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Verification of underground excavation stability during tunnel construction

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Abstract

Design of tunnels in rock mass according to the New Austrian Method (NATM), using the results of geotechnical field investigations and rock mass classifications (GSI, RMR, Q), is most often carried out in the early design stage. Verification of underground opening stability using observational methods is carried out by measuring the behaviour of the reinforced rock mass during the tunnel construction. This paper will present the verification of underground excavation stability in the Mali Prolog and Šubir tunnels on the A1 Highway Zagreb-Split-Dubrovnik, near the town of Vrgorac, Croatia. During the tunnel excavation, field investigations and observations were carried out to verify the efficiency of the primary tunnel support and the length of the installed rockbolts. During the tunnel construction, investigation drilling at the tunnel face, geophysical seismic measurements, video-prospecting of investigation boreholes, geological mapping of the tunnel face, convergence measurements, and anchor pull-out tests were carried out. The investigations were carried out in the II, III, and IV categories of rock mass (according to the RMR classification), in which excavation by drilling and blasting was envisaged. Multi-layer reinforced shotcrete and rockbolts were used as the primary tunnel support. Based on measurements and observations, it was necessary to confirm the achieved stability of the underground excavation. If the behaviour, or rather the stability of the underground excavation, significantly deviated from the design criteria, the input parameters of the rock mass were corrected, and the analysis was repeated based on test results and back-analyses. Changes in the support system resulted in additional gradual strengthening or, in some cases, reduction of the support elements of the primary support system.

Keywords

Tunnelling, Observational method, Stability verification, Observation, Support system.

1 Introduction

During the construction of numerous roads in Croatia from 2003 to the present, numerous road tunnels have been constructed in the Croatian coastal part of the Dinaric Alps, mostly in carbonate deposits (limestone, dolomites), and less frequently in some other deposits (flysch). Given the large number of tunnels that were built in a relatively short time, common procedures have been developed in exploratory works, design of tunnel excavation and support, and execution of works, and ensuring the stability of the support system based on the New Austrian Tunnelling Method (NATM) (Rabcewicz 1964; Rabcewicz and Golser 1973) and the observational method (Peck 1969; Terzaghi and Peck 1967; Nicholson et al. 1999) with an active design approach. In the initial stages of design, the results of the conducted exploratory works enable the classification of the rock mass using the geotechnical classification (RMR) (Bieniawski 1989), the Geological Strength Index (GSI) (Marinos and Hoek 2000), and, less frequently, the Q-classification (Barton et al. 1974), and the determination of rock mass categories according to the RMR classification and empirical values of strength and deformability of the rock mass. Based on the determined characteristics of the wall mass, the spreading zone, and the span of the underground opening, typical primary support systems were determined for certain estimated rock mass categories (Bieniawski 1989). In the execution phase of the works, verification of the stability of the underground opening was carried out using observational methods, carried out by classifying and measuring the behaviour of the reinforced rock mass during the execution of the works. Measurements were also carried out to reduce the impact of drilling and blasting on the rock mass around the designed underground opening. In fact, the active design phase continued already at the tunnel excavation head, and on the test sections of the tunnel excavation in the II, III, and IV categories of rock mass, the impact of drilling and blasting outside the designed tunnel excavation line on the damage to the rock mass was monitored. Measurements were carried out to identify the depth of damage to the rock mass outside the excavation profile, including geophysical measurements, investigation drilling, RQD, and video-prospecting of investigation boreholes at the tunnel excavation head. Standard geotechnical measurements included: monitoring of deformations (convergence) of the excavation profile, subgrade measurements, and formation of the plasticisation zone through anchor pull-out tests.

This paper will present the verification of the stability of the underground opening and some of the results of measurements and observations on the Mali Prolog and Šubir tunnels on the A1 Zagreb-Split-Ploče Highway, near the town of Vrgorac, Croatia, during their construction from 2007 to 2012.

