FIGHT AGAINST THE NATURE - ŽILINA TUNNEL

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Abstrakt
Dva tubusy tunelu Žilina byly plánovány pro ražbu pomocí zásad NRTM. Zastižená hornina se ukázala být vysoce stlačitelná a současně nesoudržná. Plánované primární ostění nemohlo nalézt účinnou kontrolu nad ražbou tunelu. Opatření nedokázala snížit vertikální poklesy ostění, které dosahovaly hodnot až 200 mm a nedokázala stabilizovat rozvolněnou horninu vypadávající z čelby tunelu. Ve vyšším nadloží byly deformace ostění úspěšně udržovány v bezpečných mezích vytvořením dočasného dna v kalotě a instalací tunelového dna v krátké vzdálenosti od čelby. Přestože celkové hodnoty poklesů ostění zůstaly poměrně vysoké, vyvážená podpora obou invertů umožňovala kontrolu rychlosti nárůstu deformace v ostění. Hledání spolehlivého řešení pro stabilizaci čelby se stalo rozhodujícím faktorem pro dokončení tunelu. Řešením byla kompenzační injektáž o tlaku 20 barů, která stlačením zvýšila tuhost horniny. Úspěšná opatření však vedla k pomalejšímu postupu ražeb. Bylo provedeno několik pokusů o zvýšení rychlosti. Nicméně praktické testy a analýzy dospěly k závěru, že metoda horizontálního členění výrubu s včasným uzavíráním tunelového dna a kompenzační injektáží byla nejvhodnější metodou pro dané podmínky.

Kľúčové slová
NRTM, poklesy ostění, stabilita čelby, kompenzační injektáž.

Abstract
The twin two-lane highway Tunnel Žilina was planned to be mined using the NATM principles. The encountered ground turned out to be highly compressible, and cohesionless at the same time. The planned initial support measures, could not find an effective control over the tunnel excavation. The supports could not reduce the lining large vertical settlements of up to 200 mm, and could not stabilize the cohesionless ground pouring from the tunnel face. At higher overburden the lining deformation was successfully kept within the safe limits by building the top heading temporary invert, and by installing the tunnel invert at a short distance from the face. Although the total lining settlements remained relatively high, the balanced support of the two inverts allowed controlling the speed of the lining deformation development. Finding a reliable solution for stabilizing the face became determinant for the tunnel completion. The solution was a compensation grouting of 20 bar which increased the ground stiffness by increasing the ground confinement. However, the successful support measures led to a slower advance. Several trials were performed to increase the advance speed. Nevertheless, the trials and analyses concluded that the heading and benching method with the early tunnel invert closure, and the compensation grouting, was the most appropriate method for the given ground conditions.

Key words
NATM, lining settlements, face stability, compensation grouting.

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1 Introduction

The excavation from the West portal of Žilina Tunnel was started on November 2014 as a part of the motorway D1 construction Hričovské Podhradie – Lietavská Lučka. The excavation was completed in January 2017 with the delay of 1 year.

The excavation encountered ground conditions which were not suitable for conventional tunnel construction. The conventional supports could not control tunnel deformation, and could not sufficiently stabilize the excavation face. The ground was highly compressible, and at the same time cohesionless. The tunneling works could not proceed without extensive ground improvement measures.

2 Project description

The motorway D1 is part of the international corridor E-50 (Paris - Nuremberg Praha – Brno – Trenčin – Žilina – Košice – Užhorod).

The tunnel consists of twin tunnel tubes with length of 685 m. Each tube has two lane carriageway, which is 8 m wide. The excavation width of the tunnels was 12.3 m with the excavation profile area ranging from 100 to 140 m^2.

The owner, and the operator of the motorway is the Slovak National Highway Authority (NDS), (Narodna diaľnična spoločnosť, a. s.). The Žilina Tunnel was constructed by the Joint Venture of Doprastav, a. s. and Metrostav, a. s. The tunnel construction supervision was executed by the workers of NDS.

