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(RESEARCH ARTICLE)

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Application of seismic methods for determining the depth of the rock mass damage zone around the excavation profile by blasting

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Abstract

Research and determination of the blasting damage zone around the profile of tunnels or underground structures is a platform that would serve to confirm or correct blasting parameters and underground assemblies, that is, geotechnical and main tunnel projects. From the obtained research, the stability of the excavation profile, as well as the safety risk for equipment and personnel on the construction site, would be better understood. Considering the importance of the size of the damage zone of the rock mass in terms of its influence on stability, the amount of primary support elements required to achieve the stability of the excavated space, and the additional costs of stabilization of the excavation and the safety of people and equipment, the aforementioned problem was investigated and processed for specific cases and for the rock mass characteristic for the location in question. From the described research, the change in the elastic characteristics of the rock mass and the depth of the damage zone around the tunnel excavation profile were measured and calculated.

Keywords: Excavation; Tunnel; Blasting; Damage Zone; Seismic; Cross-Hole; Refraction

1. Introduction

Excavation can be considered the most important phase of tunnel works due to the number of consequences it can cause. Excavation of tunnels in the world, including in our country, using drilling and blasting techniques is widely used due to its practical and economic nature. As a result of the excavation of the tunnel by blasting, a certain degree of damage to the rock mass occurs around the contour of the section of the excavation [1]. Low-quality excavation using the blasting technique can cause increased overbreak excavation and excessive loosening of the surrounding rock mass, as well as a decrease in the load-bearing capacity of the rock, which is the basic supporting element [2]. Drilling and blasting is usually approached with insufficient attention, which, due to unprofessional execution, causes an increase in the volume of other works, which results in an increase in the total cost of tunnel construction. The released energy of the explosive charge during blasting is spent on destroying and crushing the rock, while a part is converted into kinetic energy of seismic waves [3]. During the passage of seismic waves, ground oscillations occur, i.e. artificial earthquakes, during which zones of damage to the rock mass by blasting can occur, which, if excessive, can lead to progressive local failure, i.e. threating the stability of the entire underground opening [4]. Among the first, the investigation of the damage zone around the excavation profile using the cross-hole method was carried out in the gallery tunnels during the construction of the Martinje Dam [5]. Determining the damage zone around the excavation profile by blasting using the cross-hole method was carried out during the construction of the "Mali Prolog" tunnel, which is located on the A1 motorway on the section of the connection road between the junction "Ploče" and CP "Karamatići". The total length of

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the right tunnel tube is 1092.00 m, while the length of the underground excavation is 1074.00 m. The maximum upper layer of the right tunnel tube is approximately 155 m. The total length of the left tunnel tube is 1029.00 m, and the length of the underground excavation is 1011.00 m. m. The maximum upper layer of the left tunnel tube is approx. 140 m [6]. The spatial location of the Mali Prolog tunnel is shown in Figure 1.

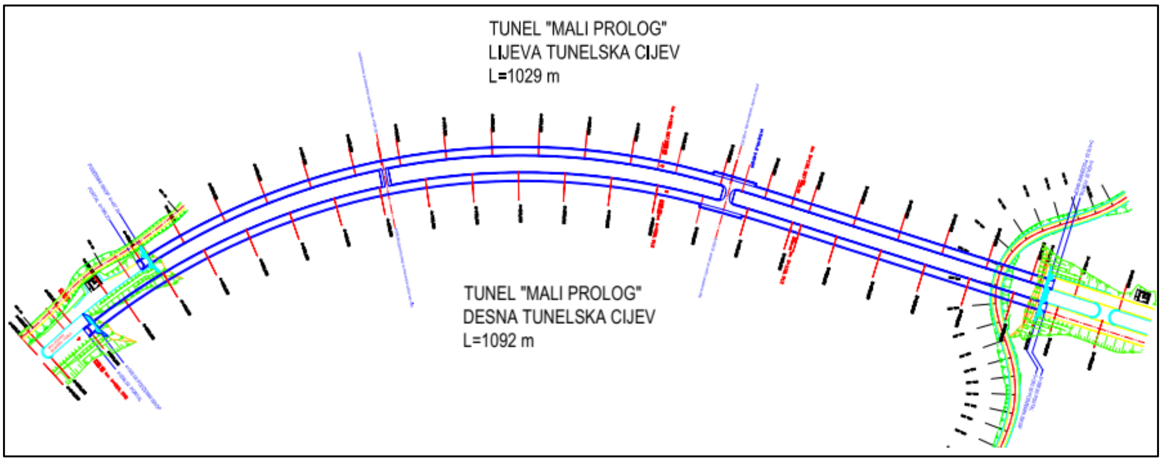

Figure 1 Mali Prolog tunnel [6]

2. Material and methods

2.1. Engineering-geological characteristics of the rock massif

The "Mali Prolog" tunnel is located in a rock massif built by Eocene foraminiferal limestones (V, E1,2) and Upper Cretaceous rudist limestones (V, K2). Rudist limestones make up the central and northern parts, or two thirds of the rock massif through which the tunnel is laid, while foraminiferal limestones comprise the southern area of the tunnel as shown in Figure 2.

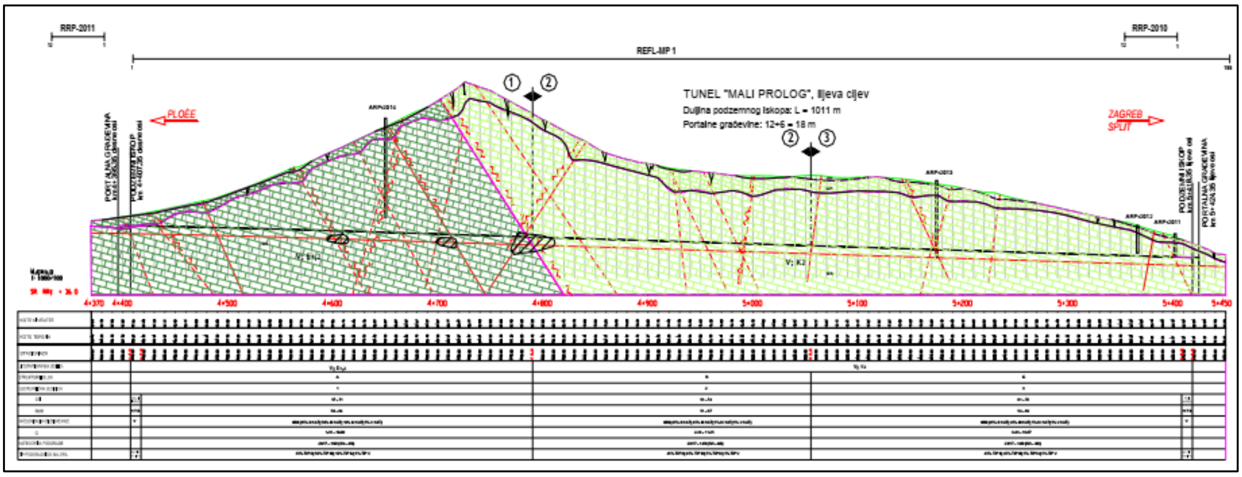

Figure 2 Engineering-geological longitudinal forecast profile of the left tunnel tube [6]

