Utility tunnel for Václav Havel Airport in Prague – project assumptions and reality

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ABSTRACT: This paper summarises the experience gained during the preparation of the detail design documentation and verifying the assumptions of the technical solution during the construction of the mined section of the utility tunnel for Václav Havel Airport in Prague, which is part of the project "Construction 1 – Hangar G – Utility tunnel".

The mined tunnel, less than 300 m long, was constructed according to the principles of the New Austrian Tunnelling Method (NATM). The utility tunnel profile, which with its area of 29 m^{2}exceeds for example the Prague metro line tunnel, was excavated with only 4 to 6 m of overburden. The excavation in difficult boundary conditions included a number of technical challenges and risks. It is especially the excavation with low overburden in clayey soil under the frequented main road and utility constructions, especially high- and medium-pressure gas pipelines.

The article describes the stage of processing the detail design documentation for the primary and secondary lining, including optimisation of the cross-section of the utility tunnel profile, modification of the excavation support class, reinforcing the stability of the excavation and the tunnel face, and specifying details of the secondary lining water-tightness. The actual start of the design work was preceded by a careful evaluation of the available information of geotechnical conditions along the tunnel route and the utility constructions in the assumed zone of influence. The modifications of the project documentation enabled significant savings in investment costs and simplified construction. The article also focuses on the experience gained during the construction from the designer's perspective and the application of the observation method in difficult geotechnical conditions of excavation with low overburden, during which further optimisations were carried out. The article also includes a description of the design and construction of secondary lining without the use of intermediate waterproofing.

1. INTRODUCTION AND BASIC INFORMATION

The mined section of the utility tunnel for Václav Havel Airport in Prague is part of the "Construction 1 - Hangar G – Utility tunnel" project. In addition to the mined section, the project also includes an “cut and cover” section, which is not the subject of this article. The utility tunnel will connect the airport's existing and planned utility routes. Design work on the detail design documentation for the mined tunnel began in December 2024. The tunnel was excavated between May and November 2025. This was followed by work on the secondary lining, which is expected to be completed at the end of February 2026.

The investor for the project is Prague Airport, a.s. The mined section of the utility tunnel is being carried out by Pohl CZ, a.s., the main contractor is Metrostav DIZ, s.r.o Technical supervision of the mined tunnel is provided by SG Geotechnika, a.s. The main designer of the entire project is Ingutis, s.r.o, and the author of the detail design documentation for the mined tunnel is the design company Sagasta, s.r.o

The profile of the mined utility tunnel is horseshoe-shaped with an invert and its excavation area of 29 m^2 exceeds for example the Prague metro line tunnel. The tunnel was excavated using the New Austrian Tunnelling Method (NATM), top heading and core were excavated separately. The secondary lining is designed of the water-tight concrete without the use of intermediate waterproofing.

The mined section of the utility tunnel route, with a length of 298 m, connects to the “cut and cover” shafts S09 and TK7. The section is divided approximately in the middle by shaft S08. The tunnel route leads straight along its entire length with two breaks L9 and L10 without any curvature of the axis (Figure 1).

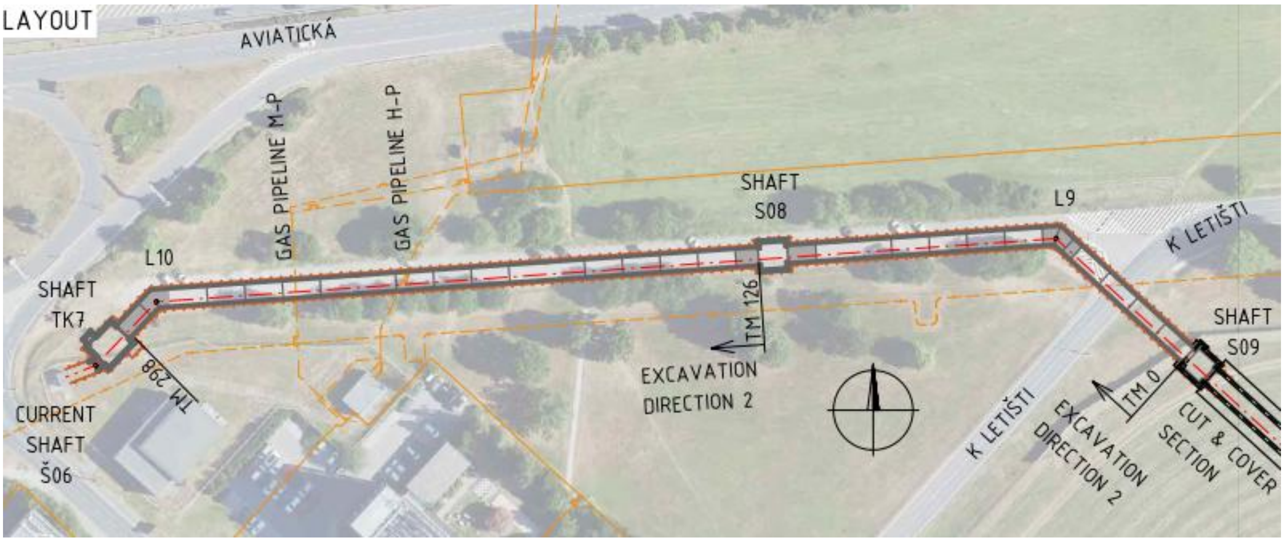

Figure 1 Layout of the mined section of the utility tunnel

The tunnel grade line rises for the entire section in order to connect to the current S06 shaft. From the start of excavation (TM 0) in shaft S09, the grade line rises at a slope of 0.6 % to shaft S08, and in the next section of excavation from S08, the grade rises at a slope of 2.5 % to shaft TK7; the elevation breaks are not rounded. The tunnel excavation was carried out at two independent work sites from shafts S09 and S08.