3 Geology

The project area was formed by paleogenic claystone which was strongly weathered or decomposed with low unconfined strength of 0.5 to 1.5 MPa of the intact fragments. In the weathered zones the claystone was changed to a clay of high plasticity with stiff to firm consistency.

Weathering reached to depths of about 10 m below the bedrock surface. Influence of weathering was more pronounced in the tectonic zones, which contained small fragments of claystone rock embedded in clayey matrix, Fig. 1.

Fig. 1 Cores of weathered claystone

Ground water table was at maximum 10 m above the tunnel crown. Ground water ingress was expected in pervious tectonic zones otherwise the claystone was considered impervious.

The depth of the tunnel crown ranged from 5 to 40 m, Fig. 2.

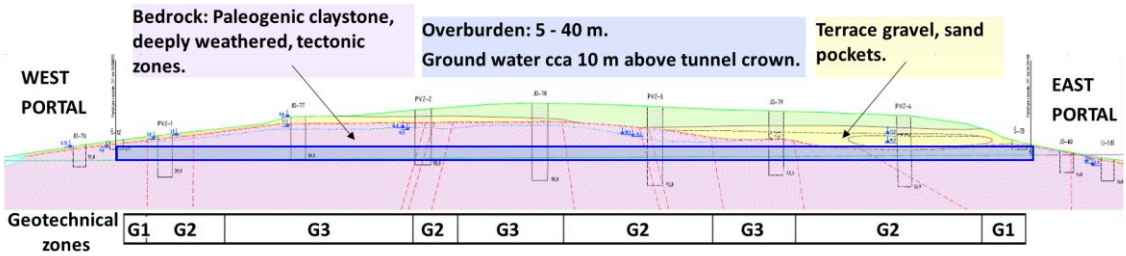

Fig. 2 Geological profile with expected geotechnical zones

At the East portal, the claystone was overlaid by quaternary deposits of terrace gravels with sand pockets, which were expected to be saturated by the ground water, and highly permeable. The terrace was expected to intrude into the tunnel crown to a depth of about 3 m into the tunnel profile.

3.1 Expected ground behavior

Geotechnical ground investigations described the claystone as a weak, and a soft ground. The description was supported by the estimated geotechnical parameters, Table 1.

Table 1. Claystone geotechnical parameters

			Geotechnical parameters			
			Expected			Encountered
Geotechnical zones			G1	G2	G3	G2, G3
Deformation modulus	(Edef)	MPa	8 - 20	20 - 60	60 - 150	20
Cohesion	(c)	kPa	10	18	25 - 50	0 - 5
Friction angle	(φ)	deg	18	24	32	20

3.2 Encountered ground behavior

In spite of the expectations the ground turned out to be highly compressible, and on top of that also cohesionless, which was an unexpected combination of ground properties.

The high compressibility was demonstrated by the tunnel lining settlements of up to 200 mm. The ground could be easily compressed by a bear hand at the open face.

The cohesionless behavior was presented by the face instability of loose ground pouring from the face, and creating overbreaks. Due to the presence of claystone fragments contained in the clayey matrix, the ground did not stick together. It was not possible to press or form a clay roll, it would easily fall apart.

The cohesionless ground at the face had always tendency to adjust the vertical excavation face into a plane with an inclination of about 55 deg. The natural face slope of 55 deg roughly indicated the friction angle of 10 deg.

Based on the ground behavior observations it was possible to conclude that the geotechnical parameters were lower than expected, both for the shallow, and the high rock covers, Table 1.

The ground water in the previous tectonic zones made the instability problems worse. Nevertheless, the groundwater water ingress was small, and water was usually dripping locally from isolated portions of the face.

A supplementary geotechnical exploration was performed, which confirmed deeper failure zone reaching below the tunnel invert.

4 Planned excavation

The gravel terrace of the East portal was considered a great risk for tunneling operations. Therefore, dewatering boreholes of 100 m long were installed to drain the terrace at the East portal, and the excavations started from the West portal

The tunnels construction was designed as a conventional excavation based on the principles of NATM (New Austrian Tunneling Method) using conventional soft ground support measures. For safe, and also economic tunnel construction the principles prescribed interaction between the ground and the support elements to activate the ground shear strength so that the ground was capable to bear a portion of the ground load induced by the excavation.