Investigations have determined the two lithostratigraphic units mentioned above, which are separated from each other by a reverse fault that runs approximately perpendicular to the tunnel route at km 4+460 (right tunnel tube) and km 4+750 (left tunnel tube). Considering the engineering-geological characteristics, the investigated rock massif is divided into three structural blocks A, B and C, which, in combination with lithostratigraphic units, describe three geotechnical units, namely:

Geotechnical unit 1 (chainage of the left tunnel tube 4+407 – 4+790; chainage of the right tunnel tube 4+308 – 4+755) lithostratigraphic unit I, foraminiferal limestones V; E1,2, structural block A.

Geotechnical unit 2 (chainage of the left tunnel tube 4+790 – 5+055; chainage of the right tunnel tube 4+755 – 5+000) lithostratigraphic unit II, rudistic limestones V; K2, structural block B.

Geotechnical unit 3 (chainage of the left tunnel tube 5+055 – 5+418; chainage of the right tunnel tube 5+000 – 5+382) lithostratigraphic unit III, rudistic limestones V; K2, structural block C.

2.2. Description of the blasting area and blasting

From the northern undercut, the excavation of both tunnel tubes was started in parallel in the full profile of the tunnel, and the excavation area varied from 71.20 – 73.46 m^2, depending on the category of the rock massif. For the excavation of the "Mali Prolog" tunnel, the application of the drilling and blasting technique with the same drilling and connection scheme for all categories of rock material was foreseen, as shown in Figure 3. Depending on the category of rock material, the projected drilling depth varied from 2.2 – 4.5 m, as well as the construction of blasting boreholes, i.e. the mass of explosives by degree of ignition.

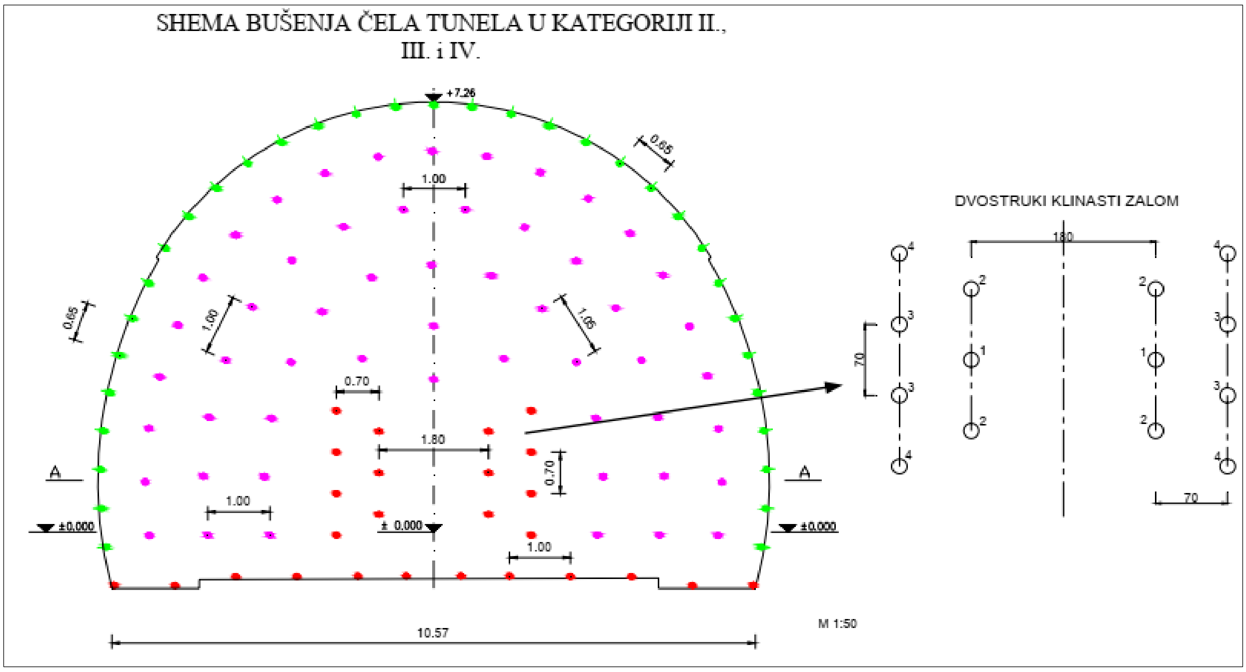

Figure 3 Scheme of the arrangement of blasting boreholes for all categories of rock material

The research was carried out on the section of the tunnel in II. category of rock massif, in which the step of advancement, ie drilling and blasting, was reduced to 3 m. The reason for such a shortened step of advancement is the tendency to reduce overbreaking excavation. The selected section of the tunnel where the research was carried out was 13.5 m long, and 5 cycles of drilling and blasting were recorded. To create a new free surface, a double wedge fracture with 14 boreholes and the drilling geometry shown in Figure 3 was used. The total number of boreholes at the head was from 106 to 123 with an explosive consumption per blasting cycle of 229 to 300 kg. Two types of explosives were used to fill mine boreholes with a diameter of 45 mm: Elexit-1 and Perunit, table 1.

Table 1 Technical characteristics of the explosives used in the "Mali Prolog" tunnel

Technical characteristics	Diameter (m)	Density (kg/dm^3)	Detonation speed (m/s)	Gas volume (l/kg)	Explosion energy (MJ/kg)
Elexit-1	0.038	1.40	5300-5800	851	4545
Perunit	0.038	1.38	6200	861	4627

Figure 4 shows the applied constructions of blasting boreholes for a drilling depth of 3 m and filled with Elexit-1 explosives.