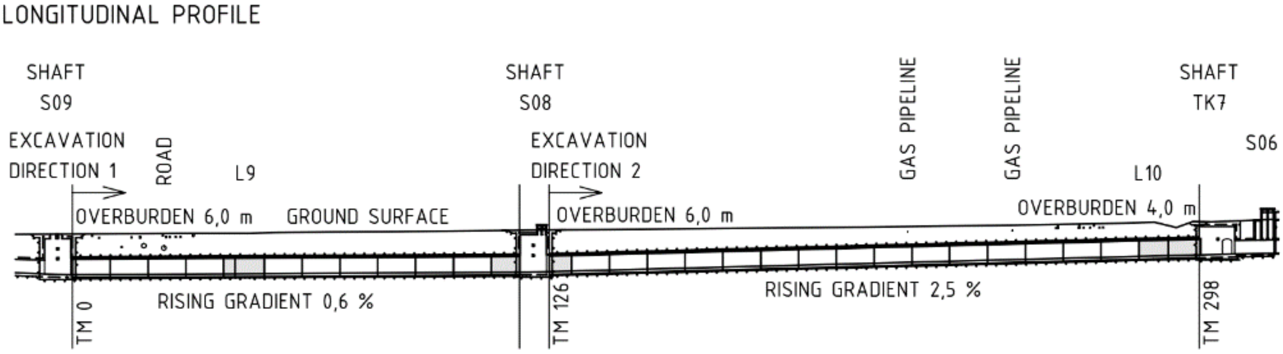

Figure 2 Longitudinal profile of the mined section of the utility tunnel

There are a relatively large number of structures in the tunnel overburden, which height is only from 4 to 6 m. The tunnel route is crossed by two gas pipelines, three sewers, a culvert and a number of communication and power cables in overburden. Two water mains and a non-functional hot water pipe run parallel to the tunnel. In terms of "sensitivity to deformation" caused by excavation, gas pipelines (high-pressure and medium-pressure) are critical. Another important structure on the surface above the excavation profile is the road "K Letišti" with frequent bus traffic. The presence of these structures, which could not be relocated, was the reason for choosing mined technology instead of the technically simpler and less risky "cut and cover" method.

2. GEOTECHNICAL CONDITIONS

Before starting the design work on the detail design documentation, the designer had the results of a geotechnical survey carried out by Chemcomex, a.s. in 2019. A total of five core boreholes with depths of 5 to 12 m were available in the area of the mined utility tunnel, which is less than 300 metres long. The geotechnical layers of ground were classified into several geotechnical types (GT) based on their properties.

The Quaternary cover, approximately 6 m thick, consists of fill, loess clays of class F6–F5 and loess (GT3b) of class F6, as well as deluvial clays and sandy clays with fragments (GT4) of class F8 to F4.

The bedrock consists of sandy marlstones of varying rate of weathering and fracturing. Strongly weathered sandy marlstones (GT5) with intense fracturing, classified as R6-R5, occur near the surface of the bedrock. Deeper down, there are weathered sandy marlstones (GT6), fractured with open cracks filled with clay, class R5-R4. The layers and cracks of marlstones are practically horizontal. The thickness of the marlstone slabs is variable and ranges from a few centimetres to a few decimetres.

The height of overburden in mined section is between 6 m (at shaft S09) and 4 m (at shaft TK7), and the invert of the tunnel is approximately 12 or 10 m under the ground surface. In these conditions, the tunnel profile near shaft S09 is located in weathered marlstones (GT6), partly also in strongly weathered marlstones (GT5), while the entire height of the overburden is formed by Quaternary cover. In the direction of the tunnel route to the shaft S08, the level of the tunnel rises. From shaft S08 towards shaft TK7, part of the top heading is excavated in Quaternary cover (GT4), also with the occurrence of loess (GT3b). Near shaft TK7, practically the entire top heading is excavated in Quaternary cover. The foundation of the top heading is always located in GT6 (R5–R4) bedrock materials through the entire mined section.

The groundwater level is located approximately 10 to 16 m under the invert and therefore does not affect the excavation.

The ground was described by geotechnical parameters according to the following table. These are the values of bulk density (γ), deformation modulus (E_{def}), Poisson's ratio (ν), effective angle of internal friction (φ) and effective cohesion (c).

Table 1 Geotechnical parameters of ground

Soil / Rock	Geotype	Classification	γ	E_{def}	ν	φ	c
			[kN/m^3]	[MPa]	[-]	[°]	[kPa]
Fill	GT2		19,0	7,0	0,40	20	5
Loess	GT3b	F6 (F5)	19,0	7,0	0,40	22	20
Clayey sediments	GT4	F6, F7, F8	20,0	15,0	0,35	22	14
Strongly weathered marlstone	GT5	R6-R5	20,0	30,0	0,35	25	30
Weathered marlstone	GT6	R5-R4	20,0	130	0,35	28	50

3. OPTIMISATION OF THE CROSS-SECTION SHAPE

In the first step, the detail design documentation proposed optimising the cross-section shape of the utility tunnel (Figure 3). This proposal was based on a complex evaluation of the available information of the geotechnical conditions along the tunnel route, consideration of the influence of the excavation shape on reinforcing of stability during excavation, and evaluation of the static function of the primary and secondary lining.

The optimised cross-section has a smaller theoretical excavation area and a more favourable shape of the primary lining, which better resists earth pressure, increases the stability of the structure during excavation and thus reduces the risk of an extraordinary event.

In the case of secondary lining, optimisation is related to adapting the dimensions of the structure to the expected load. The upper vault thickness gradually increases from the top to the bottom sides. Furthermore, the new design increases the cross-section thickness at the connection of the upper and lower vault, where the increased values of bending moments and shear forces is expected. The aim of the optimisation is to ensure more favourable loading conditions of the structure and to reduce the risk of cracks in the secondary lining. Besides the support function, the lining also ensures the water-tightness of the utility tunnel. The formation of cracks, which can only be repaired only from the intrados of the tunnel, may have a negative impact not only on water-tightness but also on the service life of the load-bearing structure, which is designed for a service life of 100 years, due to the risk of corrosion of the reinforcement.

The internal clear dimensions of the secondary lining are designed with regard to the required passageways and the number of utility lines. The clear width of the utility tunnel is 4.25 m and the clear height is 4.10 m.

