

RECONSTRUCTION OF THE DOLNOLUČANY TUNNEL, FROM INITIAL IDEAS FOR SOLVING LEAKS TO THE CONSTRUCTION OF A NEW TUNNEL

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ABSTRACT: The railway network of the Czech Republic includes a large number of tunnels built in the second half of the 19th century and in the 20th century before World War II. These are historically and technically interesting structures, but their condition often does not meet current requirements for safe operation and maintenance of tunnels. The main problems include insufficient spatial clearance, the unsatisfactory condition of the waterproofing and drainage systems, and in many cases the condition of the tunnel lining. The aim of this article is to use the example of the Dolnolučanský railway tunnel to illustrate not only the principles of technical design, but also the mechanisms of evaluation when dealing with uncertainties associated with a whole range of unknown boundary conditions and the risks arising from them. The opportunity to follow the development of the project from study to implementation makes it possible to evaluate the detail of the available documentation and the corresponding technical solution at each stage. It is also very valuable to compare the project assumptions with the actual geotechnical conditions encountered during construction and the complications that made the implementation itself difficult due to limited access to the construction site and cramped space conditions in front of both portals and in the single-track tunnel.

1. INTRODUCTION

The Dolnolučanský railway tunnel has long required special attention due to its poor technical condition in terms of ensuring safe operation in winter. It is a single-track tunnel 82.5 m long, located between the stations of Jablonecké paseky and Lučany nad Nisou, only 260 m from the border of the Jizerské Hory Protected Landscape Area. From the western entrance portal, the track passes under Stolní vrch at an altitude of 590 m and climbs at a gradient of 2.65% towards Tanvald. In summer, there are no signs of the problems that arise when frost sets in. Groundwater seeping through damaged joints in the tunnel lining quickly turns the tunnel into an ice cave. Icicles over 1 m long and water freezing between the tracks require daily inspection. The Railway Administration has therefore decided to include this tunnel in the list of structures requiring reconstruction.

2. HISTORY OF THE TUNNEL CONSTRUCTION

The tunnel is located on the line from Jablonec nad Nisou to Tanvald, which was put into operation in 1894 as part of the Liberec – Tanvald – Harrachov railway connection and served to connect Austria-Hungary with Prussia. The location of the tunnel in the area of interest is shown on the map in Figure 1.

No documentation from the time of the tunnel's construction has been preserved, and neither the tunnelling method nor any construction details are known that would provide an idea of the actual shape of the excavation, the creation of overburden, the quality of the rock mass, water inflows, etc. However, it can be assumed that a similar construction method was used as for tunnels built on this line in similar geotechnical conditions, for which contemporary documentation has been found in the archives.

The technological possibilities, especially in terms of drilling and blasting at the turn of the 19th and 20th centuries, did not allow for the achievement of a precise excavation shape compared to the current possibilities of mechanical equipment. The formation of overburden could have been significantly influenced by the blocky separation of granite and the degree of its weathering. Historical sources from the construction of tunnels on the Tanvald – Kořenov line, which were excavated in similar geotechnical

conditions, state that Nobel's dynamite, plastic dynamite and, where their effects were insufficient, blasting gelatine were used to break up the rock. The rock drills were made of hardened steel, and up to 1,800 of them had to be sharpened for every metre of tunnel length. The average daily excavation rate was 1.15 metres. The slow progress, and in particular the prolonged delay between the excavation and the installation of the lining, could lead to the loosening of the rock mass around the excavation. The activation of the lining at that time, which was only secured by grouting, does not ensure effective support of the excavation, which, in combination with water, can lead to further long-term and uncontrollable degradation of the rock mass behind the existing lining. All these facts may play a role in the reconstruction of the tunnel and the design of a method to ensure its long-term stability and water impermeability.

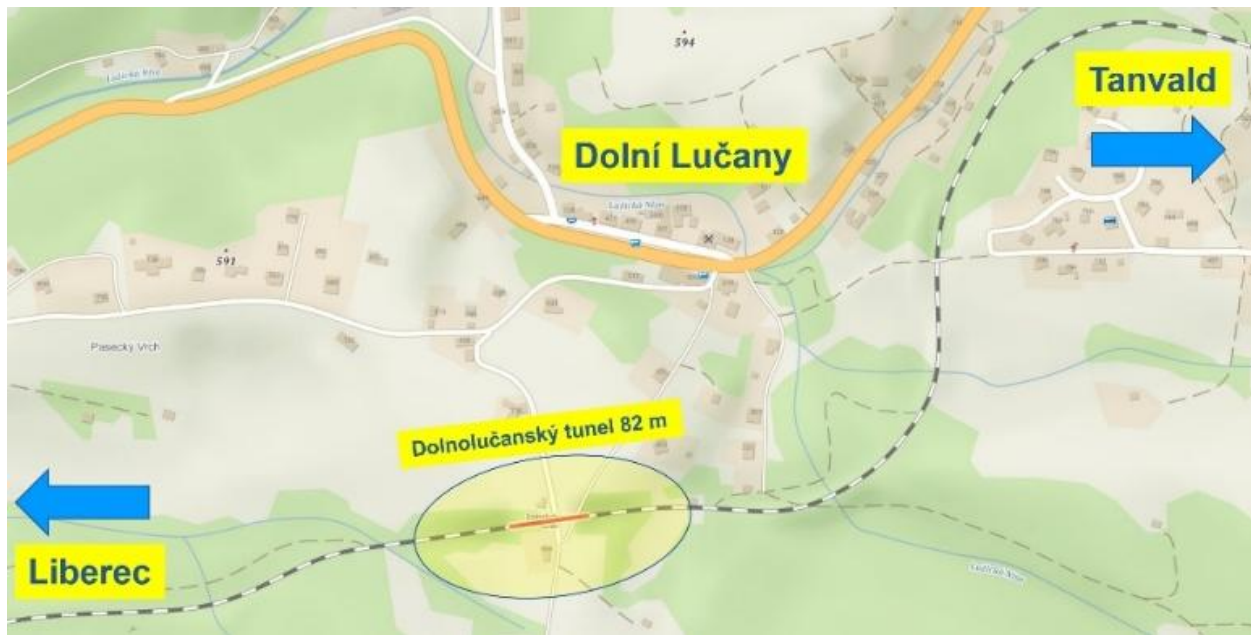


Figure 1 Location of the tunnel in the area

3. GEOLOGICAL AND GEOMORPHOLOGICAL CONDITIONS

The area of interest is located in the Jizera Highlands, which is a sub-unit of the Jizera Mountains orographic unit belonging to the Sudeten Mountains (Western Sudeten sub-system). The Liberec – Tanvald railway line crosses the flat ridge of Pasecký vrch with a developed ridge plateau approximately 80 m wide in the area of interest. The slope of the plateau is very small – in the longitudinal direction of the ridge, the plateau slopes at an angle of 1° - 3° towards the north, while in the perpendicular direction, the slope gradually increases from the central part to approximately 3° , with the horizontal part reaching a width of approximately 30 m in the area above the tunnel. The slope of the hillside is also not very steep – 10° to the west and 13° to the east. The slight slope of the hillside results in considerable length of rock cuts in front of the portals, which reach a depth of up to 21 m.

The Dolnolučany Tunnel and its cutouts are excavated in the granite of the western part of the Krkonoše-Jizera granite massif. The Krkonoše-Jizera massif protrudes in the core of the vault of older crystalline schists of Algonic and Palaeozoic age. The granite massif is a body solidified at great depth, which was later lifted by geological activity and largely exposed. Its exposed part has the shape of a horizontal figure eight, 70 km long and 8 to 23 km wide. The massif is petrographically uniform, composed of mainly coarse to medium-grained granites consisting of quartz, potassium feldspar, plagioclase and biotite, with smaller amounts of muscovite and amphibolite, especially in the southern part.

The depth of weathering of the granites in the area of interest is uneven, its variability being determined by the penetration of a considerable number of faults, along which weathering progresses to great depths. Overall, the degree of rock disruption in the investigated area is considerable – blocks of sound rock can only be found sporadically in the cuts. The walls of the cuts in front of the portals are dominated

by weathered and wind-eroded rock, and there are also sections with completely decomposed rock over the entire height of the walls. Blocks of rock are still being released from the steep walls of the rock cut, which, without securing, threaten the safety of traffic. Figure 2 shows a loose block of rock from the side of the rock cut caught in a high-strength net.



Figure 2 Rock block caught in a high-strength net on the side of the rock cut

4. CURRENT STATE OF THE TUNNEL

The entire length of the tunnel tube is secured with granite block lining. The masonry is heavily saturated, and the tunnel's waterproofing and drainage systems are non-functional. Fissure water infiltrates the tunnel and the rock cuts at both portals, manifesting as significant leaks and the leaching of mortar from the joints. Locally, this process compromises the stability of individual lining blocks. In certain sections, rock pressure causes slight bulging of the masonry on the sidewalls. However, the tunnel remains stable as a whole, and the structural integrity of the lining is not compromised, as shown in Figure 3. In portal sections P1 and P2, significant transverse cracks (up to 30 mm wide) are present in the vault, as illustrated in Figure 4. In these sections, the joint mortar has deteriorated, and the surface of the masonry blocks is weathered.



