Tunneling in karst: a case study for HPP "Dabar" tunnel

Uroš Mirković, Slobodan Radovanović, Dejan Divac, Zoran Dankov, Dejan Vučković

Jaroslav Černi Water Institute, Jaroslava Černog 80, 11226 Pinosava, Belgrade, Serbia
(corresponding author: uros.mirkovic@jcerni.rs)

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Abstract: Within this paper, an overview of the construction of tunnels in karst terrains is given from detecting various unfavorable conditions, monitoring, testing, analyses, and adapting to actual conditions on the ground. In karst conditions, fault-cavernous mediums can be filled with crumbly, clayey matter, and water. Such phenomena represent serious risks during tunnel construction in terms of excavation collapse, damage to the machines, slowing down, and, in some situations, suspension of work on the tunnel route. All these phenomena lead to significant economic consequences during the construction of tunnels. The paper presents different concepts of predicting and adapting to different conditions that can occur in karst and that can be prevented or remedied. In addition to examples from world practice, the paper also mentions the HPP "Dabar" supply tunnel as a region structure representing a hydrotechnical tunnel under pressure with a length of 12125 m and which has been under construction since 2016. The modern concept of monitoring, testing, analyses, and integrating all relevant information during construction was applied to this tunnel to adapt to actual conditions in the field.

INTRODUCTION

Karst is formed when carbonate rock types degrade in contact with water of varying temperatures and pressure. This phenomenon is common in all calcite rocks, such as limestone, dolomite, and marble. When excavating tunnels in karst rock masses, karst formations can have severe consequences for safety, the performance of works, and the environment. Depending on the chosen tunnel construction technology, there are different ways to overcome problems in karst. Generally, two technologies are used for tunnel construction: the new Austrian tunneling method (NATM) and the construction method using tunneling boring machines (TBM).

The new Austrian method implies that the load-bearing structure consists of an integrated system of built-in structural elements (primary support) and part of the mountain mass in its surroundings, where the goal is to ensure and preserve the load-bearing capacity of the rock mass to the greatest extent possible. The primary support takes on the underground pressures and stabilizes the rock mass. Additional safety is provided by installing secondary shotcrete protection [1]. When applying NATM, excavation is performed classically (drilling and blasting) or mechanically (using machines that enable excavation without significant cuts and damage to the rock structure). The primary tunnel structure mainly consists of a layer of shotcrete applied along the borders of the excavation, that is, if necessary, reinforced steel arches and a system of anchors. In contrast, the secondary structure consists of an unreinforced or reinforced concrete structure.

Tunnel boring machines (TBM) are also used to excavate long tunnels in karst. These machines excavate the rock with the help of a rotating cutting head equipped with cutting tools (discs, blades). The buckets, placed on the cutting head's periphery, collect the excavated rock mass pieces, deposit them on the conveyor, and transport the rock mass out of the tunnel. The TBM can excavate a tunnel in full profile, temporarily supporting an unstable excavation, transporting excavated materials outside the tunnel, and forming a permanent tunnel structure. The advantages of using this method of excavation are a significantly higher rate of progress (4-6 times higher compared to classic excavation by blasting), continuous work, less damage to the rock mass in the zone of the tunnel profile, greater safety at work, and the possibility of remote control and automation. The disadvantages are reflected in high initial costs, a relatively long period required for the production, transport, and assembly of the machine, the need for space for the installation of the machine, the excavation of an exclusively circular tunnel profile, restrictions regarding the minimum radius of the route and, especially, great sensitivity to changes of geological conditions.

The design and construction of engineering structures in karst are very complex due to unforeseen construction conditions and the karst dimensions, geometry, and structure. Many researchers have dealt with the problem of tunnel construction in karst. The main issues that arise are difficulties in excavation, the collapse of arches, subsidence, and water seepage. Designers have used numerous solutions to solve the problems mentioned above: relocating the construction route, constructing bypasses, filling the hollows and cracks with concrete, improving rock mass characteristics by grouting or geogrids, and controlling surface water and groundwater [2].

This paper provides an overview of the challenges that arise during the construction of tunnels in karst, as well as a description of the remediation measures taken on several examples of tunnels from broader engineering practice. An example of the construction of a tunnel in the region (supply tunnel of HPP "Dabar", Bosnia and Herzegovina) is also presented, with an overview of the design of this tunnel, the monitoring, tests, and analyses carried out during construction, as well as the technical solutions for adapting to actual conditions in the field.

The supply tunnel of HPP "Dabar" is a hydrotechnical tunnel with a length of 12125.0 m for the supply of water from the Nevesinje reservoir to the surge tank from which it will be transported through pipelines to the machine hall of the hydropower plant "Dabar" in Dabarsko Polje, where generators for the production of electricity will be located.

The dominant type of relief is of karst origin. This area is rich in all forms and phenomena that characterize the complete karst. These are karst fields, karst springs, occasional streams, estavelles, swallow holes, pits, caves, dry valleys, sinkholes, slopes, and all variations and subcategories of the forms mentioned above. The degree of karstification depends on the lithological composition, structure, age, and neotectonic movements.

CHALLENGES AND ACHIEVEMENTS DURING THE CONSTRUCTION OF TUNNELS IN KARST

During the construction of tunnels in karst, the following challenges may arise faults, caverns, rock bursts, or high groundwater seepage. These challenges often occur in combination, so it is possible to encounter cracks/caverns that are dry and filled with water or material (Figure 1).

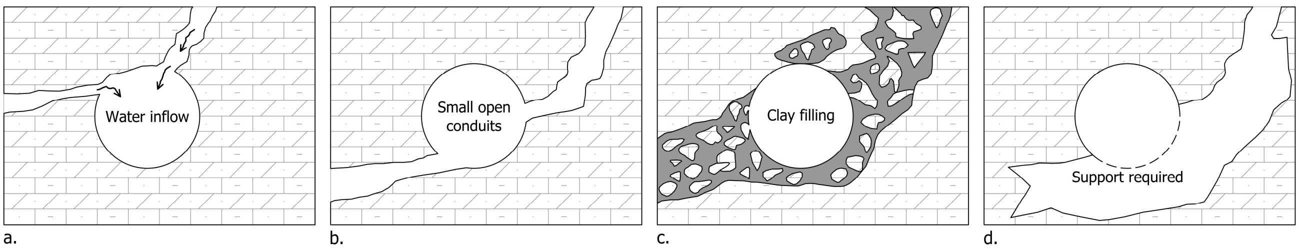

Figure 1. *Different types of karst formations a) cracks and pockets filled with water b) smaller open dry conduits c) karst cracks and pockets filled with clay material d) large karst caverns that require filling with adequate material [3]*

Faults are mechanical discontinuities with sides that have moved relative to each other by sliding. As a result of shearing, a rock mass fracture occurs, where the fault surfaces are polished and with occasional furrows. Faults produce disturbances in the rock mass and represent weakened areas. Crushed rock mass zones are often found through which surface water is drained. Active faults can disrupt or compromise planned construction.

Networks of canals and caves (caverns) of various sizes are often found in karst landscapes. In carbonate rock masses, these phenomena are caused by water that passes through the cracks and dissolves the rock mass. These cavities are often filled with water, sludge, gravel, or a combination of these elements.

Dealing with such conditions during excavation is a significant challenge that can be overcome by bridging or filling if the crack is empty, stabilizing soft material, and controlling the water before the excavation head approaches the channel or cavern [4]. The consequence of the presence of a high level of pressure, which exceeds the strength of the rock mass on the boundaries of the underground opening in rigid, brittle rock masses, is the release of stress due to brittle fracture, called a rock burst. In such circumstances, parts of the rock break off, most often on the sides of the excavation, less frequently at the top, usually suddenly, and a bang follows this. Cavities and places of significant damage in the rock mass can be the cause of rock bursts.

When water is present, which can penetrate the excavation in karst, there are negative consequences of its action: weakening of the material, washing of small particles from the surroundings of the excavation, propagation of cracks with changes in stress conditions, and an increase in load on the load-bearing structure.

Some of the mentioned karst problems also occurred during tunnel construction around the world. Santa Lucia (Italy, the 1960s) is a railway tunnel about 10.0 km long in which, due to poorly performed preliminary investigations, there was seepage of water of over 1000.0 l/s, which caused enormous problems during construction, pauses, and price increases, as well as lowering of the groundwater table in the local area with the drying up of numerous springs. A similar situation occurred during the excavation of the Gran Sasso tunnel (Italy, the 1970s) with a length of 10.0 km, in which the seepage of water and sludge was over 20.0 m^3/s. In the hydrotechnical tunnel, Furore (Italy, 1995) encountered a cavern [5]. During the construction of the Chaoyang Tunnel (China, 2018), there was a seepage of water and sludge amounting to over 57,000.00 m^3 within 40 minutes. Three lives were lost [6].

The facts mentioned above suggest a need to conduct thorough investigations before constructing tunnels in karst areas.

Challenges and achievements arising from the application of the TBM excavation method

Limestones are generally suitable for TBM excavation. This is seen from the data on world records where TBMs have been used (Table 1), with most records being achieved in carbonate rock types [3].

The use of TBMs requires knowledge of the geological/geomechanical surroundings, even in the initial stages of the project. The development and accessibility of computer technologies have led to significant progress in monitoring structures in real-time during construction, as well as in the automation of the work of the machines themselves, which are used for excavation. TBMs today have complex systems capable of collecting large quantities of operational data.

The latest achievements in the development of these machines include the detection of geological conditions in front of the excavation face using seismic and electric waves [7] or exploratory boring [3], as well as monitoring the operation of the cutting head and collecting other performance parameters [7]. Detection of conditions in front of the excavation face involves seismic (passive monitoring) and electric waves.

Table 1. *Selected world records in carbonate rock types [3]*

4.00–5.00 m diameter	**Day**	**Week**	**Month**
Record	128.0 m	477.0 m	1822.0 m
Project	SSC No. 4, Texas	SSC No. 4, Texas	Yellow River Tunnels 4 and 5
Country	USA	USA	China
10.01–11.00 m diameter	**Day**	**Week**	**Month**
Record	48.8 m	235.0 m	841.8 m
Project	TARP, Chicago	West Qinling	West Qinling
Country	USA	China	China

Passive monitoring is based on the monitoring of seismic waves generated during the operation of the cutting head (collection of signals generated during excavation) and applies to solid rock masses. Passive monitoring is carried out continuously during the operation of the TBM with the help of seismic wave receivers that are an integral part of this machine [7]. The signals received by the geophones include the wave signals from the source of their origin (the tunnel head) as well as the waves that are reflected from the surfaces located in front of the excavation face, whereby various techniques are used to remove the signal of the waves arriving from the source of origin and amplify the signal that comes from geological mediums located in front of the excavation face [8, 9].

Figure 2 shows the data collected during the construction of the Stavoli Ravorade tunnel (Italy), located in limestone. The figure shows the distances obtained based on the speed of transverse waves through different materials. The mentioned approach is suitable for identifying more significant changes in the structure and larger fault zones but not for identifying discontinuities smaller than 10.0 m [8].