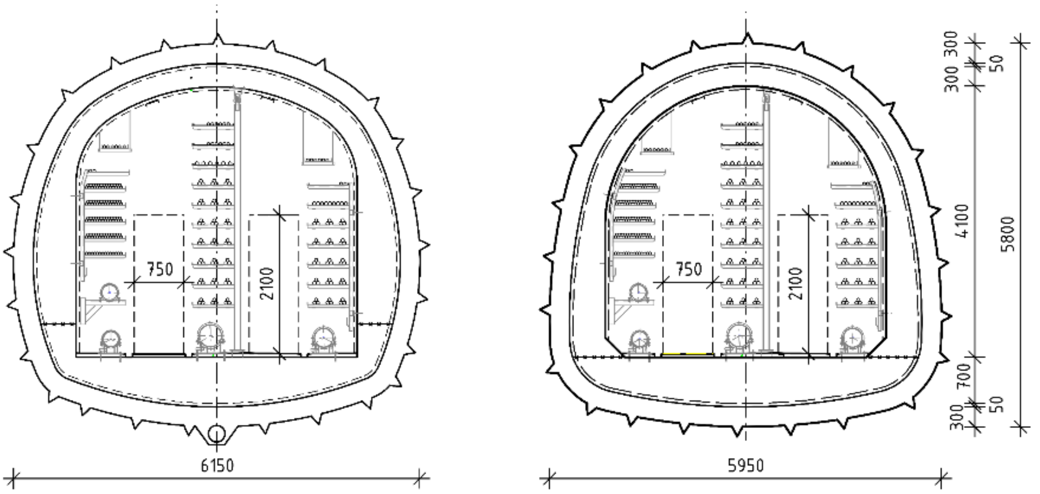

Figure 3 Comparison of the original and new cross-section tunnel profile

4. EXCAVATION AND PRIMARY LINING

4.1 ORIGINAL DESIGN

In the original design the tunnel face was divided into a top heading and a core, the top heading was in addition vertically divided into the two sequences (Figure 4). The primary lining was designed with a thickness of 300 mm with two layers of 10/100x10/100 mm wire mesh and lattice girders, the top heading was reinforced in the middle only with a 70 mm thick layer of shotcrete with one 10/100x10/100 mm wire mesh. A relatively flat invert of top heading was designed of 200 mm thick shotcrete reinforced with two layers of 10/100x10/100 mm wire mesh. The top heading face (both sequences) was reinforced with 15 fibreglass rockbolts 8 m long, and the core face with 5 rockbolts of the same type. With the required overlap of 2 m, the face with an excavation area of approx. 30 m^2 in this part was reinforced with a total of 40 fibreglass rockbolts. In the first excavation sequence of the top heading was designed 3 steel forepoles with a length of 6 m. In the second partial excavation sequence of the top heading was designed another 6 forepoles. The required overlap of the forepoles was 2 m.

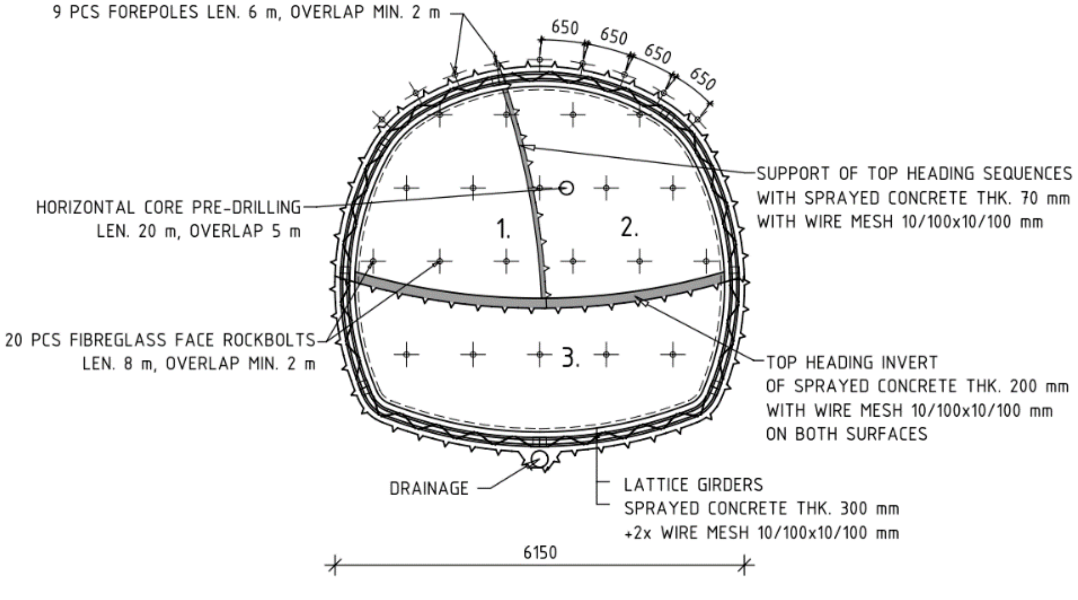

Figure 4 Original design of the excavation support class with vertical face sequencing of top heading face (tender design)

4.2 NEW DESIGN

The new, more favourable static shape of the cross-section, together with other design modifications, made it possible to eliminate the vertical division and the invert of top heading (Figure 5). The removal of these elements was important for optimising the technological process of the construction (powerful excavation, mucking and spraying machinery). In the case of unfavourable geotechnical conditions with low overburden, the speed of excavation is very important. One of the main objectives of optimising the excavation design was to allow the contractor to support the stability of the advance relatively quickly with primary lining after the excavation. Furthermore, from a static point of view, it is not optimal to connect the reinforcing lattice girders in the most stressed cross-section at the top of the vault, which would require the originally proposed vertical division of the top heading.