Figure 3 Condition of the brickwork during the dry season



Figure 4 Vertical cracks in the lining

Due to the shallow overburden and the nature of the rock mass fracturing, the entire tunnel is significantly affected by climatic conditions. Surface rainwater appears within the tunnel with a minimal time lag. During the spring thaw, when the rock mass in this mountainous area is continuously saturated, the issue becomes more persistent. The formation of icicles and icefalls, which can reach as far as the track, necessitates daily maintenance. The condition of the tunnel during the winter season is illustrated in Figure 5.



Figure 5 Massive icing of the tunnel due to groundwater seepage

This situation negatively impacts the operating costs associated with ensuring operational safety. Detailed inspections indicate that the lining in the vault is saturated, with water infiltration occasionally manifesting as heavy dripping or "raining." Given the tunnel's length and the environmental conditions at the portals, the entire interior of the tunnel freezes during winter. Consequently, the lining throughout the tunnel exhibits defects similar to those in the portal areas, specifically loose masonry joints and surface-wide seepage. The technical condition reflects the tunnel's age and the fact that neither the supporting structure nor the waterproofing and drainage systems have been renovated during its service life.

During the reconstruction of the Liberec–Tanvald line in 2015, only the central tunnel drain was renewed to collect and gravity-drain water seeping through the lining. As part of that reconstruction, a new 49 E1 railway superstructure was installed throughout the tunnel, utilizing B91S/2 concrete sleepers with flexible fastenings for 49E1 R350HT rails and continuously welded track (CWT). The tunnel conforms to the J-GC Z3 clearance gauge. The ballast bed consists of uncontaminated gravel. Communication and signalling cables run through the tunnel, and their functionality must be maintained throughout the upcoming reconstruction.

5. OBJECTIVES OF THE RECONSTRUCTION

Due to the long-term unsatisfactory condition of the tunnel—requiring demanding maintenance in harsh climatic conditions where leaks and freeze-thaw cycles accelerate lining degradation—the infrastructure manager, Správa železnic, has initiated a reconstruction project plan. Pending approval, subsequent project documentation stages are expected to proceed rapidly, with construction scheduled to commence in mid-2023.

The reconstruction design must adhere to clearly defined boundary conditions. All works must be situated on land owned by Správa železnic, s.o.; any placement on third-party property requires the Client's prior approval based on a justified proposal by the designer. The renovation aims to significantly reduce or eliminate water ingress, thereby preventing the formation of icicles, icefalls, and track icing. The proposed measures are intended to extend the tunnel's service life by at least 50 years. Furthermore, the installation of standard-compliant rescue refuges for maintenance personnel must be considered.

As the railway superstructure was reconstructed in 2015, it must not be damaged during the works. Track panels will be dismantled to the necessary extent, stored, and reinstated upon completion. Consequently, the geometric alignment of the track must remain unchanged, and the track will be re-established as a continuously welded track (CWT). Waterproof cable ducts and inspection shafts at both portals will be installed to house communication and signaling cables.

The project specifications allowed for a comprehensive reconstruction, including the removal of the existing lining, restoration of the waterproofing and drainage systems, and enlargement of the clearance gauge. Additionally, the feasibility of enlarging the tunnel profile for potential future electrification was examined.

The designer had to evaluate whether to proceed with seemingly less costly repairs or a comprehensive reconstruction. While the latter requires higher initial investment, it offers greater long-term efficiency and a significantly longer service life. Therefore, the design was preceded by a detailed variant study that weighed the known condition of the tunnel, maintenance requirements, and the criteria specified in the tender documentation. This study assessed both the repair of the existing lining and the option of a complete tunnel reconstruction.

6. BASIS FOR DECIDING ON THE CHOICE OF OPTION

According to the Client's internal regulations, structural and technical surveys to determine the current condition of the structure and rock mass cannot be conducted until the project plan is approved. Consequently, it is necessary to rely on available archival data and site inspections. To determine the optimal technical solution for the tunnel reconstruction, the following information is essential:

- a) Data on the spatial clearance of the track to evaluate the feasibility of installing additional components under the existing lining while meeting post-reconstruction requirements.
- b) Information on the geological and hydrogeological conditions along the tunnel alignment and within the rock cuts directly connected to the portals, as these influence the overall technical design. Geotechnical conditions are also decisive for designing the potential removal of the existing lining, ensuring excavation stability, and designing a new monolithic lining.

- c) Data on the dimensions and material properties of the tunnel lining to assess its load-bearing capacity and evaluate potential remediation options.
- d) Information on the tunnelling method used, specifically the actual geometry of the excavation profile behind the lining and the construction system, including the thickness and quality of the backfill. This backfill activates the lining against the rock mass and serves as part of the drainage system, directing groundwater to the foot of the tunnel vault. Furthermore, details are needed regarding the waterproofing system, the position of transverse drains discharging into the central tunnel drain, and the geometry and position of the tunnel invert relative to the railway substructure and superstructure.

Regarding spatial passability, it was only known that the tunnel complies with the J-GC-Z3 clearance gauge; however, the tolerance (clearance) between this gauge and the inner face of the lining remained unknown. Partial geological and hydrogeological data were obtained from a 1989 survey by GEOTEST Brno, which also provided information on the lining thickness. Nevertheless, details concerning the original tunnelling method, the actual position of the excavation face, and the specific groundwater inflow patterns—which subsequently manifest as seepage through masonry joints—remain largely unknown.

7. SPATIAL CLEARANCE AND LINING TOLERANCES

In accordance with the tender documentation, all remediation options must ensure the achievement of the Z-GC/J-GC or Z-GC Z3 clearance gauges. The geometric relationship between the clearance envelope and the inner face of the existing tunnel lining is critical when designing the remediation or reconstruction method. To evaluate a comprehensive reconstruction—including lining removal and the renewal of waterproofing and drainage systems—spatial clearance was verified against the tunnel clearance gauge (TPP) as per standard ČSN 737508. This was assessed both for a variant with overhead line (traction) extension (to meet electrification requirements) and a variant without extension to minimize investment costs.

For the project documentation, the actual position of the lining was precisely surveyed using point cloud laser scanning via a Trimble SX10. The georeferenced scans were merged into a single point cloud, which was subsequently processed. After defining the maximum extent of the cloud, it was cleaned of noise, and specific points representing the lining face and portals were isolated. The resulting cloud was downsampled to a minimum point spacing of 10 mm, which provided sufficient resolution for clearance verification.

A 3D model of the tunnel lining face was generated from the point cloud. A 3D track axis was integrated into the model, and the investigated cross-sections were "strung" along this axis. Following the mapping of the lining face, spatial clearance was verified for the following envelopes:

- a) Tunnel clearance gauge with traction extension (ČSN 737508)
- b) Tunnel clearance gauge without traction extension (ČSN 737508)
- c) Z-GC clearance gauge
- d) Z-G2 clearance gauge
- e) Z-GC Z3 clearance cross-section
- f) M-GC clearance gauge

The CloudCompare software was utilized to analyze the geometric relationship between the clearance envelopes and the lining face. The program performs a numerical evaluation of the entire lining surface, allowing for visual inspection on a 3D model. An example of clash detection (collision points) between the lining face and the clearance envelope is shown in Figure 6..

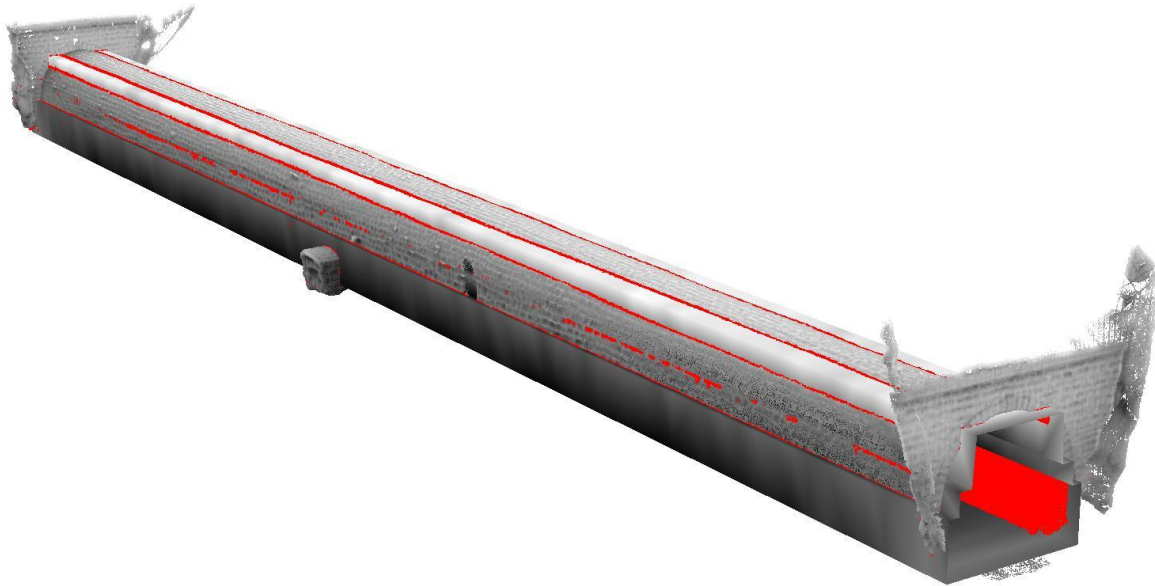


Figure 6 Evaluation of collisions between the clearance gauge and the 3D point cloud model of the lining face

This process ensures that all interferences between the clearance gauge and the tunnel are reliably detected. Due to the confined space within the tunnel, the clearance gauge was modeled with zero tolerance, even though a 50 mm tolerance is typically standard. Nevertheless, the 3D model evaluations demonstrated that without altering the geometric alignment of the track, spatial clearance cannot be guaranteed for any of the tested gauges. Since the adjacent track sections preclude any change in the track's position, it was evident that no measures could be implemented under the existing vault that would further encroach upon the clearance envelope.

