Reconstruction of the Jablunkov Tunnels – Design and Construction

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1. Basic Description of the Construction

The new Jablunkov double-rail tunnel will be 612m long, with a 564m long mined section and a 24m long cut and cover section at each portal, which will be built in a secured construction trench (Fig.1). The horizontal alignment is straight, with the exception of the last 60m long section, which is on a transition curve. In terms of the vertical alignment, the tunnel is of the summit type. The tunnel design requires an invert and a closed waterproofing system throughout the tunnel length.

The lining of the mined tunnel consists of two shells and a 3mm thick intermediate waterproofing membrane. The primary lining consists of a C 16/20 shotcrete layer, lattice girders and two layers of mesh. The top heading stability during excavation is secured by a spiling umbrella. The final lining is in C 30/37 grade, in-situ concrete, which is reinforced by lattice girders and horizontal steel bars. Traveller formwork will be used for casting of the vault. The lining within the cut and cover sections is designed as a singlepass structure in C 30/37 water retaining concrete, reinforced with tie-up rebars. It will be cast using double-sided formwork. The space between the invert and gravel ballast is filled with C16/20 concrete; the inner surface is inclined toward the central tunnel drain located on the tunnel centre line. The net height and net width of the tunnel tube is 10m and 11.4m, respectively. Service walkways lead along either side of the tunnel. Cableways and a hydrant line are only on one side. Safety recesses are on both sides, at 24m intervals, directly opposite each other.

The mined part of the new tunnel is constructed by the New Austrian Tunnelling Method (NATM). The existing single-rail Jablunkov tunnel No. II from 1917 is being relined, round by round, and converted into a doublerail tunnel. The lining of the old tunnel is being broken out, with the exception of the right-hand side wall (viewed in the direction of chainage) and the cross section is being widened to the left side, in the direction of the Jablunkov tunnel No. I, which was built in 1871. The following procedure is being used: The right-hand side wall of the Jablunkov tunnel No. II, which is to remain unbroken, is in advance stabilised by shotcrete, welded mesh and PG anchors, through which grout is injected into the loosened rock mass behind the lining. Then the top heading of the new tunnel is excavated, concurrently with breaking out a part of the old tunnel lining. The top heading is immediately stabilised by a primary lining. Subsequently the remaining part of the new profile will be excavated and provided with the primary support; the old lining extending to the excavation profile will be broken out concurrently. Then the closed intermediate waterproofing system will be installed. The final lining of the new tunnel will follow.

2. Geological and Hydrogeological Conditions

In terms of geology, the wider surroundings are found in the Outer Western Carpathians, consisting mostly of flysch sediments (alteration of claystone, siltstone, sandstone and agglomerates), which are represented by the Silesian and Racany Units. Both units form
Fig.1 Layout of Jablunkov tunnel

independent sheets, which are thrust over one another, forming the so-called Magura Overthrust. The Magura Overthrust line runs along the eastern slope of the Jablunkov Pass. The complex overthrust structure is accompanied by fault tectonics. In terms of engineering geology, the flysch complex is a typical slide area.

The tunnel route itself is found in the upper part of the Palaeogene Silesian Unit, consisting mostly of claystone with hornblende and sandstone layers (menilite series of measures). The tunnels are driven through the least favourable geology, consisting of micro-cyclic flysch series of strata with prevailing calcareous, very low to extremely low strength claystone (R5-R6 class according to CSN 73 1001). The excavation has encountered first of all folded, partially schistose to sheared, laminated dark-grey claystone, which is often faulted, with frequent occurrence of faulting polishes (glassy appearance surfaces, smoothed out as a result of dynamic movements). The claystone contains varying thickness, irregular interbeds of siltstone and sandstone. It is heavily weathered, with R6 strength prevailing; it passes to firm clay (F8) in the direction toward the cover. Once excavated, the ground disintegrates into non-cohesive soil.

Hydrogeological conditions within the location are very complex. All surface water and groundwater flowing from the sides of both adjacent hills is gathered at the lowest lying point, which is, by accident, in the tunnel area. The water table is found at the depths of 0.25 – 6.0m under the surface. Common interstitial to interstitial-fissure confined aquifers exist within the surface layers of the Tertiary ground. No more significant inflows have been encountered during the excavation; the excavation has been dry to moist. Problems occurred at the portals, where small but frequent springs caused slaking of the clay or the clayey filling of fractured claystone, resulting in reduced stability of the walls exposed by the excavation.

3. Design Changes Made During Construction Work

From the engineering geological point of view, the area of operations is extremely complicated, forming an environment which is very difficult for executing an underground construction. The quality parameters of the rock or soil environment are poor and very variable; changes in the layers and transitions between individual geological layers and interfaces take place very rapidly and unexpectedly.

Significantly worse parameters of the rock and soil than the engineering geological survey which was used as a basis for the construction design had indicated were encountered during the execution of earthwork in the entrance cut and cover section, P1. Deteriorated parameters were further encountered even during the tunnel top heading excavation. At that time the advantages of the observational method, which was applied to both the cut and cover and mined sections, showed up. The support of both the construction trench sides and the top heading was reinforced
depending on the development and magnitude of deformations; effectiveness of the implemented measures was subsequently verified by the continuing monitoring.