2 Verification methodology and research implementation

The paper will continue to describe the geological conditions and construction features of the Mali Prolog and Šubir tunnels on the A1 Zagreb-Split-Ploče Highway, near the town of Vrgorac, Croatia. Given the similar geological structure, the height of the tunnel overburden, and the tunnel cross-sections, the experience gained during the construction and verification of the stability of the underground tunnel opening enabled the application of the methods under similar geotechnical conditions.

2.1 Geological setting in the tunnels area

The Mali Prolog Tunnel is a two-tube tunnel, 1,092 m long, with a maximum overburden of 170 m. It is located within a rock massif composed of Eocene foraminiferal limestone (V, $E_{1,2}$) and Upper Cretaceous rudist limestone (V, K_2) (Užarević 2011). Rudist limestone compose the central and northern part, or two-thirds of the rock length of the tunnel, while foraminiferal limestones cover the southern area of the tunnel.

The Šubir Tunnel is a two-tube tunnel, 962 m long with a maximum overburden of 120 m. Šubir is mostly composed of Paleocene–Middle and Upper Eocene foraminiferal limestone (V, Pc-$E_{1,2}$) and Upper Cretaceous rudist limestone (Užarević 2010). In part of the tunnel, there are deposits of Middle and Upper Eocene flysch (F(L); $E_{2,3}$) in which blue-grey marls predominate, which are very fractured in contact with foraminiferal limestone.

Both tunnels have the same horseshoe-shaped cross-section, 10.65 m wide at the bottom and 8.25 m high, with an approximate area of 72.35 m^2, Fig 1a. In both tunnels, the excavation was carried out in the II, III, and IV categories of rock mass according to the RMR classification. For each of the excavation categories, the project defined primary support systems, and the selection of the support

system depended on the mapping of the face, walls, and vault of the excavation and the determination of the category of rock mass in the tunnel.

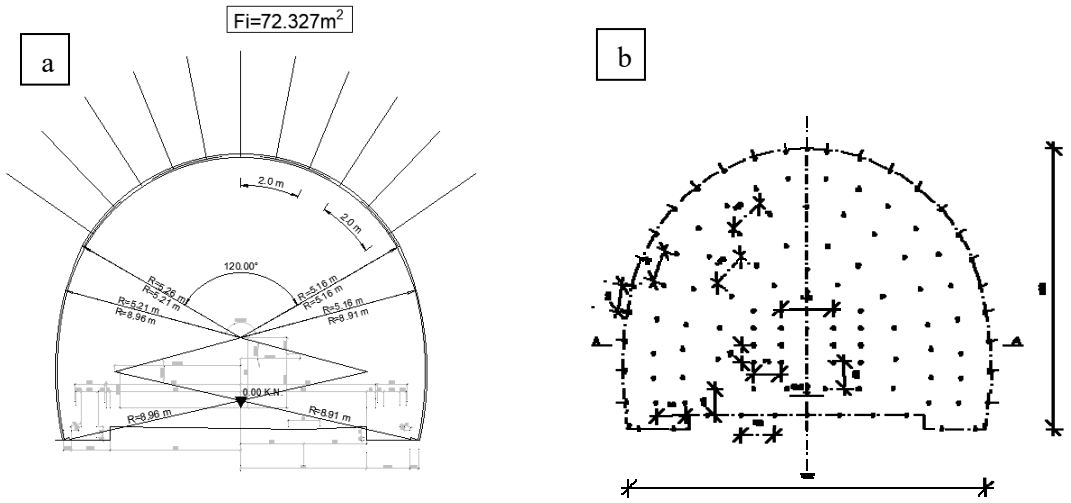


Fig. 1 Cross section of the Šubir and Mali Prolog Tunnels. a) Geometry of the cross section with schematic view of III category primary support system, b) Schematic view of the mining borehole positions at the face of the tunnel.

