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28–30 April 2014, Split, Croatia

Road and Rail Infrastructure III

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SV. ILIJA TUNNELS THROUGH BIOKOVO MOUNTAIN

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Abstract

The Sv. Ilija tunnel, 4.1 km long with longitudinal alignment gradient of 2.5%, was designed and constructed to improve transport links of the Makarska Riviera with the rest of the roads in Croatia. This traffic connection between the Makarska seaside and the A1 highway: Zagreb-Split-Ploče will carry up to 8600 vehicles/day as compared to the current number of 3000 vehicles per day (estimates for the year 2026). The longitudinal ventilation system with adequate improvement was installed by building service tunnel connected to the main tunnel tube every 700 m for fire trucks and every 250 to 300 meter distance for pedestrian crosswalks. The main building (geotechnical) project was developed by SMAGRA Ltd. Zagreb. According to the Traffic study (IGH, 1999) expected construction costs amount to 428.000.000 HRK. The estimated construction cost value based on the main construction project is as follows: a) main tunnel – 320.728.00 HRK and b) service tunnel – 64.145.000 HRK (7,5 HRK = 1,0 EUR). The main construction project for the building permit consists of eight books Phase I and thirteen books Phase II (installation projects). The building permit for the tunnel construction was issued on February 19, 2001. The construction of the tunnel was completed and handed over to the operator for use in 2013.

Keywords: tunnel, main construction project, geotechnical project, engineering geology, rock mass

1 Introduction

The Sv. Ilija tunnel, 4100 m in length and with the longitudinal alignment gradient of 2.5% connecting the Makarska coastline with the A1 motorway, was designed and constructed through the Biokovo rocky mountain in the direction of Baško Polje. Considering the complex engineering geology situation and high seismicity level (IX seismic zone), the construction process had to deal with the complex structure of the rocky massif, partially defined underground water level and changing geotechnical parameters (compressive strength, deformability module). The height of the overburden rock mass is about 1300 m, so far the highest overlaying rock mass in tunnel construction in the Republic of Croatia. Considering the length of the tunnel, it should be notified that the tunnel was dimensioned to comply with the traffic forecast of 8600 vehicles/day. By the European Tunnel Assessment Programme classification (Annex 4/2004) the tunnel is assigned to current updated Category II list. As for the traffic, the ventilation must ensure that the emission of the fuel gas is within the permissible limits. The European Union regulations (Annex 4/2004) require adequate equipment (lightning, traffic signaling, hydration system, fire alarm system) with no increase in the traffic accident risk. Having in mind the tunnel length of 4100 m, with the threshold limit value for longitudinal ventilation of 4000 m, the longitudinal ventilation system was installed to improve service tunnel connected to the main tunnel. This set up maintains traffic connections for fire trucks every 700 m and walkways for passenger evacuation every 250 to 300 m.

Figure 1 The tunnel route

2 Engineering geology, Hydrogeology, Geophysical and Geotechnical surveys

Submitting project documentation for building permit required engineering geological, hydrological and geophysical surveys as well as geomechanical laboratory sample testing. The respective geotechnical project incorporates synthesized field research findings. Engineering geological and hydrogeological investigations were conducted at the north and south portal of the Sv. Ilija tunnel on the Baška Voda-Zagvozd-Imotski road. Conducted research involved detailed geological mapping, field screening of lithological progression, element layers measurement and identification of structural tectonic elements, such as outspreading and slope of fault zones, measuring of breach systems, breach system penetrability, its stuffing and others. In addition to geological prospection, hydrogeological prospection on tunnel portal locations was also conducted. Restricted hydrogeological investigations were carried out by measuring the hole drilled in the near vicinity of the tunnel´s north portal and hydrogeological categorization of the rocks in the exploration area. Based on geological mapping specific locations for setting geophysical profiles were determined. Accordingly, tectonic systems in the portal zone of the tunnel were confirmed, i.e. geometry zone of the talus deposits determined by the geological mapping on south tunnel´s portal.
Based on geological prospecting the following lithostratigraphic segments were selected:
- well-bedded limestones and dolomites $J_{1,2}$;
- well-bedded limestones $J_{2,3}$;
- limestones and dolomites with massive limestone cartridges $^{1}K_{1}$;
- dolomites, dolomitic limestones and limestones $K_{1,2}$;
- poorly bedded bioaccumulated limestones K_2^3;
- bulbous glauconitic limestones and marl E_2;
- talus breccias Q_d.