A verification borehole was drilled in the area of the exit cut and cover section, P2, on the basis of the above-mentioned complications. The assessment of the borehole corresponded to the conditions which had been encountered till that time. When new calculation parameters had been applied to individual areas, it was proved that the support design was inadequate. For that reason the design of means and methods was modified on the basis of new structural calculations. This design has been used to date.

4. The Entrance Cut-and-cover Section, P1

In the beginning, the construction trench for the entrance cut-and-cover tunnel section was designed with sloped sides. The slopes were divided into three stages by berms. They were stabilised by C 16/20 shotcrete, two layers of welded mesh and a grid of steel nails or SN anchors. A micropile umbrella was designed to be installed in the front wall, above the future excavation.

Increased deformations of the sides of the trench were measured during the excavation. The main reasons can be determined to be the continuing saturation of clayey materials with ground water (despite a system of collecting drains), the irregular occurrence of layers of heavily decomposed claystone with zero cohesion, and even locations in which the backfill of the cut-and-cover parts of the old tunnel was found inadequate.

The response to the complications comprised a densified grid of SN anchors, increased length of the anchors (in some locations even up to 8m), installation of two tiers of continuous steel walers with a shotcrete spreading sill on the berms. The waler was tied to the slope by up to 16m long stranded anchors, which had to be drilled outside the operating single-track tunnel (Fig.2, 3).

The resultant stabilisation of deformations lasted only until the tunnel top heading excavation commenced. In addition, cracks started to develop in the shotcrete layer covering the slopes in the vicinity of the portal. The decision was made not to continue the excavation of the construction trench up to the bottom and to install a 12m long ‘false primary lining’ adjacent to the portal. The false lining is a 500mm thick, vaulted structure, which is supported by strip foundation. It will serve as sacrificial formwork for the tunnel vault structure. The lining is in shotcrete with the

Fig.2 The modification of the support of the entrance cut-and-cover section, P1
same reinforcement elements as that used for casting of the primary lining within the mined part of the tunnel. However, it is not calculated for the load which will be imposed by the backfill. The backfill will be carried out only after the completion of the final lining. When the false primary lining had been erected and the space between the springing and the side slopes of the construction trench had been backfilled, the deformations finally stabilised.

5. Mined Part of the Tunnel

The top heading excavation started from the entrance portal, P1, after exposing the old tunnel tube, which was in the beginning left in the construction trench with the aim of increasing its stability. The old tunnel was filled up to the calotte bottom level so that the width of the working platform was increased to be sufficient for construction equipment. The NATM excavation support class 5 was encountered at the beginning. It required the application of an invert between the left springing of the calotte vault and the left sidewall of the old tunnel. Right since the beginning, deformations of the left side of the top heading reached high values. They developed rapidly and stabilised very slowly. The old tunnel in its righthand part acted as a stabilising element. The remaining part of the calotte rotated around it, in the anti-clockwise direction as shown in Fig.4 (the left springing of the vault sank most of all). A series of stabilisation measures were immediately adopted and implemented. The effect was checked by the monitoring. First of the measures was a longitudinal spreading sill reinforcing the left bottom corner, and widening of the vault springing toward the rock mass, the so-called Elephant’s Foot. Further, 6-9m long micropiles, drilled down through the spreading sill, and self-drilling anchors were installed into the left side wall of the top heading. Despite temporary improvement, none of the measures had any long-term effect in the clayey materials. Supporting the vault by mighty logs approximately on the centre line of the top heading proved to be the most effective; on the other hand, it obstructed the work of excavation equipment.

When it seemed that the implemented measures had started to work and the excavation had arrived at slightly better geology, an incident took place on 4/5/2008 night, at chainage TM 54.5 (tunnel metre). A collapse happened during the excavation and installation of the top heading support. The collapse affected the top heading length of about 8m. The primary lining was damaged and a crater about 10m in diameter appeared on the surface. Fortunately, nobody was
injured. A historic collapse or a large overbreak which had developed during the excavation of the old tunnel on the right side of the vault was determined as one of the main causes. A significant local increase in the loads developed in this location; the primary lining vault structure was not able to carry them. The following measures were adopted after checking the construction: The collapse material inside the tunnel was stabilised by depositing a pile of muck and applying shotcrete with welded mesh. Further, the vault was supported by props as close to the collapse location as possible. Subsequently the bottom of the crater was backfilled with...
unreinforced concrete and compacted soil was placed on it. The collapse was dissolved using the side drifts and central pillar sequence, under the protection of a 12m long canopy tube pre-support, which was, in addition, used for the work on injecting grout into the loosened space (Fig.6). The next 35m long section was still excavated using the side drifts and central pillar sequence, under the protection of spiling. The temporary lining between the side drifts was left untouched as long as possible. This length was determined by the size and reach of the excavation equipment.