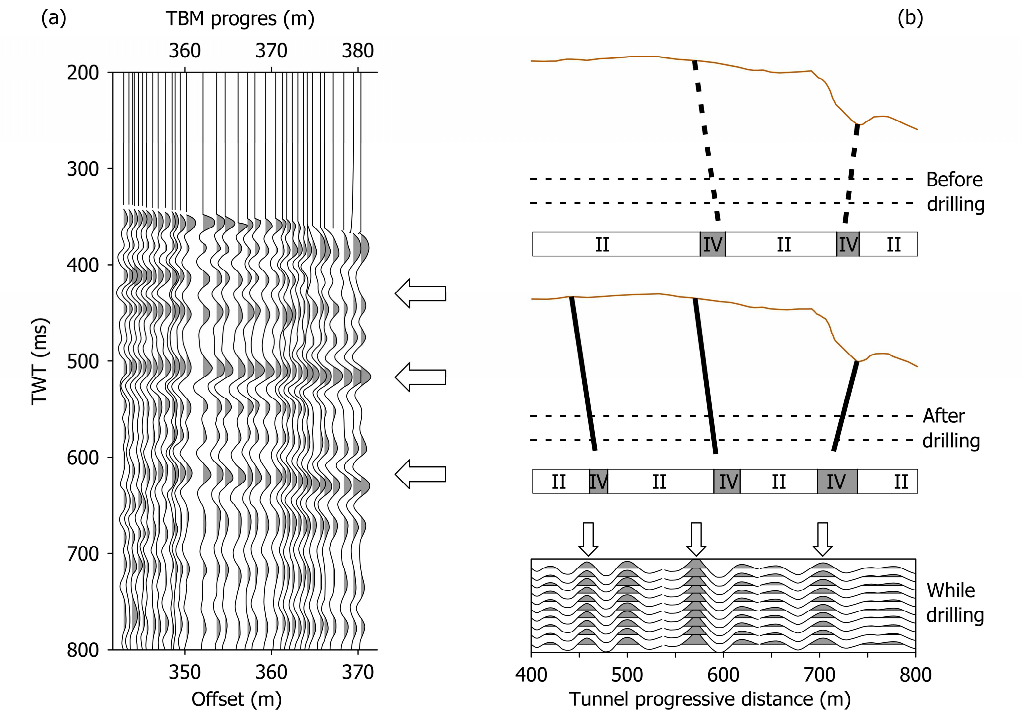

Figure 2. *Passive monitoring data collected during limestone excavation (a) time history of transverse waves in relation to the TBM progress (b) history of transverse waves in relation to fault positions according to RMR classification [7]*

The system for monitoring by measuring the electrical resistance of the soil is based on the technique of forming an electric field in the rock mass by installing electrodes and passing current, whereby the cutting head of the TBM can be used as a measuring electrode (Figure 3). Different aspects of rock mass resistance can be related to its porosity, permeability, groundwater flow, mineralogical composition, and macro features such as hollows and faults. Such a system can detect all relevant changes in the soil, including discontinuities up to 0.5 m [7]. This research also states that the system based on forming an electric field has been applied to 32 projects in Europe and Asia since 2000 and one project in North America.

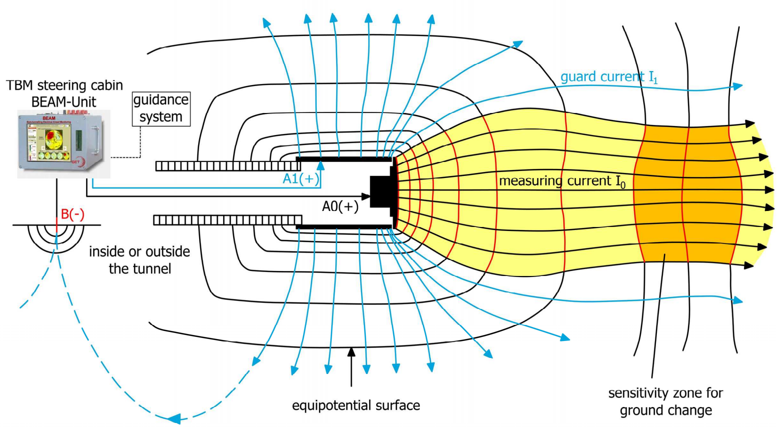

Figure 3. *Diagram of the electric field and current induced in front of the excavation face of the tunnel [7]*

Exploratory boring is less often utilized when excavating tunnels with the help of TBMs due to the risk of a negative impact on the dynamics of work execution. However, modern TBMs have boring exploratory systems where the data obtained is combined with the data collected after excavation. The result is the possibility of interpreting the collected data to define the geological model of the terrain through which the TBM passes. This system has generally proven to be very useful for understanding rock mass behavior, properly setting up the machine, and thus improving its overall performance [3].

Monitoring the operation of the cutting head involves collecting measurement data on instruments placed on the cutting tools during the operation of the TBM. These instruments are often used with rolling cutting tools for excavation in solid rock masses. They can collect data such as the force acting on the cutting tool [10] or the rotational speed of the tool [11]. In this manner, a change in geological conditions in the mediums through which the machine passes can be predicted.

Challenges and achievements arising from the application of the NATM excavation method

Sudden water seepage during tunnel construction can result in casualties and massive economic losses. As a consequence of this process, there may be a significant decrease in the groundwater table in the local area. Grouting is the primary method used to prevent the seepage of water and sludge [12-15] during tunnel excavation, and it also serves to preserve the groundwater table of the local area [16]. However, this method has its limitations – grouting is a process that displays poor compatibility at the level of different projects. Also, the long-term functionality of the produced grouting seal elements has not been thoroughly investigated [17]. As the grouting process has great spatial variability, it is essential to know and monitor the stress state of the rock mass in this process. An example of investigating the impact of grouting on the rock mass is an experiment in which the formation of a grout curtain was applied to a fault zone that was highly permeable with the help of several holes, with data collection using built-in monitoring instruments [18].

In general, cases of water seepage are divided into inflows from the fault zone that is rich in water, seepage through karst pipelines, seepage when in contact with zones saturated with groundwater, and seepage through microcracks [15]. Sudden seepage of water and sludge makes up about 17% of all incidents in karst [19].

Seepage from fault zones is often found in underground structures [18] and is variable over time. The main reasons for seepage through fault zones are the exposure of water-rich fault layers and the instability of rock plugs, where the fault zone is connected to other water-bearing units [20]. As shown in Figure 4, the control of water seepage from the fault zone is carried out through several methods: remediation through deep and shallow grouting, strengthening of the support and rock mass, and stability monitoring based on the performed remediation measures. Shallow grouting is accomplished using quick-hardening grouts [17].

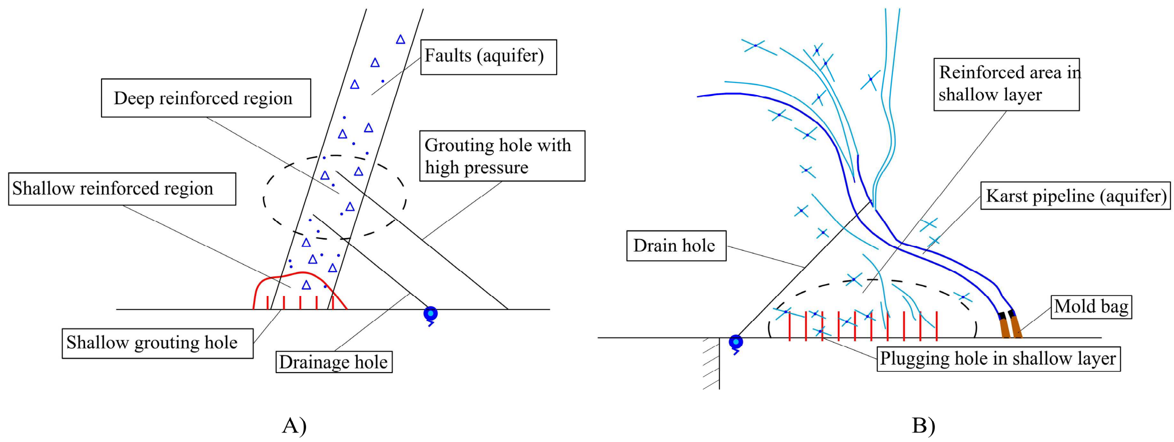

Figure 4. *Diagram of deep and shallow grouting during water seepage through a fault zone (A); diagram of water seepage control through karst pipelines (B) [15]*

Water inflows through karst pipelines can lead to serious safety problems and high risks during construction [21]. The main reasons for these inflows are the discovery of cracks rich in water and the instability of rock plugs or filling. As shown in Figure 4, to control the flow of water through the karst pipelines, it is first necessary to examine the extension of the channel and the condition of the pipeline walls and then close the channel with sealing bags while grouting with grout mass. In cases where the inflows are large, sealing is challenging, so it is suggested to separate the channel of the seepage as well as to drill control channels to examine the flow of the water seepage. For deep grouting, polymer-based grouts can swell in contact with water, while quick-hardening grouts are used for shallow grouting [17].

When encountering zones saturated with groundwater, seepage occurs by revealing zones of aquifers that are most often connected to surface water. Apart from conducting more detailed geological investigations in such zones, remediation is carried out using a combination of contact and deep grouting with monitoring of the results in real-time. As seen in Figure 5, the holes are drilled following the cracks' spatial orientation and the water's direction. Stopping water seepage is achieved by using radial wells in shallow zones and grouting the prominent cracks in deeper zones. The main cracks are selected by multi-factor optimization, where the ratio of water and grout is precisely controlled using advanced information technologies. Quick-hardening grouting compounds are used to control the seepage of water at elevated pressures and flows. Polymer-based and high-strength cement-based compounds can be used for grouting in surface areas [17].

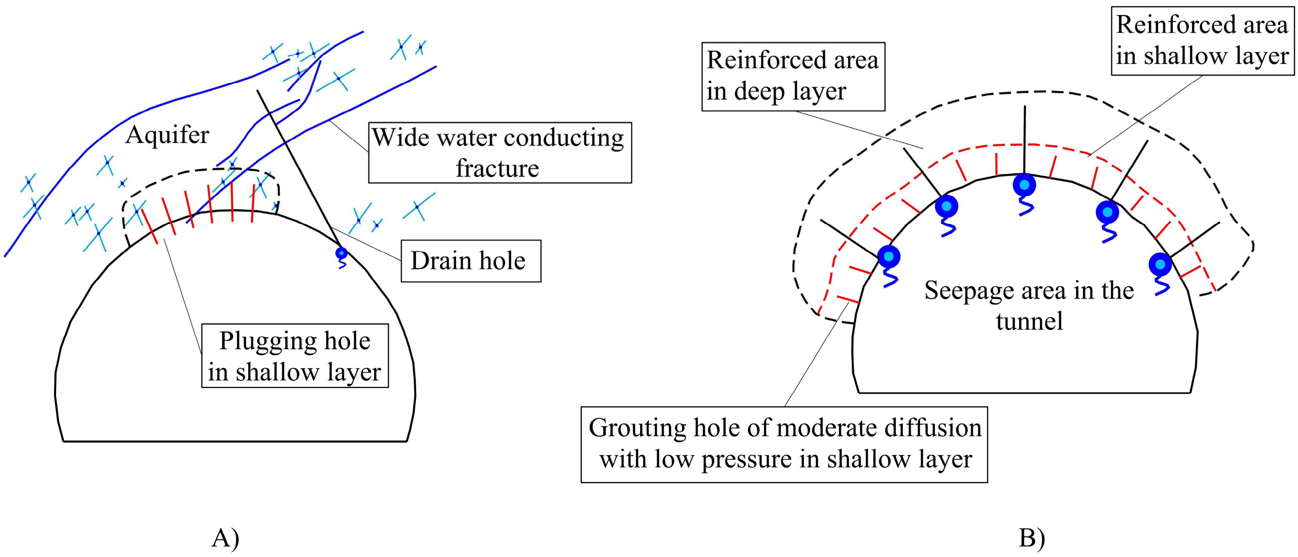

Figure 5. *A) Diagram of the control of water seepage in places of zones that are saturated with water; B) diagram of water seepage control at microcrack locations [15]*

Seepage at microcrack locations represents a significant challenge in geotechnics, especially in constructing subways, underwater tunnels, deep mines, and underground oil storage. Due to the complicated structure and small crack sizes, these seepage cases have a large flow over a long period. As shown in Figure 5, water seepage control at microcrack locations is achieved by grouting, where the distances between the grouting holes are reduced with the application of continuous grouting to accomplish a complete stop of the seepage. Lower grouting pressures are used in shallow zones to facilitate mass penetration, while higher pressures are applied in deeper rock mass zones. Additionally, it is necessary to monitor the seepage, grouting pressures, and deformations of the rock mass in real-time to adjust and optimize the grouting parameters promptly. Grouting compounds should be selected based on their possible application in a given environment, their durability, strength, and flexibility. Radial grouting parameters are usually determined based on the rock structure and local hydrogeological conditions and optimized by testing on-site [17].