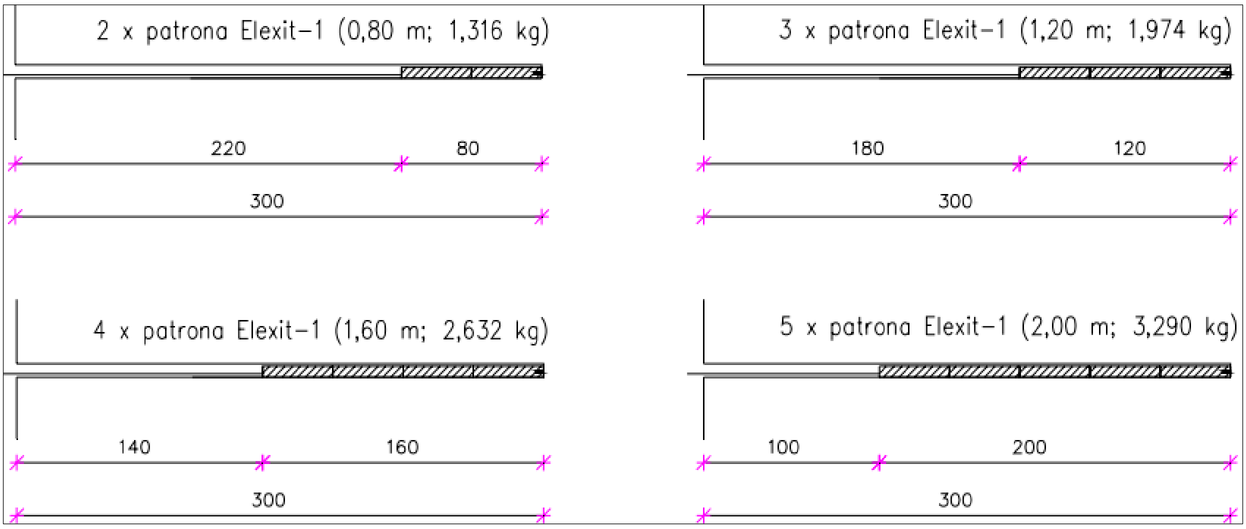

Figure 4 Construction of filled boreholes for a drilling depth of 3.0 m

In the "Mali Prolog" tunnel, detonators of the PRIMADET - LP type with a delay time were used:

- 100 ms - detonators between numbers 1 -10
- 200 ms - detonators number 12, 14, 16, 18 i 20
- 500 ms - detonators number 25, 30, 35, 40, 45, 50, 55 i 60
- 1000 ms - detonators number 70, 80 i 90

Figure 5 shows the geometry of blasting boreholes and the arrangement of detonators at the head of the tunnel excavation.

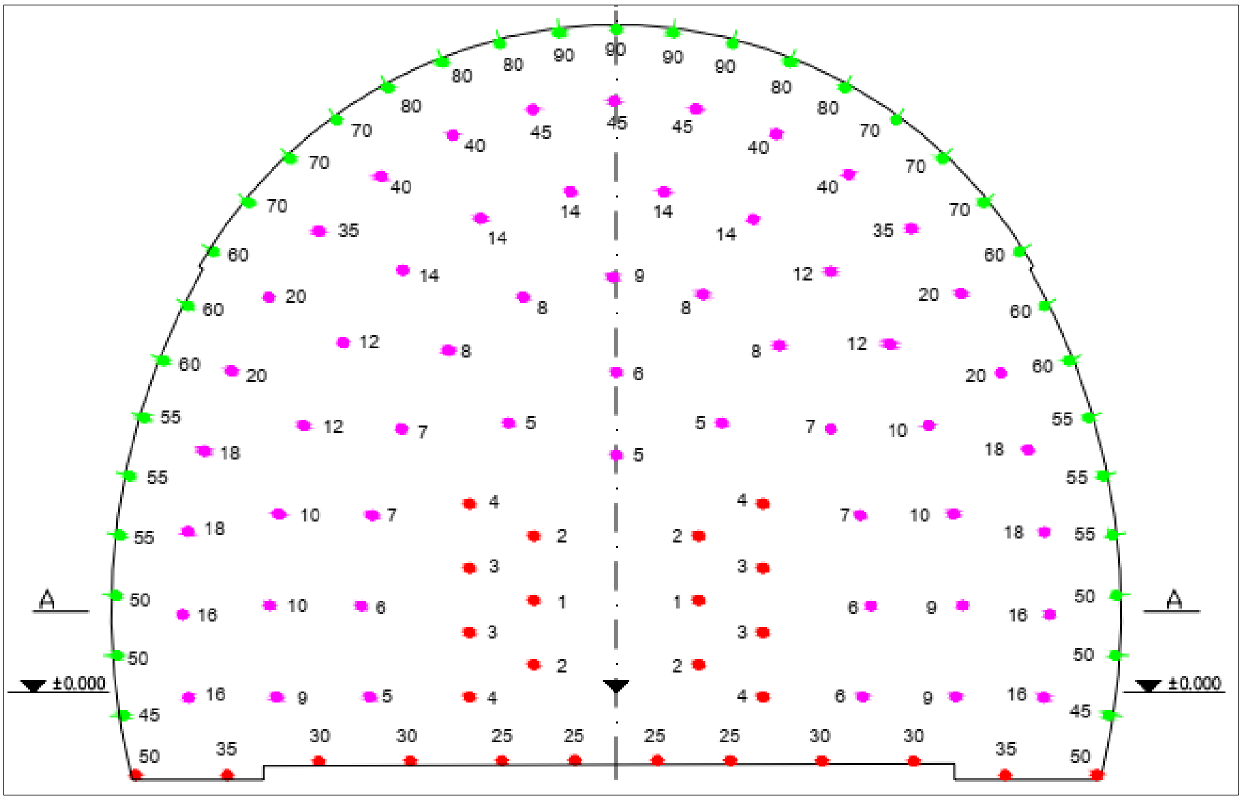

Figure 5 Scheme of initiation of blasting boreholes at the head of the tunnel for all categories of rock material

2.3. Measurement of peak velocities of oscillations in relation to the blasting area

During the excavation in the left tube of the "Mali Prolog" tunnel, where the depth of the damage zone around the tunnel profile due to blasting was investigated, it was planned to measure the peak velocities of oscillations at two locations perpendicular to the blasting site.

Figure 6 Location of observation site and measurement of peak oscillation speeds in relation to the blasting area

On the pre-determined section of the left tunnel tube, where the depth of the damage zone around the tunnel profile was investigated, five blasting cycles with a drilling depth of three meters were carried out. For each blasting cycle, the peak velocities of the oscillations were measured and the blasting parameters were recorded. The observation location (MO-1) for each blasting was vertically above the tunnel face on the ground surface at known distances from the blasting location. The distance of the observation site from the blasting site was from 21.34 to 24.16 m, and at the same time it represented the upper layer which increased with the increase of the excavation, i.e. the penetration of the tunnel into the rock massif.

Table 2 shows the results of the peak velocities of oscillations at the observation point MO-1 with the distances from the blasting site and the associated blasting parameters.

Table 2 The results of the peak velocities of oscillations at the observation point MO-1 with the distances from the blasting site and the associated blasting parameters.