8. GEOLOGICAL CONDITIONS ALONG THE TUNNEL ALIGNMENT

Information on geotechnical conditions is derived from the only available source: a June 1989 report by GEOTest Brno. The document focused on assessing the rock mass through which the Dolnolučanský tunnel is excavated, evaluating the condition of the tunnel lining, and determining the characteristics of the rock mass both within the tunnel and along the faces of both portal cuts to assess their stability.

The assessment of the rock mass behind the tunnel lining was conducted via ventilation openings situated within the lining. Out of the 81 investigated openings, the nature of the rock could be determined in 48 of them. In the remaining 33 locations, backfill or packing was found filling the void between the lining and the excavation face. The only locations where the exposed rock mass is visible within the tunnel are the rear walls of two rescue refuges, as shown in Figure 7.

The rock mass in the area of the Dolnolučanský tunnel is highly competent in its intact state; however, its strength is significantly reduced by progressive weathering. This reduction in strength and the subsequent deterioration of other key physico-mechanical properties are detailed in Table 1, which presents test results from samples collected during the survey for the nearby Josefův Důl dam on the Kamenice River. Since these tests determined the properties of the same lithological units, the values provided are also applicable to the rock mass in the investigated area of the Dolnolučanský tunnel.



Figure 7 Rock in a rescue niche

Table 1 Geotechnical parameters of the rock mass

<i>Degree of rock disturbance</i>	<i>Units</i>	<i>Undisturbed</i>	<i>Weathered</i>	<i>Weathered</i>
<i>Density</i>	<i>[kg/m³]</i>	2,620	2,620	2,510
<i>Density of solid particles</i>	<i>[kg/m³]</i>	2,670	2,610	2,640
<i>Compressive strength after drying</i>	<i>[MPa]</i>	130	70	42
<i>Compressive strength after saturation</i>	<i>[MPa]</i>	107	57	32
<i>Shear strength after drying</i>	<i>[MPa]</i>	48	32	31
<i>Shear strength after saturation</i>	<i>[MPa]</i>	20	13	14
<i>Compressive strength after freezing</i>	<i>[MPa]</i>	103	67	30
<i>Water absorption</i>	<i>[%]</i>	0.79	0.82	1.46
<i>Density</i>	<i>[%]</i>	98.12	99.24	95.07
<i>Porosity</i>	<i>[%]</i>	1.88	0.76	4.93
<i>Frost resistance</i>	<i>[%]</i>	0.21	0.36	1.62
<i>Impact resistance</i>	<i>[%]</i>	40.0	32.80	76.00

Table 2 presents the evaluation of the weathering grade based on the samples collected during the survey. Since 33 locations were found to be filled with backfill, making it impossible to assess the rock mass directly, the percentage distribution is calculated both for the total number of investigated points (81) and for the locations where the rock mass was exposed (48).

Table 2 Evaluation of probes behind the lining – degree of weathering of the massif

<i>Degree of weathering</i>	<i>Frequency of occurrence</i>	<i>Of total [%]</i>	<i>Of those encountered [%]</i>
<i>Healthy</i>	2	2.5	4.2
<i>Healthy to slightly stale</i>	3	3.7	6.3
<i>Over-aerated</i>	13	16.0	27.1
<i>Weathered to weathered</i>	3	3.7	6.3
<i>Weathered</i>	10	12.3	20.8
<i>Weathered to decomposed</i>	9	11.1	18.8
<i>Decomposed</i>	8	9.9	16.7
<i>Not found (bookmark)</i>	33	40.7	-
TOTAL	81	100	100.0

Rock that has retained its original condition and appearance is classified as unweathered. In such cases, only a minor fraction of minerals are partially weathered, and the intact minerals remain in direct contact. From this perspective, rocks with an intact skeleton account for 41% of the collected samples, while those with a degraded skeleton represent 59%. In 1989, data on the rock mass were collected only at the base of the lining (29 probes located at a height of approximately 0.5 m) and along the sidewalls up to a maximum height of 4.2 m above the track level. No information is available regarding the rock mass properties or the overburden thickness at the crown of the vault.

9. CONDITION AND DEFECTS FOUND IN THE TUNNEL LINING

The condition of the lining was also evaluated as part of the structural engineering survey. Overall, the lining was found to be in good condition, with the exception of the portal sections, where transverse cracks are visible across the tunnel tube. These are likely caused by unfavourably oriented discontinuities in the overburden, which compromise the stability of the portal faces and the masonry of the portals themselves. The stability of the rock walls in front of the portals was subsequently reinforced using high-strength wire mesh secured with SN bolts, thereby improving the shear strength parameters of the rock mass.

In the entrance portal segment (P1), two cracks were identified. The more prominent crack is located 1.6 m from the portal face; it is open up to 20 mm at the crown of the vault, narrowing to a few millimetres towards the sidewalls, and is not visible at the base. Another crack, situated 2.5 m from the portal face, runs through the left half of the vault but does not extend to the sidewalls. In the adjacent tunnel ring No. 1, a horizontal crack approximately 3.5 m long (2 mm wide) runs through a masonry joint at a height of 3.3 m.

In tunnel ring No. 2, a longitudinal crack is visible on the left side at a height of 4.5 m, also following the masonry joints. The exit portal segment (P2) exhibits similar damage to the entrance. Two transverse cracks are located near the portal; these are slightly inclined and transition from the vault into the right abutment. Another crack, 2 m from the portal face in the vault, partially extends into the abutments. Throughout the entire lining surface, the masonry suffers from deteriorated joint mortar, leading to significant seepage through the joints during periods of heavy rainfall.

10. OPTIONS FOR REPAIRING THE EXISTING STRUCTURE

Based on the available documentation and the known condition of the tunnel lining, rock mass, and waterproofing system, it was evident that achieving the required tunnel parameters without significant structural intervention was practically impossible. The rock mass is tectonically disturbed, and surface water infiltrates the lining rapidly along the entire length of the tunnel. Sealing the rock mass through grouting to ensure safe operating conditions would be prohibitively expensive, and the long-term effectiveness of such a measure would be uncertain.

The original waterproofing and drainage system relied on tight joints between the granite blocks and a backfill behind the vault, which directed groundwater along the lining to the tunnel foot and via transverse drains into the central tunnel drain. While deep repointing of the masonry could temporarily improve the watertightness of the vault, it cannot be executed to the full depth of the bed joints between the blocks.

Another potential solution—low-pressure grouting of the masonry joints—was considered, ensuring the grout does not penetrate behind the vault. However, this would likely lead to uncontrollable material consumption and could compromise the drainage function of the backfill. Furthermore, it would require meticulous grouting of all masonry joints, as addressing only the areas with the most significant leaks would likely cause water to find new paths through adjacent joints. Due to the time-intensive nature of this approach, high material costs, and the uncertain outcome, this option was rejected. Ultimately, the decision to proceed with a comprehensive reconstruction of the Dolnolučanský tunnel as the most advantageous solution was also heavily influenced by the fact that the spatial clearance did not meet the requirements for any of the monitored gauges.

11. OPTIONS FOR COMPLETE RECONSTRUCTION OF THE TUNNEL

The decision to proceed with a comprehensive reconstruction—involving the removal of the original lining, enlargement of the excavation profile, and renewal of the waterproofing system—initiated a process to optimize the technical solution. The optimization focused on two primary factors influencing investment costs: construction duration and tunnel geometry. Five variants were proposed for assessment, several of which required formal exceptions to the current standard ČSN 737508 (Railway Tunnels). In the case of a full reconstruction, this standard mandates a range of construction details and measures that would not be required for a mere renovation of the existing structure. One variant featuring a tunnel cross-section with overhead line (traction) extension (Option "0") and four variants without traction extension (Options 1 to 4), which deviated from various regulations, were submitted for evaluation.