The tunnels excavations were planned under a green field, and no strict settlement limits were set. The tunnel wall deformation warning levels of 50 mm were established by the numerical models.

4.1 Tunneling classes

The claystone bedrock at the West portal was expected to be suitable for mechanical excavation. The sufficiently spacious initial lining profile of 11.5 m I.D allowed to employ large tunneling machinery. The planned heading and benching method together with the high powered machinery were predestined to good advance rates especially in the good quality claystone in deeper tunnel sections, where minimum face support measures were expected. It was planned that about 50 % of the tunnel length would be excavated in tunneling classes not needing any face anchoring or pipe umbrellas.

On the other hand, the tunnel design was prepared for the ground behavior, including the weak ground with short standup time, and soft ground providing minimum passive support to the lining. There were seven tunneling classes planned in tender documents, and in the bid documents, which included a complete range of soft ground support measures, Fig. 3: short excavation round, circular shape of shotcrete lining, tunnel invert, top heading temporary invert, elephant feet, radial bolts, face ground wedge, partial face excavation with immediate shotcrete stabilization, face anchors, spiles canopy, grouted IBO umbrellas, grouted pipe umbrella, dewatering boreholes, chemical permeation grouting (East portal).

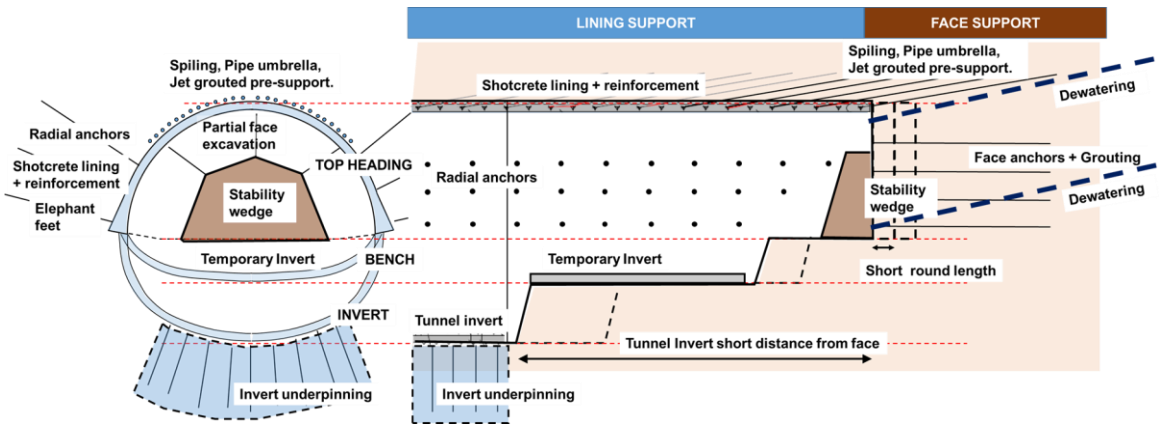

Fig. 3 Conventional soft ground support measures

4.2 Closed lining ring

The expected weak, soft ground predicted low capability of the ground to provide support at the face, and around the tunnel walls. A quick development of ground load on the lining was expected to take place in a short distance from the face. For that reason it was necessary to complete the tunnel lining with a closed invert in the shortest distance from the face. Building

the tunnel invert in a close proximity to the tunnel face was one of the main design criteria of Zilina tunnel, which assured stability of the initial lining.

The soft ground could not provide sufficient passive support for the tunnel walls, therefore the tunnel walls were designed to be oval as much as possible. A deepened tunnel invert was planned for larger ground loads of higher overburden to take advantage of an almost circular tunnel shape.

The initial lining thickness varied from 200 to 350 mm, and the excavation profile was enlarged to account for expected lining deformations.