Excavation face chainage	Number of boreholes	Blastingfield drilling depth	Total mass of explosives	Max. mass of explosives per degree of ignition	Specific consumption of explosives	Peak oscillation speed	The distance of the observation site from the blastingfield
(m)	(kom)	(m')	(kg)	(kg)	(kg/m^3)	(mm/s)	(m)
5+393.2	112	2.5	175	12.50	0.98	32.2	21.34
5+391.7	123	3.0	296	16.21	1.38	48.8	22.00
5+388.7	120	3.0	229	15.45	1.07	49.9	22.67
5+385.7	120	3.2	300	17.76	1.40	56.9	23.41
5+382.7	106	3.0	255	18.42	1.27	68.2	24.16

2.4. Investigation of the depth of the rock mass damage zone around the excavation profile by seismic cross-hole tomography

Before the start of tunnel excavation, five investigation boreholes were drilled from the ground surface above the left tunnel tube at the stake positions. The spatial position of the boreholes, as seen in Figure 7, includes the volume of the rock massif in the vertical sense from the ground surface to 5 m below the level of the tunnel, and in the horizontal sense 5.35 m from the theoretical excavation line of the sides of the tunnel.

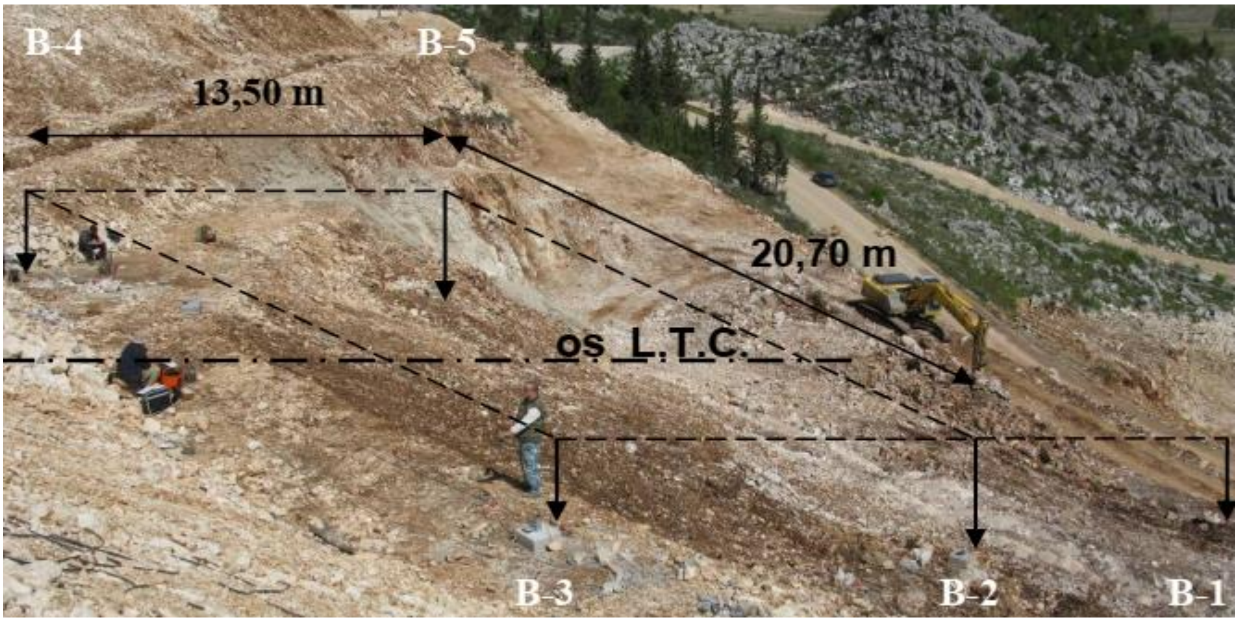

Figure 7 Spatial location of investigation boreholes on the surface of the terrain

The length of the boreholes depended on the elevation of the terrain, and ranged between 24-36 m, as can be seen from the longitudinal section of the tunnel in Figure 8.

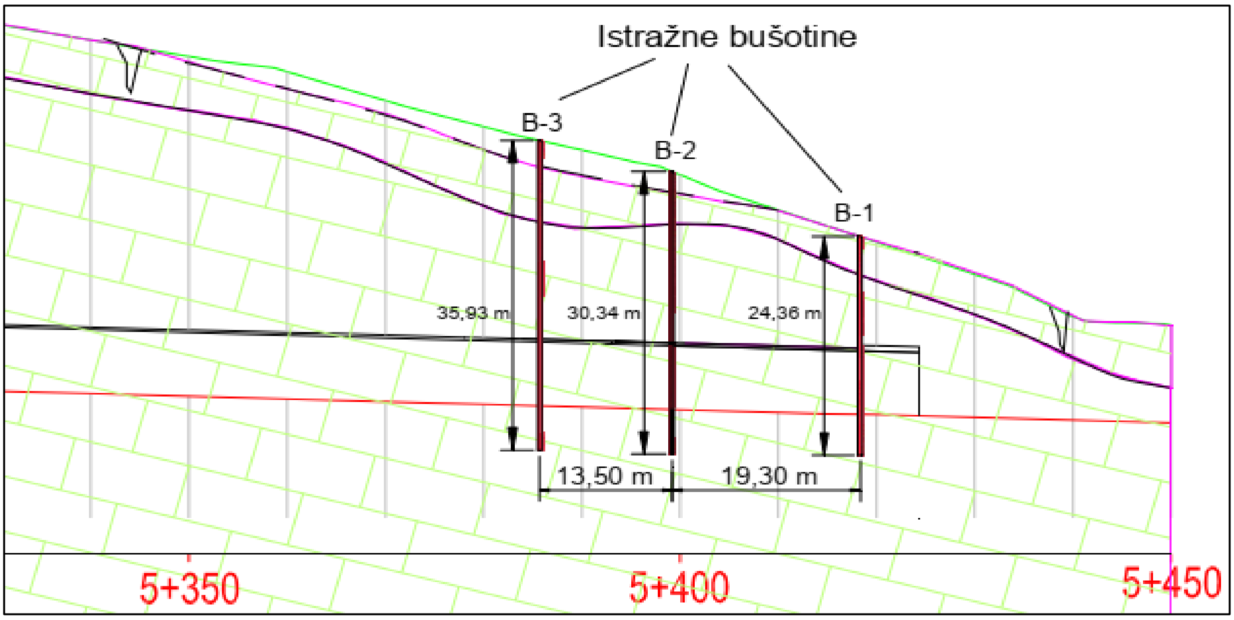

Figure 8 Longitudinal cross-section of the tunnel at the research site with investigation boreholes B-1, B-2 and B-3 drawn in the left side of the tunnel

Table 3 shows the positions of the exploratory wells that were drilled on the surface of the terrain above the left tunnel tube.

