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Single-layer load-bearing tunnel lining structure in hard rock masses

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ABSTRACT

The paper presents some of the observations made during tunnel construction with a single-layer load-bearing lining in hard rocks. The load-bearing elements of one such lining were observed, as were measures to improve the quality of the material, as well as other required actions to guarantee its stability throughout its lifespan. An extremely important measure is to limit the water inflow into the tunnel, by pressure grouting contemporary grout mixtures into the fractures, commonly referred to as pre-grouting. This paper also shows the construction time and cost of single-layer lining compared to the traditional two-layer lining. The construction expenses of single-layer tunnel lining are slightly lower compared to two-layer tunnel lining. Nevertheless, the main benefit is construction time-savings. The observations are presented using a practical example: a tunnel in the limestone rock near the Mratinja dam, between Plužine and Šćepan polje in Montenegro.

1 Introduction

Tunnels in hard rock masses are excavated and traditionally designed in south-eastern Europe (countries of the former Yugoslavia) with two-layer linings. Moreover, the initial lining is installed to preserve the rock mass against the effects of air and moisture. Later, a permanent cast-in-place concrete structure (secondary lining) is installed, designed to withstand long-term loads and meet the requirements of serviceability and durability. Waterproofing is achieved by installing a plastic membrane between the primary and secondary lining, which additionally acts as a separating layer, reducing potential cracks due to shrinkage on the secondary lining.

Single-layer tunnel structures are only used in rock masses where the tunnel can be formed in a circular shape. The single-layer lining was mainly used in a hydraulic tunnel with cast-in-place reinforced concrete.

On the other hand, in the Scandinavian countries, this type of structure has been built for decades as permanent structures for various underground structures (car parks, roads, railway tunnels, underground stations, etc. In the past few decades, significant progress has been made in the understanding and quality assurance of single-layer tunnel structure components.

The purpose of this paper is to show that, in many cases, a single-layer tunnel structure could be designed as an alternative to the traditional two-layer tunnel structure. This is discussed in the example of a tunnel, previously designed by the author as a two-layer structure. The tunnel is designed by utilizing individual parts of a single-layer tunnel structure, based on experience gained in designing tunnels for the expansion of existing metro lines [1], [2], [3], and the main underground bus station in Stockholm [4].

2 Tunnel No. 10 on the road segment Šćepan polje - Plužine

2.1 General tunnel information and basic technical data

The tunnel is planned for the road section between Plužine and Šćepan Polje in the Republic of Montenegro. The current road passes through the Canyon of Piva, above the artificial lake Piva. The lake was formed by the diversion of the Piva River in 1975, by the construction of a high hydroelectric power plant (Mratinje dam), and by flooding part of the canyon. The dam is of the arch type, made of concrete, 220 m high and 261 m long, with a usable volume of about 800 million m^3. The dam is one of the highest of its kind in Europe and an exceptional construction success for the previous country (SFR Yugoslavia). The road was originally built for the construction of a hydroelectric power plant but was later upgraded to serve as the main road. One of the features of the road is that at one section it crosses the dam, i.e., the road crosses over the dam in the crown of the dam. The existing road is narrow and winding, with geometric elements corresponding to a maximum speed of 30 km/h in some sections. In many places on the road, there are unlined rock tunnels, rockfalls, and avalanches, so driving is a constant threat to all who pass. It is also impossible to drive heavy vehicles on the road. To improve the geometric

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elements of the route and meet safety requirements, extensive design work and on-the-road reconstructions have been made. Part of the road after the dam is designed to extend straight into the mountain rather than the existing sharp twist into a 1.5 km long tunnel [5, Figure 1].

The tunnel is designed as a single tube, for two-way traffic with a maximum speed of 80 km/h. The tunnel includes two lanes, each 3,25 m wide, and two edge lanes, as well as emergency evacuation and revision paths on both sides of the tube. The height of the traffic clearance gauge for vehicles is 4,70 m, and the clear height above the revision paths is at least 2,0 m.

The theoretically useful area of the tunnel depends on the radius of the horizontal curve and is A_1 = 51.72 m^2 or A_2 = 56,40 m^2 depending on the type of construction (structure gauge type 1, R_1 = 4,85 m / structure gauge type 2, R_2 = 5,15 m). At the beginning of the tunnel, just after the dam, the tunnel extends with a left horizontal curve with a radius of 350 m. The rest of the tunnel is in a straight direction. The vertical curve of the tube is a concave curve, with a radius of 10,000 m and a maximum longitudinal slope gradient of 2,1%. The tunnel cross-section is rounded – horseshoe-shaped, with a maximum span of 11,5 m and a height of 8,7 m. The tunnel is designed with all the necessary construction safety measures: breakdown bays, fire-protection niches, hydrant niches, niches for placing electrical installations, accommodation of UPS devices, drainage niches, escape routes, etc., and everything is described in detail in the main tunnel design project [5], as well as in the paper [6].

2.2 Geological and hydrological conditions

The canyon at the dam's location is geomorphologically V-shaped, with very steep and nearly vertical valley sides. The entire right bank is made of massive Triassic limestones with an uneven distribution of fractures, Figure 2. According to their origin, the limestones are of the ridge type, influenced by significant tectonic movements in geological history. Regionally, the rock mass is part of the Dinaric karst, and they belong to the synclinal part of the large anticline (called the "Crkvička anticline") which stretches in the direction northwest-southeast, i.e., in the direction of the stretching of the Dinarides [5].

The surface of the canyon is very uneven and gouged, which is typical for thick, layered to massive limestones, affected by the tectonic effects emphasized above, as well as by frost, precipitation, and insolation. As a result of all these processes, in many places at the base of the steep slopes, larger dents were formed. Dents were later filled by loose rock fragments and debris, which were added each year, and in the end, screes were formed.

The presence of several fractures in the limestone rock mass, as well as the carbonate composition, enabled the development of karstification. Chemical erosion happens below the current riverbed and especially along larger cracks and faults. This is because the river flows out very deeply and sometimes a lot of water comes in from the mountainous