5 Tunnel response

From the beginning of excavations from the West portal the ground behavior demonstrated that the geologic environment was one of the most difficult, and the least suitable for the underground construction. The excavations suffered from the negative behavior of:

- High initial lining settlements.
- Face instability.

In the soft and cohesionless ground the NATM principles could not be fulfilled. The ground shear strength activation had little impact since the ground strength was low, and there was hardly any interaction with the stiff, elements like radial bolts, face anchors or pipe umbrellas. The strong influence of the weak ground behavior outbalanced the ground – support interaction, in which increasing of supports quantity did not have any positive effect on tunnel stability. Improving the ground parameters was much more effective than increasing the amount of supports.

5.1 Lining settlements

The ground high compressibility was demonstrated by the substantial vertical lining settlements ranging from 120 mm to 200 mm. The top heading lining footings were being punched into the compressible ground, Fig. 4.

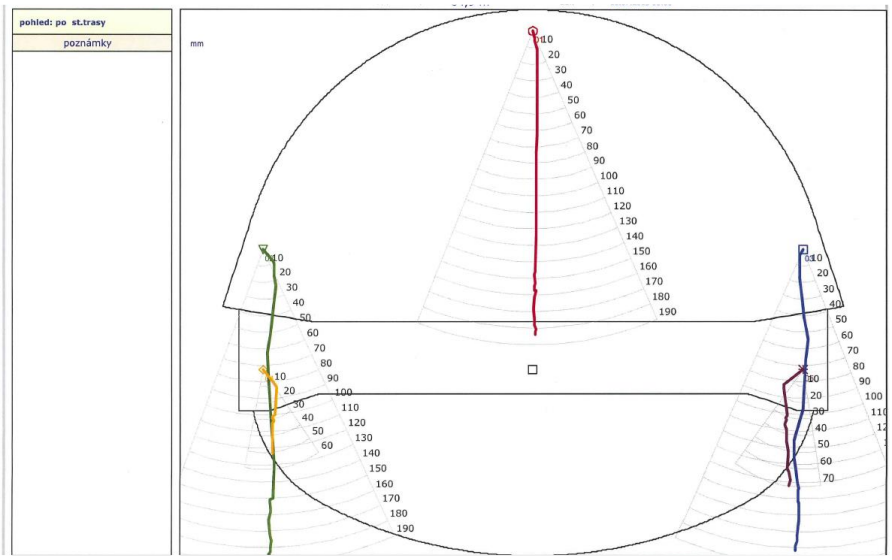

Fig. 4 Vertical lining settlements up to 200 mm.

5.2 Face instability

The excavation tunnel face suffered from frequent overbreaks which were caused by the cohesionless ground pouring down from the excavation face any time during the excavation.

The overbreaks were increasing the length of the excavation advance round, and thus exposing the umbrellas for a longer length, Fig. 5. As a result the umbrella pipes were overloaded, and bended, which allowed the ground loosening above the umbrella. The loosened ground was pouring between the umbrella pipes. That cohesionless ground behavior presented a continuous threat to the excavation, and led to two sinkholes.

The excavations did not suffer from large ground water ingress, the water, when encountered, was rather dripping at localized areas of the face. The groundwater presence certainly contributed to the face instability, however the cohesionless ground behavior remained the main source of the face instability.

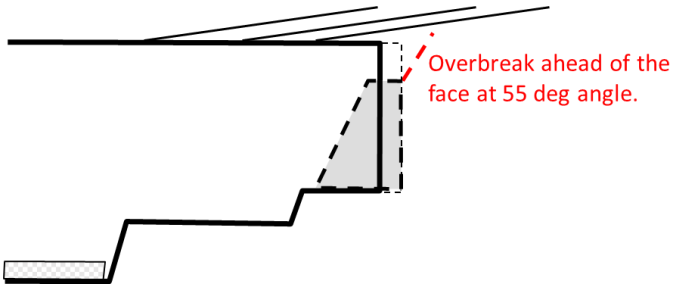

Fig. 5 Overbreak at the face perimeter increasing the advance round

5.3 Sinkholes

The two sinkholes were caused by the face instability. First the overbreaks at the top of the face exposed the umbrellas for a larger length than specified, and the cohesionless ground started pouring down between the pipes, which had a relatively small spacing of 100-200 mm. The pouring went for hours, and could not be stopped by the shotcrete. At the end the overbreak reached the quaternary sediments, and a crater propagated to the surface. Rapid inflow of material into the tunnel eventually broke the umbrella, Fig. 6.

