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Monitoring and back numerical analyses in the Konjsko tunnel

I. Arapov & M.S. Kovačević
Faculty of Civil Engineering, Zagreb, Croatia

P. Lušo
MPV. d.o.o., Zagreb, Croatia

ABSTRACT: The Konjsko tunnel is an integral part of the Zagreb-Split-Dubrovnik highway, section Prgomet-Dugopolje. It consists of two tunnel tubes having 1326.0 and 1133.8 meters in length. The greatest overburden amounts to 150 m. This paper presents geotechnical measurement results and back numerical analyses. Back numerical analyses in combination with geotechnical measurements enable safer and more rational approach to designing and performing underground constructions. They contribute to the development of cognition on rock massifs and to determining its physically-mechanical parameters, joining it with rock classifying results. They can help to verify or to modify characteristics of the primary support system, of the foreseen progress length, estimated time of non-supported ranges, as well as time and schedule of performing all works regarding the excavation stability.

1 INTRODUCTION

The Konjsko tunnel is located on the Prgomet – Dugopolje section of the Zagreb – Split highway. The tunnel consists of two tubes, each having two lanes. The length of the north tube is 1,326.0 m, while the south tube length amounts to 1,133.8 m. The thickness of the overburden near the north tube is approx. 145 m, while near the south tube 125 m.

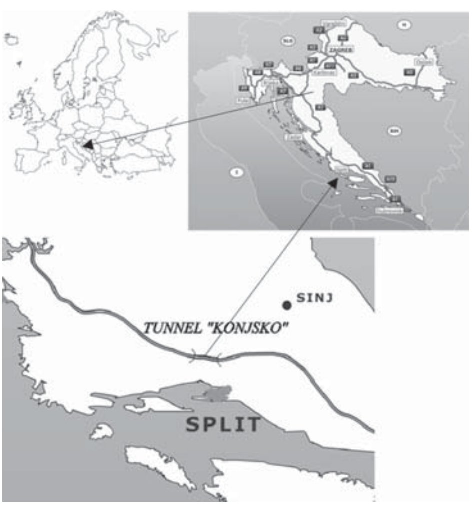

Figure 1. Croatian highway network with location of the "Konjsko" tunnel near Split.

2 ENGINEERING AND GEOLOGICAL CHARACTERISTICS OF THE TUNNEL AREA

2.1 *Investigation works*

For the purposes of the Konjsko tunnel design, geological, engineering-geological, hydro geological and geotechnical investigations have been performed in the wider area of the south and north tube route (Grabovac et al., 2004). The engineering-geological and geotechnical investigation works have been performed in order to define the composition and physical and mechanical properties of deposits, thickness of quaternary deposits, as well as elevations at which flysch deposits appear on the western part of both tubes. Positions of limestone sediments and characteristic discontinuities have also been recorded. Selected samples of marl and limestones were laboratory tested. The above mentioned investigations and obtained data provided a design engineers' base for geotechnical categorization of rock masses (according to RMR and Q systems), for designing of the tunnel support system and for selection of the tunnel excavation technology.

Investigation drilling was performed at eight points in total. Three drillings were performed at the west tunnel tube centreline and one at east portals. Drilling further could not be performed due to lack of available access. Among the geophysical investigations, seizmic down-hole measurements were performed in three drill holes, geo-electrical measurements with the application of the LIS technique, shallow seizmic reflection, as well as refraction and geo-electrical sounding were also executed. During the determination of the rock core, the rock core fragmentation ratio (RQD), inclination of characteristic discontinuities and their properties were also defined.

By mapping the ground and using data from the literature, Upper Cretaceous and Eocene limestone sediments as well as clastic sediments of Eocene flysch may be distinguished in the wider tunnel tube zone. Upper Cretaceous Senonian limestones (K_2^3) rich in rudist fauna are light grey to white with sparse dolomite intercalations. Stratified limestone rock is to be found at eastern and western parts of the site, while in the central part the limestone is massive. Two main joint systems are registered, which are vertical and inclined in relation to layer striking, and which together with the interlayer discontinuity separate the rock mass into blocks. Lower to Middle Eocene foraminiferal limestone ($E_{1,2}$) is rich in fossil remains of Miliolids, Alveoline and Nummulite. Their color is white to light brown. Their presence is registered on the southern part, outside the tunnel tube zone. Middle Eocene marly glauconitic bulbous limestone (E_2) of grey-brown color is registered in the narrow belt on the eastern part of the site (outside the tunnel zone), where they are in the fault contact with flysch sediments and Upper Cretaceous limestone sediments. Eocene flysch ($E_{2,3}$) clastic deposits are registered in the western part of the site and in the tunnel zone where they are incorporated under Senonian Upper Cretaceous limestone (K_2^3); these deposits are also registered on the eastern part of the site, outside the tunnel. They are composed of tectonized, deformed limestones which are grey to grey-blue in color with intercalations of clayey marls. In the field, these sediments are mostly covered with diluvium (Q_d) and eluvial formations (Q_{et}), and their outcrops may be noticed only occasionaly.