Tabel 3 The positions of the investigation bore

Borehole mark	Chainage	Distance from the beginning of the underground excavation (m)	Overlay (m)	Drilling depth of the investigation boreholes - 5m below the level of the tunnel (m)
B - 1	5+418.35	0.00	12.55	24.36
B – 2	5+399.04	19.30	19.21	30.34
B – 3	5+385.70	32.86	22.36	35.93
B – 4	5+385.93	32.41	22.36	35.56
B - 5	5+399.42	18.92	19.21	32.59

Tests of the rock massif were carried out using seismic methods, namely shallow refraction seismic and seismic *cross-hole* tomography as a non-destructive method [7], and based on the value of the speed of propagation of P and S waves, the wear, fracture and width of the crack systems of the rock massif were measured. Due to its practicality, the cross-hole tomography method has been widely used in many economic branches, including tunnel construction [8]. The tests were carried out in two phases:

- before the start of excavation of the left tunnel tube - "zero" state,
- after the excavation of the tunnel of the left tunnel tube using the blasting technique.

With this approach, the rock massif was examined in detail before and after the excavation of the tunnel using the same measurement methods and the positions of the investigation boreholes, except for the flanks of the tunnel, which were examined with modified cross-hole tomography adapted to the excavated opening.

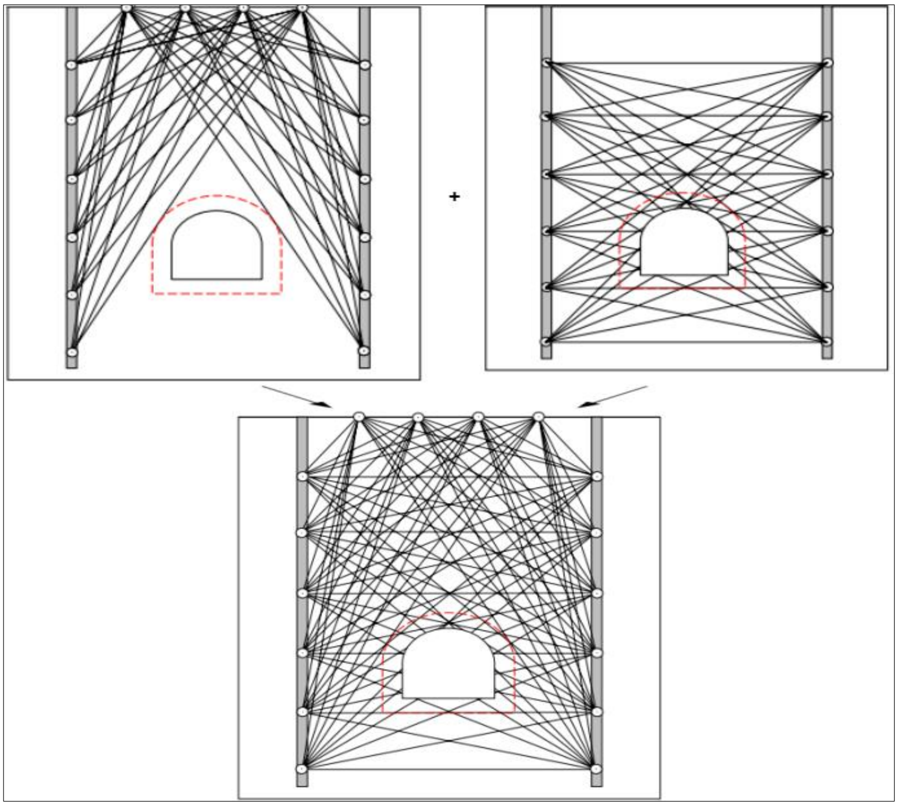

Figure 9 Schematic representation of measurements by seismic methods in the zone between the boreholes before the excavation of the left tunnel tube of the "Mali Prolog" tunnel [9 and 10]

Before the start of the tunnel excavation, the rock massif was investigated using the methods of shallow refraction seismic and seismic cross-hole tomography, Figure 9.

All measurements were made in the left tunnel tube of the "Mali Prolog" tunnel on the section from chainage 5+418.35 to 5+385.70. Between the mentioned chainages, two measuring profiles were made between the investigation boreholes B2_B5 and B3_B4. Figure 10 shows a comparative view of the measurement scheme and distribution of P-wave propagation speed on profiles B2_B4 and B3_B4.

