceed 15m, the elephant foot worked well, lining deformations (settlement predominantly) fixed fast at the distance 15-30m from the tunnel face – 2-3 tunnel cross sections. If the overburden exceeded 15m, lining deformation did not fix and continued to progress. Enlarging of the elephant foot or longer and more rock bolts were used but almost no effect was monitored. The proper extra measures were selected according to experiences in the Jablunkov tunnel (see chapter 3.2). The temporary invert closure of the top heading was executed at a distance of 6m from the tunnel face. This measure worked immediately, the lining deformation fixed immediately – within 3 days after the temporary invert closure.
Bench excavation was followed by the same progress of deformation as occurred in the top heading (Fig. 8). Deformation started to progress once the temporary invert closure in the top heading had been cut off. The settlement fixed after the tunnel invert was again closed completely. Hence, the bench and invert advance were detailed so that the entire tunnel invert closure was completed within 16m after the top heading temporary invert was cut off. Only this procedure enabled control of the lining deformations and kept them at satisfactory limits. In the opposite case, cracks appeared in the lining and deformation exceeded the threshold limits without trends to fix. Additionally, extra rock bolting was used for the bench area. This extra measure only had a subsidiary effect in decreasing the speed of deformation, not fixing it. Rock bolting was understood to be a measure that increased the cohesion parameters of the surrounding rock mass. Furthermore, cohesion of rock bolts and laminated broken claystones was doubtful. Fast invert closure of all partial excavations was the main measure able to control and stop the progress of lining deformation.

The rapid invert closure seems to be in opposition to one of the NATM principles. It puts a lot of effort on the tunnel lining, its reinforcing and dimensions. Analyzing the rock mass monitoring system – extensometers and levelling on the surface – it was determined that no natural vault developed above the tunnel in the Flysch environment. Despite the overburden height of 35m, no natural vault developed and the tunnel lining was loaded with the full weight of the overburden.

The asymmetrical loading of the tunnel lining is an additional phenomenon. After the results of the loads on lining and lining stresses, asymmetrical loading was observed. The direction of the load followed the main discontinuities and tectonic faults respectively. This phenomenon was accentuated in profiles where the intersection of several faults was observed. It is not an extraordinary phenomenon. Obviously, Flysch is fault-prone environment; especially with slicken slide problems. Instabilities develop on predisposed zones, mainly on tectonic faults or on contacts of claystones/sandstones. The change of water balance in the rock mass initiates increasing pore pressure, loss of strength in predisposed zones and the sliding movement of a large amount of ground. The communication of the ground water with surface water is significant; saturation of faults with rain water is considerable. This phenomenon was observed within periods of heavy rain or melting snow vs. dry rain-free periods, when lining loads/stresses and deformations slightly increased/fixed based on the climate.

Last but not least, the complex monitoring system was the only tool to modify the excavation in advance and the design of the primary lining during project progress. Geotechnical monitoring
was implemented both inside the tunnel tube (lining deformations, loads on lining, stresses in lining), on the surface (levelling, extensometers) and in portal areas (deformations of portal walls, inclinometers). Individual monitoring points were put together in combined profiles. It made it possible to understand the system of the rock mass – the tunnel lining in all the details: the scheme of tunnel lining loads, lining stability and capacity, deformations in surrounding ground, appearance of natural vault in rock mass, surface reaction to excavation underneath. If the monitoring had been concentrated only underground inside the tunnel the results would have been limited. Without information about lining loads, surrounding rock mass and surface deformations, the correct loading schema, lining design and excavation procedure would not have been done, especially in the case where existence of predisposed fault zones determined asymmetrical loading.

6. Conclusions

The tectonic phenomena of the rock mass, discontinuity conditions and orientation, their changes along the tunnel route, the proportional appearance of solid sandstones and broken claystones are the determining aspects for tunnel stability. The fundamental geotechnical risk is the ratio of tectonic disturbance of the rock mass that could cause total loss of the natural strength in extreme cases. Predisposed shear zones with non-cohesion and very limited shear strength go hand in hand with the fault phenomena.

Geological and geotechnical surveys for tunnel projects should be focused on the phenomena described above. It is especially convenient to focus on in-situ testing, investigate the quality and the orientation of discontinuities and determine the principal appearance and quality of fault zones. The large-scale box-shear testing seemed to be in high accordance with the in-situ tests. It is recommended that mineralogical tests are implemented to set up detailed stratigraphy research. Detailed knowledge of regional geology shows the tectonic history and ratio of disturbance of the rock mass.

The behaviour of the rock mass along the predisposed fault zones is the determining geotechnical risk during the excavation process. The loading is often asymmetrical; despite the high overburden no natural vault develops in the mass of broken claystones and the lining is loaded with the weight of the entire overburden. A natural vault only develops in a relatively undisturbed rock mass. Immediate invert closure of the entire section in all excavation steps is the only tool to control and stop rapidly-developing deformations in the tunnel. Rock bolting has limited effect with respect to poor cohesion of the bolt with broken claystone; elephant foot, sometimes supported by micropiles, has no function in a heterogeneous rock mass. Ground water is associated with discreet zones along tectonic faults or along saturated sandstone. The ability of broken and shifted claystone to saturate water slowly is one of the most dangerous phenomenon. Absorbing water, broken laminated claystone loses its shear strength up to an unexpected collapse.