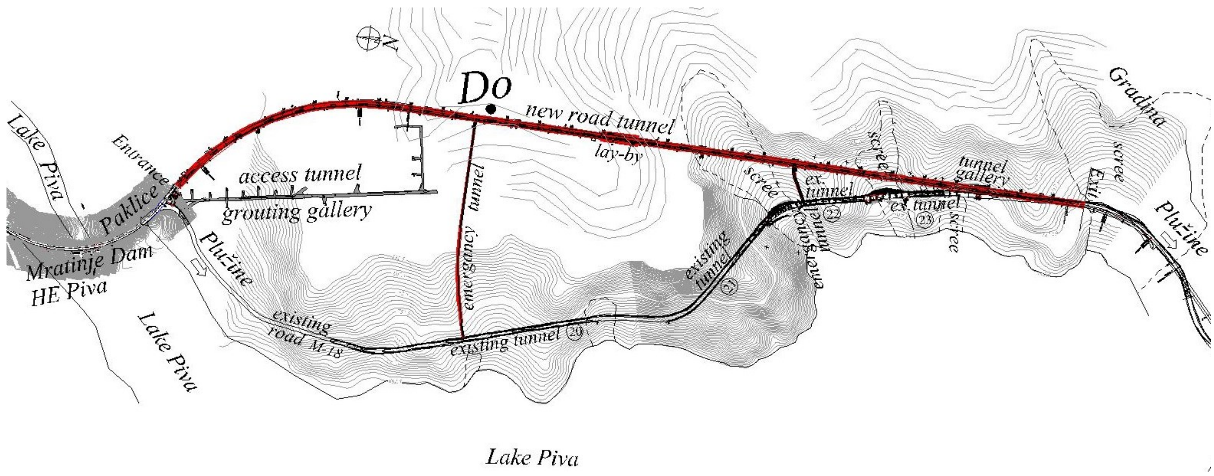

Figure 1. Layout plan of the new tunnel located after Mratinje dam

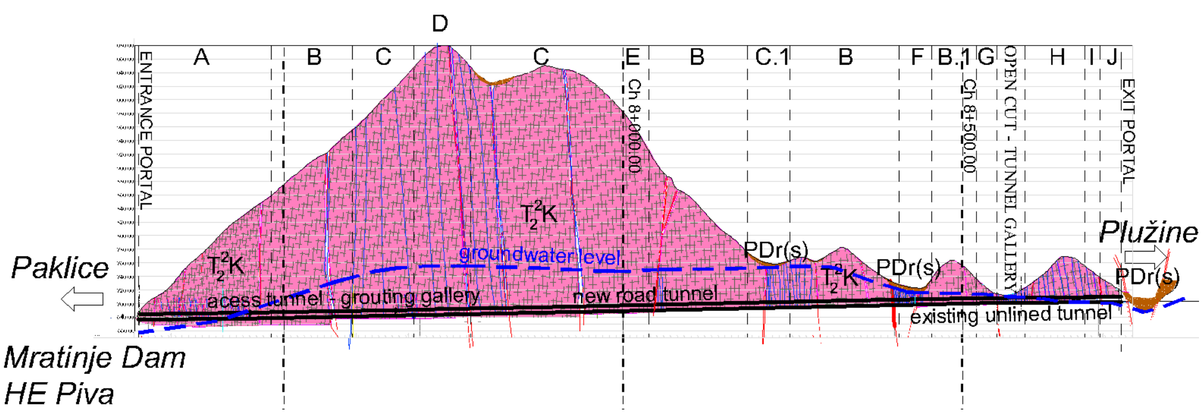

Figure 2. Longitudinal geological section of the new tunnel located after the Mratinje dam

area behind it. Limestones have secondary fissures and cavernous porosity and function as an aquifer. The conductive zone is above, and the storage zone is below the level of the Piva River.

In the vicinity of the dam, limestones have lower fracture density and low permeability ranging from 0.1 to 5 Lu. Permeability in the intervals of 5 - 10 Lu and 10 - 20 Lu occurs sporadically. A smaller number of fractures and lower permeability indicate that at this place they are shallower and less karstified [7].

Because the rocks have a lower porosity and permeability, precipitation infiltrates quickly and groundwater levels rise quickly in the event of heavy rains. In the zones of larger fractures, high conductivity can influence the formation of occasional surface and underground accumulations.

Groundwater location and regime are generally determined by hydrogeological features of the rock mass, as well as groundwater regime in the reservoir's hinterland and perception frequency. The level of the established aquifer fluctuates depending on changes in the water level in the reservoir of the dam. The level of groundwater is not significantly higher than that in the reservoir, except in cases of infiltration from heavy precipitation and after the melting of snow. This phenomenon is due to the existence of a tectonic fault in the hinterland that acts as a conductor and lowers the groundwater level. Once the water level in the reservoir is kept at the same level for a long time, a balance is established between the water level in the reservoir and the inflow of groundwater from the rock massif.

On the right flank of the dam, there is an access tunnel. The tunnel has been used to construct piles in the dam's right foundation. It also served for grouting the dam plug base, the drainage of water from the hinterland, and the execution of geotechnical and hydrological observations.

3 Conventional two-layer tunnel lining

The tunnel is designed as a double shell system, i.e., with two linings. First, the outer layer is placed - the initial tunnel lining (rock support system).

The initial tunnel lining consists of shotcrete, reinforcing steel wire mesh, lattice supports or steel arches, and rock bolts. It is installed immediately after the excavation and is designed to provide sufficient initial structural capacity and stability for the tunnel excavation. A controlled stress release and redistribution in the surrounding rock is enabled by introducing the initial tunnel lining (rock support system). The selection of the excavation and support system (initial lining) was determined based on the behaviour of the rock mass due to excavation. The behaviour (response) of the rock mass is defined without considering any stabilization measures or excavation sequences.

The tunnel has a complex geometry and unfavourable boundary conditions such as the proximity of the access tunnel, the enlarged tunnel cross-sections at the locations of the emergency stopping areas (lay-bys) and turning areas, the proximity of the existing unlined tunnels, settings with insufficient rock thickness above the tunnel crown or a steep slope on the side, or else, tunnelling under the scree deposits or near the fault. Different initial linings (support types) are developed to cover all these tunnelling conditions. Support types B, B.1, C, C.1 G, and I are light support systems distinguished by different shotcrete thicknesses and bolt spacings. They are foreseen on 62,37 % of the tunnel length. Stronger support types are denoted as A, D, E, and H. In addition to shotcrete and bolts, these support types include pre-support via a spilling bolt and lattice girders. They are planned for 29% of the tunnel's length. Heavy support types are denoted as F and J. These support types include, pre-support spiling bolts, HEA steel sets, shotcrete, and bolts, in combination with tunnel face stabilization measures. They are planned for only 5.52% of the tunnel length. A brief overview of support types is given in Table 1, and further support types are described in the paper [6] and the main project [5]. The most common initial lining (support) types are B and C. Therefore, in this paper, the tunnel sections with these types of initial linings have been considered.

The excavation of the rock mass in the case of a two-layer lining is A= 70.01 m^2/m. The tunnel is designed to be driven using the sequential excavation method, with the sequence consisting of a top heading and bench (Figure 3).

The first layer of the lining consists of shotcrete, reinforcing wire mesh, and rock bolts. A flexible load-bearing structure is formed by a combination of all these elements. The main load-bearing element is the rock mass. If there are visible water leaks on the surface of the rock mass or a higher groundwater inflow, the installation of the drainage holes is designed, before placing shotcrete. The drainage holes (weep holes) have a diameter of 50 mm and a length of 3.0 m.