The two sinkhole events demonstrated the localized nature of the collapse. The collapsed ground went through a small hole in the tunnel, while the ground stabilization wedge at the face, and the shotcrete lining remained stable and unaffected.

Fig. 6 Inflow of Quaternary sediments in the tunnel after sinkhole formation

5.4 Differential lining settlements

The lining vertical settlements, when displayed in the longitudinal profile, showed that the lining was bending towards the face. In response to every new excavation round the largest settlement increments were measured close to the face, and were diminished with increased distance from the face. The top heading lining behaved like a cantilever fixed at the tunnel invert section with a free end bending down towards the tunnel face. The longitudinal lining bending generated shear cracks. The shear cracks were not considered hazardous for the lining

bearing capacity, yet the cracks indicated a quick deformation development, which could lead to exceeding the lining settlement limits, and therefore had to be limited.

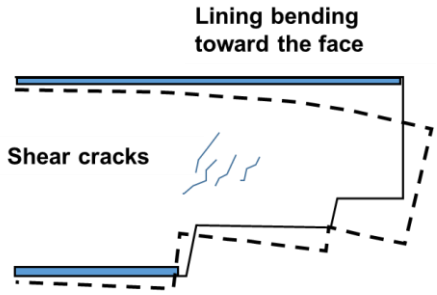

Fig. 7 Large settlement increments at the face, and lining bending

5.5 Lateral lining displacements

The lateral displacements of the top heading lining sidewalls became more pronounced with the increased overburden. The largest lateral displacements of up to 90 mm were measured on the bench sidewalls. The differential displacement between the top heading sidewall and the bench sidewall led to a formation of the longitudinal crack at the joint between the top heading and the bench. The crack could have had serious impact on the lining stability, and was monitored. Nevertheless, after closing the tunnel invert no sign of instable behavior was observed.

6 Supports effectiveness

The support members like radial bolts, elephant feet, underpinning, spiles, face anchors, pipe umbrellas were not effective in the compressible, cohesionless ground, and could not reduce the lining settlements, or stabilize the face.

On the contrary they were slowing down the excavation advance, and deteriorated the ground stability by contributing to additional ground loosening.

6.1 Tunnel invert

The early tunnel invert closure, proved to be the most effective measure to control the lining settlements and deformation. However, the tunnel invert was not able to reduce the settlement magnitudes below the design warning level of 50 mm.

The tunnel invert distance from the face was limited to a maximum of 12 - 16 m to minimize the rapid lining settlement development. The largest settlement increments coincided with the time of the excavations of the top heading, the bench, and the invert. At the section where the tunnel lining was closed by the invert, the settlements started to slow down. The ground high compressibility was demonstrated by continuation of the lining settlements for additional 50 mm even after the tunnel invert installation.

6.2 Long temporary invert

The speed of lining settlement development was reduced by building a temporary invert in the top heading. Due to the dimensions of the face wedge the temporary invert was installed 2 - 3 m behind the face.

The installation of the temporary tunnel invert was disruptive to the excavation cycle, and removal and refilling of the ramp was time consuming. Therefore it was expected that building a long top heading invert of at least 30 m behind the face would not only reduce the lining settlements, but would also help to increase the excavation speed.

For that purpose the temporary invert was designed to withstand the fully developed ground load which was expected in a distance of 2 tunnel diameters behind the face. The temporary invert was made round, and strengthened by the tapered reinforced connections to the top heading walls.