Structural-tectonic pattern of the whole Biokovo area is characterized by a high degree of tectonic disturbance as a result of geological movements from older Mesozoic until today. The structural unit of Biokovo is clearly bordered with two strong dislocations from the southwest and northeast side. In the southwest the Jurassic and Cretaceous limestones are much drawn to the Eocene detrial Makarska units. As to the northeast the upper Cretaceous limestones of the Slivno structural units are partly drawn to the equivalent upper Cretaceous sediments of the Biokovo units.

Hydrogeological relations at the location are determined by structural-tectonic interrelation, relief and hydrogeological properties of the represented lithologic segments. The relief is characterized by steep slopes made up of limestones. The additional relief expression is caused by fault and fissure systems with occasionally occurring vertical sections. At the north portal location heavily karstified limestones with numerous karst formations and open fracture system are recorded. The emergence of caverns and individual large cave objects in this part of the field was anticipated. Following the measured data, the depth of karstification was not approximately 30 m below, except in places where, due to tectonics, deeper chemical limestone abrasion was encouraged. Chemical limestone abrasion (karstification) spreads over the investigation area. Under such conditions, surface humus layer fails to occur. Consequently, the infiltration of rainwater into underground occurs entirely in the absence of superficial outflow. Numerous springs on the coastal area, as well as the total absence of wells and surface runoffs, confirm the appearance of underground storm water runoff.
Locations for profiles of geophysical testing were determined according to field screening and data analysis. Two refraction profiles in the southern portal and three refraction profiles in the northern portal were derived. Rank interpretation of speed amount is up to 6500 m/s, and reached interpretation depth is between 20 and 40 meters. Based on the refractive research results, the grade levels of both tunnel tubes at the northern portal are located in the area of compact carbonate rocks (seismic wave velocity > 4000 m/s) of which only the deviation of the area around chainage 3 + 700 was recorded – the spot where strongly weathered fault zone carbonate rocks were observed. The soil difference between the main and service tunnel tube was not evidenced by the cross section interpretation results. The interpretation of geophysical profiles at the tunnel tube grade level at the south portal revealed the compact carbonate. However, narrow portal sites were rugged with distinct fault zones.

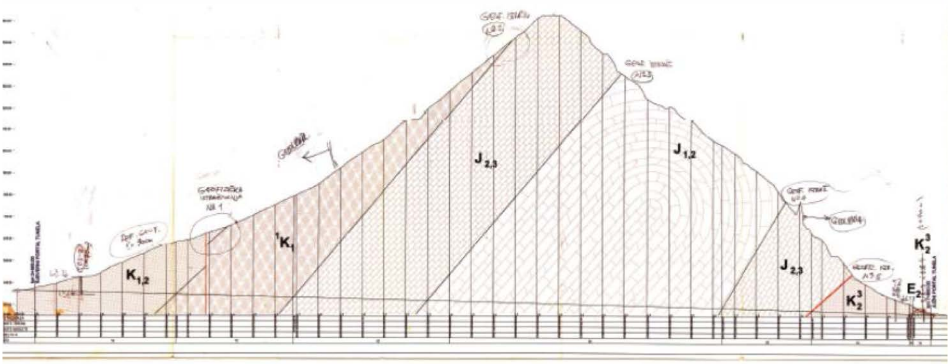

Figure 2 Engineering geological cross section

The 23.8 m deep exploration well was drilled in the hinterland of the north portal. Core drill was determined in the field, RQD and relatively small quantities of groundwater levels measured. After determination of cores for laboratory testing samples were taken to the depth of 8.9 m. Thereafter, laboratory engineering mechanic testing was performed at the Faculty of Civil Engineering, University of Zagreb. The values of compressive strength (σ_{tl}) increase with sampling depth ranging between 62 and 133 MPa, while the values of tensile strength (σ_{vl}) obtained by the Brazilian test range between 7 and 19.5 MPa. Also, elasticity module E of 69 GPa value was determined.
The carried out surveys and collected results were used for designing the major geotechnical project of the main and service tube of the Sv. Ilija tunnel.