The shotcrete is designed with a compressive strength of C25/30 and is to be applied in two layers, with a final thickness of 10 cm. After the first layer, the fully grouted SN-rock bolts (Store-Norfors) are installed at a c/c distance of 2.0 m. The rock bolts are 25 mm in diameter and 4.0 m long. A welded reinforcing wire mesh Q188 (6 mm / 215 x 600 cm) is attached to the rock bolts, and subsequently, the second layer of shotcrete is applied.

Following the completion of the initial lining, the final (secondary) lining is executed. The final lining (inner layer) is made from cast-in-place reinforced concrete (Figure 4). It is designed by assuming that the entire load (or some portion of the load) carried by the initial lining, at the end of its life, is transferred to the secondary lining. The secondary lining is designed to be load-bearing for long-term loads from rock mass loads, water pressure, installation loads, rheological effects in concrete, temperature changes, etc.

Before the construction of the final lining, a layer of geotextile (500 gr/m^2) is placed, followed by a 2 mm thick layer of waterproofing made of an LDPE (low-density polyethylene) sheet. The seepage water collected behind the waterproofing is conducted by perforated plastic sidewall drainage pipes, placed at the tunnel bottom elevation. The drainage pipes have a diameter of 160 mm, placed on each side. The pipes are backfilled with drainage concrete C12/15. The collected seepage water is conveyed to the tunnel`s main roadway sewage system.

The secondary lining is C30/37 reinforced concrete, with exposure classes XC 3 and XF3 [8]. The lining is 30 cm thick in the tunnel crown, and increases to 40 - 51 cm on the sides. The foundation of the secondary lining is also C30/37 reinforced concrete with a 40 cm thickness. The foundation is built on a 10 cm lean concrete layer.

Table 1. The distribution of the initial tunnel linings (rock support systems) in %

A	B	B.1	C	C.1	D	E	F	G	H	I	J	Open cut
13,47	29,39	4,56	20,51	4,32	5,80	3,87	3,31	2,07	6,09	1,52	2,21	2,90

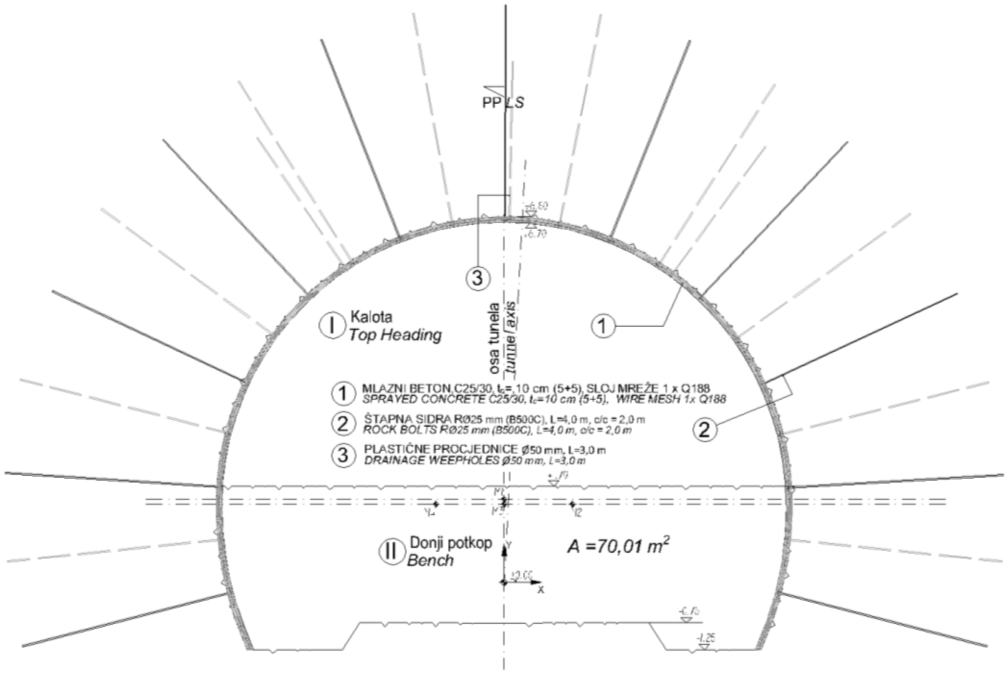

Figure 3. Typical cross-section of the tunnel with the initial lining (support system)

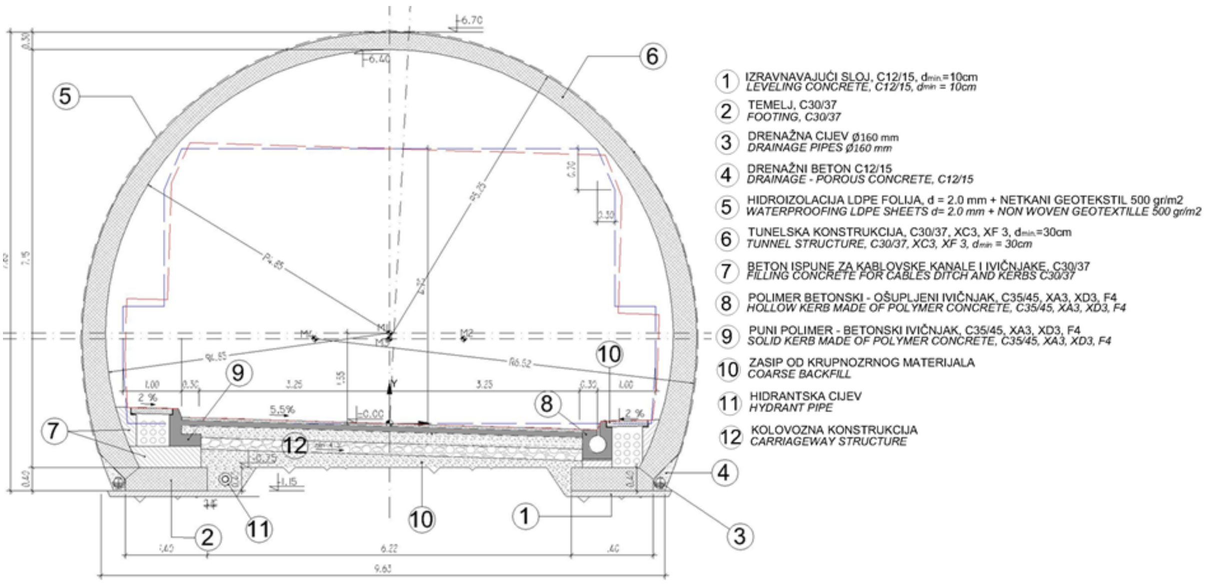

Figure 4. Typical cross-section of the tunnel with the final (secondary) lining

The waterproofing system and secondary lining of the tunnel are designed for a life span of between 80 and 100 years.

Drainage of the tunnel roadway is carried out by a system of hollow curbs. The curbs are collecting water from the roadway and conveying it further via syphon outlets and transverse PVC drainage pipes to the sewage inspection shafts. The curbs are made of C35/40 polymer concrete, with exposure class XA3, XD3, and XF4 [8]. A solid curb is designed on the other side of the roadway. The solid curb is L-shaped and made of polymer concrete.