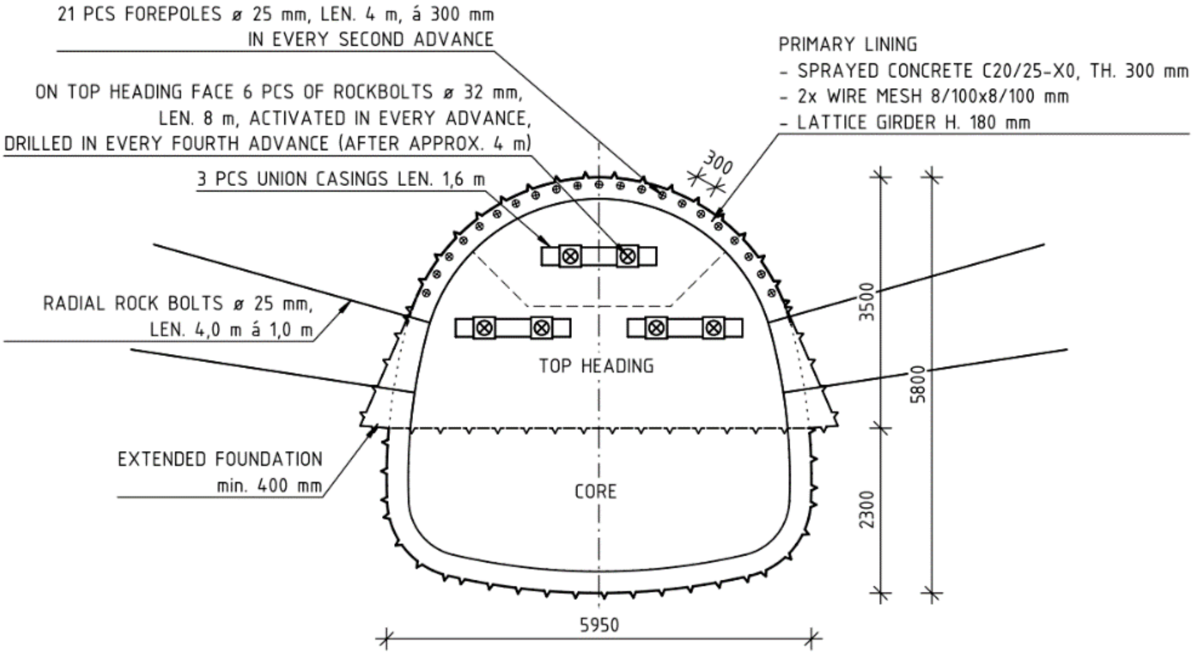

Figure 5 Optimised design of the excavation support class

In the new design, the function of the lower vault of the top heading was replaced by a combination of radial rockbolts and an extended foundation of top heading. The four selfdrilling (grouted) radial rockbolts per advance were designed to limit horizontal deformations. The purpose of the extended foundation of top heading is to reduce the settlement of the primary lining. Another advantage of extended foundation of top heading compared to temporary invert is a more favourable effect during the core excavation. The phase of demolishing the temporary invert introduces new load into the mature top heading primary lining.

Another important modification of the new design is a change of the tunnel face reinforcement. This modification is designed to reduce the risk of face instability. In case of a loss of face stability, there is a risk of uncontrolled extension of excavation advance and subsequent breakthrough of the overburden. In the new design the face stability is improved with selfdrilling steel rockbolts activated with cement mortar. The steel rockbolts has higher shear strength than fibreglass rockbolts. The external thread of the selfdrilling rockbolts rods also allows the UNION casings to be tightened and activated after each excavation advance. Therefore, the fibreglass rockbolts in the face were replaced with 8 m long selfdrilling steel rockbolts (min. overlap 4 m) with active face support using UNION casings. In the new design, 20 fibreglass rockbolts have been replaced by 6 selfdrilling steel rockbolts.

To stabilise the excavation profile in advance are designed forepoles. In the new design, nine 6-metre-long Ø 25 mm forepoles are replaced by 21 forepoles of the same type with a length of 4 metres. The overlap of the forepoles in the longitudinal direction is 2 m and remains unchanged from the original design. The reason of this modification is to minimise over-excavation with regard to the length of the advance and the drilling angle of the forepoles.

All drilling for rockbolts and forepoles in the given geotechnical conditions was performed with air. The use of drilling with water would cause degradation of the rock mass properties and worsened conditions for further excavation. Due to this reason, horizontal core pre-drilling was also cancelled in the new design, and basic information of the conditions in the excavation foreground was continuously obtained from geotechnical monitoring during the drilling of face rockbolts.

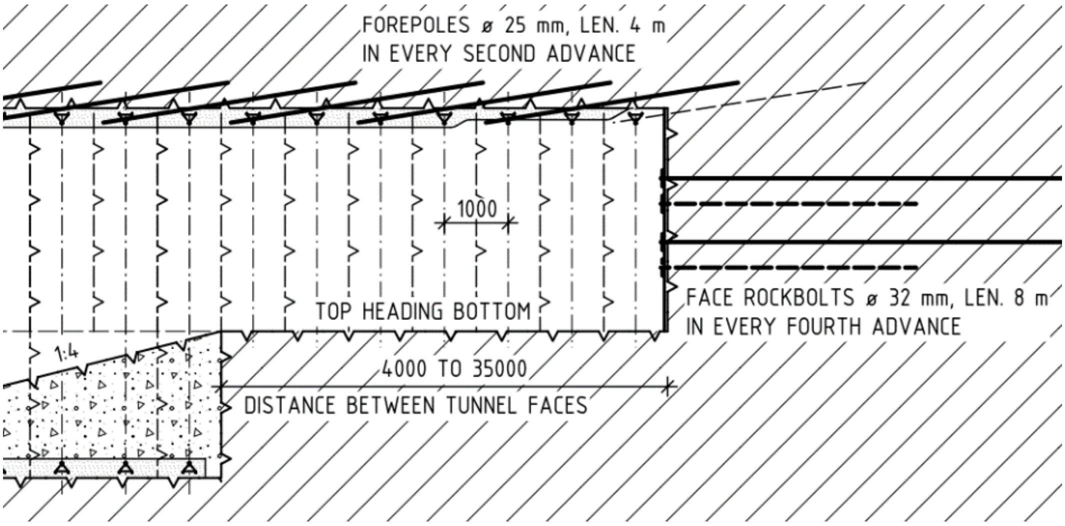

Figure 6 Longitudinal excavation procedure

Another significant optimisation was the modification of lattice girders and wire meshes. In the original design stage were designed rectangular frame cross-sections with 6 bars of ø 32 mm. Based on the new, more favourable static shape of the tunnel cross-section and the refinement of the load, more subtle frames with 4 bars ø 28 mm were designed, with a triangular cross-section achieved by placing the lower bars next to each other (Figure 7). This allows easier access for drilling forepoles with minimal deviation from the excavation outline. The reinforcement wire meshes of the primary lining were lightened from type 10/100x10/100 mm to type 8/100x8/100 mm. The design of more subtle frames and wire meshes is also suitable in terms of handling and installation in the tunnel.