One of the examples of the application of grouting in the control of water seepage is on the section of the railway line Divača - Koper in Slovenia. Grouting was carried out in two ways - pre-grouting (intended to limit the seepage of water into the tunnel during excavation) and post-grouting (intended to reduce the permeability of the rock mass after tunnel excavation). A diagram of the grouting is given in Figure 6 [22].

The application of grouting in the construction of tunnels can also be used to consolidate heavily fractured rocks and karst formations filled with sediments, as well as in the construction of bypasses [23].

Apart from sudden seepage of water during tunnel construction, cracks/caverns that can be dry and filled with water or material are also significant issues. Such problems can be solved by major remediation measures, bridging, or the construction of bypasses.

Major remediation measures were applied during the construction of the Yujingshan Tunnel in China, which encountered the largest cavern in the history of Chinese railway construction. The methods of filling the cavern, strengthening the cave arch, strengthening the tunnel structure, and diverting and closing the groundwater flow were all used during the remediation.

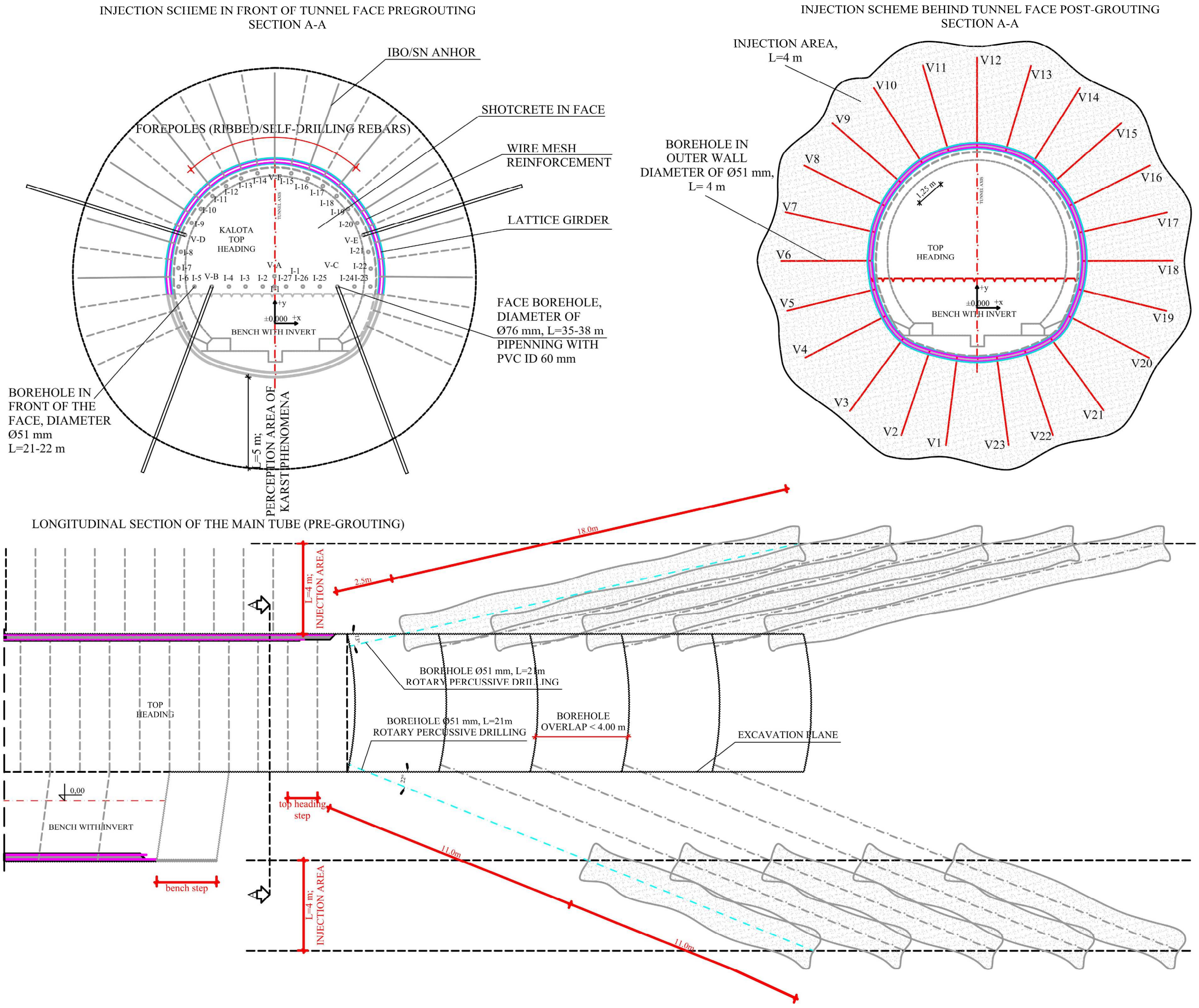

Figure 6. *Diagram of grouting works on the Divača– Koper tunnel [22]*

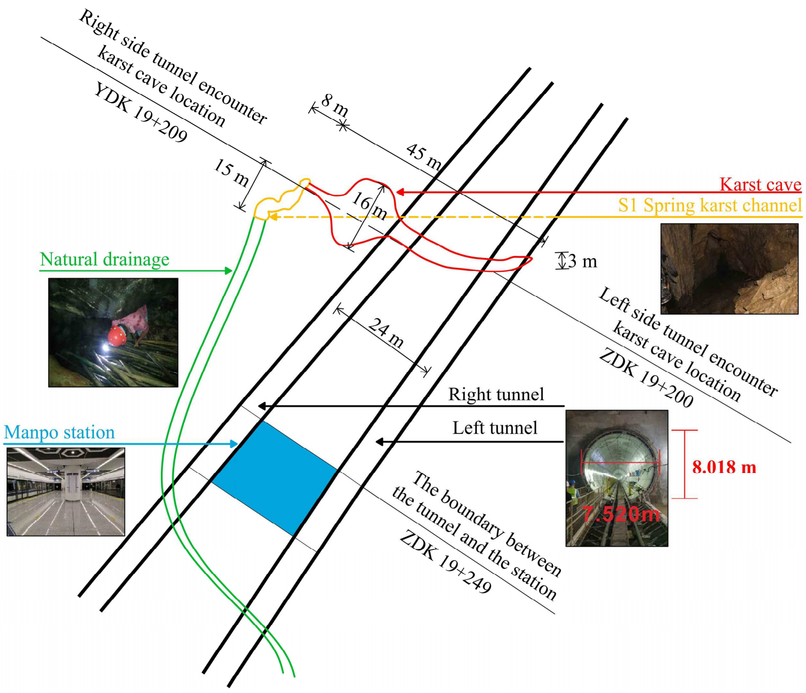

Figure 7. *Diagram of the passage through the cavernous zone during the construction of the Yujingshan tunnel [24]*

During the tunnel excavation, the surrounding rock was in relatively good condition, and there were no signs of leakage at the tunnel face. However, on August 8, 2014, a large cavern was discovered with a massive inflow of water and sludge in the left pipe of the railway tunnel. The maximum water flow at the excavation head was 42000 m^3/h. Remediation primarily involved considering several possible solutions [24].

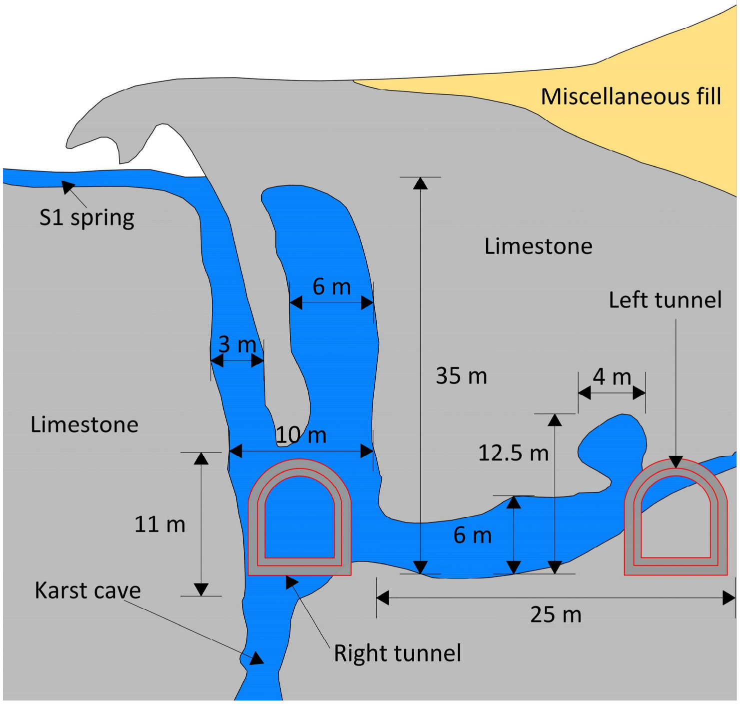

Figure 8. *Diagram of the passage through the cavernous zone during the construction of the Yujingshan tunnel [24]*

The first solution entailed installing piles in the primary rock mass and filling the cavity above the tunnel structure with gravel. The problem with such a solution is the potential possibility of leakage because no measures were foreseen for groundwater drainage. Another solution involved securing it with anchors and installing a drainage pipe. The problem with this solution was the proper sizing of the drainage pipe due to the impossibility of estimating the amount of water drained in this way. The third solution involved grouting in the full profile, which would be carried out from the excavation face. The disadvantage of this solution was the increased risk of collapse of the constructed structure, which was sensitive to increased groundwater seepage. The fourth solution involved creating a bypass tunnel with the help of which natural groundwater drainage would be carried out. The disadvantage of this solution was the increased construction time and the costs of building the bypass tunnel (Figure 9).