Sidewalk paths are formed by placing backfill made from regular concrete and fitting the covers at the final stage. The covers are made from prefabricated reinforced concrete. Thus, under the sidewalk paths, the empty rooms are shaped. This space serves to install high- and low-voltage power lines and other tunnel utilities.

In the final stage, water supply and hydrant pipes are installed for firefighting, followed by backfilling of the tunnel bottom, the construction of the pavement structure, protection of the lining surface, and the installation of other required tunnel equipment, etc.

4 Single-layer load-bearing tunnel linings

4.1 General preferences

Single-layer tunnel linings are avoided as permanently load-bearing structures in South-Eastern Europe (countries of the former Yugoslavia). It is because there are many concerns regarding the behaviour of the lining, which make it difficult to assume that it can guarantee system durability over its whole service life. However, such concerns disregard the fact that, in many cases, it has been proven that the secondary lining remained effective and unloaded even 30 years after construction [9,10].

First, the conditions should be distinguished under which the lining is loaded. In hard rock, there is a certain degree of self-supporting behaviour of the rock mass after excavation. Due to the self-supporting capacity of the rock mass, stand-up time can take weeks or even years. This feature of rock mass is decisive. In the case of weaker and more fractured rock masses, the tunnel lining is stressed instantly after installation. Shotcrete has not yet developed the required early strength, and cracks appear and spread more rapidly due to the bending and stressing of the material. In this case, the durability of the system cannot be guaranteed. On the other hand, in hard rocks, this is not the case, so the shotcrete of the initial lining achieves its strength at the right time and assures long-term durability.

A further issue is the presence and control of groundwater. Groundwater is a principal factor that can lead to the decomposition of load-bearing elements of the rock support system. Shotcrete of any kind is not watertight. Long-term groundwater control is uncertain with a single-layer shotcrete lining, despite significant progress in developing more water-resistant mixtures in recent decades. However, much can be done to limit the inflow of groundwater, i.e., by grouting jointed rock with contemporary cement or chemical mixtures.

Based on the above, the application of a single-layer tunnel lining can be successful, but careful work must be assured first. Its application includes two main phases:

- pre-grouting, to seal the rock mass and prevent or reduce the groundwater inflow into the tunnel
- the execution of load-bearing lining, i.e., all elements of the system such as shotcrete, bolts, etc.

In the next few chapters, the parts and features of both phases will be explained, but in the reverse order.

4.2 Load-bearing elements of the single-layer lining

Given the last few premises from the previous chapter, it is possible to design the same tunnel with only a single-shell load-bearing lining, Figure 5. First, it is feasible to accomplish a decrease in the excavation profile of the tunnel. In the case of a tunnel with a single-layer lining, the excavation area is reduced to A= 65.26 m^2/m. That is nearly 7% less compared to the two-layer tunnel lining. That is minor but decent inception.

The load-bearing elements of the system are identical to the initial tunnel lining in the case of a tunnel with a double-shell lining. The difference is in the quality of work and the material properties.

The shotcrete is envisioned to be applied in a single layer, with a thickness of 12 cm. The quality of the shotcrete properties is significantly higher in this case. Shotcrete is designed with a compressive strength of C32/40. The micro-reinforcement has been designed to reinforce the shotcrete. Discontinuous steel fibres are added to the shotcrete to improve the tensile strength of the shotcrete (MAMB). The addition of fibres affects the mixture in both its fresh and solid states. The fibres have tensile strength (yield strength) > 1000 - 2000 MPa, and Young's modulus E > 210,000 MPa. The incorporation of fibers contributes to the quick activation of fibres, even at small crack openings, and the development of crack-bridging stresses. The shotcrete does not reduce its load-bearing capacity after the appearance of the initial cracks. The stress relaxes, and tensile stresses are redistributed and transmitted over the crack, i.e., the fibres are taking the actual tensile stresses. The result is a significant increase in flexural toughness and ductility compared to shotcrete without fibres. The typical quantity of steel fibres ranges from 20 to 60 kg/m^3 of fresh concrete mix.

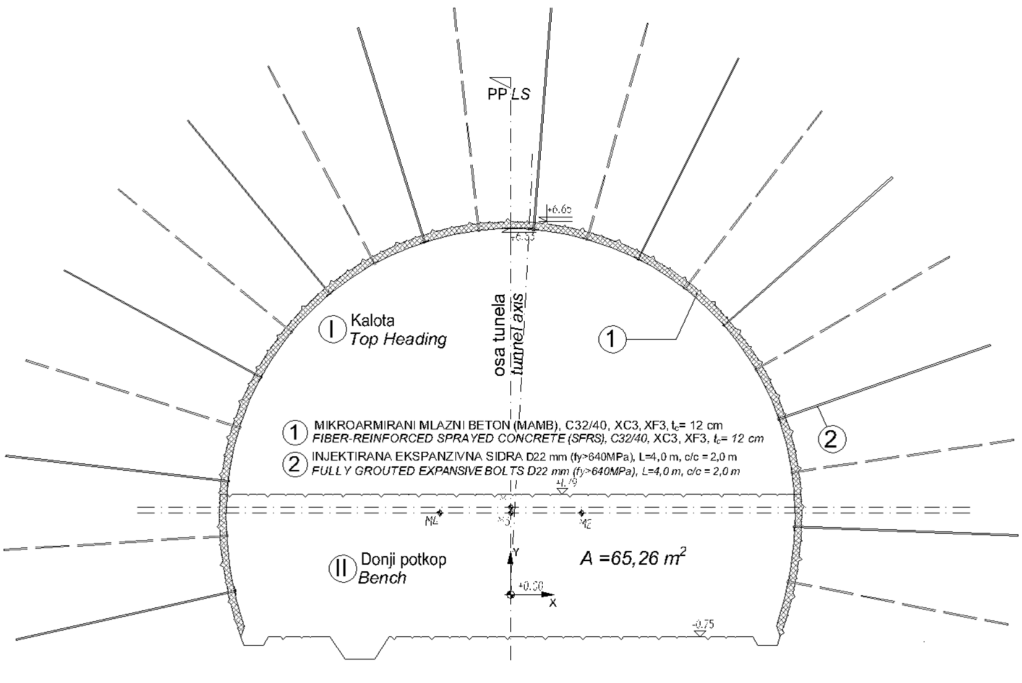

Figure 5. Typical cross-section of the tunnel with the single-layer lining system

In addition to fibres, shotcrete mix is made with mineral additives. First of all, with silicate fume it contains at least 85 - 90% of amorphous silica (SiO_2). The average size of the particles of silicate powder is 0.1 - 0.2 µm and they have a large specific surface area. In the silicate fume, there are also small amounts of iron oxides, magnesium, and alkaline oxides. Silicate fume reacts with free calcium hydroxide, which is formed due to the hydration of cement, forming calcium silicate and aluminum hydrates. These compounds increase the strength, thicken the cement mass, and fill the voids and pores in the concrete, whereby the formed crystals connect the space between the cement particles and aggregate grains. The result is the reduction (or absence) of segregation and a decrease in the permeability of shotcrete. This can be observed in Figure 6. The figure shows an extremely homogeneous and compact concrete structure with pore sizes ranging from 7- 8 µm [11,12]. This mineral additive significantly improves the quality of the mixture, reduces rebound, improves the dispersion of ingredients, and enhances the properties of the mixture in the solid state, including resistance to freezing and thawing and chemical attack [13]. The result is an increase in the overall durability of shotcrete.