Hydrogeological relations result from lithological composition and tectonic relations generated in the field. Quaternary silty-clayey sediments of small thickness and unequal participation of rock fragments on the western side of the site are characterized by intergrain porosity. Fracture porosity is characteristic for flysch and for limestone sediments, fracture and cavern porosity. Regarding permeability, the characteristics of Quaternary sediments are changeable depending on the portion of clayey component in them. Limestone sediments have a very good permeability while clayey marls are practically watertight. During the investigation drilling, the presence of ground water without the connected water level was detected in the west tunnel in flysch sediments, as well as at the overthrust contact. The level varies depending on the quantity of precipitation.

The results obtained by these investigations and research have made possible the elaboration of the geological and engineering-geological map of the narrower tunnel area at the scale M…1:5000 and two prognostic longitudinal engineering-geological sections along the centerlines of the tunnel tubes at the scale M...1:1000 (Figure 2). By blast monitoring, about 450 engineering-geological cross sections were made at the scale M...1:50 for the area of both tunnel tubes. Results of previous field investigations have been supplemented during the works in the tunnel by constant monitoring and recording of changes.

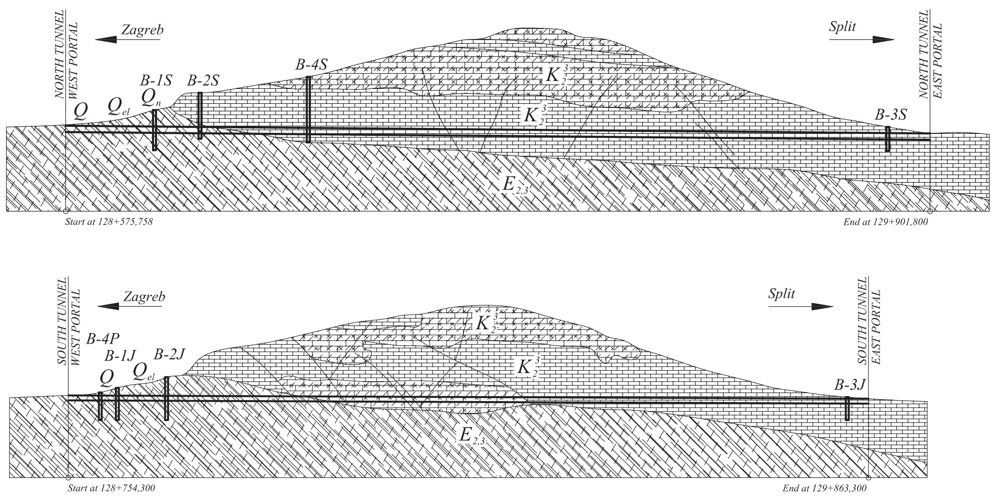

Figure 2. Engineering and geological longitudinal section of the north and south tunnel tubes, (Grabovac et al., 2004).

Table 1. Presentation of important investigation results.

Material	σ_{ci} [MPa]	GSI [–]	RQD [%]	$CaCo_3$ [%]	Categ. [–]
Marl	0.3–5.3	15–40	25–80	30–60	V
Limestone	30–60	20–60	25–100	83–95	II–V

3 GEOTECHNICAL MONITORING

3.1 *Introduction*

Geotechnical measurements along with the back numerical analyses represent a fundamental part of the interactive designing concept (Kovačević, 2003) or the second stage of designing, as the tunnel practice calls interactive designing. Back numerical analyses with geotechnical measurements and observations enable to define the actual mechanical characteristics of the soil where the underground excavations are performed. They enable the verification or modification of properties of the primary tunnel support system elements, forecasting the length of progress, assessed time of stability of unsupported spans, and the duration and sequences of all works to be executed concerning stabilization of the excavation (Kovačević et al., 2006).