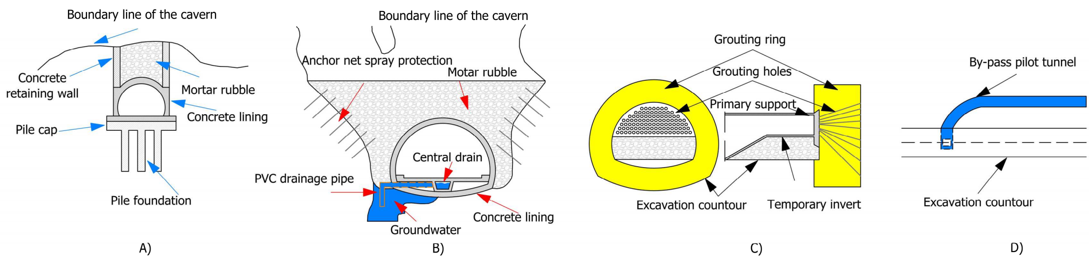

Figure 9. *Diagram of the considered solutions [24]*

The adopted solution involved the construction of an independent drainage system to discharge groundwater at the same surface source as before the incident (Figure 10). In this way, the environmental impact was reduced to a minimum. The procedure consisted of the following steps: the construction of a shaft and a connecting passage, the installation of the bottom beam that closes the karst channels and the filling of the surrounding karst cavities with concrete, and the production of a waterproof lining dimensioned so that it can take on the water pressure, the strengthening of the surrounding rock mass by grouting so that losses of water do not occur due to increased pressures [24].

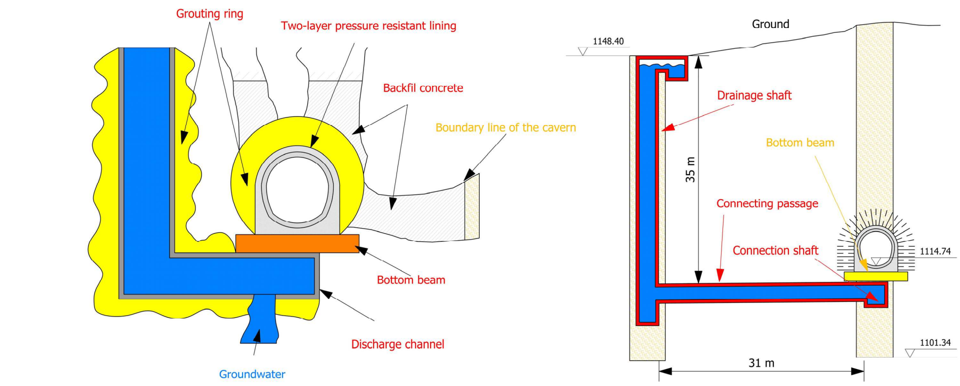

Figure 10. *Diagram of the adopted solution [24]*

Another way of passing through caverns in karst conditions is shown in the example of the Huaguoshan Tunnel in China [25]. In this tunnel, remediation was carried out by backfilling, constructing a protective arch, constructing a bridge structure over the cavern, and forming a secondary lining. The diagram is given in Figure 11.

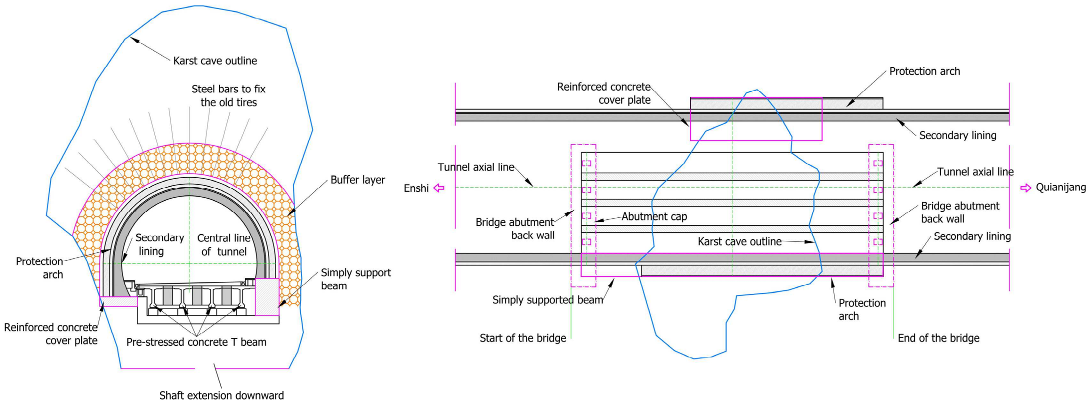

Figure 11. *Diagram of the passage through the cavernous zone during the construction of the Huaguoshan tunnel [25]*

Our region also has examples of tunnel construction in karst conditions. In the text that follows, several examples are presented - supply tunnel RPP "Čapljina" (Bosnia and Herzegovina), Fatničko Polje - Bileća reservoir tunnel (Bosnia and Herzegovina), and Sveti Ilija - Biokovo tunnel (Croatia).

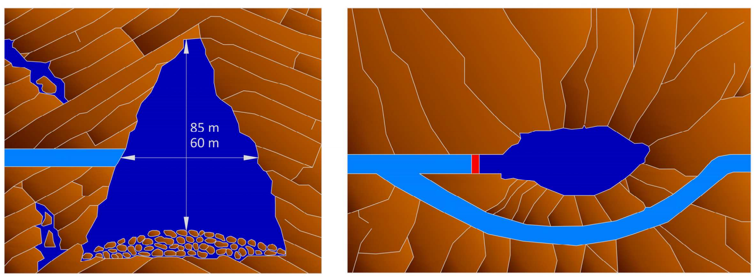

Figure 12. *Bypass around the cavern during the construction of the supply tunnel of RPP "Čapljina" [26]*

During the construction of RPP "Čapljina", the work on the construction of the tunnel to the hydropower plant was accompanied by constant battles with groundwater. The divers closed the karst channels through which the water penetrated, and when that was not enough, the rock through which the excavation was carried out was previously grouted. During the excavation of the supply tunnel at 2637 m from the entrance, the builders were stopped by a large cavern (Figure 12). The investigation showed that the cavern length was 60.0 m, the height was 85.0 m, and the volume was 150,000.00 m^3. The possibility of bridging this cavern with a pipe structure was rejected due to the danger of the stone blocks caving in, so it was decided to bypass the cavern on the right side with a 180.0 m long tunnel. With this solution, the total length of the supply tunnel was increased by 15.0 m compared to the designed one.

The bypass was shaped for the smallest flow resistance. Due to difficult ventilation, the excavation of the supply tunnel after the bypass was constructed was accomplished with a reduced diameter to break through the tunnel as soon as possible and thus establish natural ventilation [26, 27].

The Fatničko Polje - Bileća Reservoir tunnel serves the purpose of controlled transfer of water from Dabarsko Polje and Fatničko Polje into the Bileća reservoir to eliminate flooding in the fields and obtain greater energy effects at the power plants built downstream. The length of the tunnel is 15,650 m, with an excavation diameter of 7.10 m. The tunnel is divided into three sections from the technical side of construction. The first (up to the chain. km 3+535) was executed using NATM (classic excavation using pneumatic equipment and a milling machine), the second (up to the chain. km 12+625) with the use of a TBM, and the third (up to the chain. km 15+650) using NATM (classic excavation - application of rail equipment).

The disadvantages of using pneumatic equipment were: cutting, additional cutting, and increased looseness of the rock mass. By applying this excavation method, using equipment and machinery with internal combustion engines and additional gases released by the explosive, the conditions for carrying out the work were challenging, despite the installed ventilation system.

Excavation with a TBM presented problems in encountering fault zones and caverns (when the machine was not used for more than 5 months). Entering the fault zones interrupted the work with the TBM. The remediation of faults took place in phases, with the large degree of participation of qualified workers and the use of adaptive equipment.

Mechanical excavation with a tunnel boring machine, the "milling machine", had more advantages compared to classic conventional methods, which were reflected in better stability of the tunnel excavation, parallel removal of excavated material with the excavation, minimal cuts, greater progress, and reduction of construction costs. The application of this method did not require major preparatory work. The machine also had good mobility during flood evacuation [28].

The Sveti Ilija - Biokovo tunnel is located on the road that connects the Zagreb - Dubrovnik highway with the Adriatic Highway. Three vertical caverns with a width of 12-20 m and a height of up to 260 m appeared during the tunnel construction. The caverns were bridged by a semi-prefabricated composite steel-concrete structure [29].

REVIEW OF THE SUPPLY TUNNEL DESIGN OF HPP “DABAR”

Water is one of the most important natural resources in Eastern Herzegovina, mainly belonging to the Trebišnjica River basin. The term "Upper Horizons of the Trebišnjica River" (Figure 13) refers to the broader area of the Trebišnjica River basin, which is located above the level of the existing reservoir Bileća (the reservoir's elevation is 400 masl). The term also includes karst fields – Nevesinjsko Polje, Gatačko Polje, Dabarsko Polje, Fatničko Polje, and Bilećko Polje.

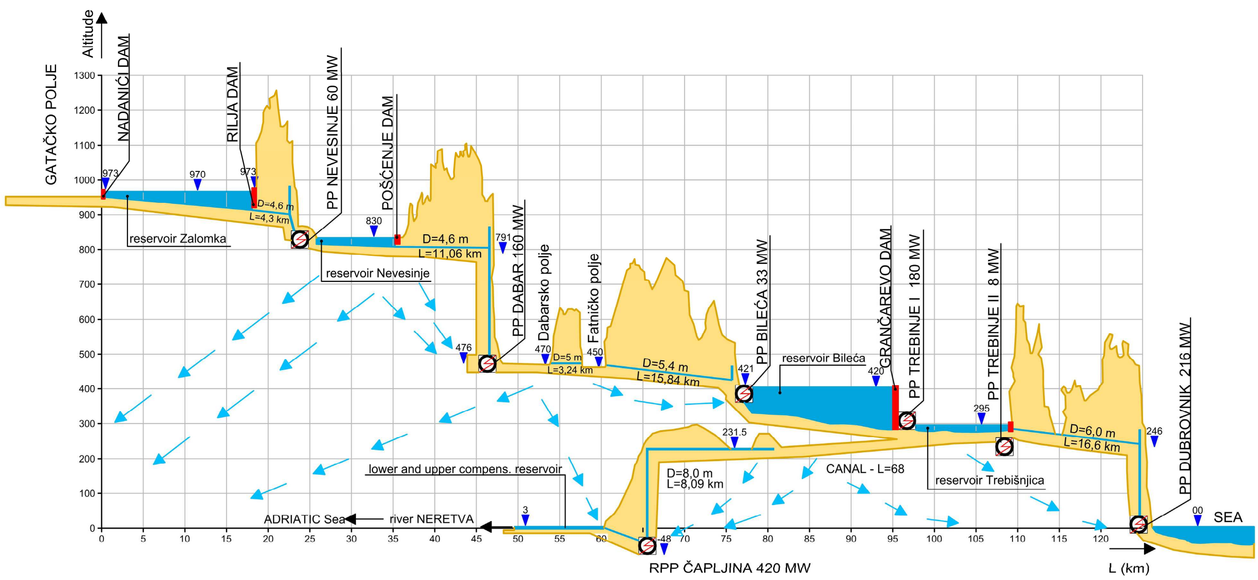

Figure 13. *Longitudinal profile of the Trebišnjica hydropower system [30]*

The area of "Upper Horizons" is characterized by significant water resources, which have abundant precipitation in the flood period as an essential feature (October - May) with long-lasting floods in the karst fields and a significant deficit of surface water in the summer period when the watercourses completely dry up. Due to the pronounced karstification process, the groundwater table is very low in the summer and unsuitable for exploitation.