11.1 OPTION "0" WITH TRACTION EXTENSION

Enlarging the tunnel to accommodate a traction extension would increase the excavation area from 42 m² to 63 m² (an increase of 21 m²) and raise the excavation face at the crown of the vault by 1.15 m. Over the entire length of the tunnel, this would require the excavation of 1,666 m³ of rock. Furthermore, it would increase seismic effects during drill-and-blast operations and significantly impact the portal slopes, as the height of the overburden would be reduced by the required profile elevation. This option meets all standard requirements; therefore, no exceptions were necessary. The cross-section is illustrated in Figure 8.

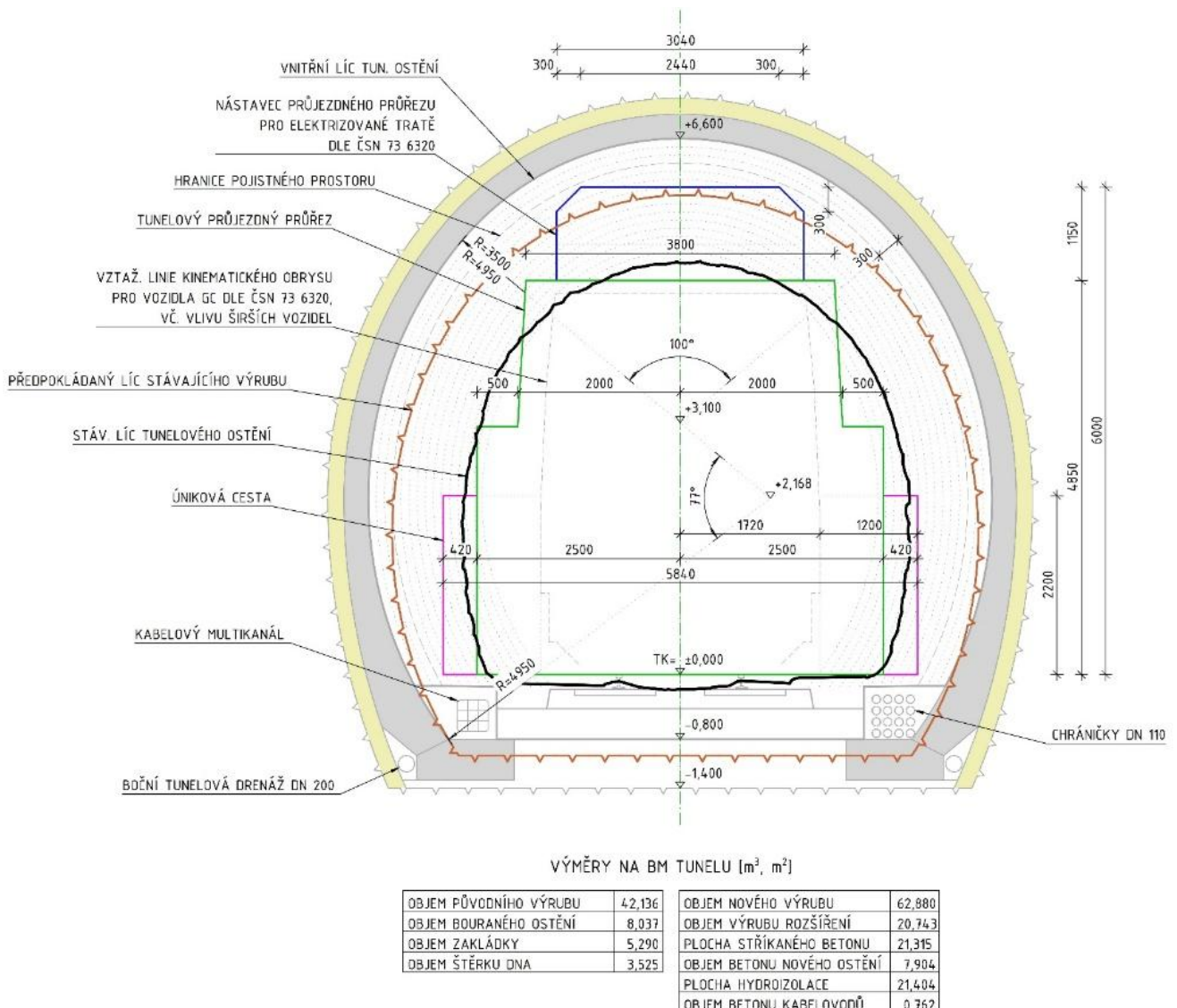


Figure 8 Cross-section for option "0" with traction extension

11.2 OPTION NO. 1

In Option No. 1, the cross-sectional geometry is designed to meet the tunnel clearance envelope according to ČSN 737508 without a traction extension. The tunnel features an optimal shape that ensures sufficient space for walkways and cable ducts. The standard width of the safety clearance is 300 mm. The footings, 0.5 m high and 1.15 m wide, are positioned 0.8 m below the track centre line at a distance of 2.0 m from the track axis.

The tunnel design incorporates lateral drainage and an umbrella waterproofing system. Water drainage from the formation level is ensured by a central tunnel drain (see Figure 9). This proposed technical solution fully complies with the requirements of ČSN 737508 and all applicable regulations.

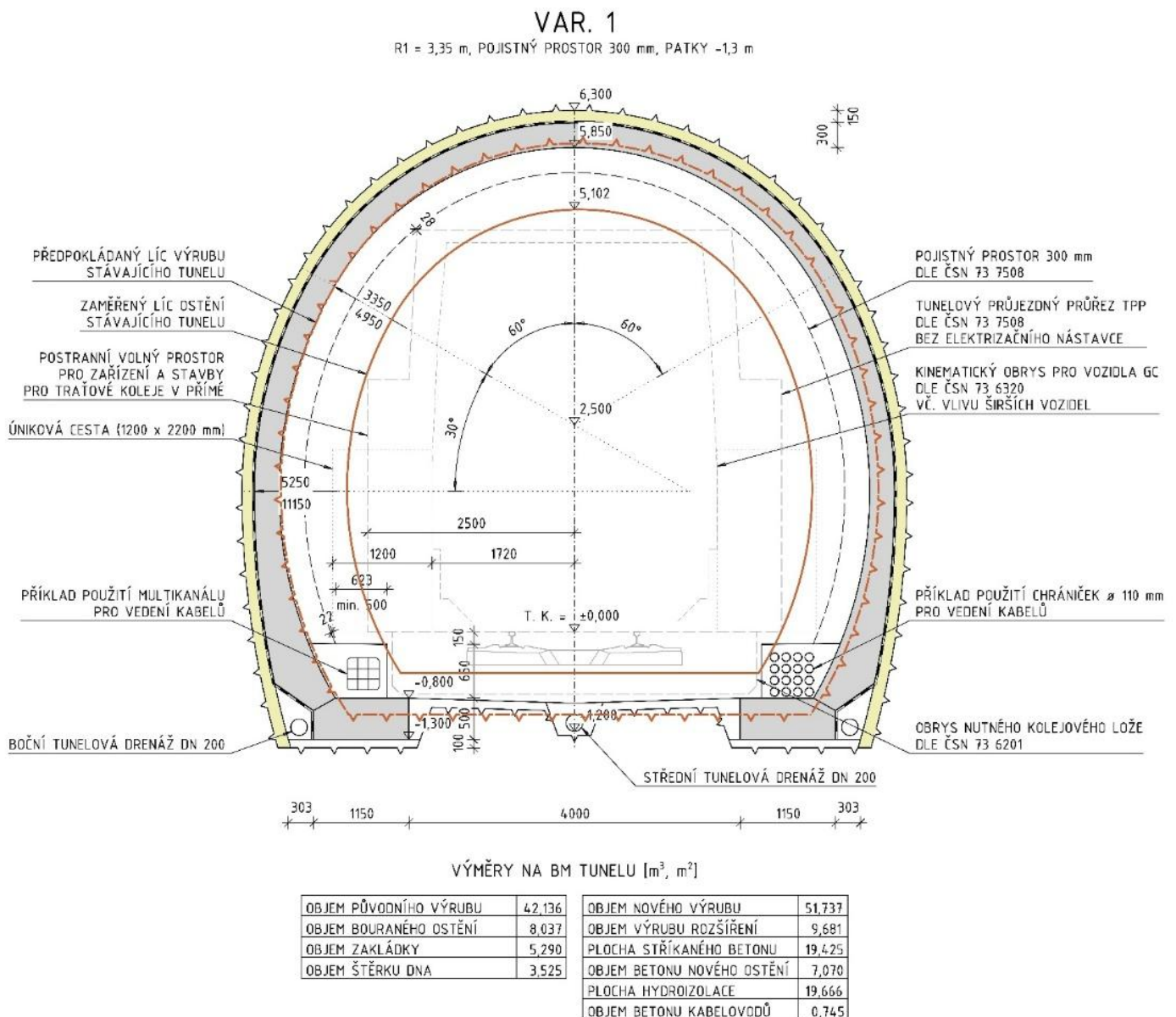


Figure 9 Sample cross-section – variant 1

11.3 OPTION NO. 2

Option No. 2 is a modification of Option No. 1, featuring a reduced safety clearance width of 150 mm. While the geometry of the excavation floor beneath the track remains identical to Variant 1, the width of the footing is reduced to 1.0 m. This is achieved by shifting the inner face of the lining towards the tunnel axis by 150 mm (see Figure 10). The resulting savings per running metre of the tunnel, compared to Variant 1, amount to 0.25 m³ of secondary lining concrete and 2.95 m³ of excavation volume. These savings are achieved at the cost of an exception from the ČSN 737508 standard regarding the minimum

width of the safety clearance. All other requirements of the relevant standards and regulations are maintained.

