

OKRUHLIAK TUNNEL – GEOLOGICAL EXPECTATIONS AND REALITY: THE DESIGNER'S EXPERIENCE

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ABSTRAKT: The Okruhliak Tunnel is a twin-tube motorway tunnel located in the second section of the northern bypass of Prešov, a regional capital in eastern Slovakia, on the R4 expressway. The tunnel is driven through a flysch belt and has a length of nearly 2 km. Tunnelling operations commenced in the spring of 2024. This paper presents the fundamental technical data of the tunnel along with the geological assumptions used for the development of the final design documentation. Subsequently, a comparison is provided between the actual geological conditions encountered during excavation and their impact on the tunnel's technical solutions.

1. INTRODUCTION

The Okruhliak Tunnel is a key component of the proposed R4 expressway section, Prešov – Northern Bypass. This section forms part of a major North-South transport corridor, connecting the Slovak/Polish border (Vyšný Komárnik) with the Slovak/Hungarian border (Milhost'). The expressway is designed to provide a strategic link between the D1 motorway and the R4 Kapušany – Giraltovce section. The Okruhliak Tunnel is situated within the second stage of the R4 Prešov – Northern Bypass, which seamlessly continues from the first stage (including the Bikoš Tunnel), completed and opened to traffic in 2023. The tunnel alignment follows a West-East orientation.

1.1 TUNNEL LOCALIZATION

The Okruhliak Tunnel is situated within the Okruhliak massif (390.7 m a.s.l.), specifically in its southwestern to southeastern part. The area is characterized by a gentle, hilly relief, partitioned by wide depressions into prominent and continuous ridges oriented in a general N-S to NNW-SSE direction. The massif includes elevation points 379 and 384 in the southwestern to western part (forming a more pronounced and narrow ridge) and elevation point 383 in the southern to southeastern part (forming a wide and flat ridge).