2.2 Excavation methodology

For the excavation of tunnels in II, III, and IV categories of rock mass, the application of drilling and blasting techniques was envisaged, and the excavation area varied from 71.20–73.46 m^2, and the advancement from 1.0 to 1.5 m (Užarević 2010, 2011). Depending on the category of rock mass, the drilling depth of the mining wells varied, as did the construction of the mining wells, i.e., the mass of explosives per individual ignition stage. For the excavation of each section, the total number of wells at the face was from 106 to 124, with a drilling diameter of 45 mm and a depth of 1.0 to 1.5 m, Fig. 1b.

At the beginning of the tunnel's excavation, test sections were selected for confirmation or possible corrections of the designed drilling and blasting scheme, and at the same time, extended research was carried out on the mentioned sections to verify the stability of the underground excavation.

2.3 Over-profile excavation

Before the start of tunnel excavation, the designed drilling and blasting scheme was tested and confirmed or refined. The drilling of mine boreholes was done by computer and manually. In order to ensure the designed profile of the tunnel, the contour row of mine boreholes is drilled at a minimum angle related to the axis of the tunnel. This angle cannot be smaller than that allowed by the approach of the drilling machine carriage to the already excavated tunnel profile, and this is a technological over profile that cannot be avoided. The drilling machine carriage must be positioned in the over profile of the previous step of blasting so that the drilling angle of the contour row of mine boreholes from the designed excavation contour would be as small as possible, yet sufficient to create an over profile by blasting, that is, space for positioning the drilling machine carriage for the next blasting.

In addition to the technological over profile, excessive over profile due to excessive damage to the rock mass, due to careless drilling and blasting, is a very common occurrence. Sometimes, a certain excessive over profile is a more acceptable cost to the Contractor than a sub profile that must later be mechanically excavated, and mechanical excavation of profile finishing can also cause an over profile excavation. In both cases, unnecessary financial costs and disruption of work dynamics are created.

At the beginning of the tunnel excavation, drilling and blasting were monitored, and the over profile excavation was geodetically recorded to define the technological over profile. The average value of which is usually approximately 25 cm from the designed excavation line. A justified geological over profile is acceptable in the construction costs.

Over profile excavation, apart from its technological significance and cost impact, has a significant impact on the stability of the underground opening. Firstly, it is a real extension of the tunnel span, which is rarely taken into account in stability analyses. Furthermore, it represents additional damage to the rock mass outside the design profile of the tunnel, and can have consequences in reducing stability and increasing deformations in the rock mass.

2.4 Geodetic survey

The part of the verification procedure the stability of the tunnels includes geodetic measurements due to over profile excavation and rock mass damage at the excavation contour, and the need for additional reinforcement of the support lining.

During the excavation of the tunnels, control measurements of the displacements (convergence) were performed at the contour of the tunnel profile. The measuring profiles were installed every 50 to 100 m, and each measuring profile has five measuring points; one in the crown of the tunnel and two on each of the tunnel walls. The frequency of measurements was every day in the first seven days after excavation, and the construction of the primary support. After 7 days, measurements were carried out once a week and after the displacement had subsided, once every six months. The measured values were up to 2 mm, which met the requirements of the Geotechnical Design of the tunnels (Užarević 2010, 2011).

2.5 Engineering geological mapping of the tunnel face

After each blasting in a tunnel, geological mapping and categorization of the rock mass were performed in order to obtain the most detailed picture of the geological structure on the tunnel route. For each tunnel face along a tunnel, the most influential discontinuity systems were measured and plotted. In order to define the quality of the rock mass according to RMR, categorization was performed, and all relevant parameters were determined (rock mass strength, RQD, assessment of the impact of discontinuities, discontinuity spacing, state of discontinuities, mutual influence of discontinuities, groundwater, etc).

Based on results of engineering geological mapping a developed geological profile of the right tunnel tube and geological mapping of the left tunnel tube face of the tunnels were established, on that the rock mass classification were determined. Layering was not identified or is weakly pronounced. According to the RMR classification, the rock mass was classified to adequate tunnelling category.