After completing the shotcrete layer, rock bolts are installed. Rock bolts with a length of 4.0 m are drilled with a center-to-center spacing of 2.0 m. Rock bolts are installed with a diameter of 22 mm. Corrosion is the main reason for the decomposition of bolts. Corrosion in the underground occurs due to oxygen, water, and humidity. Due to the humid environment, a chemical interaction occurs between the surrounding media and the steel. Strong acids are formed in contact with water and hydrogen sulfide. The acids readily react with steel material, causing severe damage to bolts. Special mechanical rock bolts have been designed [14,15] see Figure 7. They combine two methods of force transmission to the stable rock mass. First, the bolt force is transferred into the stable rock mass by point anchoring using the expansive shell (sleeve). On one bolt end is a conical core that slides inside the sleeve, and at the other end, there is a hemispherical, half-dome that serves to transfer the load from the bolt-bearing plate. Once the torque is applied to the bolt, the rod rotates, and the shell (sleeve) expands over the thread, fixing the rock bolt. On the hemispherical dome, there is also a space for grouting. It is an opening with a diameter of 16 mm. Cement mortar is grouted through the hole, fixing the bolt in its final position. Subsequently, by grouting, the rock bolt is anchored by friction (adhesion) and performs as a fully bound (grouted) rock bolt. The bolt is protected by a corrugated plastic tube. The plastic tube prevents the infiltration of corrosive substances through the cracks in the cement mortar and minimizes the corrosion that could occur on the bolts. On the tube, there are also small buttons (bulges). The bulges ensure improved friction with the grout and serve as spacers/centering rings.

This type of rock bolt is characterized by a double protection system against corrosion, with a plastic tube and cement mortar. The result is an increase in the overall durability of the anchors. The service life of rock bolts is at least 50 years, based on accelerated corrosion tests (ACT) [15].

After the construction of the single-layer lining system, which functions as a permanent structure, an additional - "protective" layer of shotcrete could be designed (Figure 8). This layer is not part of the load-bearing structure. First, a levelling layer of 5 cm is applied. Then, a layer of geotextile weighing 300 gr/m^2 is placed, followed by the installation of a waterproofing layer (LHDPE foil). The waterproofing layer has a thickness of 1.50 mm. The plastic pipes are installed in drainage concrete with a reduced diameter of 100 mm to drain seepage water. The groundwater issues have been resolved by grouting; therefore, the properties of the layer of geotextiles, waterproofing, and drainage pipes are lower compared to the solution with a two-layer lining.

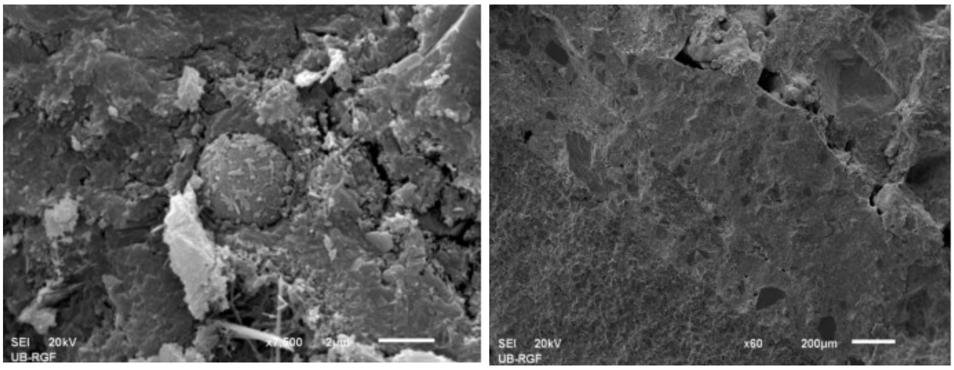

Figure 6. Scanning electron microscopy (SEM) of the structure of concrete with silicate fume [11,12]

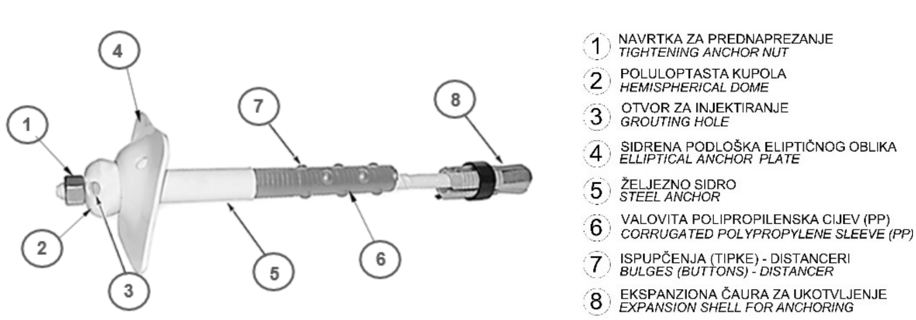

Figure 7. Illustration of the rock bolt with the combined mode of anchoring, point-anchoring, and friction-anchoring (fully bounded)