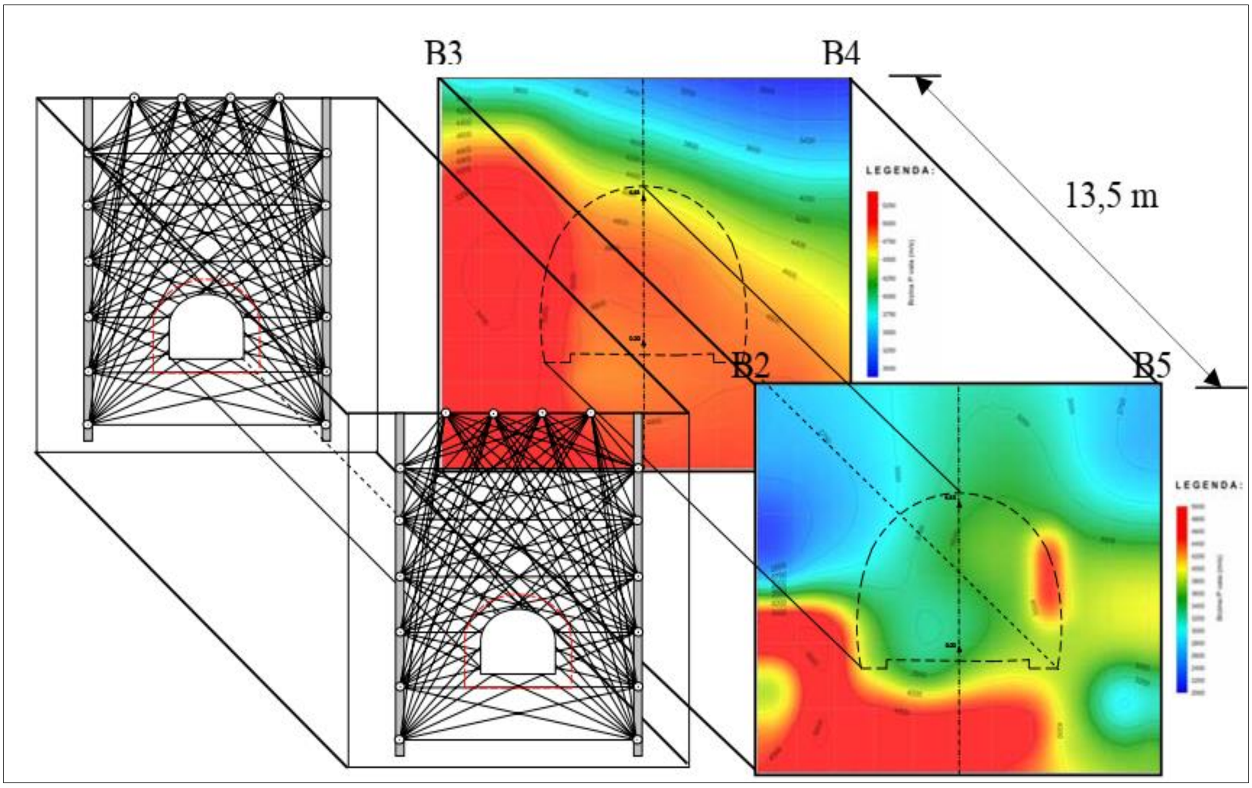

Figure 10 Comparative view of the scheme of measurement and distribution of P-wave propagation speed on profiles B2_B4 and B3_B4 before tunnel excavation

Figure 11 Drilling boreholes for installing anchors and initiating signals on the anchor [9]

After the excavation of the tunnel, investigations were carried out of the damage zone of the rock massif in the section between profiles B2_B5 and B3_B4. The seismic cross-hole tomography method and its modification were used in the investigation of the damage zone of the rock massif. Before the beginning of the measurements, the stations of profiles B2_B5 and B3_B4 were geodetic transferred from the surface of the field to the tunnel profile. On the research section in the tunnel, a profile was selected from among the transferred profiles from the surface of the terrain, Figure 11.

On the selected and geodetic recorded profile, the conditions were prepared for the application of measurements using the modified method of seismic cross-hole tomography. This type of modification was applied transversely to the tunnel opening.

Figure 12 Anchor before installation, after installation and during measurement [9]

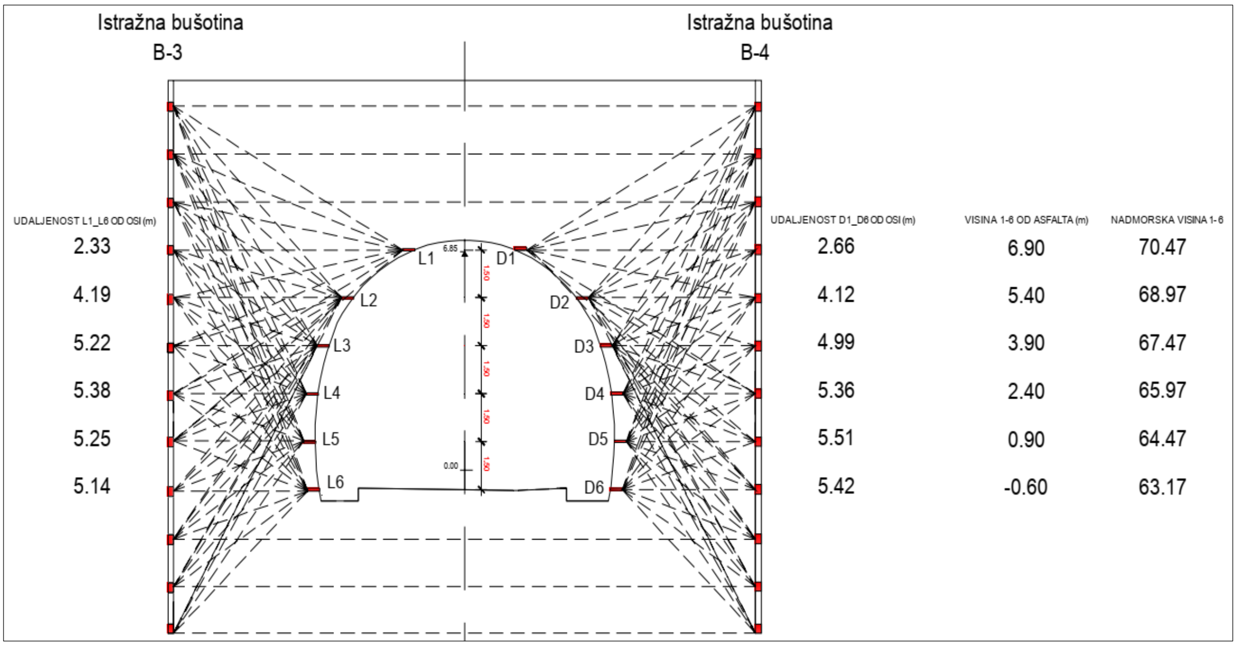

Figure 13 Display of the measurement scheme between the tunnel contour and monitoring boreholes with measured and calculated distances between the initiation site and monitoring boreholes

Preliminary work for the application of this method was geodetic marking, drilling and installation of short anchors along the tunnel excavation contour. The embedded anchors along the contour of the excavation served as points of signal initiation towards the investigation boreholes B3 and B4 in which the geophones were located. On the left and right side of the contour of the tunnel excavation, six anchors were installed on a grid of 1.5 m with the markings of the initiation locations L1_L6 and D1_D6. For each point of initiation along the contour of the tunnel, the speed of propagation of P and S waves was measured every meter from listening boreholes from a depth of 15-36 m. For the best possible signal reception in the listening boreholes, a device was used to attach a 3D geophone to the wall of the borehole. Each initiation point on the excavation contour was geodetically recorded to calculate the distance to the listening borehole, as shown in Figure 13. The measurement procedure includes the controlled excitation of elastic waves with a hammer over the anchor along the profile contour for signal transmission and precise measurement of the time required for the wave to travel to the three-component geophone (3D) in the listening borehole.

Figure 14 shows the results of measuring the distribution of the propagation speed of P and S waves from the contour of the excavation to the listening boreholes.