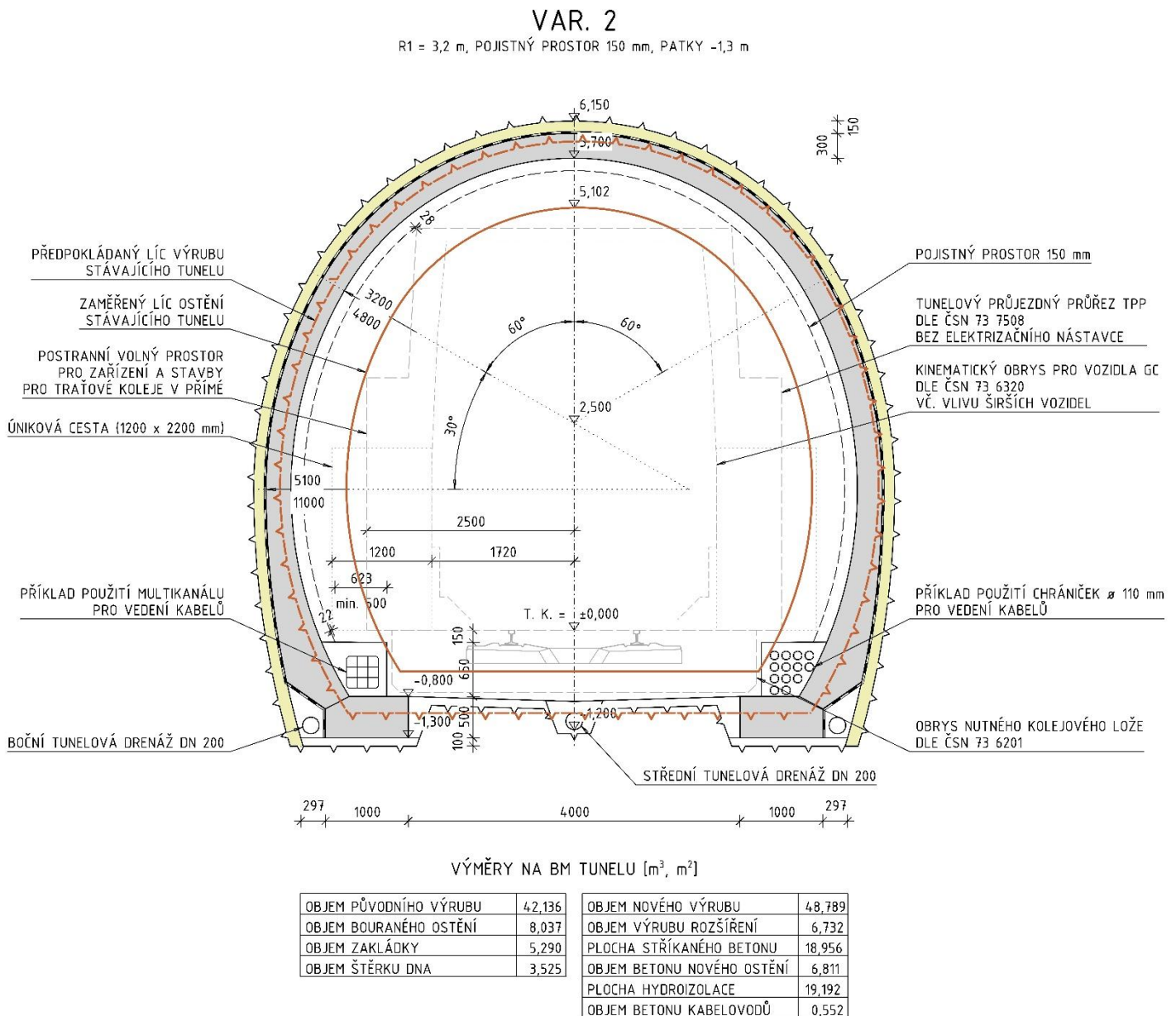


Figure 10 Sample cross-section – variant 2

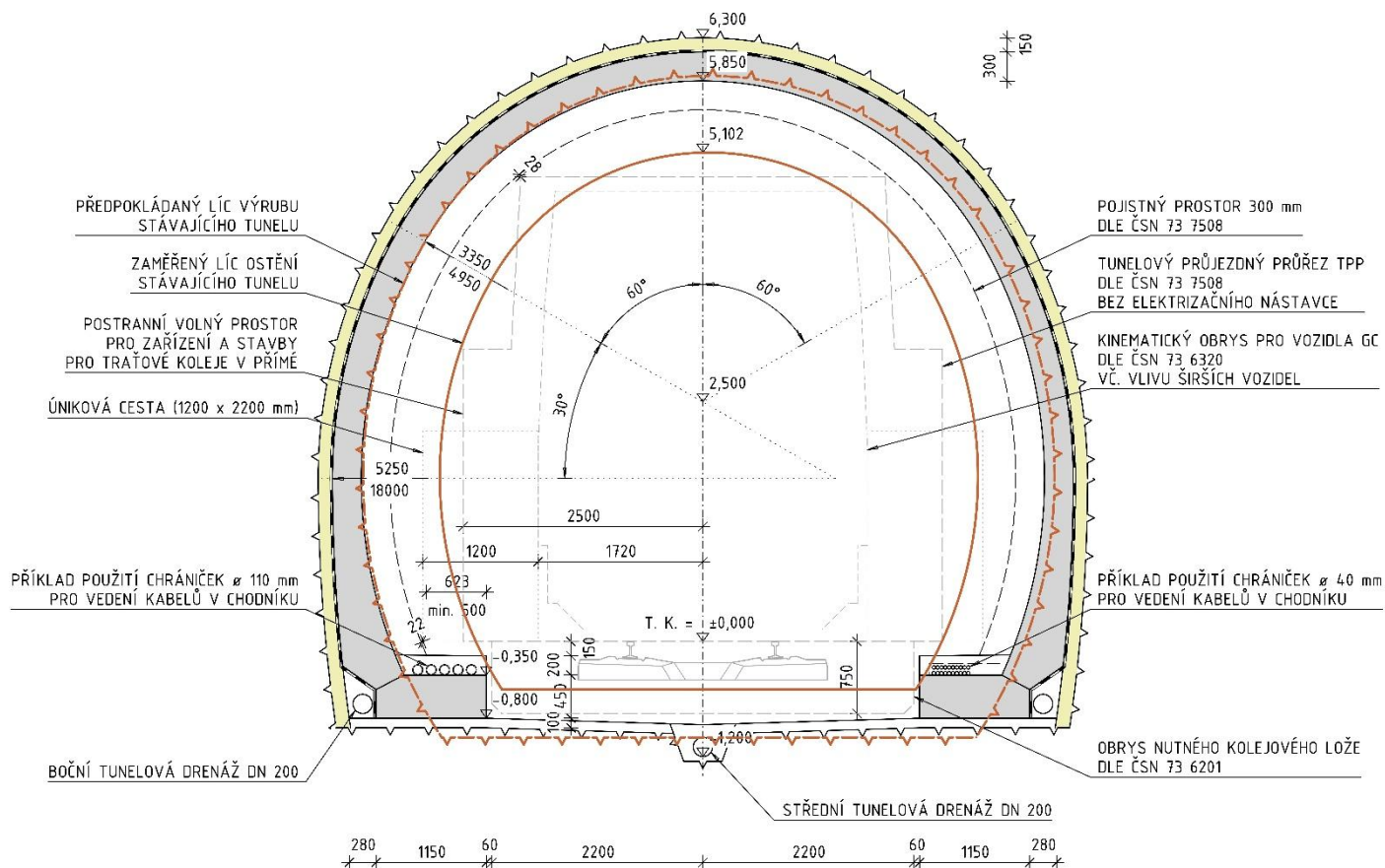
11.4 OPTION NO. 3

Variant No. 3 aims to achieve savings by reducing the space required for cable ducts, thereby decreasing the excavation volume in the tunnel floor area. This variant is based on the assumed level of the existing tunnel floor, which would require significant deepening in the previous options. The foundation level of the secondary lining footings is raised to -0.8 m below the rail level (RL)—compared to -1.3 m in Variants 1 and 2—aligning it with the formation level beneath the ballast bed.

The height of the footing is reduced to 450 mm to allow for the installation of protective conduits beneath the walkway pavement. This leaves a 200 mm concrete layer for the pavement, accommodating either a single row of DN 110 mm conduits or two rows of DN 40 mm conduits. The technical solution is illustrated in the cross-section in Figure 11. Cable chambers (pull boxes) can only be situated within rescue refuges, where the tunnel footing can be interrupted to provide the necessary space. While the standard safety clearance width of 300 mm is maintained, and the solution generally complies with ČSN 737508, the design requires verification of sufficient duct bank capacity, given the limited routing space above the footing, including provisions for future use.