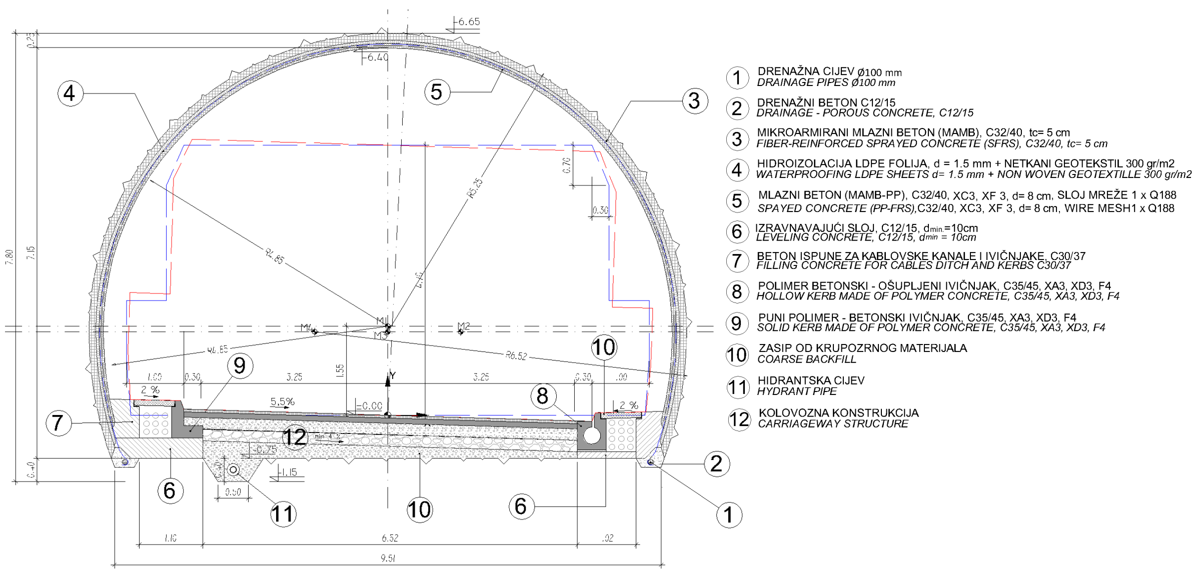

Figure 8. Typical cross-section of the tunnel with the single-layer load-bearing lining

Before the placement of the final shotcrete layer, a thin reinforcing wire mesh (Q188) is installed and fixed. Then, the protective layer of shotcrete is sprayed, in a single layer, with a thickness of 8 cm. The protective shotcrete layer is designed with a compressive strength of C32/40. Polypropylene monofilament fibres (PP) are added to the wet shotcrete mix. By incorporating PP fibers, a network of uniform thickness and surface texture is created within the shotcrete, improving its fire resistance properties (MAMB-PP, micro-reinforced concrete with PP fibers). PP fibres also prevent microcracks in young shotcrete and the particularly explosive spalling of shotcrete during a fire. Explosive spalling is the most extreme and dangerous form of breakdown that occurs during the first 20-30 minutes of a fire when the temperature in the concrete is raised to the range of 150-250 °C. The softening temperature and melting point of polypropylene fibres are relatively low, at about 160 - 170°C, after which they decompose [16,17]. Due to the thermal decomposition of the fibres, microchannels (cavities) and microcracks occur in the shotcrete, through which moisture and gas can transfer within the shotcrete, moving away from the fire. The moisture and gas transfer without restriction, and therefore there is no rise in pore pressure during a temperature rise (heat). The typical quantity of polypropylene fibres ranges from 2 to 6 kg/m^3 of fresh concrete mix. The protective layer of shotcrete doesn't need to be placed all over the tunnel. It is an additional provision if the grouted rock and single-layer (or load-bearing) shotcrete slacken at some point or are placed at the tunnel sections with a greater fire risk.

Drainage of the tunnel roadway is carried out in the same way as in the case of the solution, with a two-layer lining, a system of hollow curbs, siphon outlets, transverse drainage pipes, and sewage inspection shafts. Curbs, sidewalk paths, covers, ducts for the high and low-voltage power lines, etc., were also resolved in the same way as in the case of a two-layer lining system.

The same goes for water pipes and hydrant pipes, backfilling, the structure of the pavement, protecting the surface of the lining, etc.

4.3 Pre-grouting of the rock mass

The pre-grouting is used to reduce the permeability of the rock mass and control the groundwater inflow to a tunnel. The grouting technology has existed for more than 60 years, and substantial experience has also been gained in these regions during the construction of large hydro-technical facilities [18,19], i.e., grouting the dam plug base or hydropower tunnels. Since then, grouting has mostly been designed by using an empirical design approach based on the rule of thumb and without a theoretical or analytical perspective.

Typically, grouting projects began with a thinner mix and lower pressure and progressed to a higher pressure and thicker mixture. But grouting technology has improved and grown over the last 20 years, especially in Scandinavian countries where groundwater can flow in a lot.

Contemporary grouting design is based on estimating the spread of the grout and the trend of grout flow. The grouting is designed by establishing that the grout spreads sufficiently and fills the joints of a particular fracture aperture. The entire grouting process is observed over time to find the appropriate time required for a sufficient penetration length of the grout, grouting time, and pressure.

The grouting design process starts by examining the hydraulic properties of the rock mass and each fracture/joint, i.e., estimating the hydraulic aperture of joints [20]. The hydraulic aperture of the joints is a significant parameter for determining the penetration of the grout, i.e., for choosing a suitable grouting technique. It is presumed that water flow through the joints is proportional to the hydraulic aperture and the hydraulic gradient. The water flow is proportional to the hydraulic aperture to the cubic power, i.e., the traditional concept of "cubic law" is assumed [21]. The equations for fluid dynamics are solved by assuming steady one-dimensional laminar flow through a smooth fracture between two infinitely parallel plates. By applying "cubic low" and solving the basic equations for fluid dynamics, the relation between transmissivity and hydraulic aperture is obtained:

$$T = \frac{w \cdot b^3}{12} = \frac{\rho_w g \cdot b^3}{12 \cdot \mu_w} \quad (1)$$

where T is the transmissivity of the rock mass, ρ_w is the density of water, μ_w is the viscosity of the water, and b is the so-called hydraulic aperture of the joint.

The grout is considered a viscoplastic material, i.e., the grout will not start to flow until the pressure exceeds a certain critical value. Furthermore, it is assumed that the joint aperture is constant (flow through uniformly open plates) as well as the grout properties over time. Then, analytical methods are used to determine the grout penetration length, grout properties, and the filling of the joints.

A brief description of the pre-grouting tunnel design is presented in the following few paragraphs, without going further into theoretical details of the grout flow model through the joints and the penetration length of the grout, as this explanation goes beyond the scope of this work and requires a separate one.

The hydraulic conductivity and aperture of joints were determined using the hydraulic properties of the limestone joints in the dam area and water inflow measurements in the right flank (drainage gallery). The problem is solved by the inverse procedure. Grouting design entails creating a grout mix that can fill joints with the smallest hydraulic apertures of 0.04 mm and the largest hydraulic apertures of 0.4-0.6 mm, then hardening in the joints. The grouting procedure must be carried out with a high grouting pressure of 25 bars, not exceeding the grouting time of 30 minutes. It seals the rock mass for the entire length of the tunnel (8.0 m) and allows minor water into the tunnel. The maximum cement consumption should be 380 kg/borehole, which is about 15.2 kg/m^1 of the borehole.

The pre-grouting is to be performed by using 25 m-long drilled grouting holes, spread around the tunnel in each grouting profile (Figure 9). The grouting holes are designed with a diameter of 42 mm, inclined to the surrounding rock mass by 18°. The tip distance of the grouting holes along the tunnel perimeter is 4.0 m, as is the end of the grouting fan. The grouting fan in the longitudinal direction is 15 m, so there is an overlap of approximately 10 m between the fans.