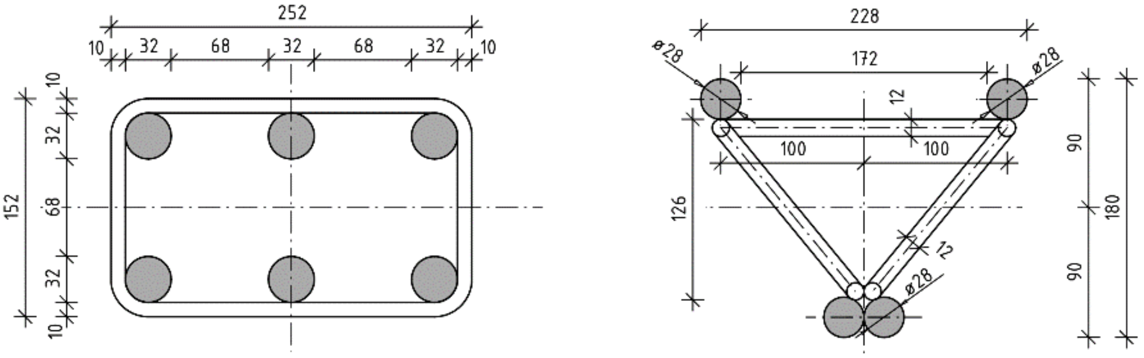

Figure 7 Comparison of the original and new cross-section of the lattice girders

4.3 APPLICATION OF THE OBSERVATION METHOD

The tunnel was excavated according to the principles of the New Austrian Tunnelling Method (NATM). Due to the low overburden, the size of the excavation profile and the ground conditions, only one excavation support class was designed. The support class covered the most unfavourable expected conditions in the entire mined section. Therefore, the observation method could only be used to a limited extent under the given conditions.

In the initial section of the excavation, the top heading excavation was carried out in three stages in each advance, with only one UNION casing removed at a time (Figure 8). After dismantling the UNION casing in the relevant part of the face, continues excavation of this part, spraying the temporary shotcrete, and finally installing the UNION casing back to a pair of selfdrilling face rockbolts. The installation of outer wire meshes and lattice girders could only be carried out after the complete execution of all three parts of the top heading, which was relatively time-consuming.

Figure 8 Tunnel face support and excavation sequence in the initial section

During excavation, the rock mass was monitored in detail, especially during the tunnel excavator works. This monitoring, from which the designer regularly took video recordings, together with the results of geotechnical monitoring, provided a complex view of the stability conditions of the tunnel face and the primary lining. Based on the experience gained with the specific conditions of excavation, a further adjustments of the primary lining were undertaken (Figure 9).

Based on the relatively favourable behaviour of the ground during the excavation, the top heading excavation procedure was modified to a one sequence and the requirement of systematic UNION casing installation was removed. These UNION casings remained ready on site and were installed only in the event of local instability at the tunnel face. Furthermore, the number of face rockbolts was reduced from 6 to 2 in every fourth advance. Only two face rockbolts were installed in the crown, on which it was possible to install a UNION casing if necessary. Radial rockbolts were also removed and the maximum permissible advance length was slightly extended. The actual advance length was always adapted to the specific conditions at the each face and ranged between 0.7 m and 1.1 m.

The horizontal marlstone slabs and the cohesion of the clayey soils can be considered as the main favourable factors. Any sandy lenses with water were not encountered during the excavation.

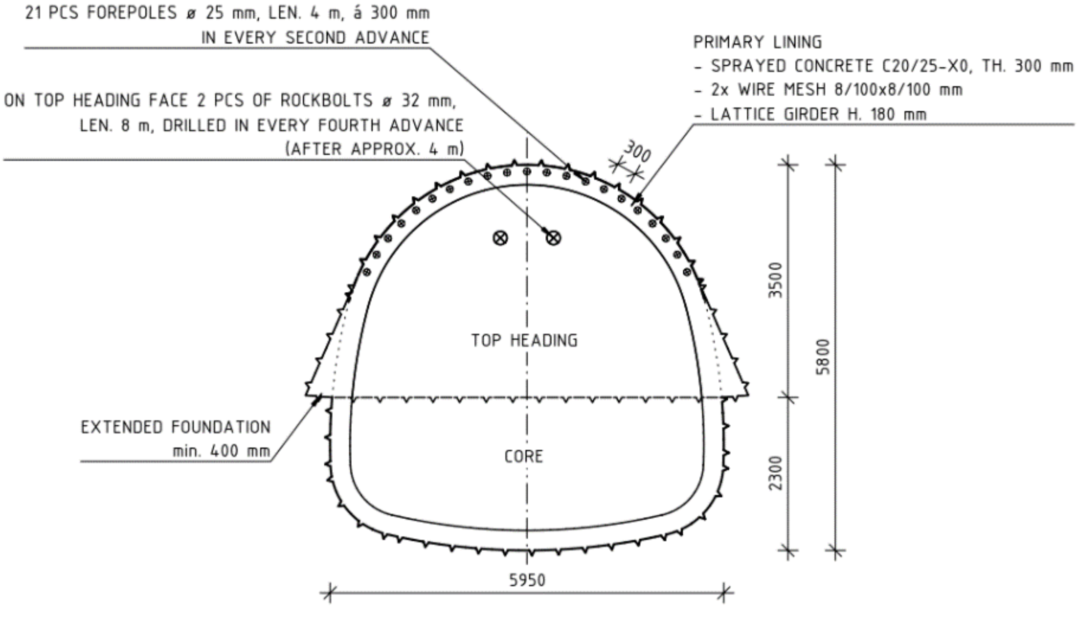

Figure 9 Primary lining adjusted based on the actual behaviour of the ground during excavation

Figure 10 Top heading face consisting of slightly weathered marlstone (TM 66)

Figure 11 Top heading face consisting of clayey soil (TM 264)

Figure 12 Core and invert face consisting of horizontal marlstone slabs (left), completed section of primary lining after profiling (right)

4.4 GEOTECHNICAL MONITORING AND EVALUATION OF ITS RESULTS

Complex geotechnical monitoring was carried out during the construction of the tunnel by GeoTec GS, a.s. Monitoring took place both on the surface and in the tunnel and involved geotechnical description of the faces in each advance, measurement of deformations of the primary lining and settlement on the surface.

The excavation of shafts and mapping of tunnel faces during excavation confirmed the geological conditions of the ground predicted by the survey. The bedrock level is located at a depth of approximately 6 m below ground level.

Due to the low overburden of the tunnel, measurements of ground surface settlement were taken across the entire mined section. Levelling points were arranged in profiles perpendicular to the longitudinal axis of the tunnel, the distance of the profiles was 20 m. These cross-sections were used to determine the width and shape of the settlement trough. In order to obtain complex information, points for determining the longitudinal settlement trough were added in the area of the road “K Letišti”, the section with gas pipelines and the ending section of the tunnel route with the lowest overburden.