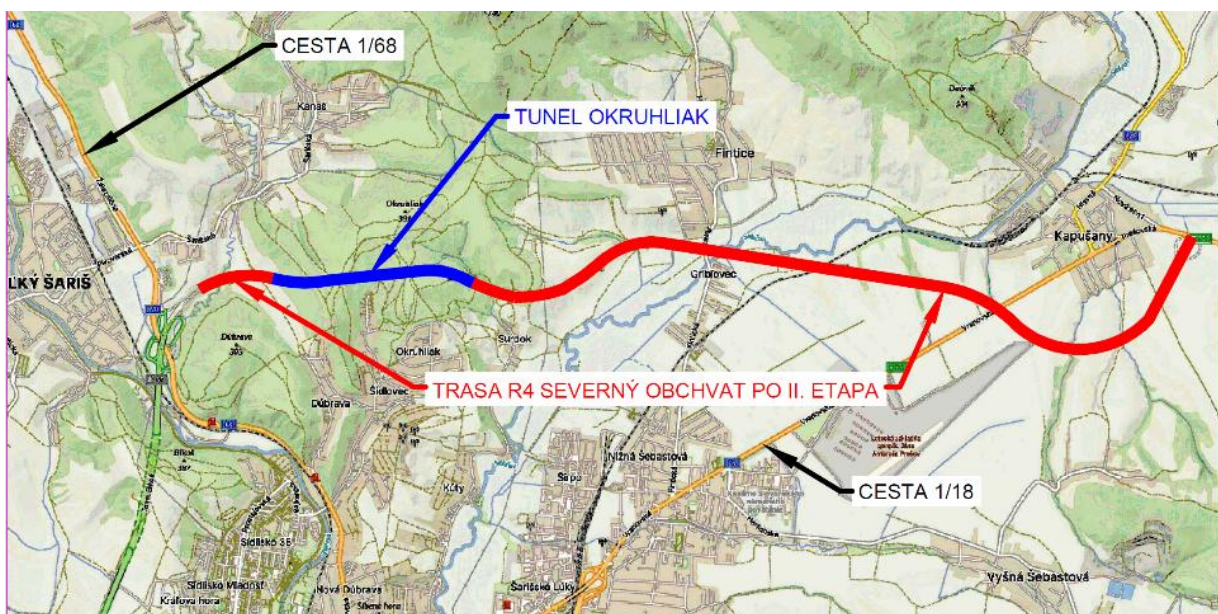


Figure 1: Location of the Okruhliak Tunnel

Besides these partial ridges, the massif is dissected by depressions of varying widths and orientations. The slopes of deep erosional gullies, which segment the flat relief of the Okruhliak massif, are affected by slope deformations. The eastern edge of the massif (near elevation point 383) in the vicinity of the Surdok settlement is also significantly disturbed by slope instabilities. The areas preceding the portals, as well as the terrain above the tunnel alignment, consist of forests and meadows. There is no surface development located directly above the tunnel or in the proximity of the portals.

2. TECHNICAL SOLUTION OF THE TUNNEL

The Okruhliak Tunnel is designed as a twin-tube tunnel, with one tube for each traffic direction, category 2T – 7.5. The vertical clearance of the tunnel is 4.8 m, the sidewalk width is 1.0 m, and the overhead clearance above the sidewalk is 2.2 m. In terms of motorway chainage, the tunnel follows a West-East orientation. The Right Tunnel Tube (RTT) corresponds to the Southern Tunnel Tube (STT), and the Left Tunnel Tube (LTT) corresponds to the Northern Tunnel Tube (NTT).

The total construction length is 1,854 m for the LTT (NTT) and 1,846 m for the RTT (STT). Based on the construction technology, the tunnel tubes are divided into mined (bored) and cut-and-cover sections.

Cut-and-cover sections will be built in open excavation pits at both portals. Their lengths at the western portal are 30 m for the RTT (STT) and 22.3 m for the LTT (NTT); at the eastern portal, they are 17.5 m for the RTT (STT) and 20.15 m for the LTT (NTT).

The mined section of the Okruhliak Tunnel is 1,798.5 m long for the RTT (STT) and 1,812 m for the LTT (NTT). The mined portion is executed according to the principles of the New Austrian Tunnelling Method (NATM) using mechanical excavation with a tunnel excavator. Drill-and-blast excavation is used where necessary in more competent rock mass sections. The tunnel lining in the mined section is designed as a double-shell system, consisting of a primary and a secondary lining with an arch shape. The primary lining consists of reinforced shotcrete throughout the entire tunnel length. The secondary lining is also reinforced along its entire length. The tunnel foundation is designed with an invert (bottom arch) in sections with poor geotechnical parameters and on strip foundations in sections with better geotechnical parameters, which may, under certain circumstances, be supplemented by a plain concrete bottom slab. The vertical alignment of the tunnel tubes rises in the direction of the chainage, with a uniform gradient of 1.09% for the RTT (STT) and 1.08% for the LTT (NTT). The horizontal alignment begins with a left-hand curve with a radius of 3,800 m for the RTT (STT) and 4,800 m for the LTT (NTT), followed by a straight section, and concludes with a right-hand curve with a radius of 1,300 m for the RTT (STT) and 950 m for the LTT (NTT).

The Okruhliak Tunnel features one one-sided emergency lay-by in each tube and six cross-passages of identical profile. Five cross-passages are for pedestrians, while cross-passage No. 4, located between the emergency lay-bys, is designed for vehicular access. The lining of the cross-passages will be double-shelled, similar to the main tunnel tubes. Ventilation is designed as a longitudinal system.

The tunnel includes drainage cleaning niches, combined CV+PV niches (drainage cleaning + fire hydrant), and combined SOS+CV niches (SOS cabin + drainage cleaning). Fire hydrant (PV) niches will be located at the cross-passages, shaped like CV+PV niches, but with the cleaning manhole situated outside these niches within the adjacent cross-passage stubs.

The primary lining thickness is designed according to support classes ranging from 150 to 300 mm. It consists of C20/25 strength class shotcrete combined with lattice girders and wire meshes. During excavation, the rock mass will be secured using rock bolts, steel needles (spiles), and, at least in the portal sections, micropile umbrellas.