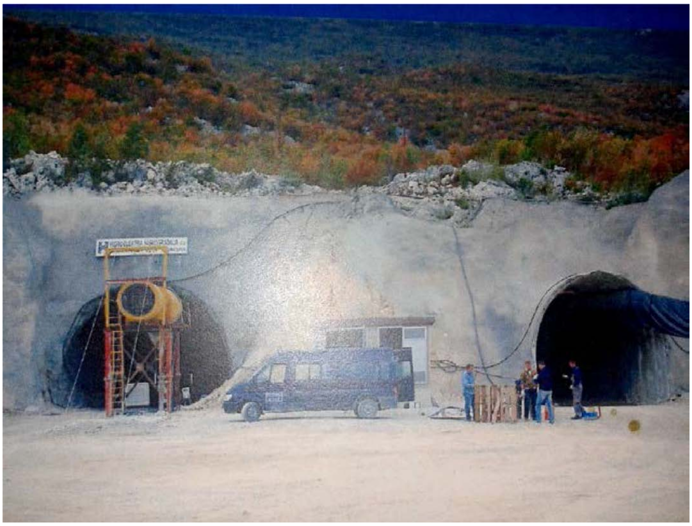

Figure 3 The north portal of the main and service tunnel tube

3 Main construction project for obtaining the building permit

The main construction project and project solution are designed for the Sv. Ilija tunnel between north and south portals on the Baška Voda-Zagvozd-Imotski road. Major construction project follows project solutions derived from Expert Elicitation for issuing the location permit. Documents required for obtaining a location permit were prepared by Geoprojekt Company in November 2004. The location permit was issued by County Road Administration. The lack of visibility in the tunnel portal zones is considered to be the basic disadvantage of the demonstrated project solution. The north portal lies within a 500 m radius, and the south portal within 300 m horizontal radius zone. The north portal is 363.70 m above the sea level, and the north portal 263.02 m above sea level with unilateral longitudinal slope of 2.50%. According to the European Agreement (Annex 4/2004) tunnels are categorized into classes. Constructing measures for respective traffic volume (vehicles/day) are specified by the same agreement as follows:
- fire roads;
- emergency exit;
- rescue emergency cross passages every 1500 m;
- escape windows;
- middle lane crossing;
- emergency roadside stop.

As for the groundwater coming from direction of Imotski, the designing of one-side slope of the roadway tunnel was considered necessary according to the conducted research. [Normal] The use of SI units is strongly recommended and mixed units are to be avoided.

3.1 Proposal for amendments to the main construction project

The change in the tunnel layout position was anticipated in the south portal due to the exact location of faults and talus deposit zone. In this view it is important to consider also the high level of seismicity (IX zone). The south portal should be moved closer to settlement, i.e. east of the talus and not towards the quarry. Due to the possible rising of groundwater above the tunnel alignment, additional explorations in the northern portal zone were required prior to executive project finalization.

3.2 Rock mass geotechnical data

Lithostratigraphic units described in previous section and certain geotechnical parameters were singled out on account of geological mapping, exploration drilling in portal zones, geophysical surveys (seismic refraction) on individual longitudinal and transverse profiles and aircraft shot analysis of wider Biokovo area.