To meet sealing requirements, the grout mix must contain cement with a maximum grain size of 30 m, as well as a low viscosity and yield point. The cement is grouted in a liquid state. The water-to-cement ratio is w/c = 0.8 - 1.0. It is preferable to make a grout mix with mineral additives, i.e., silicate powder, and add limited amounts of accelerators and superplasticizers. Thus, good stability is obtained in combination with the low viscosity of the grout mixture, which is essential for good penetration into the joints.

The grouting procedure is performed before the rock excavation, first by drilling grouting holes. Then, the holes are cleaned and washed from the rock particles and dirt accumulated during drilling. Before grouting, the water ingress measurement is completed. After grouting all the holes, the additional control holes are drilled and the water measurement is accomplished again. The grout mixture then starts to harden. After grouting all the holes, the additional control holes are drilled, and the water measurement is accomplished again. If the grout take is 2.0 l/min in 5 minutes of grouting with the designed and stable grouting pressure, the grouting is stopped, and the controlled holes are sealed. The grouting starts from the bottom of the profile, moving to the left and right and toward the top.

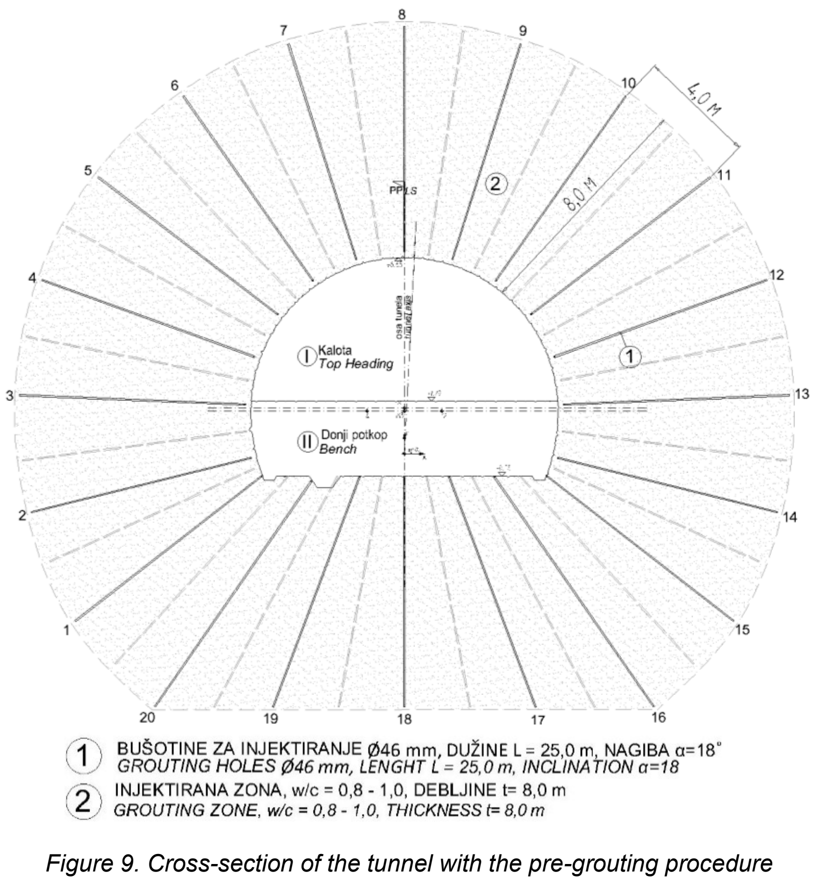

Figure 9. Cross-section of the tunnel with the pre-grouting procedure

5 Evaluation of construction time and costs for both alternatives

Based on the previous chapters, both solutions can be compared in terms of time and cost savings. In Figures 10 and 11, the average construction time is presented for the excavation round cycle and the construction of cast-in-place concrete, respectively.

According to the evaluations, the single-layer lining construction time is 14.80% less than the initial lining of the two-layer lining system. This construction time is without considering time for grouting work. However, the construction time of the single-layer lining, including the grouting works, compared to the two-layer lining, is 16% longer. On the other hand, the construction time of the “protective” shotcrete layer of the single-layer lining is 42.6% shorter than the construction time of the secondary lining of the two-layer tunnel lining. Finally, it was recognized that applying a single-layer liner results in an overall time savings of 28%.

A reference unit cost was defined to make comparing both alternatives easier. The excavation of a two-layer lining section of 1.0 m is set as a reference unit cost. Figures 12

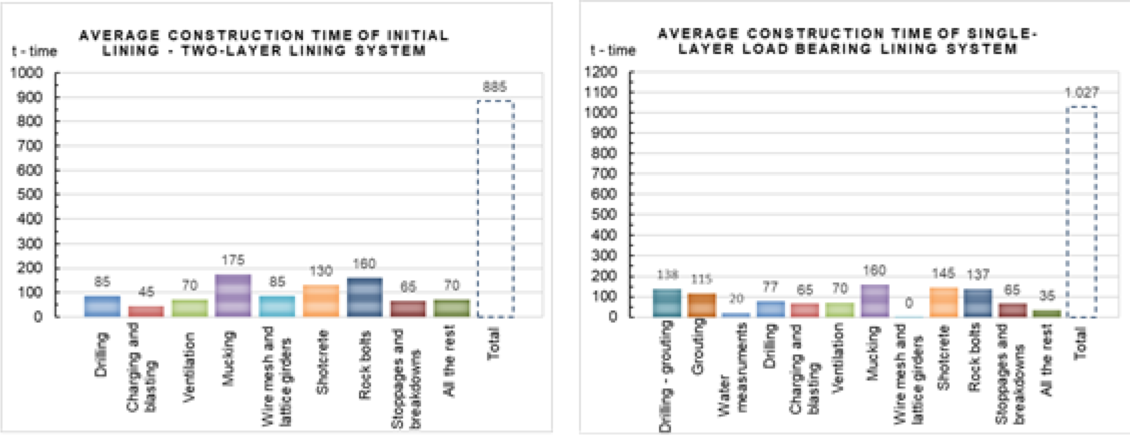

Figure 10. Average construction time (in min.) of initial lining and load-bearing structure in the case of two-layer and single-layer tunnel lining