The secondary lining is designed with a thickness of 300 mm, or 450 mm in the emergency lay-bys. In sections with poorer geotechnical parameters, an invert with a horizontal upper edge and variable thickness (739–1196 mm or 928–1465 mm) is designed. The lining is made of reinforced C30/37 concrete. The standard block length is 12 m.

The cut-and-cover sections will consist of a load-bearing arch structure (450 mm thick) and a foundation slab (1000 mm thick). These structures are designed from monolithic reinforced C35/45 concrete. The portal block faces will be beveled at an angle of approximately 45°.

The tunnel features a drainage system and pavement drainage. Lateral drainage consists of perforated PP DN 250 mm pipes located at the arch footings between the primary and secondary linings. Drainage cleaning niches are placed along the lateral drainage path at a maximum distance of 72 m. A central drainage collector is not utilized.

Inside the tunnel, the only fluids considered are those from accident or fire management, tunnel lining cleaning water, and minimal amounts of water brought in by vehicles during precipitation. These fluids are drained via the pavement's transverse slope (achieved by rotating the tunnel profile) to the lowest point at the sidewalk into a slot gutter, leading to an emergency collection tank at the western portal.

Cable conduits for technological equipment are designed on both sides within the tunnel sidewalks. Additionally, a fire water main will be located in the sidewalk on the side adjacent to the cross-passages.

Backfilling of the excavation pit will be performed with a front slope of 1:1.75. The portal slope will transition seamlessly into the original terrain at the top. The area around the tunnel tube outlets will be lined with 300 mm thick gabion baskets. A drainage ditch made of concrete slabs set in 100 mm of bedding concrete is located above the tunnel tubes. Geogrids and the backfill will be covered with a 200 mm humus layer and treated with hydroseeding.

3. GEOLOGICAL CONDITIONS

3.1 ASSUMPTIONS OF ENGINEERING-GEOLOGICAL AND HYDROGEOLOGICAL INVESTIGATION (MARTINČEKOVÁ ET AL. 2014)

The massif intersected by the Okruhliak Tunnel consists of underlying Neogene rocks. These range from decomposed to highly weathered claystones, characterized as soils of classes F2-F4-F5-F6-F7-F8-G5 and R6-R5. However, the massif is predominantly composed of slightly weathered to fresh rocks of classes R5-R3 (claystones and siltstones) and R3-R2 (sandstones).

Along the entire projected tunnel alignment, significant volume changes were identified. Swelling pressures ranging from 0.977 to 1.620 MPa were recorded, with a borehole in the eastern section showing swelling pressures as high as 2.600 – 2.922 MPa. These values were obtained from tests on reconsolidated samples taken from the massif.

Eleven quasi-homogeneous sections were identified along the alignment. From a geotechnical perspective, these sections can be categorized into three fundamental types:

Type 1 – Portal sections of the tunnel.

Type 2 – Environment predominantly composed of claystones, siltstones, and marlstones.

Type 3 – Environment predominantly composed of sandstones.