VAR. 3

R1 = 3,35 m, POJISTNÝ PROSTOR 300 mm, PATKY -0,8 m
CHRÁNIČKY V CHODNÍKU



VÝMĚRY NA BM TUNELU [m³, m²]

OBJEM PŮVODNÍHO VÝRUBU	42,136	OBJEM NOVÉHO VÝRUBU	50,128
OBJEM BOURANÉHO OSTĚNÍ	8,037	OBJEM VÝRUBU ROZŠÍŘENÍ	8,258
OBJEM ZAKLÁDKY	5,290	PLOCHA STŘÍKANÉHO BETONU	18,361
OBJEM ŠTĚRKU DNA	3,525	OBJEM BETONU NOVÉHO OSTĚNÍ	6,521
		PLOCHA HYDROIZOLACE	18,512
		OBJEM BETONU KABELOVODŮ	0,298

Figure 11 Sample cross-section – variant 3

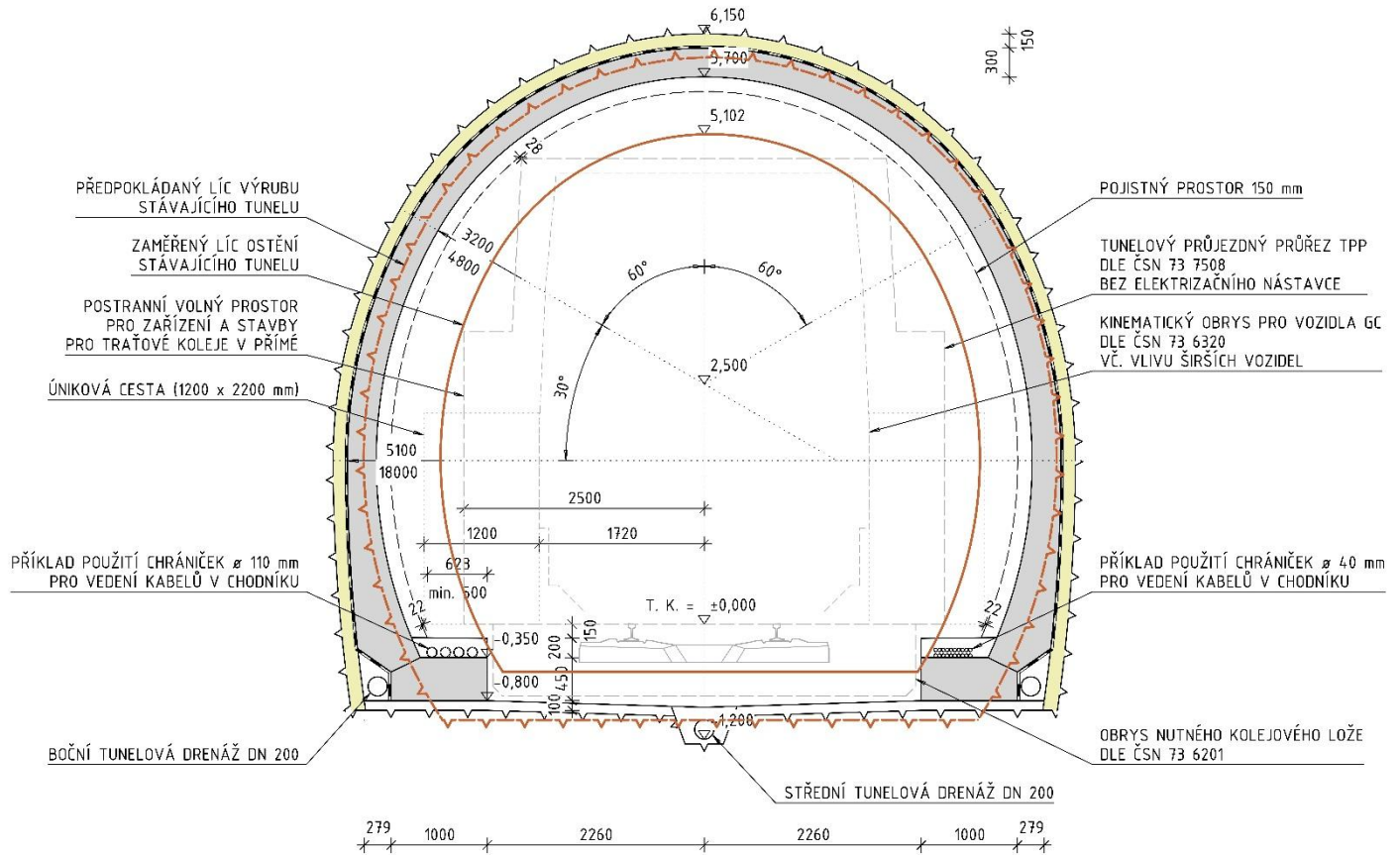
11.5 OPTION NO. 4

Option No. 4 is a modification of Option No. 3, featuring a reduced safety clearance width of 150 mm. Compared to Option No. 3, the combination of the narrower safety clearance and the reduced height of the secondary lining footings results in a saving of 3 m³ of excavation volume per running metre of the tunnel (see Figure 12).

The technical solution requires not only a verification of sufficient duct bank capacity—given the limited space for conduit routing above the footing—but also a formal exception from the ČSN 737508 standard regarding the width of the safety clearance. All other requirements of the relevant standards and regulations are met.

VAR. 4

R1 = 3,2 m, POJISTNÝ PROSTOR 150 mm, PATKY -0,8m
CHRÁNIČKY C CHODNÍKU



VÝMĚRY NA BM TUNELU [m³, m²]

OBJEM PŮVODNÍHO VÝRUBU	42,136	OBJEM NOVÉHO VÝRUBU	47,342
OBJEM BOURANÉHO OSTĚNÍ	8,037	OBJEM VÝRUBU ROZŠÍŘENÍ	5,471
OBJEM ZAKLÁDKY	5,290	PLOCHA STŘÍKANÉHO BETONU	17,889
OBJEM ŠTĚRKU DNA	3,525	OBJEM BETONU NOVÉHO OSTĚNÍ	6,265
		PLOCHA HYDROIZOLACE	18,041
		OBJEM BETONU KABELOVODŮ	0,239

Figure 12 Sample cross-section – variant 4

11.6 COMPARISON AND EVALUATION OF VARIANTS

For each variant, based on simplified assumptions regarding the back of the existing lining, the projected excavation face, and the geometry of the tunnel floor, a calculation was performed for the primary areas and volumes that significantly impact investment costs. The results are summarized in the following tables. Table 3 presents the key measurements per running metre of the tunnel.

Table 3 Main items for evaluating variants per 1 running metre of tunnel

Item	Units	Option "0"	Clearance cross-section without traction extension			
			Option No. 1	Variant No. 2	Variant No. 3	Option No. 4
Clearance envelope (new tunnel)	[m ²]	62,880	51,737	48,789	50,128	47,342
Excavation for enlargement	[m ³ / m]	20,743	9,681	6,732	8,258	5,471
Primary lining area (Shotcrete)	[m ² / m]	21,315	19,425	18,956	18,361	17,889
Upper vault concrete	[m ³ / m]	6,782	5,959	5,854	5,530	5,411
Concrete footings	[m ³ / m]	1,122	1,110	0,957	0,991	0,854
Total concrete volume	[m ³ / m]	7,904	7,070	6,811	6,521	6,265
Waterproofing area	[m ² / m]	21,401	19,666	19,192	18,512	18,041

Option No. 1 was selected as the baseline variant for the proportional evaluation of all options, broken down by individual items. The values for the remaining options are expressed as a percentage of this baseline, which is assigned a benchmark value of 100%. The results of this evaluation are presented in Table 4.

Table 4 Evaluation of items as a percentage of option 1

Item	Option "0"	Clearance cross-section without traction extension			
		Option No. 1	Option No. 2	Variant No. 3	Option No. 4
Clearance envelope (new tunnel)	121.54 %	100.00	94.30 %	96.89 %	91.51 %
Excavation for enlargement	214.27 %		69.54 %	85.30 %	56.51 %
Primary lining area (Shotcrete)	109.73 %		97.59 %	94.52 %	92.09 %
Upper vault concrete	113.81 %		98.23 %	92.80 %	90.80 %
Concrete footings	101.02 %		86.16 %	89.24 %	76.90 %
Total concrete volume	111.80 %		96.34 %	92.24 %	88.62 %
Waterproofing area	108.82 %		97.59 %	94.13 %	91.74 %

Another indicator used to compare the individual variants is the ratio of the enlargement area (required to expand the tunnel profile to the target dimensions) to the total excavation area of the tunnel after expansion, expressed as a percentage. The results of this evaluation are summarized in Table 5.