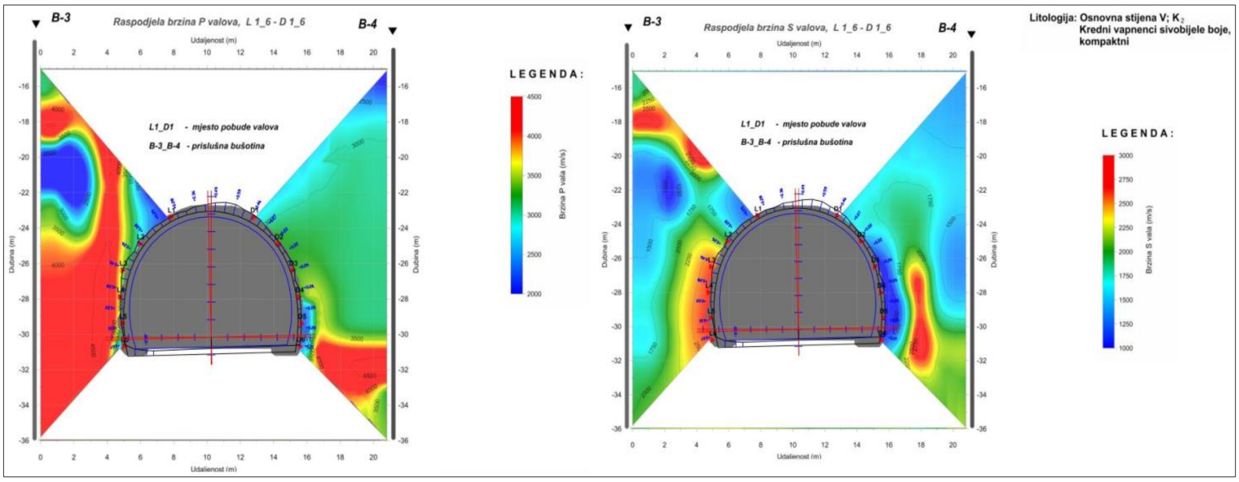

Figure 14 Presentation of the measurement results of the distribution of the speed of propagation of P and S waves between the sides of the tunnel and listening boreholes

The processing of measurement results of the *cross-hole* method includes:

- correction of the arrival time of longitudinal and transverse waves,
- calculation of longitudinal and transverse wave speeds,
- determination of dynamic modules from elastic wave speeds.

When an excitation occurs in an elastic material, waves are immediately generated starting from the excitation point, and they progress with a decrease in the amplitude of the disturbance oscillations. Each of these waves behaves in a characteristic way, and these characteristics depend on the elastic properties of the material. The obtained and processed velocities of elastic waves, which depend on the elastic properties of the material through which the wave propagates, which is shown in the equations for the propagation velocity of P and S waves, are used for the calculation of dynamic elasticity constants:

$$v_p = \sqrt{\frac{K+\frac{4G}{3}}{\rho}} = \sqrt{\frac{E(1-v)}{\rho(1+v)(1-2v)}} \quad (1)$$

$$v_s = \sqrt{\frac{G}{\rho}} = \sqrt{\frac{E}{2\rho(1+v)}} \quad (2)$$

where:

vp - speed of primary (longitudinal) P-waves (m/s),
vs - speed of secondary (transverse) S-waves (m/s),
ρ - density (10^3 kg/m^3),
E - Young's modulus (GPa),
G - shear modulus (GPa),
K - volume compressibility modulus (GPa).

Knowing the propagation speed of elastic waves through the rock mass and the density, it is possible by solving two equations with two unknowns to calculate two dynamic constants that can be used to determine, or calculate, the others as well.

$$a = \frac{v_p}{v_s} = \sqrt{\frac{2-2v}{1-2v}} \tag{3}$$

$$v = \frac{\left(\frac{v_p}{v_s}\right)^2 - 2}{2\left\{\left(\frac{v_p}{v_s}\right)^2 - 1\right\}} = \frac{a^2-2}{2(a^2-1)} \tag{4}$$

$$E = \rho v_p^2 \left(\frac{(1+v)(1-2v)}{1-v}\right) \tag{5}$$

$$G = \rho v_s^2 \qquad K = \rho v_p^2 \frac{1+v}{3(1-v)} \tag{6}$$

where:

v - Poisson's ratio,
ρ - rock density. (kg/m^3).

Using the above-mentioned formulas, the dynamic constants of elasticity were calculated for each measurement position, and the density was calculated from the value of the longitudinal (vp) wave speed according to the well-known Ansty relation shown in equation no.7.

$$\rho = 0{,}31\sqrt[4]{v_p} \tag{7}$$

3. Results and discussion

Methods of investigating the depth of the blasting damage zone in the "Mali Prolog" tunnel were carried out before and after the excavation. Research methods before excavation included seismic *cross-hole* tomography, shallow refraction seismic, and after excavation, modified seismic cross-hole tomography was applied. On the *cross-section* of the tunnel, at chainage 5+385, extensive research was carried out using the method of modified seismic *cross-hole* tomography. Interpretation and systematization of the results for research depths of 24, 26, 28 and 30 m from the surface of the terrain in Figure 15, a summary diagrammatic representation of the dynamic elasticity constants around the profile of the tunnel excavation is given.

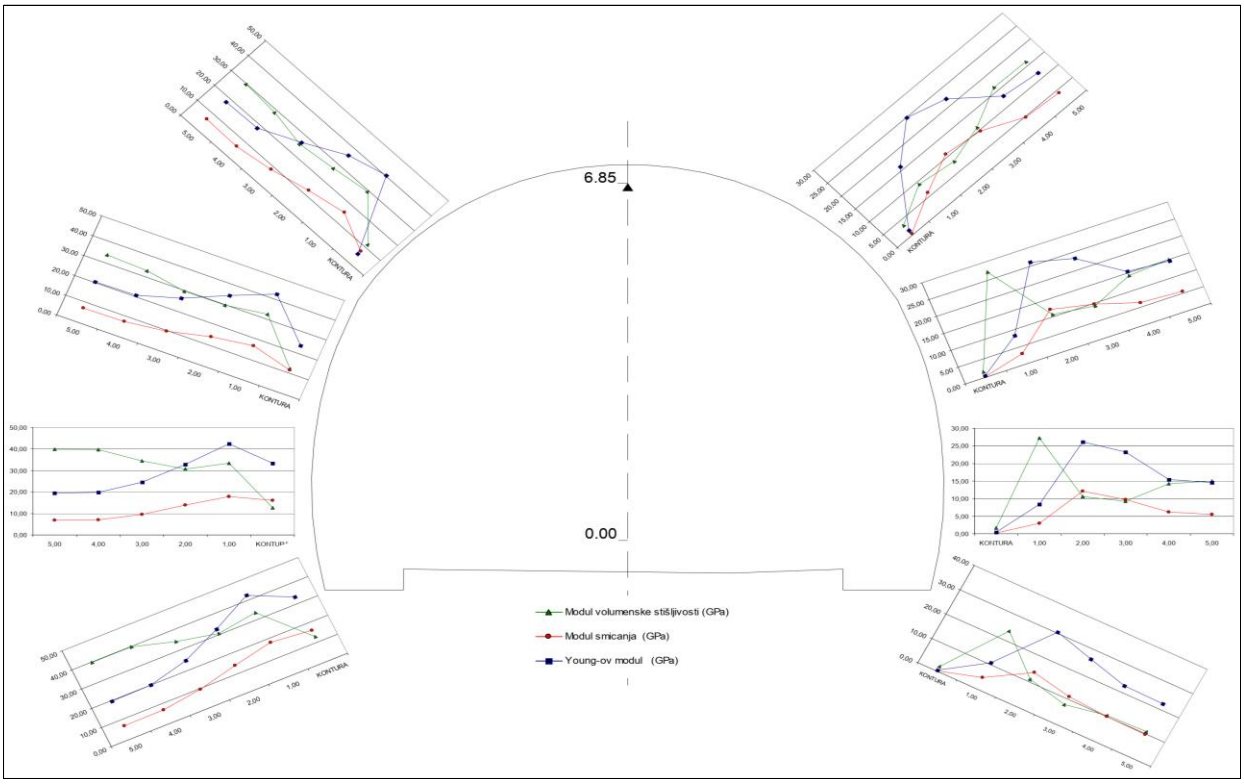

Figure 15 Diagrammatic representation of the dynamic constants of elasticity of the rock mass around the "Mali Prolog" tunnel excavation profile