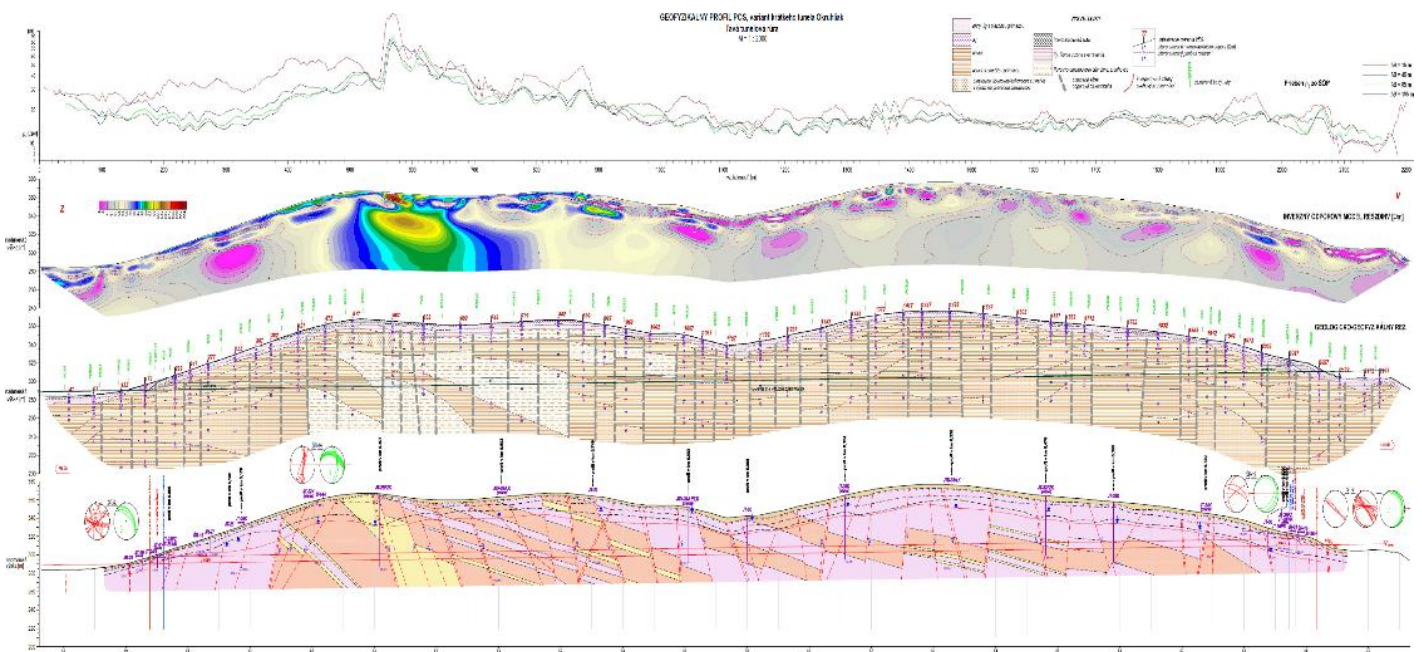
Type 1 occurs at the portal areas and is characterized by a transition into the deeper massif through surface layers of weathered to decomposed rocks (weathering grades 3 to 5, occasionally 1 to 2). Lithologically, this type mainly consists of claystones, siltstones, and marlstones, with sandstones occurring only sporadically. In these sections, the tunnel gradually moves from surface weathered, low-strength rocks into deeper layers with a slow improvement in strength and deformation characteristics. The weathering zone extends to depths of 30–50 m. In tectonically disturbed zones, weathering may reach the tunnel level, causing local deterioration of the rock mass. The width of tectonic disturbances ranges from several meters to tens of meters. Strength and deformation parameters show high dispersion due to the transition from low-strength, decomposed rocks at the start of excavation to more competent rocks deeper in the massif. According to the longitudinal profile from the investigation report (IGHP), deformation modulus values range from 8 to 140 MPa, and unconfined compressive strength (UCS) values reach 0.15 to 5.6 MPa. Cross-sections indicate that even in the blocks further from the portals, the tunnel tubes were expected to be situated in fresh to slightly weathered claystones (UCS 1.25–11.43 MPa near portals, 1.66–8.83 MPa along the alignment). Deformation moduli are reported

between 50–250 MPa for the western portal and 60–600 MPa along the alignment (derived from STN standards and literature).

Type 2 represents the majority of the massif along the tunnel tubes. It is dominated by fresh and slightly weathered claystones, siltstones, and marlstones in varying ratios, with local occurrences of sandstones or conglomerates. Fault zones with widths in the order of units of meters are interpreted, where groundwater inflows into the tunnel tubes are anticipated. Similar to Type 1, this environment exhibits a high dispersion of strength and deformation values due to lithological heterogeneity. Longitudinal profile data show deformation moduli between 100–800 MPa and UCS values from 2 to 43 MPa. Detailed report tables specify UCS values of 2.91–6.04 MPa for claystones and 5.71–23.28 MPa for siltstones, with deformation moduli of 60–600 MPa and 200–1200 MPa, respectively.