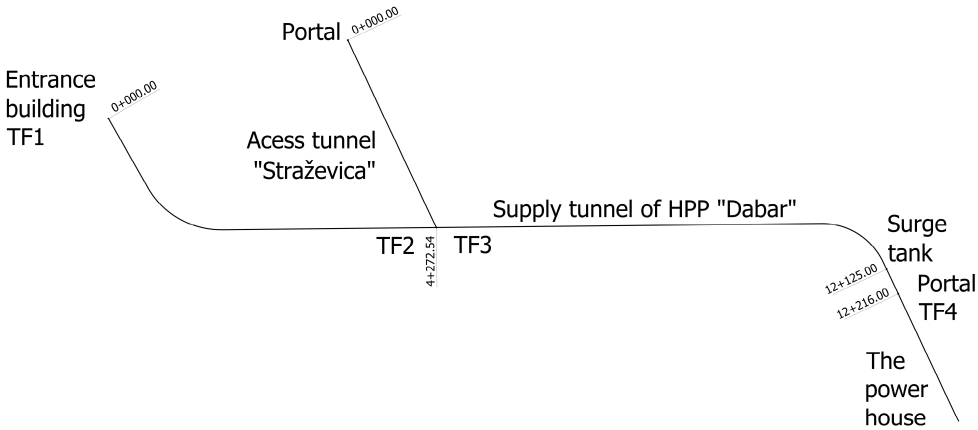

Figure 14. *Location plan of the supply tunnel of HPP "Dabar" [30]*

The "Upper Horizons" subsystem implies the construction of many structures on the mentioned karst fields with the primary goal of transferring part of the available water onto the existing Bileća reservoir. This subsystem involves the construction of 3 hydropower plants: HPP "Nevesinje" (installed power of about 60.0 MW), HPP "Dabar" (installed power of 160.0 MW), and HPP "Bileća" (installed power of 32.0 MW). These additional water quantities have a significant impact on the power plants of the constructed part of the system (Pribran HPPs "Trebinje I" and "Trebinje II", derivation-accumulation HPP "Dubrovnik" and RHS "Čapljina"), which achieves excellent energy effects [30].

The supply tunnel of HPP "Dabar" (Figure 14) is 12125.0 m long, has a daylight opening of 4.60 m, and has a secondary lining that is 30.0 cm thick. Following the Main Design [30], it is planned to be built using the New Austrian Tunneling Method (NATM). The tunnel has 4 access points (TF): the first is located at the entrance to the tunnel where the entrance building is located (TF1). The fourth is at the tunnel exit at the future surge tank (TF4) location. The remaining two access points are at the junction of the access tunnel "Straževica", which connects with the supply tunnel at the chainage of 4+272.54 of the supply tunnel (TF2 - from the junction of the access tunnel to the entrance building and TF3 - from the junction of the access tunnel to the surge tank). Work on the HPP "Dabar" tunnel began in 2016, and approximately 500 m of the tunnel remains to be excavated. The planned completion of all the work is at the end of 2024.

The area in question is part of the area of Old Herzegovina, which is located in the southeastern zone of the Republic of Srpska. The terrain is part of a high mountain area with average altitudes between 800.0 and 1100.0 m and prominent mountain massifs, between which there are plains and karst fields.

In general, the supply tunnel is in a complex geological medium made up of two types of sedimentary rocks: conglomerates with interlayers of sandstone and marl up to the chain. Km 2+650, and the remaining part of the tunnel is made up of Paleocene, Lower Eocene, and Upper Cretaceous limestones. Therefore, 23.0% of the length of the tunnel is made up of conglomerates with interlayers and 77.0% of limestone [30].

SELECTION OF EXECUTION TECHNOLOGY

General considerations

According to the design documentation, it is planned for the tunnel of HPP "Dabar" to be built using NATM, while the Contractor is given the option to choose the TBM. The Contractor chose NATM and excavation based on the application of blasting techniques for the construction of the HPP "Dabar" tunnel. Excavation is carried out in full profile by blasting at four access points through three points: access from the direction of the entrance building, access tunnel "Straževica", and access tunnel "Vodostan". Blasting is carried out cyclically, marking the boreholes, drilling, filling with explosives, and activation. After blasting the rock mass, the material is loaded into dump trucks and taken outside the tunnel to a landfill.

Additionally, analyses of the optimal technology selection were carried out on the section from km 4+272 to km 12+125. In the research conducted by Mirković [31], a comparative analysis of the different solutions for the construction of this section of the tunnel was performed, in which 4 construction solutions were analyzed, namely: the use of milling machines, the use of a TBM with grippers, the use of a TBM with a double shield, a combination of the use of a TBM with grippers and a milling machine.

Under the design documentation [30], it was kept in mind that the entire supply tunnel of HPP "Dabar" should be constructed in slightly less than 4 years (47.0 months). In the first year of works, all preparatory works and works on the "Straževica" access tunnel would be carried out, and work would begin on the construction of the supply tunnel with all 4 points of access. In the second year, excavation work would continue on the supply tunnel from the access point "Vodostan" and the entrance building, and excavation work would begin on the access point of the "Straževica" tunnel. In the third year, work on the excavation of the supply tunnel would be completed, and work on concreting the secondary lining of the tunnel would begin. In parallel with the tunnel concreting works, grouting works would begin. Concrete and grouting works would be completed in the fourth year. Also, the works on the entrance building would be carried out during the fourth year [30].

According to Solution 1, in the considered section, the supply tunnel is built using NATM (New Austrian Tunneling Method) with the help of two milling machines. The tunnel structure consists of the primary and secondary lining. The primary support structure is designed to take on hill pressures and effects caused by groundwater. The secondary lining is 30.0 cm thick and is designed to take on internal water pressures. The cross-section of the tunnel excavation is circular.

Different types of primary support were defined based on the stress-strain analyses carried out per the methodology presented in [32]. During the stress-strain investigations and the selection of support types, the characteristics of the rock mass (composition, cracking, degree of faulting, and degradation), the initial stress state (height of the upper layer), and the like were taken into account. After the completion of the excavation and the securing of the complete tunnel with the primary support, a secondary reinforced concrete lining with a thickness of 30.0 cm is added, in combination with stress grouting [33], which results in the final shape of the tunnel with a diameter of the daylight opening of Ø 4.60 m.

Solution 2 involves the excavation of the supply tunnel with the help of a TBM with grippers. This machine is suitable for applications in rock mass where the support of the excavated cross-section in the area of the excavation head and protection of the machine itself is not required or can be achieved with less effort and elements such as anchors, steel elements, and shotcrete, and they can also be applied locally in the tunnel calotte. It is used in stable rock masses with low groundwater penetration.

To solve the problem of encountering caverns, it is planned to build bypass tunnels of smaller lengths around the excavation head of the TBM. This would allow direct access to the problematic location for it to be remediated manually or by machine.

The envisioned diameter of the TBM is 5.20 m, and it was obtained based on the analysis of the stability of the excavation when encountering weaker rock masses. The secondary lining of the tunnel for Solution 2 is the same as the secondary lining of Solution 1. Still, due to the limited excavation diameter of the TBM of 5.20 m, which was obtained from the mentioned condition, it also contains elements of the primary support. This lining has the role of support during construction but also a role in taking on the load from hydrostatic pressure in the exploitation phase.

Solution 3 involves the excavation of the supply tunnel with the help of a TBM with a double shield, which is among the most technically developed machines for tunnel excavation. In stable geological conditions, this machine allows the installation of concrete segments in parallel with tunnel excavation while achieving very high performance. This powerful technology is, therefore, perfect for excavating long tunnels in solid rocks.

In ideal rocks, a TBM with a double shield can operate without adding lining segments [34].

Solution 4 of the supply tunnel section of HPP "Dabar" excavation is a combination of Solutions 1 and 2. According to this Solution, the excavation is carried out with the help of a milling machine at the access point TF3, and at the access point TF4, the excavation is carried out with the help of one TBM with grippers. The primary support of this Solution is, based on the facts mentioned above, the same as the primary support of Solution 1 on the part where the excavation is carried out with the help of a milling machine or Solution 2 on the part where the excavation is carried out with the help of a TBM with grippers.

The secondary support of this Solution is, based on the facts mentioned above, the same as the secondary support of Solution 1 on the part where the excavation is carried out with the help of a milling machine or Solution 2 on the part where the excavation is carried out with the help of a TBM with grippers.

Techno-economic analysis of the presented solutions

Analyses of the prices of individual work positions are given based on the current prices of machinery and equipment, spare parts, construction materials, and labor. Computations were made following construction and mechanical standards and based on data obtained from manufacturers of machines and equipment.

For each of the Solutions, the initial or "zero" values of the construction costs were computed for the speeds of progress, which were obtained from the conditions of work completed on the supply tunnel of a known length in a period of 47.0 months (according to the project), namely: the milling machine (Solutions 1 and 4) – 7.0 m/day, TBM with grippers (Solution 2) – 12.91 m/day, TBM with a double shield (Solution 3) – 7.81 m/day, TBM with grippers (Solution 4 for the most probable chainage of the point of contact between the TBM and the milling machine) – 8.30 m/day. It was taken into account that the TBM with a double shield in Solution 3 shows a significant reduction in the time required for excavation because the segment lining and grouting are carried out in parallel with the excavation, while for the remaining Solutions 1, 2, and 4, it was necessary to foresee additional time for the mentioned procedures. For Solutions 1, 2, and 4, it was adopted that the execution of grouting works and installation of reinforced concrete lining is 8.0 m/day and 12.0 m/day, respectively.

Possible savings on the tunnel construction were further considered in terms of achieving higher speeds of advancement of the TBMs, as well as based on the earlier commissioning of the supply tunnel.