Table 1 Adopted uniaxial compressive strength of homogeneous sample rocks σ_{ci}, Hoek-Brown constants m_i and geological strength index GSI for lithostratigraphic units passed by the tunnel

Lithostratigraphic units	$K_{1,2}$	$^1K_1$	$J_{2,3}$	$J_{1,2}$	$J_{2,3}$	$K_{2,3}$	E_2	K_2^3
Stationing [km]	3+500-4+148	4+148-4+684	4+684-5+299	5+299-6+643	6+643-7+055	7+055-7+506	7+506-7+519	7+519-7+600
σ_{ci} [MPa]	92	118	80	70	80	92	10	60
m_i [l]	10	10	10	10	10	10	10	10
GSI [l]	14	70	60	55	60	54	20	34

Estimated values of GSI (Geological Strength Index) based on the speed of longitudinal waves in the northern and southern sides are expressed by equation (Bienawski, Barton, 1991):

$$GSI = 9 \ln Q + 4 \tag{1}$$

$$Q=10[(Vp-3500)/1000] \tag{2}$$

where:

Vp velocity of longitudinal waves;
Q quality of the rock mass classification by NGI classification.

Table 2 Firmness and rock mass deformability parameters on the alignment level of tunnel by lithological units

Lithostratigraphic units	$K_{1,2}$	$^1K_1$	$J_{2,3}$	$J_{1,2}$	$J_{2,3}$	$K_{2,3}$	E_2	K_2^3
Stationing [km]	3+500-4+148	4+14 -4+684	4+684-5+299	5+299 6+643	6+643 7+055	7+055 7+506	7+506 -7+519	7+519 -7+600
Max. Overlay height (m)	237	460	879	1319	734	361	45	41
Cohesion c' (MPa)	1,933	4,926	4,086	4,439	3,704	2,377	0,1283	0,3884
Friction angle ρ (°)	46,50	47,09	37,40	31,90	36,77	43,51	31,39	49,64
Frimness σ_{cm}(MPa)	9,67	25,29	16,53	15,96	15,44	11,06	0,46	2,11
Def. module Em(MPa))	12075	31622	15905	11157	15905	12075	562	3083

Uniaxial rock mass firmness σ_{cm} from the Mohr Colomb diffraction criteria according to the equation (Hoek, Brown, 2002):

$$\sigma_{cm} = \frac{2c' \cos\varphi}{1-\sin\varphi} \tag{3}$$

where c '(cohesion) and φ (friction angle) are determined from the nonlinear relationship σ_n-τ for values of normal overlay strainings.

Rock mass deformation module in lithostratigraphic units was determined using the equation (Hoek, Brown, 2002):

$$E_m = \left(1 - \frac{D}{2}\right)\sqrt{\frac{\sigma_{ci}}{100}} \times 10^{[GSI-10]40} \quad (4)$$

where D stands for mass deformation factor in contour surroundings depending on excavation technology.

3.3 Calculation of stability

Calculation of stability is based on the numerical analysis of various forms and dimensions of the tunnel support system (Hoek, Marinos 2000)

Table 3 Primary tunnel support and excavation method following geomechanical classification for horseshoe tunnels ranging up to 10 m

Rock mass category	Excavation	Primary support		
		Passive anchor diameter 20mm	**Shotcrete**	**Steel arches**
RMR=81-100 I Very good rock	Excavation throughout the profile. 3 m excavation step	General support not required, except sporadic single anchoring		
RMR=61-80 II Good rock	Excavation throughout the profile. 1-1.5 m excavation step. Finish support 20 m away from the front.	Sporadic anchoring in the roof. Anchors 3 m length at 25 m distance. Partially steel mesh.	50 mm on roof where necessary	Without arches
RMR=41-60 III Favourable rock	Excavation in two phases. 1.5-3 m step excavation into the roof. Support setting after every single tunnel lining excavation. Finish suppport at 10 m distance from the front.	Systematic bolting anchors 4m length at 1.5- 2 m distance in roof and walls. Steel mesh in roof.	50-150 mm on roof and 30 mm on walls.	Without arches
RMR=21-40 IV Poor rock	Excavation in two phases. 1-1.5 m step excavation into the roof. Support setting along with excavation.	Systemic bolting anchors 4.5 m length at 1-1.5 m distance in roof and walls. Steel mesh in roof and walls.	100-150 mm on roof and 100 mm on walls.	Light to medium spaced 1.5 m where necessary
RMR‹20 V Very poor rock	Elaboration of the excavation profile in the roof 0.5-1.5 m. Support setting along with excavation. Installation of shotcrete immediately after excavation.	Systemic bolting anchors 5-6m length at 1-1.5 m distance in roof and walls. Steel mesh in roof and walls. Undershot anchor roofing.	150-200 mm on roof, 150 mm on walls and 50 on front	Moderate to severe at 0.75 m distance. Steel props roofing if necessary. Closed undershot roofing.