Table 5 Evaluation in terms of the ratio of the excavation area of the extension to the total excavation area

Item	Variant "0"	Clearance cross-section without traction extension			
		Variant No. 1	Variant No. 2	Variant No. 3	Option No. 4
Tunnel excavation for enlargement	32.99 %	18.71 %	13.80 %	16.47 %	11.56 %

All variants guarantee the watertightness of the lining and a service life equivalent to that of a new tunnel—specifically, zero seepage and a 100-year service life for non-replaceable structural components. After evaluating all aspects, Option No. 1 was selected as the optimal solution for the project.

The variant with a traction extension was excluded due to the low probability of future electrification, considering other artificial structures on the line that would also require cost-prohibitive structural modifications. Variants with a reduced safety clearance were deemed unsuitable as they would limit the capacity for additional installations—a restriction unacceptable given the mandated 100-year service life. Similarly, the limited capacity of cable ducts, lacking reserves for additional cabling, was considered a major constraint for future use, especially since expansion would be impossible without interfering with the tunnel's load-bearing structure.

In agreement with the client, it was proposed to extend the tunnel from the original 82.5 m to 100 m as part of the reconstruction. By moving the portals away from the steep and unstable portal slopes (which currently require reinforcement with anchors and steel mesh), the risk of rockfall onto the track will be significantly reduced. The typical conditions at both portals are illustrated in Figure 13.



Fig. 13 Rock cut walls in front of the portal

12. UNCERTAINTIES, RISKS AND THEIR MINIMISATION

In new tunnel design, the primary uncertainty lies in the geotechnical conditions. Although described by a geotechnical survey, these conditions are inherently based on "point" information interpreted for the entire tunnel alignment. In the case of tunnel reconstruction, this uncertainty expands significantly if original design documentation or archives are unavailable. During the design of the technical solution for the Dolnolučanský tunnel, the following uncertainties and associated risks were identified:

1. **Accuracy of Excavation Volumes:** Determining the precise excavation face required for the profile enlargement depends on the estimated volume of rock to be removed. This can only be calculated using simplifying assumptions regarding the position of the existing excavation surface. When calculating earthworks and landfill capacities, it is necessary to account for the currently unknown volume of backfill behind the existing lining and the volume of overbreak (degraded rock) that may fall out once the existing lining and packing are removed.
2. **Shotcrete Volumes for Primary Lining:** The imprecise position of the existing excavation surface also affects the estimated quantity of shotcrete required for the primary lining to stabilize the rock mass after widening. This volume is further influenced by the chosen waterproofing system. When using a sheet waterproofing membrane (PVC foil), the substrate's shape and curvature must strictly comply with the manufacturer's flatness specifications; any overcuts must be filled with shotcrete. Conversely, with a sprayed-applied waterproofing membrane (SAWM), substrate geometry is limited only by permissible deviations in the secondary lining thickness. However, SAWM requires specific conditions, including a relatively dry environment and, ideally, a fine-grained substrate layer to minimize material consumption.
3. **Waterproofing Material Quantities:** The actual geometry of the primary lining (the perimeter of the secured excavation) directly dictates the amount of waterproofing material. Without knowing the exact excavation face, it is difficult to accurately determine the total waterproofing area and associated costs.
4. **Secondary Lining Concrete Volumes:** An uncertain primary lining geometry negatively impacts the calculation of monolithic concrete for the secondary lining. When using shuttering (formwork), the inner face is fixed; therefore, the concrete volume between the primary lining and the formwork depends entirely on the actual position of the primary lining face.

A major complication in the "project intent" phase of a historic tunnel reconstruction is the inability to conduct a comprehensive geotechnical survey. Boreholes drilled through the existing lining, which would reveal the condition of the rock mass and the actual excavation face, are typically not feasible at this stage.

Therefore, the proposal for enlarging the excavation, ensuring stability, and selecting a waterproofing system was based on a predicted excavation face, its irregular shape, and a combination of mechanical excavation and drill-and-blast methods. To minimize investment costs, it was proposed to level the new excavation profile using a sprayed waterproofing membrane, which does not require the same substrate flatness as a waterproofing foil. The wire-mesh reinforced shotcrete of the primary lining would follow the excavation contours, serving as the substrate for the membrane, while any remaining irregularities would be filled during the placement of the monolithic secondary lining.

13. FURTHER DEVELOPMENT OF THE PROJECT

Only during the subsequent stage of project documentation was it possible to execute 12 boreholes, each approximately 5 m long, drilled toward the crown of the vault where the maximum overburden was expected. A borehole camera survey was conducted to assess the condition of the backfill behind the lining and the quality of the rock mass.

Contrary to initial fears, significant overcuts were not confirmed. It became evident that through the profile enlargement process, the designed excavation face could be achieved without major irregularities. Consequently, a primary lining reinforced with lattice girders (truss frames) and steel

wire mesh was designed, ensuring a surface geometry that meets the strict requirements for a sheet waterproofing membrane.

In all boreholes, a void was identified between the back of the existing lining and the excavation face. This indicated that the lining had not been "activated" at the crown; the excavation had remained stable for 125 years even without direct support from the lining in that area. The camera survey revealed rock fragments within the cavity, some originating from the original packing (backfill) and others apparently resulting from spalling off the unsecured excavation face. Figure 14 presents two of the twelve images from the camera survey behind the tunnel lining. Core samples taken from the boreholes were used to determine the geomechanical strength characteristics of the rock mass, which served as the basis for the final stability design of the enlarged excavation.



Fig. 14 Cavities captured by the camera behind the original tunnel lining

The geotechnical survey results enabled a refined prognosis of the rock mass condition behind the lining, allowing for the optimization of the construction sequence and the stabilization methods during profile enlargement. The core approach adopted was the New Austrian Tunnelling Method (NATM), defining three technological excavation classes. The primary lining consists of lattice girders (truss frames) and steel wire mesh, with the rock mass secured by SN anchors and the excavation contour supported by steel spiles (needles).

In the portal sections (Excavation Class 4), a primary lining thickness of 300 mm was designed, decreasing to 200 mm and 150 mm in the subsequent classes. Similarly, the advance length (excavation step) was adjusted from one metre in the poorest geotechnical conditions to two metres in more stable rock. Rescue refuges were incorporated into the tunnel design, housing inspection shafts for the maintenance of the tunnel drainage system.



Fig. 15 Cladding of the new portal with natural stone and integration of the tunnel into the landscape

The railway superstructure was designed with a single-sided cross-slope, allowing the drainage system to be positioned along the footing of the secondary lining. This configuration facilitates cleaning via the shafts shared with the lateral tunnel drainage. The waterproofing system was designed as an umbrella system, utilizing gravity drainage to direct groundwater toward the lower portal. Situated in the

picturesque foothills of the Jizera Mountains, the designer successfully advocated for the use of natural stone cladding on the new portals. Consequently, the tunnel serves not only as a vital engineering structure but also as a landscape-defining element, as illustrated in Figure 15.

14. TUNNEL CONSTRUCTION

The reconstruction of a tunnel on an operational line requires meticulous planning of individual activities to minimize the duration of the closure. In the case of the Dolnolučanský Tunnel, the schedule was coordinated with the simultaneous closures associated with the reconstruction of four other tunnels on the Tanvald–Kořenov section.

The total closure was scheduled from 15 April 2025 to 26 September 2025. Within this timeframe, it was necessary to complete not only the tunnel reconstruction itself but also the removal of the railway superstructure, the temporary relocation of cables (followed by their installation into new cable ducts), and the construction of two culverts adjacent to the tunnel portals.

Since the railway superstructure in this section had been renovated only 10 years prior, the client requested the reuse of both the ballast bed and the track panels following the works. The contractor, SUBTERRA, had to manage difficult site access and extremely cramped conditions at the portals—which are situated in deep rock cuts—while operating within the constraints of a single-track tunnel (see Figure 16).



Fig. 16 Cramped space conditions at the tunnel portal in a deep rock cut

These constraints complicated both the transport of materials and the installation of the working platforms required for fixing the waterproofing membrane, reinforcing the secondary lining, and positioning the sliding formwork. Before the tunnel enlargement could commence, it was necessary to dismantle the high-strength netting securing the unstable rock faces at the portals, demolish the historic portal structures, and widen the rock cuts to accommodate the new tunnel profile.