The initial or "zero" price values for each Solution, obtained based on preliminary measurements and estimates, are equal to:

- Solution 1 (two milling machines): EUR 34 227 726.85
- Solution 2 (TBM with grippers): EUR 38 344 720.44
- Solution 3 (TBM with a double shield): EUR 39 301 284.11
- Solution 4 (TBM with grippers and one milling machine): EUR 40 547 656.34

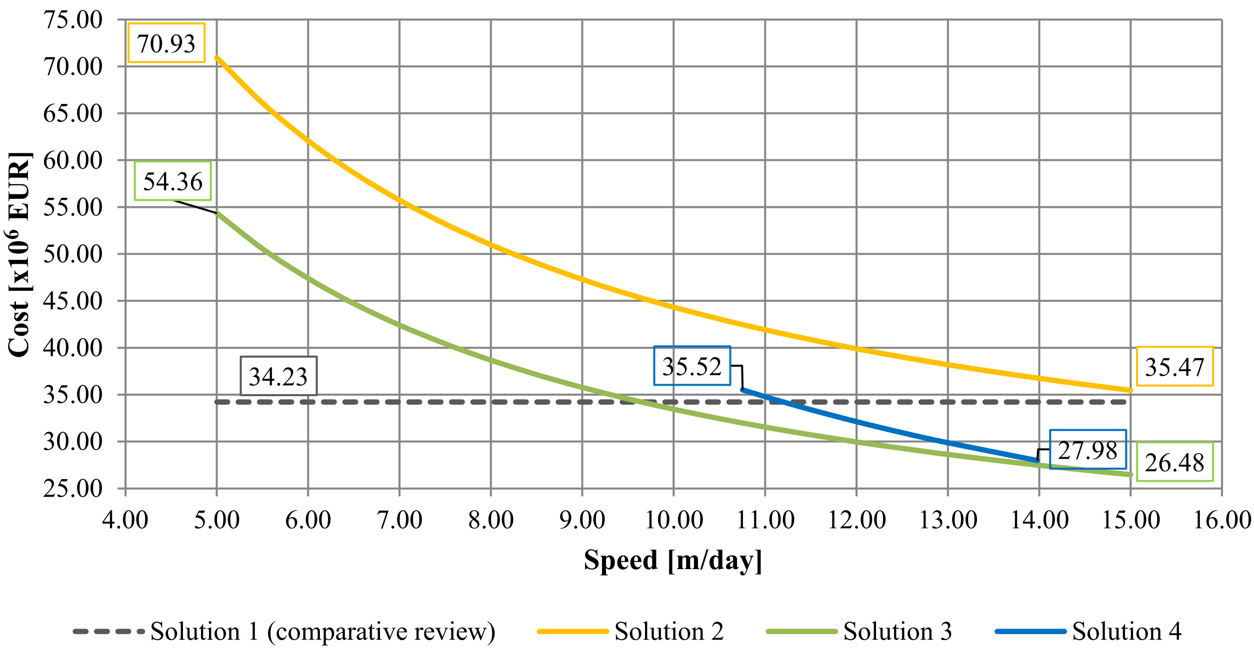

Figure 15. *Dependence of the price of tunnel construction on the speed of progress [31]*

Savings at higher speeds of advancement of TBMs in all Solutions include the cost of workforce, mechanization, and energy necessary to operate them. Total monthly mechanization costs are based on machines working 30 days per month in three 8.0-hour shifts, with breaks in each shift of 1.0 hours for fuelling and rest. For the last

Solution, it is essential to note that the savings computation was carried out for the most likely chainage of the meeting point of the TBM with the milling machine (km 6+455.58) and different speeds of advancement of the TBM with grabbers. According to the schedule, this is possible because the milling machine takes a break in its work due to the seepage of water into the tunnel, which is why its work is impossible during that period. The obtained results are shown in Figure 15.

Based on the values above, at first glance, it can be concluded that the Solution provided by the Main Design (Solution 1) of the supply tunnel is the most economical solution that requires the least investment for the construction period of 47.0 months.

However, when analyzing the prices depending on the achievement of higher speeds of excavation progress with the help of TBMs and the shortening of the construction period of the structure in question, the following conclusions can be drawn:

- Solution 2 will show a reduction in costs compared to Solution 1 for TBM advancement speeds greater than approximately 13.50 m/day,
- Solution 3 will show a reduction in costs compared to Solution 1 for TBM advancement speeds greater than approximately 8.0 m/day,
- Solution 4 will also show a cost reduction compared to Solution 1 for TBM advancement speeds greater than approximately 10.50 m/day.

In accordance with this, Solution 3 (excavation with the help of a TBM with a double shield) is the most favorable solution for constructing the considered section of the tunnel. This Solution has a slightly increased initial investment compared to the other solutions. Still, due to the reduced construction time caused by the installation of the segment lining and grouting in parallel with the excavation, it offers the best opportunity to reduce costs based on savings (workforce, mechanization, energy for work) concerning Solution 1. In reality, the speeds of advancement that would lead to reductions are very achievable. However, this Solution may have additional problems regarding the possibility of setting up the necessary facilities (here, we are primarily referring to the concrete segment factory) on the field itself, which would lead to an increase in costs. Such problems are, in reality, due to inaccessible terrain, very possible.

The economic justification of using a TBM with a double shield is particularly pronounced when considering the potential electricity production at hydropower plants in the lower part of the system (energy produced by the earlier commissioning of this supply tunnel).

Solution 4 is a moderately good solution with realistically achievable advancement speeds. This type of excavation, with two points of access, with the use of a TBM machine and a milling machine, also requires somewhat more significant initial investments but very quickly leads, similar to the previous Solution, to a reduction in construction costs compared to Solution 1.

Solution 2 implies the lowest initial investments compared to the previous two Solutions but also gives a reduced (but not unrealistic) probability of cost reduction compared to Solution 1 because it requires the highest speed of progress.

The most significant disadvantage of TBMs is in terms of the geological composition of the terrain. Upon encountering fault zones and caverns, the TBM comes to a complete standstill and continues to operate after a unique solution is chosen regarding technology and organization of work, their remediation, and the creation of conditions for continued excavation.

OBSERVATION, RESEARCH AND MONITORING DURING CONSTRUCTION

The concept of observation, research and monitoring

During the construction of the HPP "Dabar" tunnel, the envisaged concept of observation, research, and monitoring includes a constant collection of relevant information about the rock mass (EG mapping, geophysical measurements, laboratory tests, in situ tests, etc.), continuous monitoring of deformations on the measurement profiles, and also the collection of all other relevant data in the construction process. Based on the obtained results, appropriate stress-strain analyses are performed, and the proposed types of support are verified. As necessary, corrections and adaptations of technical solutions are made in line with actual conditions in the field. Verification of proposed solutions during construction includes: control of excavation methods and phases, control of the period when the support is added and the capacity of the support, as well as considering the need to modify the solution. The verification of the proposed solutions is analyzed using mathematical FEM models that must be calibrated. Model calibration entails determining the model parameters so that the model can simulate the actual situation in the field in the best way. In-situ cross-sectional tests with straining beams, point load tests, mapping, and measurement results on measuring profiles and profiles where convergence is measured are taken into account integrally. Based on the calibrated model, the behavior of the rock mass in the excavation zone and the interaction of the structure-rock mass system are predicted.

Engineering-geological mapping

During the tunnel excavation, data on the rock mass is collected by constant monitoring, i.e., by carrying out engineering and geological mapping of the face of the excavation of individual rings of the tunnel's progress, including the face and walls of the excavation itself.

During the engineering-geological mapping of the face and walls of the excavation of the tunnel sections, the petrological determination of the rock mass is performed, as well as the recording of lithological and mechanical discontinuities with the mapping of phenomena with the accompanying written description of the cracks and their filling. The rock monolith's strength and the cracks' walls are determined. Also, the category of rock in terms of quality is determined based on strength, fragmentation of the rock, disintegration, and frequency of cracks, while defining parameters on the state and properties of the cracks (length, gap, roughness, filling, disintegration), the influence of the discontinuity orientation on the stability of the excavation, as well as the influence of groundwater. Based on data collected by mapping, along with data on the uniaxial strength of the rock and laboratory tests and point load tests on rock samples, the preliminary value of RMR is determined.

The results of the engineering-geological mapping of the individual face of the excavation of the tunnel section are presented in separate reports on the EG mapping of the face, which is made immediately after the mapping. This report, in addition to the general part with data on the chainage of the face, contains a graphic representation in the form of a vertical profile of the face of the excavation, a developed profile for displaying the results of mapping the phenomena on the face and walls of the excavation ring, and a legend of the mapped phenomena. The report also contains a table of parameter values according to RMR classification and total RMR value, as well as a specific category of rock at the face, according to the GN206 categorization for underground excavations.

Based on EG mapping, established data defined RMR and category according to GN-206 classification; a proposal is provided in addition to verifying the type of primary support.

The results of EG mapping are displayed in their final form on the developed engineering-geological excavation profiles. All observed discontinuities, lithological-structural type of rock mass, occurrences of groundwater, type of filling, and other parameters are applied to the profiles. At the HPP "Dabar" tunnel, more than 3100 faces have been mapped so far.

Installation of instruments and monitoring

With the aim of monitoring deformations of the tunnel caused by the progress of the excavation face, the installation of appropriate measuring equipment is planned in the tunnel: benchmarks for measuring convergence, extensometers for measuring deformations in the rock mass, extensometers in concrete for measuring deformations in the concrete lining, cells for measuring pressure at the point of contact of secondary and primary linings, and anchors for strain measurement. In the tunnel, it is planned to install 126 measuring profiles with benchmarks for measuring convergence and 24 measuring profiles for installing appropriate measuring instruments while constructing the primary and secondary lining. It is planned to install a total of 53 rock extensometers, 69 cells for pressure measurement, 194 electroacoustic extensometers for measuring dilations in the concrete of the secondary lining, 172 measuring anchors, and 3 devices for measuring the volume change in the tunnel. Figure 16 shows the layout of wall extensometers and stress cells in one measuring profile.

Benchmarks for measuring convergence are installed incrementally as the excavation progresses according to the proposed layout along the tunnel. Distances between pairs of benchmarks are measured, and by subtracting them from the results of the previous series of measurements, the relative values of the distance change are obtained, which are used to monitor the behavior of the tunnel over time. The remaining measuring profiles in the tunnel are installed in the intended locations. Their positions can be corrected depending on the geological conditions in the field and other investigative works. All measurements on measuring profiles are carried out at appropriate time intervals from when the instruments are installed. Figure 17 shows the results of measurements on one rock extensometer with three measuring bases in the measuring profile M11 and on one measuring anchor in the measuring profile M8.

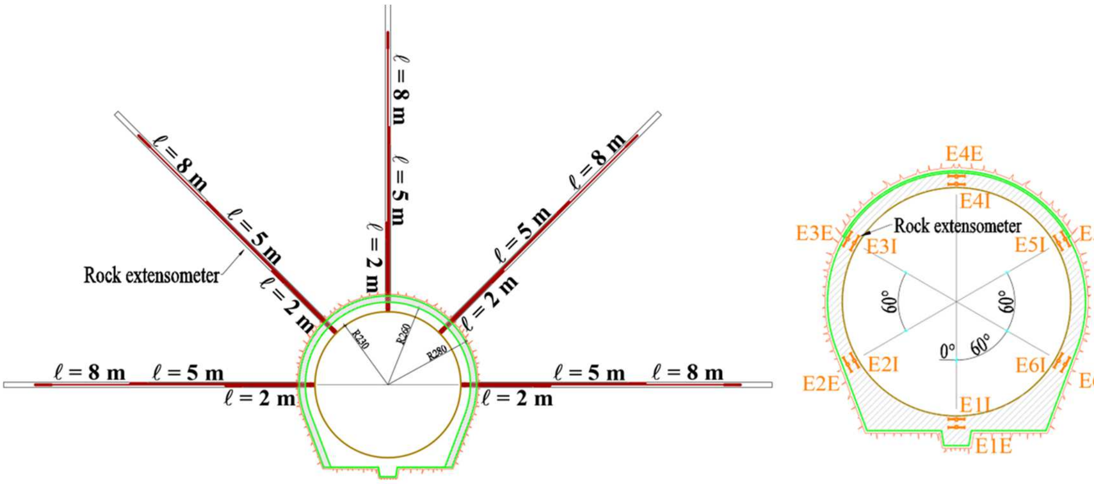

Figure 16. *Layout of rock extensometers in one measurement profile (left), layout of extensometers in concrete in one measurement profile (right)*