According to the directives of the International Tunnelling Association, the tunnel construction procedure to be permanently adjusted to the progress of works can be implemented through combining design methods, empirical designing and immediate interpretation of the in-situ measurements. In this case, the in-situ measurements of rock massif deformations, as well as the measurement of deformations and stresses in the support system are always used for the approval of the design or for its modification. Interpretation of obtained values provides us with an insight into the behavior of the rock massif as a reaction to the advancement of the tunnel. It is not just the stability and the applied design computing model that are verified by the in-situ measurements, but also the basic concept of massif rock reaction to the execution of the tunnel opening and the efficiency of safety structural elements.

Measurements in tunneling are generally divided into three groups (John, 1977):

1. Control measurements regarding deformations of underground excavations.
2. Support system measurements regarding rock mass displacements around the underground excavations, as well as deformations and stresses in support system elements.
3. Stabilization measurements regarding deformations and stresses of the secondary concrete lining.

Measurements of the vertical and horizontal soil displacements have been performed around the underground opening in the zone of the Konjsko tunnel in order to verify the applied excavation and support work methodology, as well as for the purposes of verification of numerical model results by back analyses. The geotechnical measurement results of the south tunnel tube shall also be presented and commented for the purposes of back analyses. The measurement results shall be interpreted depending on the excavation and support work stages, and shall be compared with the numerical model results in relation to considerations on safety.

3.2 *Terrain surface measurements of vertical and horizontal soil displacements in the tunnel zone*

Measurements are performed in order to record soil displacements in front, during and after the tunnel face passage through the measuring profile zone (Kovačevic et al., 2008). As per the design items, the measuring profile at chainage 128 + 829.00 was executed with three drill holes, of respective depths of 24, 20 and 13 m (Figure 3). Vertical displacement measurements were performed by means of a sliding micrometer with a measuring probe of ϕ47 mm, while measurement of horizontal displacements was performed by an inclinometer. The measurements were performed until the complete disappearance of displacements (Figures 4, 5, 6).

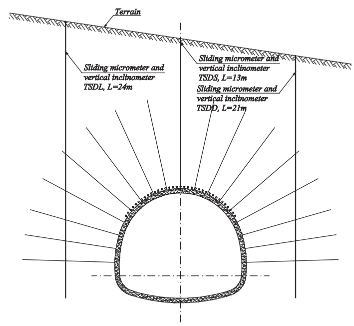

Figure 3. Characteristic tunnel and measuring equipment cross section at the chainage 128 + 829,00.

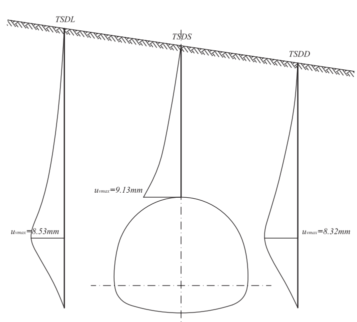

Figure 4. Measurement results of rock vertical displacement in the tunnel zone performed by a sliding micrometer.

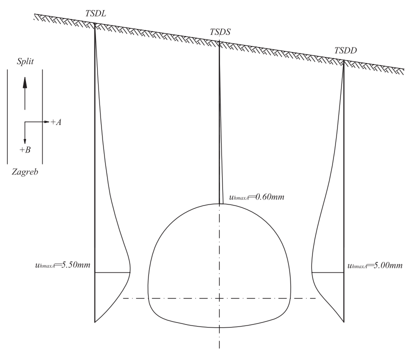

Figure 5. Measurement results of horizontal rock displacements in the tunnel zone performed by a horizontal inclinometer, direction “A”.

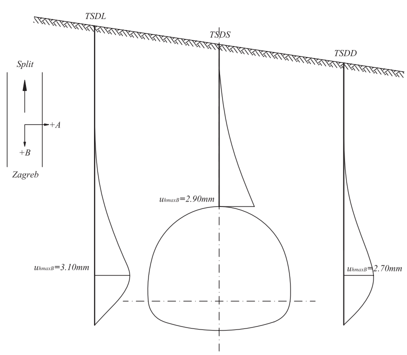

Figure 6. Measurement results of horizontal rock displacement in the tunnel zone performed by a horizontal inclinometer, direction “B”.