Type 3 is characterized by the dominance of fresh and slightly weathered sandstones or conglomerates at the tunnel level. It is primarily located in quasi-homogeneous block No. 4. This environment is the most favorable for tunnel construction, offering the highest strength and deformation characteristics. Deformation moduli range from 250 to 1,500 MPa, and UCS values range from 4 to 95 MPa. For slightly weathered to fresh sandstones, UCS values are reported between 9.88 and 94.57 MPa, with deformation moduli between 600 and 3,500 MPa.

Overall, the rock mass is **extremely heterogeneous**. Parameter ranges within single quasi-homogeneous units often show differences of several fold, and order-of-magnitude differences are not uncommon.



VYSVETLIVKY

- Kvartér**
- Delúvium nečlenené
- Neogén**
- Prešovské vrstvy-eggenburg, ilovce, prachovité ilovce, siltovce s ojedinelým výskytom piesčitých polôh a pieskovca
 - Prešovské vrstvy-eggenburg, ilovce, siltovce, pieskovce
 - Prešovské vrstvy-eggenburg, pieskovce, zlepence, siltovce

Figure 2: Longitudinal engineering-geological profile of the Okruhliak Tunnel – left tunnel tube

From a hydrogeological perspective, the pre-Quaternary basement is critical. It consists of the Prešov Formation (Eggenburgian), represented by clays and claystones with layers of siltstones, sands, and sandstones, often with carbonate cement. The Neogene molasse sediments do not provide favorable conditions for groundwater circulation or accumulation. Due to complex filtration heterogeneity and low permeability of the cover formations, only a small fraction of precipitation and Quaternary groundwater

enters the Neogene complex. Limited, shallow circulation occurs via more permeable layers. More favorable conditions exist in decomposed/weathered zones and tectonic faults (oriented N-S and NW-SE).

Groundwater at depth is more highly mineralized due to slow circulation. In boreholes, water is tied to sandstone layers and claystones with significant gypsum content in fault zones. While intergranular permeability varies in weathered zones, fracture permeability dominates in fresh rock. Significant flushing fluid loss (up to 100%) during drilling indicated intense fracturing and open discontinuities. Transmissivity is very low (to). Permeability of completely weathered claystones (to) classifies the environment as insignificantly permeable (Class VIII). No springs are recorded directly above the tunnel corridor; however, depressions remain waterlogged due to the impermeable surface clay layer. Scattered groundwater discharges are registered in a NW-SE depression on the SW slope, west of Surdok, within a large area of block landslides.

3.2 SUPPORT CLASSES AND METHODOLOGY FOR THEIR APPLICATION

In the Okruhliak Tunnel, six support classes (SC) were designed for excavation of the standard profile, along with one specific support class for the emergency lay-by. The designation of the support classes and their predicted application were as follows:

- **SC III** – For use in the most favorable geological conditions within sandstone formations, RMR > 40;
- **SC IV** – For use in siltstone and claystone formations with better parameters, RMR 32–40;
- **SC V** – For use in siltstone and claystone formations with poorer parameters, RMR 28–32;
- **SC Va-S1 and SC Va-S2** – For use in tectonically disturbed zones, RMR 22–30;
- **SC Vb** – For use in portal areas, RMR 15–25;
- **SC Va-NZ** – For use in the emergency lay-by, RMR 30–36.

The **RMR (Rock Mass Rating)** index is the key factor for assigning a specific excavation round to a support class. This index is evaluated at every tunnel face, with its ranges originally determined during the engineering-geological investigation. In addition to the data provided in the investigation, the tunnel face is also evaluated using the **GSI (Geological Strength Index)**. The GSI was not part of the initial investigation but is utilized in structural stability calculations when applying the **Hoek-Brown failure criterion** for the rock mass material model. As part of a Value Engineering proposal (Improvement Proposal) for the primary and secondary lining, and in cooperation with the geotechnical monitoring and geological service providers, GSI index ranges were established for each support class and subsequently used in the structural analysis.