Unfortunately the ground compressibility behavior confirmed the strong impact on tunnel behavior, and showed that the solution with the long temporary invert did not lead to settlement reduction. On the contrary, it contributed to the settlement increase. The settlement increase was due to doubling the effect of the ground loosening, and the consequent ground compaction which was taking place under the invert. Installation of the temporary invert represented simply an additional excavation which induced an additional settlement increment.

The top heading excavation closed with the temporary invert advanced to a distance of 30 meters. Although the settlement development was slower, at that distance the total lining settlement was already 120 mm, which included the temporary invert settlement of about 50 - 70 mm. The ground load was fully developed, and the ground below the temporary invert was compacted. However after removing the temporary invert with the compacted ground, the tunnel invert was installed into a newly loosened ground, and additional ground compaction took place below the invert, and settlement increased up to 160 mm. In comparison to the lining without any temporary invert, and under the same overburden, the settlement was 120 mm

As a result of the temporary invert impact, the top heading lining had to be re-profiled by removing parts of the shotcrete arch which protruded into the profile of the final lining.

The tunnel invert installation speed was limited to comply with the time of the invert concrete stiffening. During that time the operations in the top heading were not allowed to proceed, which cancelled the potential advantage of increasing the excavation speed.

The concept of long temporary invert was abandoned since it did not reduce the lining settlements, and did not contribute to higher advance speed.

6.3 Radial bolts, elephant feet, underpinning

Radial bolts, elephant feet, and invert underpinning were ineffective. The compressible ground with low deformation modulus formed a large influence zone around the tunnel. Not only that the supports were not embedded in the firm ground, but also were displaced along with the ground deformation of the influence zone.

6.4 Dewatering

The dewatering boreholes installed 20 m ahead of the face were seldom effective, since it was difficult to intercept the ground water source with the drain. It was also difficult to improve the ground properties by draining the water since the clayey ground would require long time to create a complete draw down in the heterogeneous ground. The drains installed above the tunnel crown were cancelled to avoid bringing the ground water to the face.

6.5 Ground wedge

The face ground stabilization wedge was the most effective measure to keep the large portion of the face stable. It was sufficient that the wedge had a slope close to the natural ground slope which was about 55 deg. The wedge remained stable even during the two sinkhole events.

The areas along the perimeter of the face, which could not be supported by the wedge, and which were vertical for the installation of the lattice girder, were those areas which suffered from the overbreaks.

6.6 Face anchors

The fiberglass anchors were helpless in stabilizing the cohesionless ground. The 1 m anchors spacing was too large, and the ground was pouring between the anchors. On top of that the drilling for the anchors installation with air flushing disturbed the ground. The air pressure evidently loosened the ground, and increased the risk of overbreaks.

In an attempt to improve the anchors effectiveness the drilled anchor diameter was increased from 70 mm to 170 mm. The drilling was performed by Cassagrande machine using the cement flushing to minimize the ground disturbance, Fig. 8. The large diameter anchors filled with the cement grout had an effect like the method of ground replacement, however, in the end the number of 90 bolts in the face was not sufficient to prevent overbreaks which became more frequent at higher overburden of more than 20 m.

Fig. 8 Drilling of 170 mm diameter face anchors by Cassagrande machine

6.7 IBO grouted umbrellas

The IBO 51 mm umbrellas were realized in variable lengths and overlaps. The maximum number of umbrella pipes was achieved for the pipe length of 9 m pipe, and overlap of 5 m, which created a roof protected with 5 layers of umbrellas above each other. Yet, increasing the number of umbrella pipes did not lead to increasing the support effectiveness. The problem was the low quality grouting, and the ground loosening by the air flushing. Either the perforated IBO pipe was clogged with the ground drillings, and the grout did not reach the end of the pipe, or the pressurized grout was lost in the weak ground. The grouting did not lead to ground compaction around the umbrella pipes.

6.8 Face grouting

Chemical and cement grouting was attempted in the ground of the face, and also through the perforated IBO umbrella pipes. The grouting results were declared not satisfactory, since the ground behavior remained cohesionless both in the face, and between the umbrella pipes.