Table 2. Presentation of important measurement results.

Profile	u_{vmax} [mm]	u_{vmaxA} [mm]	u_{vmaxB} [mm]
TSDL	8.53	5.50	3.10
TSDS	9.13	0.60	2.90
TSDD	8.32	–5.00	2.70

3.3 *Measurements of soil displacement around the underground excavation*

Soil displacement measurements around the underground excavation were performed at five drill holes per measuring section (Kovačević et al., 2008). Two measuring sections were executed at chainages 128 + 841,0 and 128 + 921,0 (Figure 7). Displacement measurements were performed by a sliding deformeter, and the measurements were undertaken until the complete displacement disappearance (Figure 8).

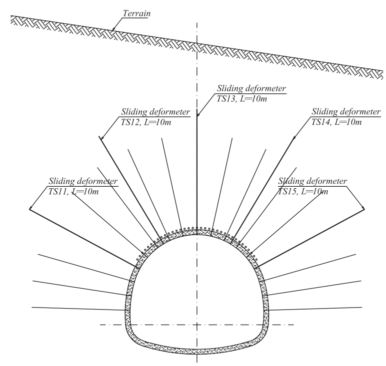

Figure 7. Characteristic tunnel and measuring equipment cross section at the chainage 128 + 841,00.

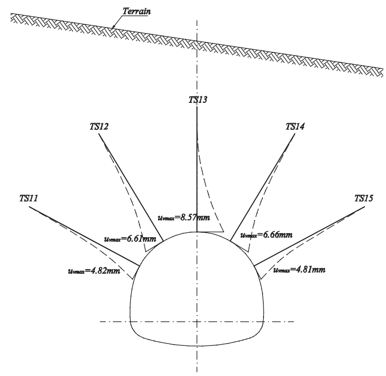

Figure 8. Measurement results of rock vertical displacements in the tunnel zone performed by a sliding deformeter.

Table 3. Presentation of important measurement results.

	Profiles				
	TS11	TS12	TS13	TS14	TS15
u_{vmax} [mm]	4.82	6.61	8.57	6.66	4.81

4 BACK NUMERICAL ANALYSES

Back numerical analysis has been performed for the measuring profile at the chainage 128 + 84 (Figure 7). At the chainage under consideration, the basic rock consists of crushed marl gray to grayish-blue in color, with occasional intercalations of sandstone and marly dust. These are poorly hardened, clastic fine-grained sediments. The support system comprises the 40 cm thick primary support, self-propelled 8 m long IBO bolts 32/20 and a pipe roof. The overburden height amounts to 14.40 m. Figure 9 shows the measurement results of the TS13 vertical deformeter in the arch crown at the measuring profile.

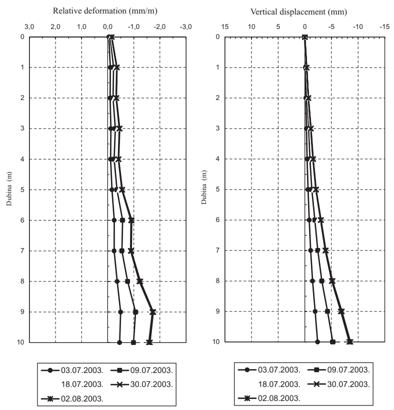

Figure 9. Measurement results of the TS13 vertical deformeter at the TS1 measuring profile.

Numerical simulation was performed by FLAC program. FLAC is a computer program designed for behavior simulation of the constructions made of soil, rock and other materials which may generate plastic flow. The material is being discretized to elements by a formation of a network adjusted by the user depending on the modeled structure form. Should the stresses be large enough to entice flow in the material, the network shall be deformed along with the material in conformity with the final difference method. In order to adjust the calculation as much as possible to the real situation, the program feature shall be applied allowing for the relaxation of imbalanced panel loads in the wanted percentage, thus achieving simulation of tunnel execution's time sequence. In such a way, 10% of unbalanced tractions may be allowed to relax in the moment upon the excavation and before the installation of the support system, taking into consideration that the opening has been unsupported for some time. This is done by estimating the tractions along the excavation edges, which on the other hand are generated by the excavation itself. Other tractions of opposite directions and weaker by a certain percent than the estimated tractions oppose them.