3.3 COMPARISON OF ACTUAL GEOLOGICAL CONDITIONS ENCOUNTERED WITH INVESTIGATION FORECASTS

As previously stated, both tunnel tubes were divided into 11 quasi-homogeneous units along their length. Based on the data available at the time of writing, it is evident that the actual geological conditions encountered across the majority of the tunnel length are less favorable than those predicted by the investigation results. Table 1 provides a comprehensive comparison of the most critical parameters from the investigation, the planned assignment of quasi-homogeneous units into support classes, and the support classes actually implemented during construction.

The assignment of individual units into support classes based on investigation forecasts was indicative. It was assumed that in fault zones and tectonic lines, SC Va-S1 would be locally applied in units primarily using SC IV, and SC Va-S2 in units using SC V. It was primarily expected that the tunnel would be driven through fresh to slightly weathered rock mass, except for the end quasi-homogeneous units, where the tunnel gradually transitions from ground level through surface sediments into deeper, more competent parts of the massif.

Comparison of the table data with the defined support classes clearly shows that, based on face classification using the RMR system, the encountered geological conditions were worse than predicted (except for the marginal units). This resulted in a significantly higher actual proportion of heavier

support classes compared to design assumptions, leading to increased consumption of materials and support elements.

Table 1: Selected characteristics of quasi-homogeneous units in the Left Tunnel Tube according to the investigation and their assignment to support classes

QHU	1	2	3	4	5	6	7	8	9	10	11
Section length (m)	103	76	160	100	220	170	240	260	310	80	93
Weathering grade	(1-2) 3-5	(3-2) 1-0	1-0	(1)0	(1)0	(1)0	(1)0	(1)0	(1)0	(3-2) 1-0	(5)4- 3-2(1- 0)
RMR	15- 25	22-32	34-40	38- 45	37- 42	33-40	30-42	36-40	(23)30- 35(40)	22-32	15-25
Predicted SC (Final Design)	Vb	V	IV	III	III	IV	V (Va-NZ)	IV	V	V	Vb
Actually implemented SC	Vb	Vb (Va-S2)	V (Va-S1)	IV (V)	V	V (Va-S2, Vb)	Vb (Va-S2, Va-S1, Vb-NZ)	V (Va-S1)	Va-S2 (?)	Vb	Vb

During excavation, extraordinary situations occurred, manifesting either as large geological overbreaks or exceedances of expected convergences. Convergences were exceeded in sections where rocks with significantly lower strength than the average strength for the given round were encountered in the bench footing and the bench (or generally in the lower part of the excavation). Heterogeneity in these areas often manifested in all three dimensions, where differences in rock strength (PLT - point load test converted to UCS) reached several-fold multiples within a single face in the transverse direction (right/left, top/bottom) or across several rounds in the longitudinal direction. In these areas, the only reliable measure to limit the magnitude of convergences was the rapid closure of the tunnel profile, which led to a swift stabilization of deformations. Where technological reasons prevented rapid full-profile closure, a temporary invert in the top heading (crown) was used, which subsequently had to be removed during bench excavation.

The most challenging event occurred in the emergency lay-by of the Right Tunnel Tube (RTT) at the location of cross-passage No. 4. Water inflows from a tectonic fault caused rapid degradation of the rock strength parameters at the top heading footing on the right side. This resulted in a significant settlement of the lining on that side during bench excavation—reaching approximately 40 cm locally at the fault—leading to transverse cracking of the lining. Control measurements revealed that significant excessive deformation (over 200 mm) occurred over a section of approximately 20 m. A similar, though less severe, incident occurred in the area of cross-passage No. 4 in the Left Tunnel Tube (LTT). Given these extraordinary events and the subsequent need for lining reprofiling, the support design for the cross-passage and its stubs was re-evaluated, resulting in significant lining reinforcement and increased use of supplementary support elements.