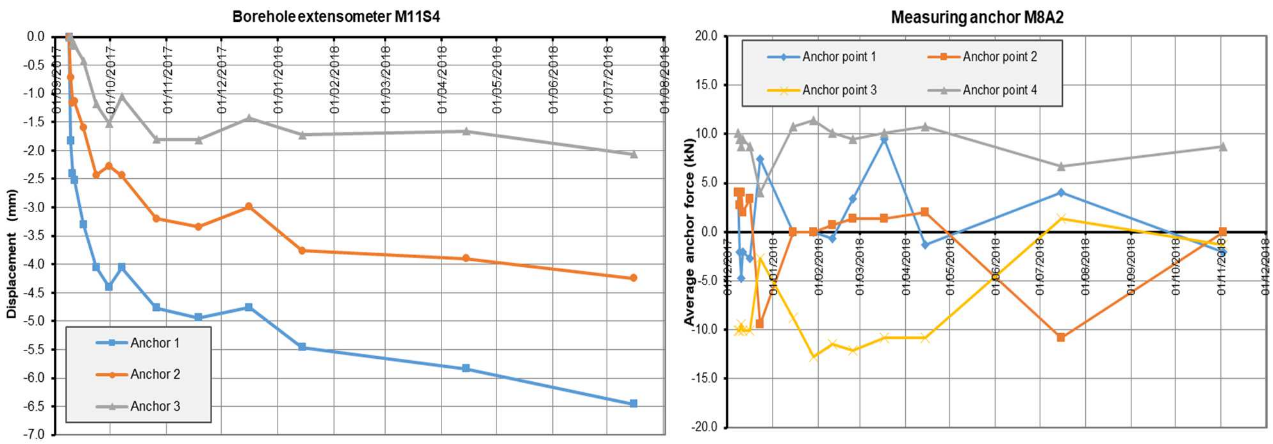

Figure 17. *Displacement of the base of the rock extensometer installed on the M11 profile (left), force in the anchor in the M8 measuring profile (right)*

In situ and laboratory testing

These tests include tests using the point load method (on each face of the excavation, a total of 24,000 samples are expected to be tested), deformability testing of the rock mass using a straining beam (16 straining beams at eight locations - 2 vertical and horizontal tests at each site), uniaxial compressive strength tests on rock mass samples (a total of 80 samples are planned to be tested and samples will be taken from the tunnel walls), and anchor testing (trial and verification, trial testing are planned on a total of 30 anchors, while verification testing is planned on 300 anchors in total).

The Point load test is an index test for the classification of rock strength. It can also be used to evaluate other strength parameters with which it is correlated, for example, in assessing a rock monolith's uniaxial compressive and tensile strength. Rock samples for testing using the Point load method can be processed as cylinders (diameter and axial test) in the form of a prism (block test). Finally, rock samples of an irregular shape can be used, which were used in the investigated tunnel. This method's testing equipment is mobile and easily applicable for work in the field. It consists of a system for applying loads (hydraulic pump, press, frame, and loading cone), a system for measuring the breaking strength of the sample (manometer), and a system for measuring the distance D between the loaded points on the sample. So far, about 22,000 samples have been tested in the tunnel.

In addition to the Point load test, direct laboratory tests of the uniaxial strength of the rock mass are also performed on the samples taken out in the tunnel. In contrast to the point load tests, these tests provide direct values of the uniaxial compressive strength of the rock mass. However, these tests are significantly more expensive and complicated, unlike the point load test, which is very practical for field conditions. Still, they are suitable for verifying the results obtained by field point load tests.

The deformability of the rock mass is determined with the field test using a plate loading test. The test consists of loading and unloading the rock mass and measuring its strain. The load is applied with a hydraulic press and transferred to the contact surface via rigid bracing (thick-walled steel pipes) and rigid transmitter (steel plates). Basic data on the deformation characteristics of the rock mass are obtained from the measured strains using three flexometers, which measure the displacements of the rigid plate in the direction of the force. The test results define technical solutions for the primary and secondary lining and when stress grouting the tunnels. Figure 18 shows the results of the tests (pressure-displacement diagram) of the plate load test at one test location.

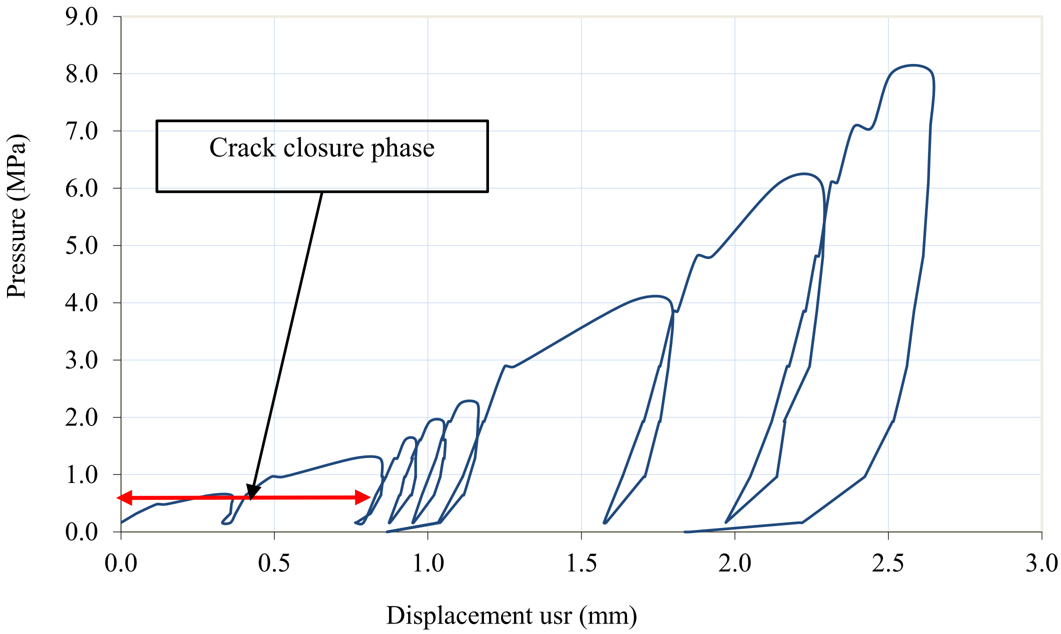

Figure 18. *Pressure-deformation diagram of one of the plate load tests*

Combining computational models and results of measurements and tests

The essential characteristic of tunnel construction using NATM is the flexibility that enables constant monitoring of the tunnel excavation during construction and the adaptation of technical solutions to the actual conditions in the field. This approach allows us to obtain a wide range of data on the behavior of the rock mass itself, based on the results of the mapping, measurements, and other investigative works during the excavation of the tunnel, which cannot be taken into account in the design phase. In connection with this, the possibility of subsequent verification and optimization of support systems during the tunnel construction is left open when a larger pool of on-site data on the rock mass is obtained.

The choice of support during tunneling is based on the following: the chainage where the excavation face is located is known (and therefore, the corresponding height of the upper layer also is known), the face of the excavation was mapped in the field and the engineering-geological classification was performed (values of RMR points as parameters of the rock mass classification according to Bieniawski), an interpretation of the geotechnical conditions (values of the parameters of deformability and strength of the rock mass, i.e., fracture conditions) was performed, an appropriate stress-strain analysis was performed for the conditions in the field (height of the upper layer and values of the parameters of deformability and strength of the rock mass), appropriate measurements were performed on main measurement profiles and convergence measurement profiles that allow the measurement results and the results obtained by the computational model to be connected, thus allowing us to verify the parameters of the material model.

The verification of technical solutions is carried out using numerical FEM models that take into account the structure and mechanical properties of the rock mass, construction technology, and all other relevant information necessary to perform the stress-strain analysis of the tunnel structure. For FEM models to be used for stress-strain analyses of tunnel structures, they must be calibrated based on all the information and results of tests and measurements in the tunnel. The data, in this case, are the results of EG mapping, laboratory and in situ tests, and observations on measurement profiles.

During investigations [35] on the example of the HPP "Dabar" tunnel, a stress-strain analysis was conducted using a three-dimensional FEM model, considering the results of measurements and tests. A 3D model was created in the PAK software package [36] (Figure 19 left) following research recommendations [37]. The Hoek-Brown constitutive model was adopted to model the mechanical behavior of the rock mass. The FEM model was estimated so that the results obtained from the computations in the FEM model best match the results measured on the measuring profiles (Figure 19 right), taking into account the results of other research (mapping, Point Load, bracing). Based on the thus obtained estimated FEM model, safety analyses of the primary support were performed.

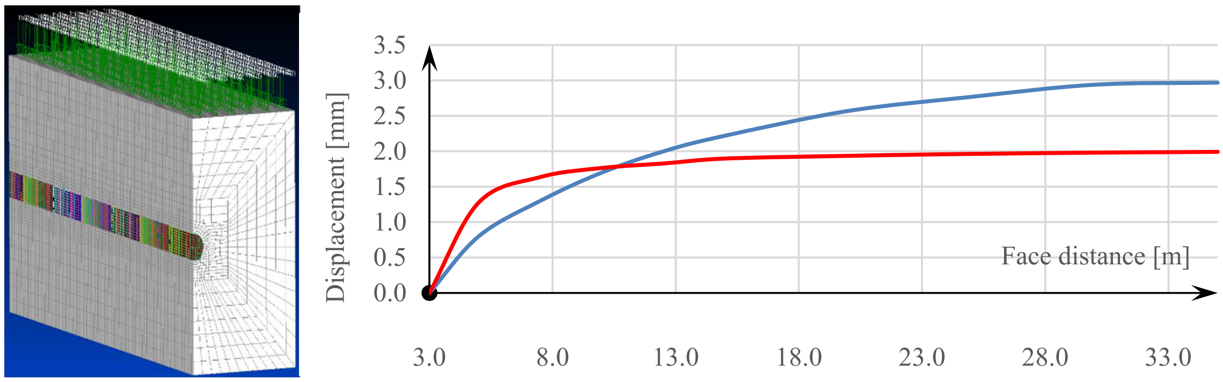

Figure 19. *3D model of the analyzed section of the tunnel (left), a comparison of the results of measurements (blue) and computations (red) of displacements obtained with an extensometer [35]*

TECHNICAL SOLUTIONS FOR ADAPTING TO ACTUAL CONDITIONS IN THE FIELD

Primary support

The primary support in the Dabar tunnel consists of a combination of different structural elements: shotcrete, anchors, reinforcing meshes, steel arches, and reinforcing spears. Based on the stress-strain analysis, 12 different types of primary support were defined. When choosing the types of supports, particular attention was paid to the characteristics of the rock mass (composition, cracking, degree of faulting, and degradation), the initial stress state (height of the upper layer), and the like. Support systems are determined on the spot depending on the determined quality of the rock mass and the performed analyses and computations. For rock mass with better mechanical properties, support with a lesser shotcrete thickness and a smaller number of anchors is used. At the same time, for complex geotechnical conditions, steel arches are also added to take on the load from the rock mass. Figure 20 shows the types of primary support with shotcrete and anchors (left), shotcrete and steel arches, and spears (right).