At the depths of the research, the following depths of the rock mass damage zone by blasting in the "Mali Prolog" tunnel were registered:

- depth 24 m: - left side of profile: 0.92 m, of which overbreak excavation is 0.30 m - right side of profile: 1.92 m, of which overbreak excavation is 0.30 m
- depth 26 m: - left side of the profile: 0.75 m, of which the overbreak excavation is 0.40 m - right side of the profile: 0.90 m, of which the overbreak excavation is 0.20 m
- depth 28 m: - left side of the profile: 0.22 m, of which the overbreak excavation is 0.22 m - right side of the profile: 1.55 m, of which the overbreak excavation is 0.20 m
- depth 30 m: - left side of the profile: 0.10 m, of which the overbreak excavation is 0.10 m - right side of the profile: 1.79 m, of which the overbreak excavation is 0.20 m

The measuring profile was in the position between two blasting cycles (the end of one cycle and the beginning of the other), where the maximum mass of explosives per ignition stage was from 17.76 to 18.42 kg at a drilling depth of 3.2 to 3.0 m and in progress from 3.0 to 2.8 m.

4. Conclusion

With the construction of underground infrastructure, there is a need to carry out careful blasting, that is, blasting that, on the one hand, must break the rock, and on the other hand, damage the rock mass as little as possible and preserve its physical and mechanical properties outside the blasting zone. The concept of the New Austrian Tunneling Method (NATM) is based on preserving as much as possible the integrity of the surrounding rock mass outside the excavation line, from which the concept of this method is based on the rock around the underground opening, which becomes the main load-bearing component through the activation of the load-bearing "ring" around the underground opening. Failure to preserve the load-bearing "ring" leads to an increase in damage and a decrease in the physical and mechanical characteristics of the rock mass around the profile of the tunnel excavation, as a result of which the unsupported opening should be brought to a state of stability with additional supporting elements. By applying seismic methods, changes in the speed of propagation of longitudinal and transverse waves were measured and the values of the dynamic constants of elasticity of the rock mass were calculated. In this way, the demarcation of the damaged and undamaged zone was determined by the depth of the rock massif. Based on the comprehensive results of the investigation of the depth of the damage zone, the following can be concluded:

- When excavating tunnels using conventional blasting methods in II. category of rock mass, the depth of the damage zone can extend up to approximately 3 m into the rock massif, while when choosing the appropriate drilling depth of blasting boreholes and associated blasting parameters, the depth of the damage zone is reduced to approximately 2 m into the rock massif from the theoretical excavation line.
- By choosing the optimal excavation method, damage zones, which are inevitable during blasting excavation, can be minimized in the sense that the strength and stiffness of the rock mass around the contour of the excavation is minimally reduced, which has the effect of preserving the bearing capacity of the rock mass as the most important "supporting" element. Thus, excessive damage to the contour of the excavation, which manifests itself in the form of an increased overbreak and depth of the damage zone, increases the costs of the primary support of the contour of the excavation compared to the standard supporting system provided for the respective category of rock massif.

Compliance with ethical standards

Disclosure of conflict of interest

No conflict of interest to be disclosed.

References

[1] Antičević H., Dobrilović M., Perković H., Zone of damage to the rock mass during tunnel excavation by blasting, Mining-geological-petroleum collection, Original scientific work, Zagreb, 2012.

[2] Antičević, H., Determining the zone of rock mass damage during tunnel excavation by blasting, Doctoral thesis, University of Zagreb Faculty of Mining, Geology, and Petroleum, Zagreb, 2011.

[3] E. Bektaševič, H. Antičević, F. Osmanović, S. Kadrić, Đ. Žutić, J. Konta, Blasting technology during the excavation of the evacuation tunnel "Vranduk II" on the main road M-17, Electronic collection of works, Faculty of Civil Engineering, University of Mostar (Scientific paper), (page 49-60), (ISSN 2232-9080 (Online) i DOI: 10.47960/2232-9080), Editor-in-chief: dr.sc. Mirna Raič.

[4] E. Bektaševič, R. Kadrić, K. Gutić, S. Kadrić, Determining the optimal method of excavation of the Zenica tunnel as a function of minimal damage to rock mass of poorer quality outside the excavation profile, Journal of faculty of mining, geology and civil engineering, University of Tuzla (Scientific paper), (page 09-23), (ISSN 2303-5161 (Online) and DOI 10.51558/2303-5161.2023.11.11.9), Editor-in-chief: Prof.dr.sc. Sunčica Mašić.

[5] Kujundžić, B., Jovanović, L. & Radosavljević, Z. (1970): Solution du revêtement d'une galerie en charge par applicationd' injections à haute pression. Proc. of 2nd ISRM Congress, Belgrade. II: Theme 4–66. Belgrade.

[6] Mustapić, I.: Tunnel construction project „Mali Prolog", Institut IGH d.d., Zagreb, 2010.

[7] Dong, L.; Chen, J.; Song, D.; Wang, C.; Liu, X.; Liu, M.; Wang, E. Application of Long-Range Cross-Hole Acoustic Wave Detection Technology in Geotechnical Engineering Detection: Case Studies of Tunnel-Surrounding Rock, Foundation and Subgrade. Sustainability 2022, 14, 16947. https://doi.org/10.3390/ su142416947.

[8] Cheng, F.; Peng, D.; Yang, S. 3D Reverse-Time Migration Imaging for Multiple Cross-Hole Research and Multiple Sensor Settings of Cross-Hole Seismic Exploration. Sensors 2024, 24, 815. https://doi.org/ 10.3390/s24030815.

[9] Mitrović, G. (2010): The procedure for measuring the speed of propagation of elastic waves between boreholes - using the crosshole method. Institut IGH d.d., Institute of Geotechnics, Zagreb, page 1-10.

[10] Mitrović, G. (2010): Shallow refraction seismic. Institut IGH d.d., Institute of Geotechnics, Zagreb, page 1-10.