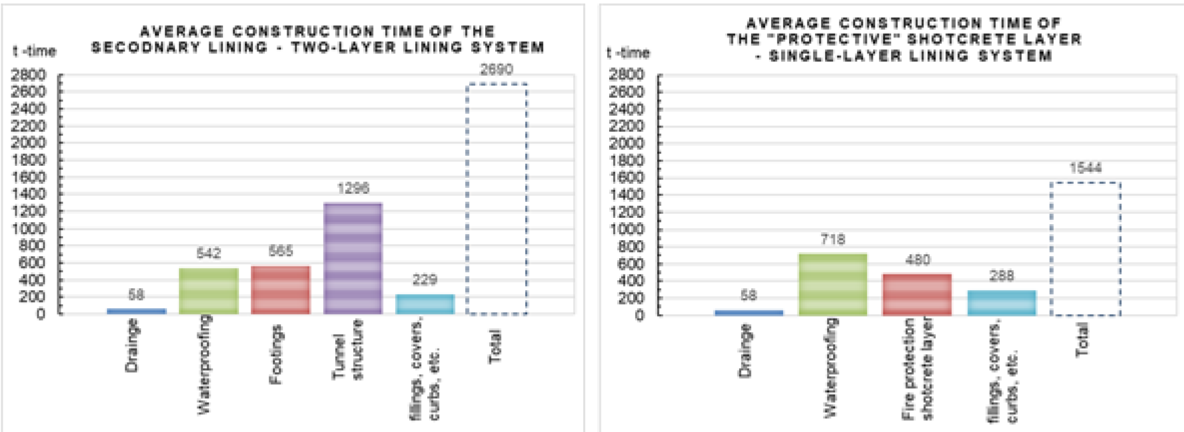

Figure 11. Average construction time (in min.) of the secondary lining and “protective” shotcrete lining, respectively, in the case of two-layer and single-layer tunnel lining

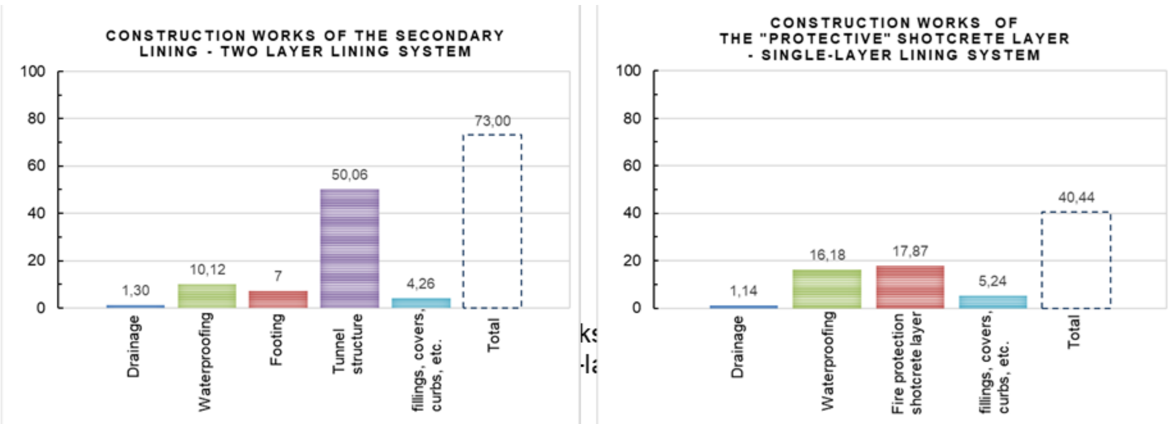

Figure 12. Approximate costs of works of initial lining and load-bearing structure in the case of two-layer and single-layer tunnel lining

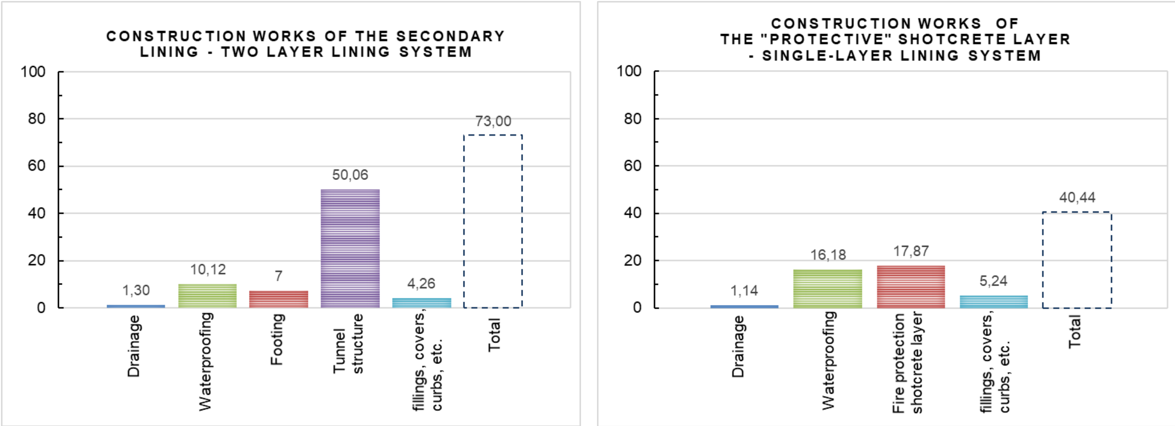

Figure 13. Approximate costs of works of secondary and "protective" shotcrete lining, respectively, in the case of two-layer and single-layer tunnel lining

and 13 present the average construction costs for both solutions under normal construction conditions and minor disruptions or breakdowns.

It has been established that the work costs of the single-layer lining are lower. The work costs for completing the load-bearing elements of a single-layer lining, compared to the similar elements of the initial lining of a two-layer system, are higher by 19.08%, on average. These work costs include the pre-grouting work.

On the contrary, the cost of construction work on the "protective" shotcrete layer of a single-layer lining is lower by 44.61% than the cost of construction work on the secondary lining of a two-layer tunnel lining system. The overall savings achieved by a single-layer lining compared to a two-layer tunnel lining is 2.47%. In the tunnel sections where the "protective" layer of shotcrete is not required, the overall savings could rise to 10.8%. In that case, this construction time would also be further reduced.

6 Conclusion

The paper compares two fundamentally different solutions for tunnel lining in hard rock, single-layer, and two-layer tunnel lining. The load-bearing elements of both alternative solutions are identical, but there is a difference in the quality of work and the material properties of the systems.

It explains the progress in the quality of the material in tunnelling with a single-layer lining, which guarantees the stability and durability of its load-bearing elements. It is also emphasised how critical the pre-grouting procedure is to the overall success of the approach.

The construction of a single-layer lining is more effective. It was determined that the costs of the single-layer tunnel lining are slightly lower, i.e., by around 2.47%, than the usual solution of the two-layer tunnel lining. Higher savings of 14.78% can be achieved if minor water ingresses are observed during tunnel driving. In this case, cost reduction is realized by neglecting or optimizing the previous pre-grouting design.

The main benefit of the single-layer tunnel lining is construction time-savings. Construction time is reduced by about 28%, as is the use of significantly less concrete and steel, and the number of labourers involved in the construction process. Nevertheless, the severe differences in costs and construction time emphasized in the paper [22] were not initially observed. However, the applicability of the solution is not reduced by this observation. On the contrary, the significant difference in costs and construction could be achieved by reducing sections with grouting works and a "protective" shotcrete layer of single-layer lining.

Finally, for the successful application of solutions with single-layer load-bearing tunnel lining, the accessibility of materials, the experience of labour with the new technologies, and the positive attitude of stockholders towards the use of an innovative solution are equally important.

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