Many negotiations regarding the incident were held, resulting in a modification of the excavation support specifications to correspond to the experience obtained till that time. The NATM excavation support class was divided into three sub-classes: 5a, 5b and 5c. The geometry of the temporary invert of the top heading was changed to correspond to the invert of the whole tunnel; self-drilling anchors were added to the left sidewall and micropiles were installed into the left-hand springing. Because the micropiles did not acquit themselves too much, they were installed only in the beginning. The closing of the temporary invert within as short time as possible yielded the greatest effect on reducing deformations under acceptable limits.

Class 5c corresponds to the procedure applied to dissolving of the collapse, while class 5b with spiling is for the side drifts and central pillar sequence, which was used beyond the collapse location. Starting from chainage TM 90, the excavation support class 5a was applied (Fig.5), which means full face driving of the top heading under the protection of spiling umbrellas (of course, with immediate closing of the invert and using self-drilling anchors). This class was applied to the major part of the tunnel – 446m in total. Complications were encountered while passing through several historic overbreaks of various sizes, and in the section between chainages TM 278-330, in which acts of sabotage were conducted during World War 2. From TM 482, the closing of the temporary invert ceased to be carried out because of the fact that very strong quartzose sandstone was encountered, gradually rising up and filling the entire top heading face area. At TM 505 even the drill and blast technique had to be used. The transition from hard sandstone to weathered, locally decomposed sandstone took place literally within several metres. At TM 530 the decision was made that the mechanical disintegration and closing the invert according to the specification for class 5a was to be resumed. An overbreak happened after a mere 6m. A rather large volume of rock and soil material fell inside the excavation. The main cause was determined to be the occurrence of disturbed materials in a part of the excavation face, combined with an unfavourable trend and dip of the bedding. After a thorough assessment of the whole situation and with respect to the character of the rock mass, the reducing thickness of the tunnel cover and the presence of a road with traffic passing in the vicinity of the exit portal, the specification for excavation support class 5a was modified for the excavation of the remaining section ending at the exit portal. The full face excavation with closing of the invert continues, but overlapping canopy tube pre-support fans are installed instead of the spiling.

6. The Exit Cut-and-cover Section, P2

The construction trench for the exit cut-and-cover tunnel section was originally designed as a trench with sloped sides, with the means of support identical with those used in the entrance section, P1. Because of the existence of the local, traffic carrying road, a twelve metres long stretch adjacent to the mined portal was to be constructed using a top-down (cover and cut) technique (the so-called tortoise shell technique). The principle and shape is similar to that of the false primary lining; the difference is only in the construction procedure. The terrain is excavated and treated to assume the shape of the inner lining, serving as a mould for the inner vault. Subsequently, the concrete vault is backfilled and the tunnel is excavated underneath.

When the parameters had been determined on the basis of the verification borehole, the construction trench support was reassessed. The support as well as the tortoise shell structure were found inadequate. The entire cut-and-cover section was redesigned according to the
modified values as shown in Fig.7. The support system was changed to a system comprising anchored soldier beam and lagging walls, divided in terms of the height into two parts, with the higher wall designed for the location adjacent to the road. The soldier beam and lagging wall is supported by four tiers of up to 23m long stranded anchors, passing through steel walers. The anchors had to be directed outside the existing single-rail tunnel. Because of the small depth, the upper tier of anchors was replaced by sub-horizontal steel tie rods, which were fixed at the other end in a short soldier beam and lagging wall, which was constructed as far as beyond the existing tunnel. The soldier beams supporting the front end of the trench are installed taking into consideration the requirement that they must not extend into the future tunnel cross section. This space is stabilised by means of GRP anchors and a 12m long micropile umbrella above the circumference of the future tunnel excavation.

Fig.8 The exit cut-and-cover section, P2
The variability of geology in the portal area was well obvious when the soldier beams were being driven in. In some places the driving was very fast, while it posed serious problems in other places even when several boreholes had been pre-drilled. Till now, the excavation of the trench has reached the bottom level and no problems associated with deformations of the walls have been registered.

7. Further Construction Procedure

After the top heading breaks through, the excavation of the remaining cross section will continue in the opposite direction, i.e. from the exit portal, P2, toward the entrance portal, P1. The installation of the waterproofing membrane and erection of the final lining will follow at some distance behind the excavation face, throughout the length of the mined part of the tunnel. Finally the cut-and-cover portals will be constructed and the tunnels will be backfilled. In addition, a germ of an escape gallery will be excavated at chainage TM 258, toward the operating single-rail tunnel. When the installation of all new tunnel equipment is completed, the double-rail trackwork will be laid. However, the rail running through the old single-rail tunnel will continue to be in service and the other operating rail will lead through the new double-rail tunnel until the construction work in the adjacent station, Mosty u Jablunkova, is finished. When both rails are re-routed into the new tunnel, the remaining part of the cross passage to the old single-rail tunnel will be driven. The cross passage, together with the part of the old tunnel leading to the entrance portal, will serve as an escape gallery, while the remaining part of the old tunnel (toward the exit portal) will be closed.

Fig.9 Visualisation of the entrance portal, P1

Fig.10 Visualisation of the exit portal, P2