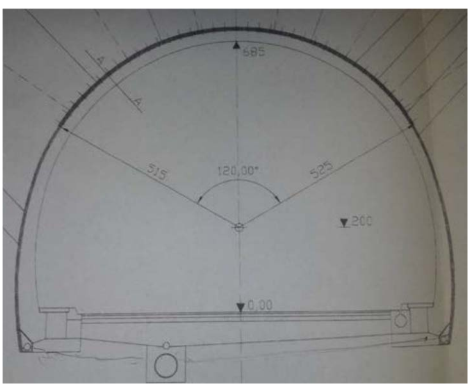

Figure 4 Support systems of the main tunnel – type III

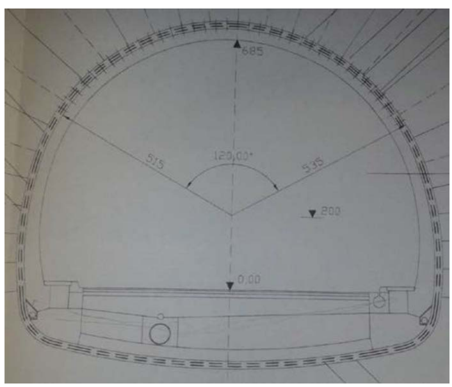

Figure 5 Support systems of the main tunnel – type V

3.4 Geotechnical measurements

During the tunneling convergence measuring on triangle sections (one point in calotte and two in hips) is obligatory. Convergence measurements between points in the cranium and hips should be performed immediately after installing the adequate support system.

4 Conclusion

Standard methods for main and service tunnel design were applied. Considering the insufficient exploration of the rock massif (engineering geological aspect and hydrogeology), potential hazards and increasing risk may appear.
Design and construction of this long and deep tunnel (the largest overlay 1300 m) in the karst area of the Adriatic coast were carried out largely on account of the assessment of geological, engineering geological and hydrological parameters with lots of uncertainties especially in terms of caverns and sudden groundwater penetration.
As to the morphology of the surface, it was impossible to set up the longitudinal geophysical profile, leading to unsatisfactory results. This was confirmed in terms of bridging over large caverns in the service tunnel near the northern portal. For derived refractive profiles, the roadway level was not scored in most parts of the tunnel.
Hydrogeological research studies were not sufficient enough especially in the northern zone of the portal and southern portal of the fault zone. Additional hydrogeological investigation is strongly suggested in the project documentation.
The vertical pressure at the tunnel outbreak on the edge of the rock hole is 70 Mpa, which may increase the risk in case of adverse loads ((hydrostatic pressure and earthquake).
Connecting of drainage pipes, set on the edge of the support system and DN 150 rock mass on to the DN 600 central drainage pipe in the middle of the tunnel, increases the risk of groundwater penetration.
The tunnel construction started in 2008 and finished in 2013. Construction work was performed by Hidroelektra, and Konstruktor Split, companies.
Building permit for Sv. Ilija tunnel was issued on February 19, 2001. Responsible for obtaining necessary regulatory permits are as follows: Cad Com, Zagreb; Geoaqua, Zagreb; Moho, Zagreb; Smagra, Zagreb; Faculty of Civil Engineering University of Zagreb; Promel project, Zagreb; and Dalekovod, Zagreb.

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