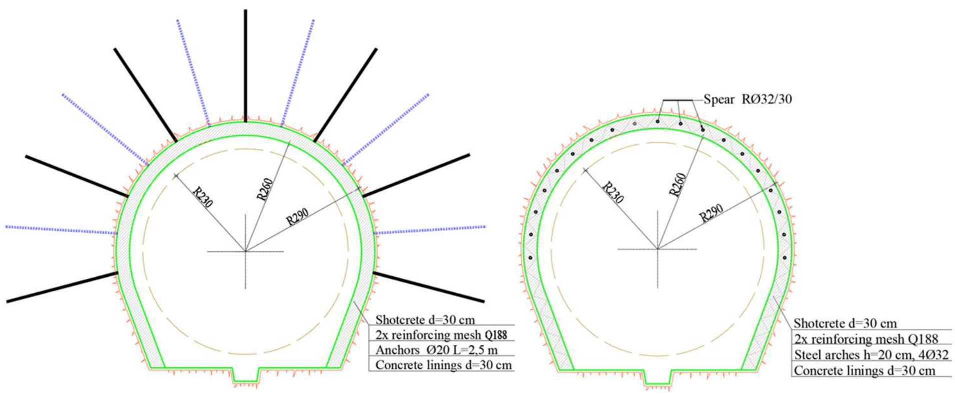

Figure 20. *Tunnel type with shotcrete and anchors (left), shotcrete, steel arches and spears (right)*

Remediation of faulted and cavernous zones

The Dabar tunnel is being constructed in areas where fault-cavernous zones exist. For these zones, special safety measures are applied that depend on the size of the cavern, filling, presence of water, location, etc. During the tunnel's construction, landslides occurred in the form of tens of cubic meters of rock mass from the fault zones, with the falling of large rock blocks that ultimately broke the installed steel arches (Figure 21).

Solving such problems in the tunnel is usually done by applying measures to protect the face of the excavation, which is realized by driving steel pipes along the boundaries of the tunnel (pipe umbrella), grouting through the pipes, and placing steel arches on which the pipes rest, and adding shotcrete (Figure 22). In this specific case, fill had to be added in front of the excavation face before the installation of the pipe umbrella, which protects the works in front of the excavation face in case of landslides and falling out of larger pieces of the rock mass. After the slope is formed, the space inside the landslide and above the calotte is filled with concrete. A pipe is placed on the top of the calotte, and two pipes are placed on the sides, through which concrete is pumped to fill the space inside the landslide on the sides and above the calotte. In this way, concrete reinforcement is formed inside the landslide. After the concrete has hardened, excavations and securing of the tunnel begin with installing steel arches at 50-100 cm intervals. Simultaneously with the installation of the steel arches, the excavation of the material and the installation of reinforcement "spears" are also executed. The spears need to be at least 4 m long in the cross-section and at a distance of about 20 cm, and the steps of progress must not be greater than 50 cm. After placing the centers, shotcrete with a thickness of 30 cm is applied in layers. In addition to reinforcement spears, IBO anchors are also used to protect the face of the excavation, especially when the reinforcement spears are deformed.

Figure 21. *Falling of large blocks from the fault zone during the construction of the tunnel at chain. km 1+839*

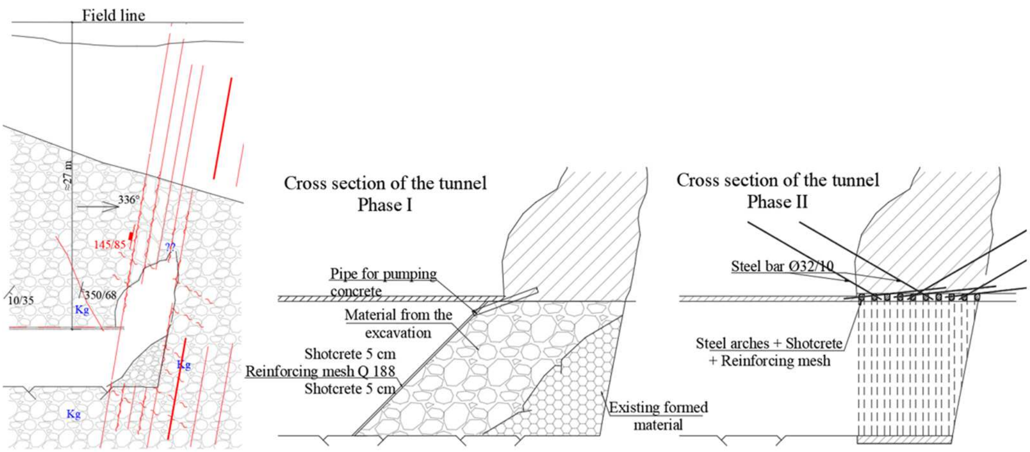

Figure 22. *Remediation of the passage through fault zones in the supply tunnel of HPP "Dabar" at chain. km 1+839*

Secondary lining

After the completion of the excavation and ensuring the excavation of the complete tunnel, the tunnel's concrete lining is added, based on which the final shape of the tunnel is obtained. The design envisages that the daylight opening of the tunnel will be circular with a diameter of Ø 4.60 m; however, during the execution of the works, due to technical reasons of more accessible transport in exploitation and corresponding savings in the amount of concrete, the daylight opening of the tunnel was modified (Figure 23 left). The lining is 30 cm thick and made in 12 m rings. The tunnel's lining is designed to meet the structure's load-bearing capacity requirements, functionality, and durability. The most important influences for the dimensioning and verifying the load-bearing capacity of the secondary lining are hydrostatic water pressures ranging from 2.9 to 5.9 bar and temperature effects. To take on tension stress that is a consequence of the effect of hydrostatic pressure in the tunnel, the stress grouting technique is used, which was successfully applied in our region on the hydrotechnical tunnel of the Pumped Storage Hydropower Plant "Bajina Bašta" [33]. According to this technique, boreholes are drilled for stress grouting after the concreting of the secondary lining is finished. The grout is added under high pressure (in this case, around 12 bar), which creates pressure on the concrete lining and pre-stresses the lining. The minimum principal stress field is shown in Figure 23 (on the right) for the conditions when there is a total water pressure of 5.9 bar in the tunnel and a grouting pressure of 12 bar. The obtained tensile stress values are acceptable from the point of view of the quality of the concrete lining.

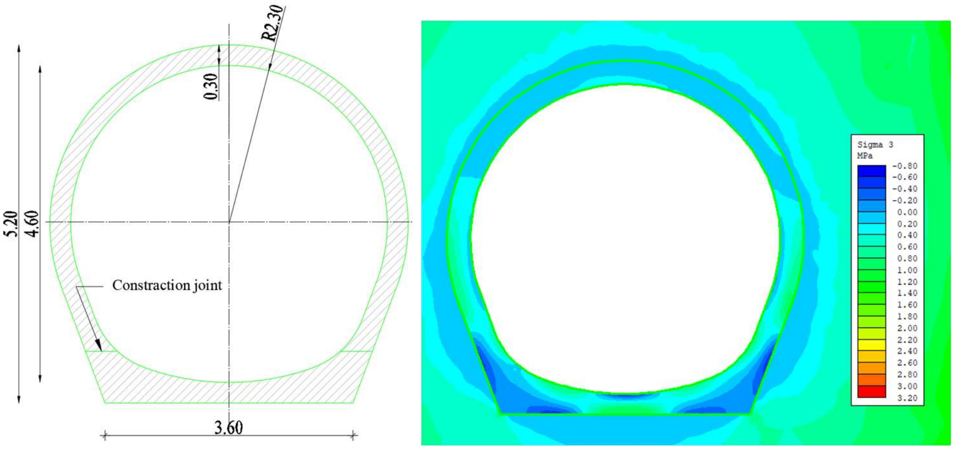

Figure 23. *Cross-section of the secondary lining (left), stress field in the concrete of the secondary lining obtained for the hydrostatic pressure of 5.9 bar and grouting pressure of 12 bar (right)*

CONCLUSION

The construction concept shown on the concrete example in this paper can be useful to engineers when creating projects and constructing tunnels. It is very important to come up with the concept of monitoring and construction of tunnels in the design phase so that all the challenges that may arise can be overcome.

The paper presents the challenges that arise when excavating tunnels in karst terrain. The challenges are related to the problems that appear during excavation using the New Austrian Tunneling Method (NATM) and during excavation using tunnel boring machines (TBM). In addition to a general overview of the challenges, specific examples with remediation methods from world practice and the region are also provided.

The paper also provides an overview of the design and execution of the supply tunnel of HPP "Dabar". The tunnel with a length of 12125 m was a challenge from the point of view of choosing an adequate excavation method and its application in karst. The choice of technology for carrying out works on the HPP "Dabar" supply tunnel was made through a techno-economic analysis with 4 solutions that included a mechanized method of excavation (milling machine) but also the application of different types of TBMs (with grippers and with a double shield), as well as their combination. The techno-economic analysis also included possible financial losses due to work pauses resulting from unforeseen geological conditions in the field.

During the tunnel excavation, various tests and measurements are carried out to define the technical solutions and adapt them to actual conditions in the field.

The application of the construction concept of the HPP "Dabar" tunnel, which is based on permanent monitoring, research, measurement, and collection of all information used to optimize the work execution process and define optimal technical solutions, proved to be efficient and very useful for this particular project. Integrating all the information collected in the tunnel can prevent various phenomena and occurrences during the execution of the works (for example, landslides, fault zones, floods, etc.) and ensure optimal and safe implementation of the works.

The construction concept shown in this paper can be helpful to engineers when creating projects and constructing tunnels. It is essential to develop the concept of monitoring and constructing tunnels in the design phase so all the challenges that may arise can be overcome.

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CONTEMPORARY WATER MANAGEMENT: CHALLENGES AND RESEARCH DIRECTIONS

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Prof. dr Srđan Kostić

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PREFACE

Institute of Hydrology was established in 1947 within the Serbian Academy of Sciences. The Hydraulics Laboratory was established that same year within the Federal Ministry of Electricity, a predecessor of the later Hydropower Institute created in 1950. These two institutions were soon merged under the auspices of the Serbian Academy of Sciences into the Hydrotechnical Institute Eng. Jaroslav Černi. This Institute merged with the Serbian Water Management Institute in 1959 to create today's Jaroslav Černi Water Institute.

Over the past decades, the Institute has been the backbone of scientific research in the field of water in Serbia and the former Yugoslavia. The international scientific conference Contemporary Water Management: Challenges and Research Directions is organized to celebrate 75 years of the Institute’s long and successful history. The Scientific Board selected 26 papers to provide readers with the best view of the current research results, as well as the further scientific research directions and potential challenges in the future. Selected papers are classified into six conference topics according to the corresponding research field, although one should note that most of the presented works is multidisciplinary, which is after all a characteristic of a modern problem-solving approach in the field of water. Hence, the chosen conference topics and corresponding papers represent only one possible way of classification of the presented works.

We wish to express our gratitude to the International Scientific Board and the Organizing Committee of this international conference for their efforts in selecting the papers, reviewing, and organizing the conference. We also wish to express our gratitude to all the authors of selected papers for the time they spent presenting the results of their research in a way suitable for this conference, and for contributing to the celebration of 75 years since the establishment of the Jaroslav Černi Water Institute. Respecting the importance of jubilee and wishing to express gratitude to previous generations of scientific workers, the Honorary Committee was also formed.

Following the path of previous generations, the Institute's present and future staff remain privileged, and under duty and obligation to continue and improve the scientific and research work of the Institute in the years and decades to come.

Belgrade, October 2022

Editors

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