The grouting firms correctly recognized the ground as non-groutable, nevertheless grouting tests were attempted with cement grout and chemical grout to achieve fracture grouting. Although it was possible to pump the chemical grout in sufficient volumes the grout many times was not found in the ground. It would make its way into unknown location in the weak rock.

The tests achieved high grouting pressures, but due to a small number of the test grouting holes, the impact on the ground behavior was not noticeable.

The grouting tests should have been performed with the focus on ground compaction rather than focusing on changing the ground parameters by the parameters of the grout.

6.9 Short temporary invert

With increasing overburden the speed of lining settlements was increasing, and lateral displacements of the top heading sides were more pronounced.

The temporary invert was effective in slowing the top heading settlements, and at the same time restricted the lateral displacements of the lining walls. Thus the temporary invert contributed to eliminating the shear cracks in the top heading lining, which were caused by the differential lining settlements in the longitudinal profile. However, the temporary invert just like the tunnel invert was not able to limit the total lining settlements below the design warning levels of 50 mm.

The short temporary invert was designed as a temporary structure. It did not form a round structure with the top heading, it was flat, and did not have the bearing capacity which would allow fully developed ground loads. Therefore the temporary invert, after it fulfilled its short term stabilizing function, had to be removed and replaced by the round tunnel invert.

6.10 Chemical grouting at the East portal.

Chemical permeation grouting was planned to be performed from perforated pipes drilled between the umbrella pipes ahead of the face to stabilize the saturated and the highly permeable gravel terrace at the east portal.

The terrace sediments which were considered risky for the tunnel roof stability turned out to be easily and sufficiently grouted to provide stable tunnel roof.

On top of that the claystone rock below the terrace was stronger than at the west portal with compressive strength of up to 15 - 25 MPa.

The ground conditions at the East portal offered better conditions for tunnel excavations, which proceeded in higher pace than from the West portal.

7 Adopted solution

Frequency of the face instability events was increasing with the increasing overburden, which indicated that successful completion of the Žilina tunnel depended on finding reliable face stabilization measures. In addition, excavations from the East portal had to be started to mitigate the delays caused by the instable ground behavior.

The unsuccessful attempts to stabilize the face with an increased number of umbrellas and face anchors indicated that the planned measures were ineffective and that ground improvement measures to change the ground parameters were needed to take place.

The encountered soft and cohesionless ground behavior called for a solution which would increase the ground shear strength, and the ground stiffness. That was achieved by ground compaction realized through the compensation (fracture) grouting. In addition to the improvement of the ground parameters, the compaction pushed out the groundwater, and made the ground of the permeable tectonic zone impervious.

The grouting criteria were set to control the grout pressure at 20 bar. If the pressure was not achieved, and the grout volume exceeded the limit, the grouting was interrupted, and continued at another location. After the time period which allowed the grout to stiffen, the grouting procedure was resumed at the same location to achieve the prescribed pressure of 20 bar. Once the fractures were created in the soil they were immediately expanded by the influx of grout.

The face had to be supported with a thick, mesh reinforced shotcrete wall, to achieve the grouting pressures of 20 bar without face extrusion. The compensation grouting was performed both through the pipe umbrella, and through the manchette tubes at the face.

The compensation grouting made the excavation safe, and free of overbreaks, however, it was lengthy, as the grouting took from 9 to 11 days including the umbrella installation.

7.1 Compensation grouting through pipe umbrella

The umbrella pipes of I.D. 114 mm were installed in one piece by the Cassagrande machine using the cement flushing. The pipe was grouted all at once along the full length. For that purpose the pipes were fitted with rubber nozzles at a distance of 0.5 m, Fig. 9.

Fig. 9 Umbrella pipes of 12 m / 114 mm with grouting nozzles

7.2 Compensation grouting of the face

The compensation grouting of the face was performed through perforated manchette pipes. Packers were used for stepwise grouting of 1 m pipe sections, Fig. 10.

Fig. 10 Compensation grouting of the face

8 Trials to increase advance speed

The compensation grouting procedure slowed down the excavation advance. Methods which would stabilize the face, and still would allow good advance rates were searched.