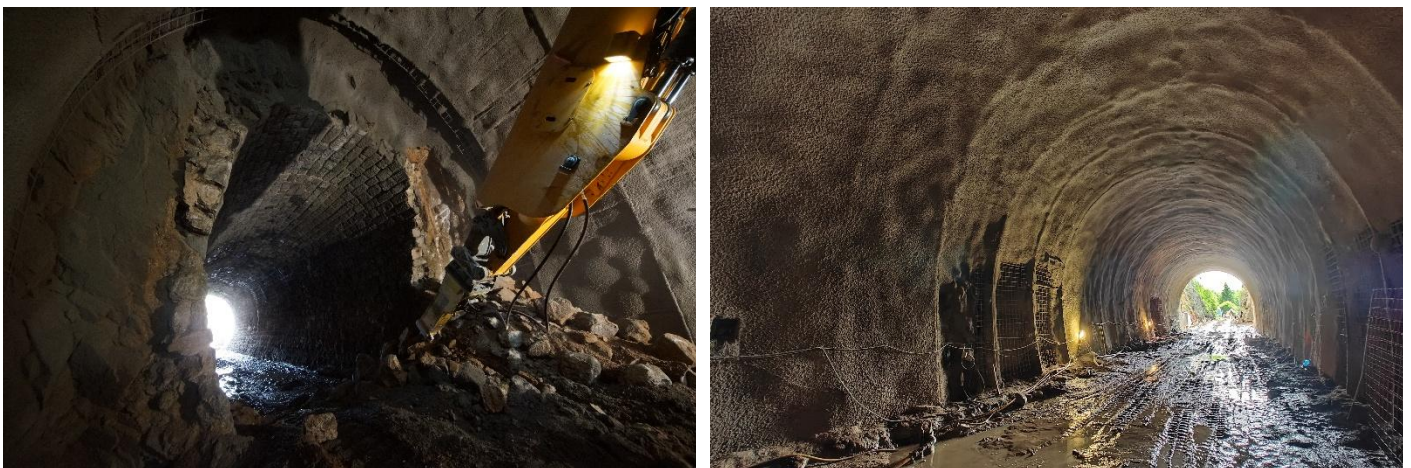


Fig. 17 Demolition of the existing tunnel lining and securing the enlarged excavation with primary lining

All operations had to be conducted with access to only one portal, meaning the opposite portal could only be reached through the tunnel itself. The excavated ballast and track panels removed from the railway superstructure had to be transported to landfills outside the construction site. The exposed formation level (after the removal of the superstructure) served as parking for construction machinery and as the primary access road to the portal. During the widening of the rock cuts, the predicted presence of fractured and weathered granodiorite was confirmed; nevertheless, it was possible to maintain the excavation geometry as envisioned in the design. The enlargement of the rock mass to the projected excavation profile was initially performed using a tunnel excavator. However, as the distance from the portal increased and the rock quality improved, drill-and-blast methods had to be employed. Works progressed from the Tanvald portal. The use of lattice girders (truss frames) and steel wire mesh ensured the structural integrity and geometry of the primary lining (see Figure 17).

Profile enlargement was completed in mid-June 2025, immediately followed by the placement of bench concrete at the tunnel floor and the casting of the secondary lining footings. These footings served as a stable base for the working platforms used for fixing the waterproofing membrane and installing the self-supporting reinforcement for the secondary lining.

Lateral tunnel drainage was installed behind the lining footings. Within the rescue refuges, inspection shafts were constructed for cleaning the lateral drainage, including connections to the railway superstructure drainage system. Due to the restricted space, the formwork carriage for the secondary lining had to be lowered in sections by crane from the area above the portal and assembled on-site, partly within the tunnel itself. While the insulation and reinforcement works proceeded from the Tanvald portal toward the Liberec portal, the concreting of the secondary lining—which commenced on 12 August 2025—was carried out in the opposite direction, from the Liberec portal toward the Tanvald portal (see Figure 18).

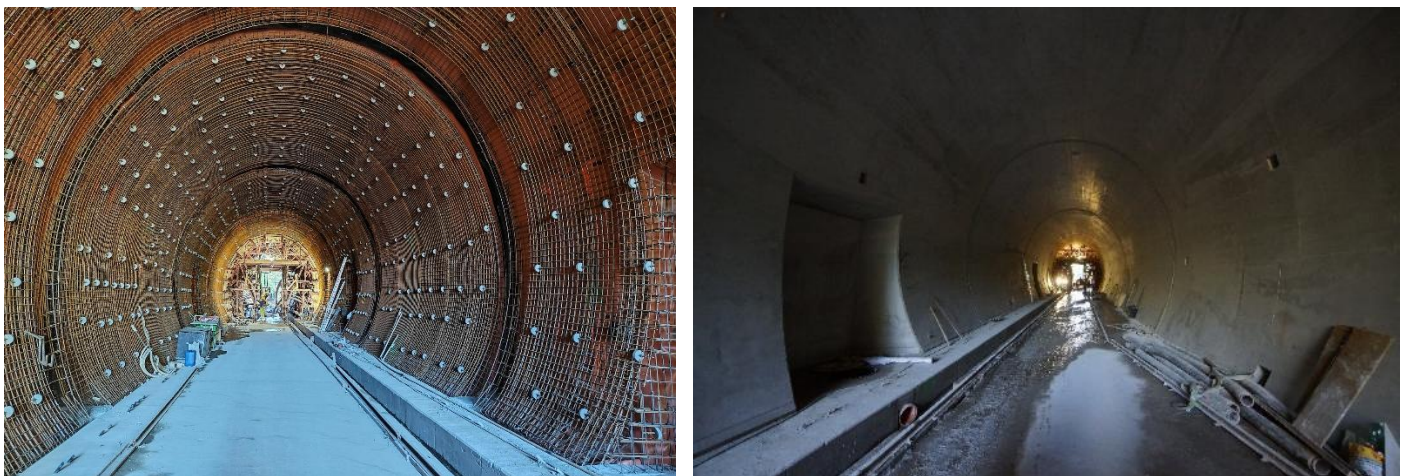


Fig. 18 Reinforcement and concreting of the secondary lining

A total of nine concrete blocks, each with a length of 10 m, were cast. The secondary lining, with a thickness of 300 mm, was reinforced along the entire length of the tunnel and constructed using C 25/30 XC1 XF1 concrete. The portal collars (P1 and P2) were designed using C 25/30 XC2 XF3 XA1 watertight concrete.

The formwork for the portal collars was partially formed by a false shotcrete shell (an extension of the primary lining). Because the gap between this false primary lining and the rock slope was too narrow to allow for the compaction of backfill material, the space was filled with a fly ash-cement grout. This mixture was pumped from tanks positioned above the portal directly behind the false shotcrete structure.

Following the completion of the secondary lining, new cable ducts and service walkways were cast on both sides of the tunnel. Communication and safety equipment cables were pulled through, and the emergency lighting system was commissioned. In keeping with the original historical portals, the new portals were finished with natural stone cladding and topped with stone cornices. Following the completion of the interior fittings, the installation of safety signage, and the restoration of the railway superstructure, the tunnel entered trial operation on 15 September 2025.

15. PROJECT ASSUMPTIONS AND REALITY

To verify the technical design assumptions, geotechnical monitoring and observations were conducted throughout the construction period. Following the controlled demolition of the historic tunnel lining, it was revealed that the original excavation face—executed using late 19th-century techniques—was more accurate than anticipated, with minimal voids and backfill behind the lining.

Monitoring points were installed on the rock cut slopes to detect any spatial displacement. Similarly, measuring profiles were established within the tunnel to monitor 3D deformations of both the primary and secondary linings. Comprehensive geological documentation was also maintained for the rock slopes and the expanded tunnel face.

Despite the shallow overburden and long-term seepage of surface water through the fissure system (which had essentially turned the historic tunnel into a massive drain), the rock mass remained stable during the widening process. No significant over-excavation occurred, and measured deformations remained below expected values, in the order of millimetres. Although core samples had initially indicated weathered rock with low strength, blasting proved necessary for the profile enlargement. Concerns regarding the excessive opening of discontinuities or rock blocks falling from the new excavation face were not realized.

16. CONCLUSION

The initial production committee meeting for the Dolnolučanský Tunnel reconstruction project took place at the end of January 2021. The design phase spanned nearly four years, followed by a construction period of approximately five months. Developing the technical solution required the evaluation of numerous variants, each accounting for the current state of knowledge and the uncertainties regarding the historic structural design and the rock mass. It was essential to assess the risks arising from these uncertainties and propose a reconstruction method that would mitigate them effectively.

The original concept—sealing the joints of the existing lining and restoring its waterproofing—evolved over time into a decision to completely rebuild the structure, creating a new tunnel that meets all contemporary technical and safety standards. The proposed technical solution, submitted as a project prospectus (project intent), met all criteria set by the client in the tender documentation. However, as it was drafted without precise data on the rock mass quality or the original excavation geometry, the initial proposal included a spray-applied waterproofing membrane and a stabilization system designed to handle significant over-excavation. Only after supplementary geotechnical surveys were conducted in the subsequent design stage was the solution refined, allowing for the use of a standard sheet waterproofing membrane to ensure the tunnel's watertightness.

The construction of the new tunnel represents a significant qualitative leap compared to the originally envisioned scope. The project involved a complete overhaul of the waterproofing and drainage systems, the installation of high-capacity cable ducts, and the creation of rescue refuges to ensure the safety of personnel during operational inspections. These improvements extend the tunnel's service life to 100 years. Furthermore, the initial investment will be offset by reduced operating and maintenance costs. The construction methods respected all environmental and landscape protection requirements; the use of natural stone on the portals sensitively integrates this modern engineering work into its surroundings. The investor (client) for the reconstruction is Správa železnic, s.o., and the general contractor is SUBTERRA a.s. The documentation for all stages—from preliminary design to as-built documentation—was authored by SAGASTA s.r.o.

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