In the selected profiles, a comparison of the results of static calculations and measured surface settlement during excavation was performed (Figure 13 and Figure 14). In general, these are very low measured values, so it is necessary to take into account the accuracy of the measurements, which is in the order of millimetres. A negative factor that certainly partially influenced the measurement results was the frequent movement of heavy construction machinery above the excavated profile. In the case of the profile with the lowest overburden, only points directly above the tunnel were available due to the work being carried out in the area of shaft TK 7. These results can be used to compile a longitudinal profile (Figure 15). The results of the measurements on the longitudinal profile show that a relatively large part of the deformations occurred after the excavation of top heading. Looking at the results of all measurements, it can be stated that the achieved ground surface settlement is basically close to the project assumptions. Practically none of pre-defined trigger limit was exceeded during the excavation.

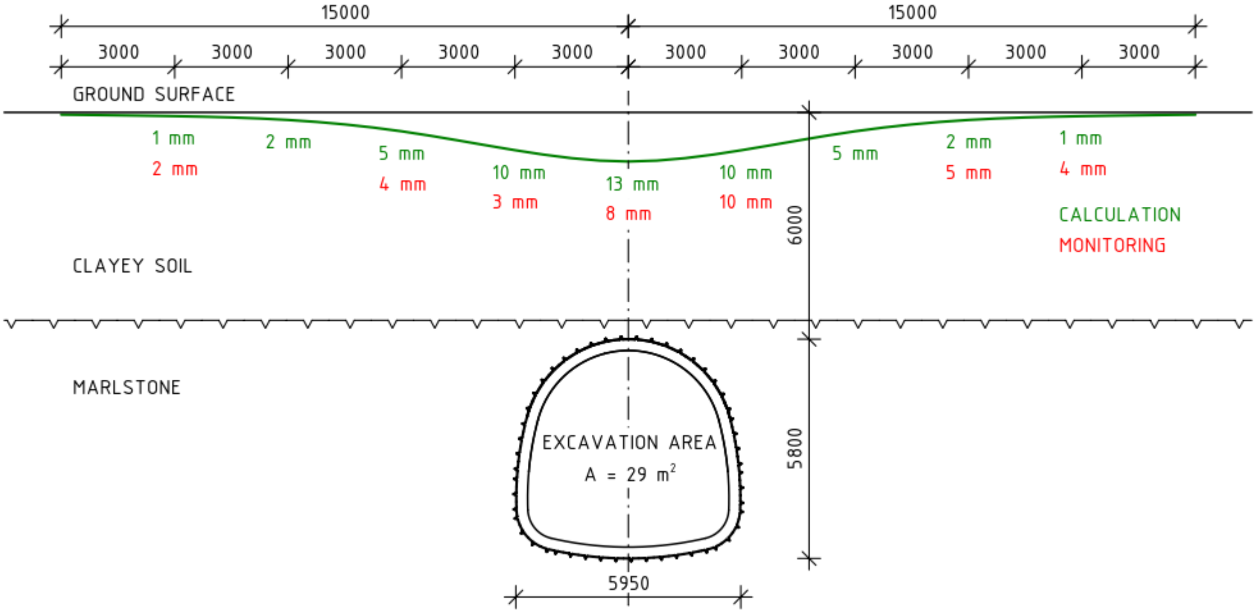

Figure 13 Settlement trough in the section with the highest overburden (NIT 7, TM 110)

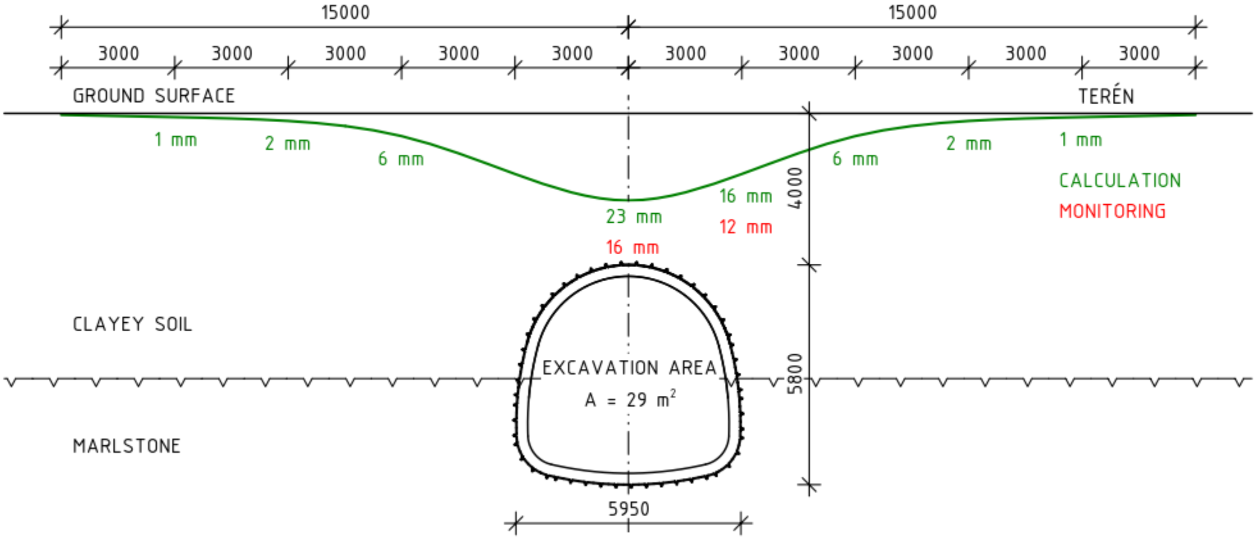

Figure 14 Settlement trough in the section with the lowest overburden (NIT 17, TM 280)

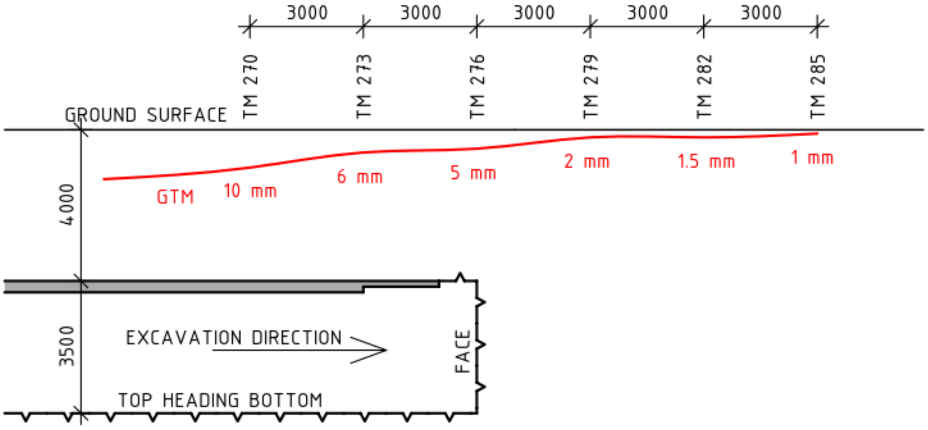

Figure 15 Longitudinal settlement trough in the section with the lowest overburden (NIT 21)