A separate issue was the predicted occurrence of swelling rocks, which caused considerable concern during the preparation phase. However, during excavation, this problem did not prove to be as severe. Critically, the vast majority of the tunnel was free of groundwater inflows or had minimal seepage, which lacked the potential to trigger the swelling process. During excavation, samples were taken for swelling tests from areas not previously explored, supplementing the detailed investigation data. The tests determined the maximum swelling pressure and, more importantly, the time-dependency of swelling, which had not been presented in the original investigation results. Supplementary tests revealed that for all samples, nearly full swelling pressure (over 90%) was reached within several weeks, and stabilization at the maximum value occurred within a few months. These data indicate that swelling primarily affects the primary lining; its impact on the secondary lining may only manifest after the degradation of the primary lining, provided the primary lining can sustain the swelling load without forming plastic hinges.

Pressure cells were installed to monitor contact stresses in areas where swelling was expected. The highest value recorded so far was approximately 250 kPa at the transition from the claystone to the sandstone sequence in QHU No. 4. This value is significantly lower than the maximum swelling pressures identified during the investigation for that section.

A major factor prior to the start of excavation from the eastern side was the activation of a landslide during the construction of the portal cut, occurring on a slip surface that was identified as much smaller during the investigation. The landslide activation necessitated an immediate suspension of work on the portal cut, supplementary geotechnical investigation, and the design of additional stabilization measures for the portal pit. These measures included a pile wall at the bottom of the portal cut in the southern slope area and the portal wall itself, connecting the originally designed anchor beams at the lower level, which required increasing their cross-section and significantly reinforcing the anchoring system. Piles were also designed at the tunnel tube locations, where the pile capping beam captures the tubes and utilizes the tunnel lining as a stabilizing element. These additional activities delayed the start of excavation from the eastern portal by approximately six months. Monitoring of dynamometers on the reinforced anchors showed that tunnel excavation still influenced the residual movements on the slip surface, which finally stabilized only after the tunnel tubes passed beneath the slip surface or reached beyond the landslide edge. All additional measures for the stabilization of the eastern portal were designed and implemented as permanent.

4. CONCLUSION

Based on the facts described in this paper, several conclusions can be drawn. The engineering-geological investigation for the Okruhliak Tunnel was conducted through a network of boreholes supplemented by geophysical surveys. However, due to the high dispersion of the rock mass material characteristics, the data obtained provided only a framework for understanding the conditions in which the tunnel was to be excavated.

Consequently, the original design carried significant uncertainties, allowing for a wide range of design variability depending on the safety factor considered. During construction, these uncertainties required frequent cooperation between the contractor and the designer to correct the primary lining design based on the actual geological conditions encountered at the tunnel face.

It is also evident that this scope of investigation in the Carpathian Flysch is insufficient for an adequate determination of conditions to be encountered during excavation. By principle, it is unable to capture the extreme variations in rock strength parameters that were observed during the excavation of the Okruhliak Tunnel, even within a single face. In the longitudinal direction, it was not uncommon for excavation rounds in relatively hard rock (requiring difficult mechanical excavation) to alternate with rounds where excavation was fast and easy, but where excavation stability became the primary concern.

An interesting finding was that in sections where the tunnel was predicted to pass through sandstone sequences, a heavier support class had to be applied. This was due to the composition of the sandstone in this zone being different from the sandstone beds found within segments dominated by claystones and marlstones. The strength of these isolated sandstone beds was significantly higher than the strength of the sandstones in the primary sandstone zone—an area where, due to the investigation's scope, no borehole had been placed and which was identified solely based on geophysical data.

In recent times, geotechnical investigations for future tunnels are often limited to boreholes and geophysics. The excavation of the Okruhliak Tunnel demonstrates that even for shorter tunnels with moderate overburden situated in the Carpathian Flysch, it is advisable to consider the use of an exploratory adit. If not for the entire tunnel length, then at least partially in sections where complicated geology and significant heterogeneity are expected. Despite the smaller cross-section of an exploratory adit compared to a full tunnel tube, the information gained would significantly improve the understanding of the rock mass state and quality. This would lead to more realistic construction cost estimates—which tend to increase considerably in such cases—and would mitigate negative impacts on the project completion schedule.

LITERATURE

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