8.1 Jet grouting

Horizontal jet grouting was valued as a method which would have equivalent positive effect on the ground behavior like the compensation grouting or even better. The jet grouting pressure would compact the ground, and in addition, the grouted columns would replace the ground. Notwithstanding, that the umbrella of jet grouted columns would act as a pre-support structural arch, which would allow a full face excavation in the ADDECO style.

Two jet grouting tests were performed to build a 12 m long umbrella. Unfortunately the tests were interrupted due to the tunnel face extrusion induced by the jet grouting, and the tests were not completed.

8.2 Face division

The NATM style excavation of large tunnel profiles divides the profile into smaller openings (side galleries) to minimize the face instability problems, and minimize the ground surface settlements. Also for the Zilina tunnel the face division was proposed, one with vertical division, and the other with horizontal division. The face division was discussed, and due to the uncertain outcome the face it was not practically tested.

Excavation of the small galleries did not guarantee that it would prevent the cohesionless ground pouring, and overbreaks formation at the face. The weak and cohesionless ground behavior had stronger impact on the tunnel behavior than the positive effect of a smaller tunnel face. A simple Ground Convergence Curve analysis demonstrated that the size of the excavation opening of 6 m or 12 m had no influence on the convergence curve of the tunnel in cohesionless ground, Fig. 11.

The localized character of the face overbreaks, and the two sinkholes accidents confirmed that making the face smaller would not likely prevent the face instability, and that the heavy use of face anchors and umbrellas would be still required. In other words dividing the face would not bring any advantage, and would not be a substitute for the compensation grouting.

On the contrary many negative features associated with the face division would lead to a risks increase. E.g.: ground loosening caused by the gallery excavation to the neighboring openings, safety hazard of working the arch joints, small space for large equipment. The main disadvantage would be the increased lining settlement caused by the increased number of excavation openings.

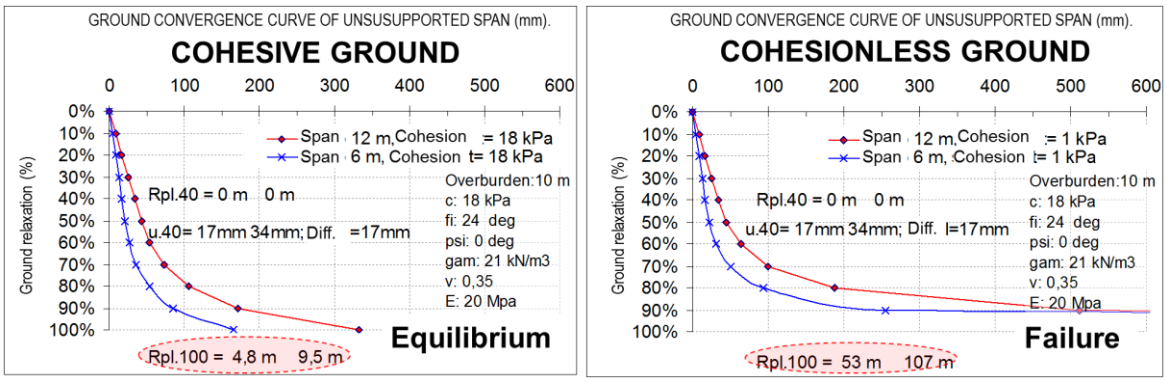

Fig. 11 Ground convergence curves demonstrating the dominant impact of the cohesionless ground compared to impact of the tunnel span

9 Conclusions

The encountered ground conditions demonstrated highly compressible, and cohesionless behavior, which could have led to serious tunnel instability problems unless careful implementation of the tunnel invert closure, and the ground modification measures was exercised.

The exceptionally weak and soft ground properties had dominant impact on the tunnel behavior, which made some of the conventional tunnel supports ineffective.

The demand for an almost immediate closure of the tunnel lining by the invert, and the necessity of adopting a lengthy procedure of the compensation grouting justified the reduced excavation advance rate, which corresponded to the implementation of the proper support measures required in the extreme ground conditions.