Another set of measurements results, which can be partially compared with the project assumptions, are the deformations of the primary lining (Figure 16). These measurements were carried out with 3 points in the top heading and 2 points in the core in each tunnel profile. The distance of the profiles were approx.. 10 m. An important aspect in evaluating these results is the timing of the installation of geodetic points. Due to the technological reasons, installation of these points usually takes place during the installation of internal wire mesh, which corresponds to a distance of approximately 2 to 3 m behind the actual tunnel face. For this reason, this type of measurement provides only limited information about the total deformation values. Most of the deformation of the ground occurs before the zero measurement at the installed points, and therefore the trend of the results is decisive.

In all profiles, the measured values stabilised reliably and the results were within the range of expected deformations specified in the project. The graph below also shows the relatively small influence of the core on the deformation of the lining in the top heading.

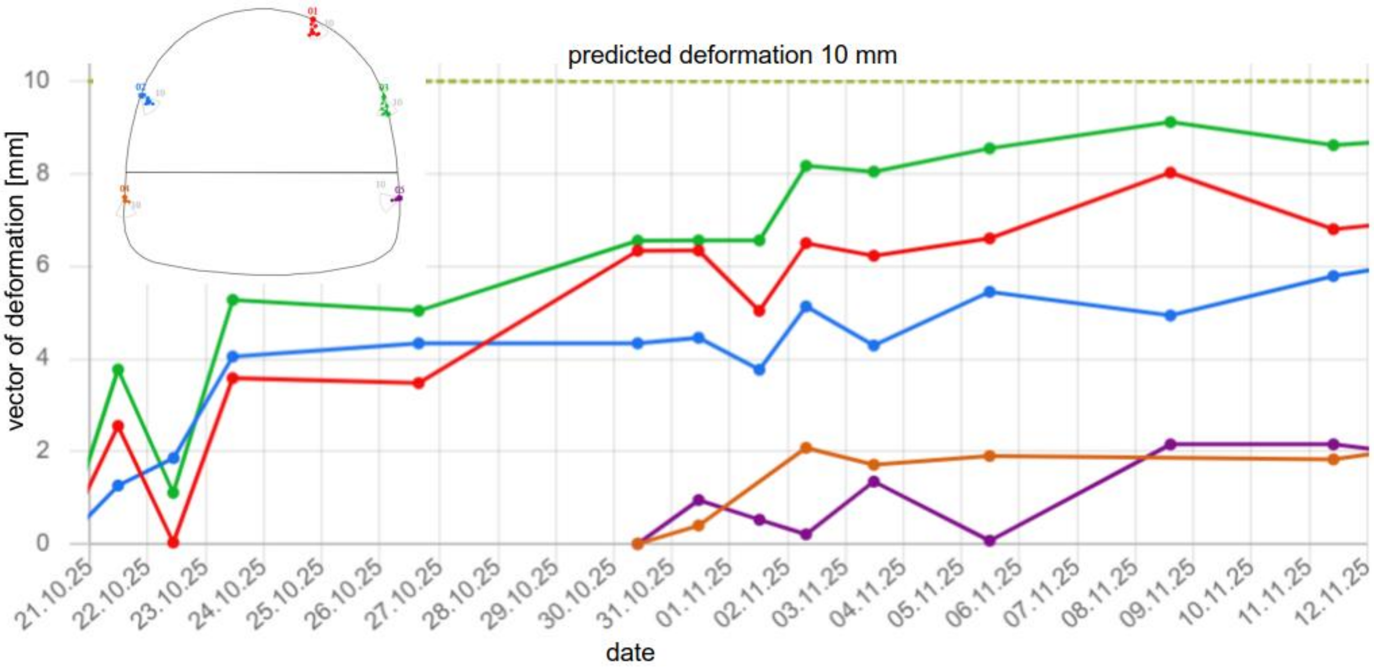

Figure 16 Deformations of the primary lining (TM 265)

The technical challenge was undoubtedly the excavation under the gas pipelines (high-pressure and medium-pressure). Before excavation in the given section, dug probes were carried out to verify the exact position and condition of the gas pipelines. Based on the results of the diagnostics, trigger limits were set for permissible pipe deformation caused by tunnel excavation. The gas pipeline was exposed in advance before the tunnel excavation in this area. On the exposed pipeline were carried out measurements of its deformation during the tunnel excavation. At the same time, a system of pipe hangers was prepared on site to compensate any excessive deformation of the overburden.

Figure 17 High-pressure gas pipeline (left), exposed pipeline (right)

5. SECONDARY LINING

The secondary lining is designed as cast-in-place concrete, concreted into sliding formwork. The thickness of the upper vault lining is at least 300 mm and widens towards the sides. Due to the required water impermeability of the secondary lining and the impossibility of gravitational drainage of groundwater, a closed pressure system is designed, which requires a lower vault (invert). The water impermeability of the lining is ensured by the use of leak-resistant concrete with sealing of working and expansion joints.

The design of secondary lining made of leak-resistant concrete without the use of intermediate waterproofing involves a number of specific requirements. These mainly include stricter criteria for the permissible width of cracks and the maximum permissible water leakage through the concrete. These requirements are then reflected in the design of the concrete mixture, where the reduction of the negative effects of hydration heat and shrinkage is important. Furthermore, a higher rate of reinforcement of the structure is designed with regard to limiting the width of cracks. At the same time, it is also important to maintain a sufficient clear distance between the reinforcement bars to allow high-quality concreting. In the area above the working joint between the lower and upper tunnel vault is designed additional longitudinal reinforcement to limit cracks caused by different shrinkage of structures that are concreted at different time. Another important aspect is the stricter permissible deviation from the theoretical thickness of the lining, which is defined by regulations as a maximum of 100 mm. For this reason, laser scanning of the primary lining and its profiling was carried out in several stages before the construction of the secondary lining.