2.6 Video prospecting of the investigation borehole

At the tunnel face, exploratory boreholes with a diameter of 102 mm were drilled in the side of the profile with a deviation of 200 from the designed excavation contour towards the rock mass Fig. 2a. The boreholes are located at a height of 4.0 - 4.5 m from the vertical alignment, and the length of the boreholes is equal to the length of the mining boreholes drilled in the tunnel face (Antičević 2011). Video prospection was carried out before (Fig. 2b) and after the blasting of the excavation of the tunnel face to detect damage to the rock mass, namely: over-profile excavation, spalling, and expansion of existing cracks, and the formation of new cracks. The imaging data showed the formation of new cracks reaching a depth of 0.61 m, while the spalling and expansion of existing cracks reached a depth of 1.08 m, Fig. 2c.

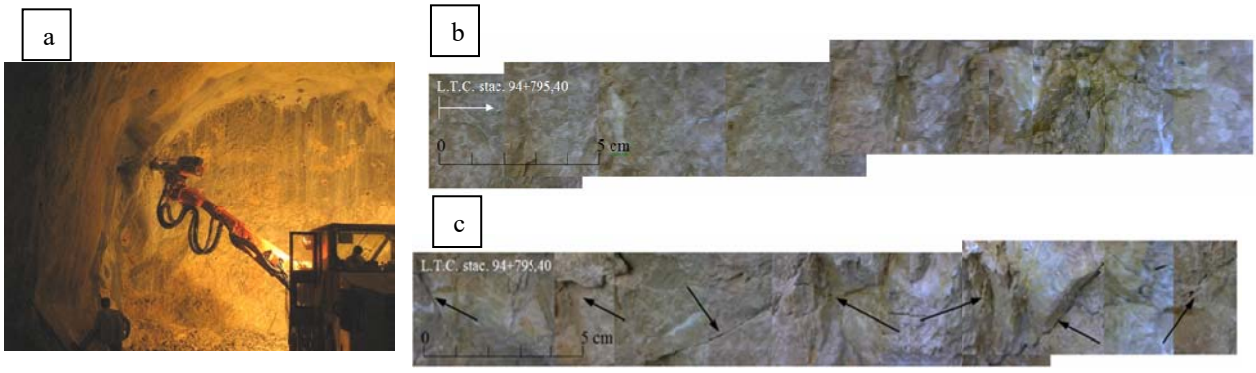


Fig. 2 Video prospecting of the investigation borehole. a) Position of video prospecting borehole, b) Photo of the borehole wall before blasting, c) Photo of the borehole wall after blasting. Black arrows indicate positions with expanded existing cracks or spalling.

2.7 Geophysical measurements in the tunnel

Geophysical measurements included seismic cross-hole tomography in the tunnel side at the location where video prospecting measurements of the exploratory boreholes were performed. Seismic cross-hole tomography measured changes in the velocity of longitudinal (primary) p-waves and transversal (secondary) s-waves propagation from the tunnel excavation contour towards the rock massif. More significant damage in category II of rock mass reaches a depth of 1.6 m, Fig. 3, with a reduction in the velocity of p-waves propagation by 20% compared to the undisturbed rock mass, from3036 m/s

to3753 m/s). In category III rock mass, the damage zone is deeper by 2.5 m, but the reduction in the propagation velocity of p-waves is smaller by 15% from 2700 m/s to 2295 m/s.

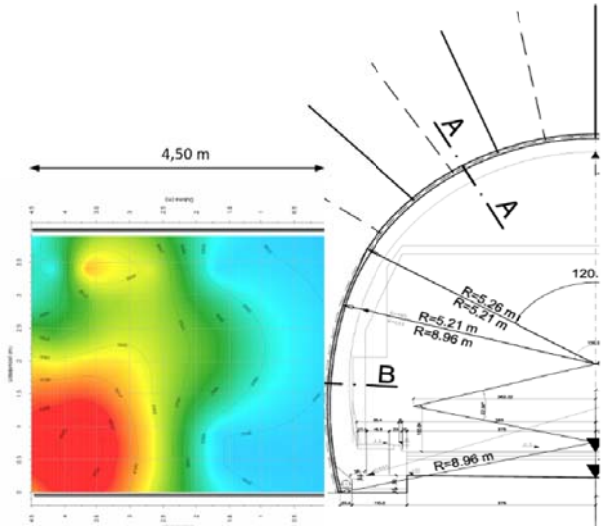


Fig. 3 Results of seismic cross-hole tomography in the wall of the Tunnel Šubir. In blue p-waves from 3.100 to 3.300 m/s: in green p-waves from 3.300 to 3.750 m/s; in yellow p-waves from 3.800 to 3.900 m/s; in orange p-waves from 3.900 to 4.100 m/s; in red more than p-waves from 4.100 m/s.

2.8 Rockbolt pull-out tests

During the excavation of the tunnel, preliminary and control tests of rockbolts are foreseen. The bearing capacity of the anchors was controlled by a pull-out test according to ISRM's Suggested methods for rockbolt testing (ISRM 1974). Preliminary tests or proving the usability of rockbolts include determining the bearing capacity of rockbolts in geological soil conditions similar to those expected during tunnelling. The tests were carried out before the start of the tunnel excavation on the tunnel pre-cut. The rockbolts are installed in the same way as they will be during tunnel construction. Total of 5 rockbolts with a length of 3 m were tested for force until failure (max 150 kN). The time between the installation and testing of the rockbolts was not shorter than 7 days. During the excavation of the tunnel, control tests of rockbolts were carried out, which were tested up to the demanding load capacity of 120 kN. The test frequency was 5 pieces for the first 100 installed rockbolts, and thereafter 5 pieces for every next 200. More than 95% of the tests of the control rockbolts met the required load capacity with deformations within 10 mm, and rarely up to 25 mm.

3 Discussion and conclusion

This paper presents the methods used in the verification of the stability of the underground excavation during the construction of the Mali Prolog and Šubir tunnels on the A1 Zagreb-Split-Ploče Highway, near the town of Vrgorac, Croatia. Given the similar geological settings, the height of the tunnel overburden and the cross-sections of the tunnels, as well as the partially simultaneous construction of both tunnels, the experience gained during the construction and verification of the stability of the underground tunnel opening enabled the verification of the application of the methods to both tunnels constructed in similar geotechnical conditions. During the tunnel construction, investigations were carried out to verify the efficiency of the length of the rockbolts of the primary tunnel support. During the tunnel construction, exploratory drilling, geophysical seismic measurements, video-prospecting of exploratory wells, geological mapping of the tunnel face, geodetic convergence measurements and anchor pull-out tests were carried out in II, III and IV categories of rock mass according to the RMR classification). All measurements taken generally confirmed the values and behaviour of the rock mass according to the requirements of the geotechnical project, with minimal deviations in the rock mass category and therefore changes in the sub-work systems applied in the tunnels. As a method of checking the effect of blasting on the rock mass, the method of video-prospecting of exploratory boreholes in the tunnel face, carried out before and after blasting, was used. The results of the visual prospecting of investigation boreholes showed damage to the rock mass to a depth of 1.08 m from the walls of the tunnel profile (Antičević 2011). Geophysical measurements showed deeper damage, in the II category of rock mass up to 1.6 m, and in III category up to 2.5 m. The reason for this difference between visual and geophysical measurements lies in the fact that there are microcracks that are not

visible, and the propagation and change in the speed of p-waves can detect them. Using the methods of video-prospecting of investigation boreholes at the tunnel head and geophysical seismic measurements at the tunnel side, which are not standard methods within the framework of observational methods, make a significant contribution to determining the damage to the rock mass due to the execution of the works (Antičević 2011). Both methods indicate damage values, which allow a clearer interpretation of the degree of damage to the rock mass in stability analyses, as well as for the purposes of better assessment of technological conditions for reducing over-profile excavation

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