During the preparation of the detail design documentation of the secondary lining, a number of changes were made compared to the previous stage of documentation. In accordance with foreign regulations, the maximum length of the concreting block was extended from 8 m to 10 m. The reason for the design change was to minimise the number of working joints between the tunnel blocks and reduce the risk of water leaks.

Furthermore, to ensure the water-tightness of the joints between the concrete tunnel blocks, the external sealing strips were replaced with internal sealing strips with an injection system. This system allows the injection of all working joints after concreting.

To reduce friction between the primary and secondary lining structures, a separation foil is installed between these structures. The application of this foil significantly reduces tensile stresses in the lining caused by temperature changes and concrete shrinkage, which leads to a reduction of the risk of cracking. The separation foil also prevents water from being drawn from the concrete of the secondary lining into the mature shotcrete of the primary lining. This foil is installed around the entire tunnel profile with overlap between the individual parts (without welding).

The reinforcement of the secondary lining was designed as a combination of welded wire meshes and bar reinforcement in the most stressed cross-sections. The reinforcement is designed as self-supporting with lattice girders, which parts are connected with simple couplers.

As the concreting takes place in winter, conditions are ensured during construction to prevent the concrete from cooling down rapidly. These measures include the transport of the concrete mix, its storage in the formwork and ensuring the required temperature during concreting in the tunnel. The tunnel portals in the shafts are closed to limit the flow of cold air, and the tunnel is also heated. After formwork removal, the concrete is treated by watering and the application of a protective material. The temperature is monitored during the construction. The aim of these measures is to limit the temperature difference between the surface and the core of the structure.

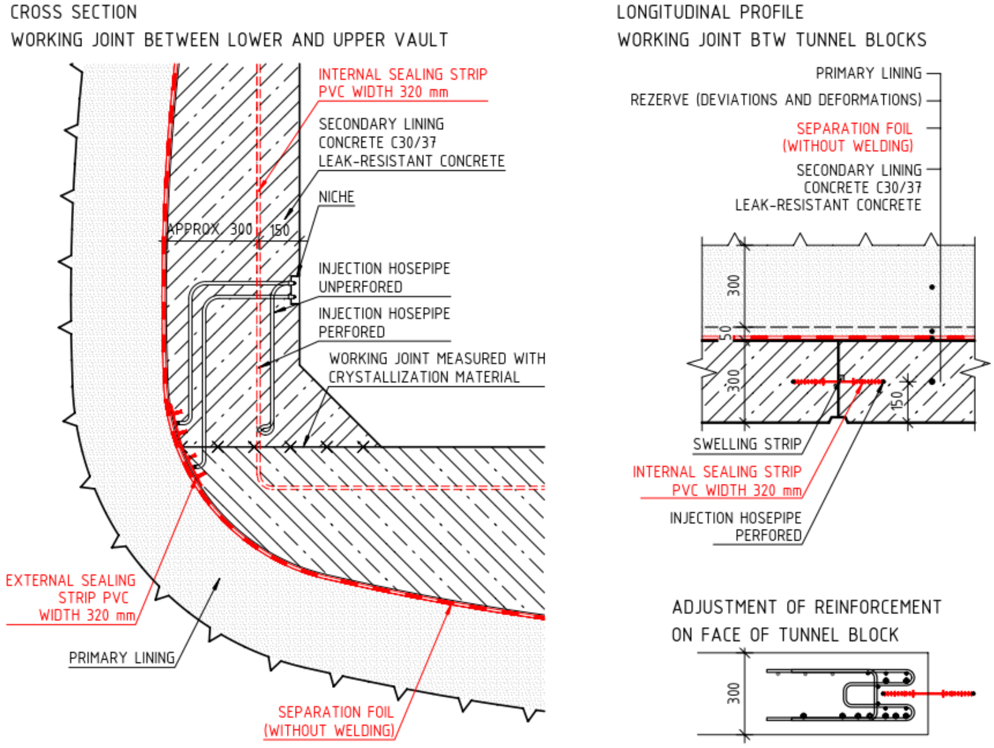

Figure 18 The designed system of sealing the working joints of the secondary lining

Figure 19 Separation foil and reinforcement of the secondary lining

Figure 20 Formwork (left), completed secondary lining (right)

6. CONCLUSION

During the preparation of the detail design documentation were made a number of optimisations for the mined utility tunnel for Václav Havel Airport in Prague. These optimisations have a positive impact on the service life of the structure. Furthermore, these changes enabled significant savings in investment costs and simplified construction, which was important due to the tight time schedule.

The construction of the utility tunnel, which exceeds the size of the metro line tunnel, is proceeding without any significant problems or complications. The tunnel excavation phase, which is the most risky part of the project due to a number of uncertainties, has been successfully completed at the time of writing. The tunnel excavation was extremely challenging due to the low overburden, adverse geotechnical conditions, the presence of important operated road and a number of utility networks above the tunnel. During the tunnel excavation the height of the overburden gradually decreased from 6 m to just 4 m. The overburden consists of clayey soils with sandy admixtures and fillings. During excavation, these soils even appeared at the tunnel face, and their proportion gradually increased as the overburden decreased. At the end of the section, the top heading was excavated only in these soil materials, rock bedrock consisting of marlstone slabs of variable quality was encountered in the core of the tunnel. In these conditions, the mined section under the important and highly frequented road "K Letišti" was a particular technical challenge in terms of minimising risks and limiting the negative effects of tunnel excavation on the overburden. This main road is particularly important for public transport between the “Nádraží Veleslavín” metro station and Václav Havel Airport, which is served by heavy electric trolleybuses with battery at relatively short intervals. Another technically challenging mined section was the excavation under the gas pipelines that supply the entire airport and were located less than 3 m above the top of the excavation profile. The route of the tunnel with low overburden also passed under several sewers, culverts and a number of other utility networks.

During the construction of the tunnel, the results of geotechnical monitoring were regularly evaluated. Based on the actual behaviour of ground during excavation, even in these complicated conditions, the stability of the excavation was relieved in accordance with the principles of the observation method, which resulted in further savings.

At the time of preparing the article, the upper vault of the secondary lining was being constructed in the utility tunnel (01/2026). Secondary lining is designed without intermediate waterproofing, which is unusual for mined tunnels in local conditons. The advantage of this system is that any leaks can be easily identified and repaired, the leaks occur at the exact area of defect. In case of intermediate waterproofing, the location of defect may not correspond to visible leaks inside the tunnel and repairs of these defects are practically impossible.

SOURCES

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