Numerical simulation was performed in three steps. First, the tunnel excavation was simulated and the relaxation of 10% unbalanced forces was permitted. Next, the installation of the support and rock bolts with further relaxation of 30% was simulated. After that, the deformeters are "installed" and the relaxation of unbalanced tractions was simulated until the achievement of balance. These relaxation degrees have been obtained from the experience and the back analyses performed on the previous tunnel sections. In order to analyze stresses and deformations conditions occurring in the support system elements on the measuring tunnel cross section, quasihomogenous linear elastic soil model was applied. The determination of actual modulus of deformability where the measured deformation shall be approximately equal to the estimated ones is extremely important for determination of the degree of utilization or for reaching the bearing capacity of the reinforced shotcrete and rock bolts. In the geotechnical design of the "Konjsko" tunnel, the modulus of deformability of $E_m = 200$ MPa was anticipated. During excavations, the rock mass of the V cathegory was determined, but there was no data on RMR. In the area of the measurement section, the data on GSI index and uniaxial compressive strength value range was recorded, the values of which are presented in the Table 1.

The back analyses have been elaborated in a way that the deformability model was being decreased until the tunnel arch crown and vertical deformeter TS13 top displacement coincided with satisfactory engineering precision. The design and measured values and distribution of deformations along the sliding deformeters are shown in the Figure 10, while the Figures 11 to 14 present the results of numerical analysis elaborated in the FLAC program.

The module obtained in this way amounted to 87 MPa being 2.3 times less than the values used in the original design and on the basis of which the primary support system elements have been dimensioned.

As it is evidenced in the Figure 10, the displacements in all deformeters are being decreased considerably faster than the design displacements, as the deformeter line moves away from the opening. This happens due to the simplified linear and elastic soil model which does not take into account the fact that the soil hardness significantly changes as deformations are increased, or in other words that the soil at small deformations is up to 10 times harder than at larger deformations (Kovačević, 1999). Non-linear soil model would generate faster deformation decrease. Moreover, it may be noted that the design displacements at ends of other deformeters are smaller than the measured displacements (5–12%) and that they are not completely symmetrical, as

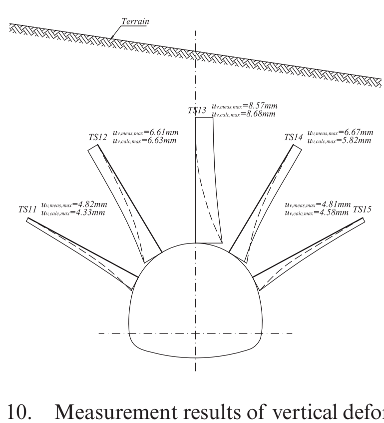

Figure 10. Measurement results of vertical deformeters and estimates made at measurement section.

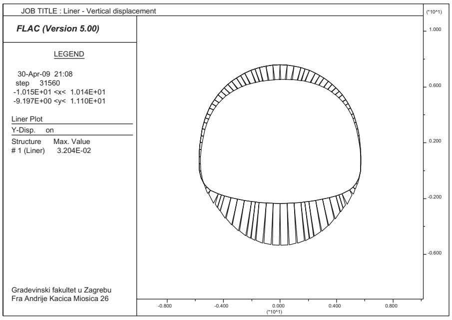

Figure 11. Vertical displacements of the primary support.

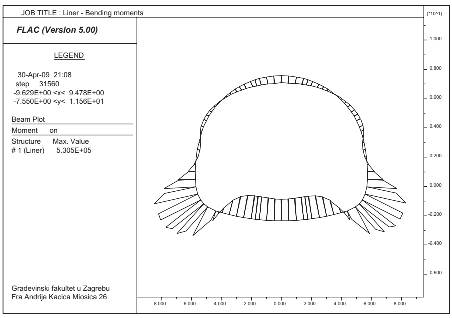

Figure 12. Bending moments in support system.

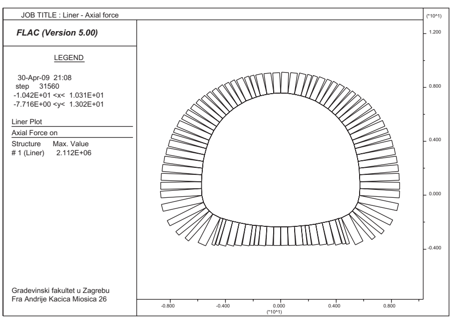

Figure 13. Longitudinal forces in support system.

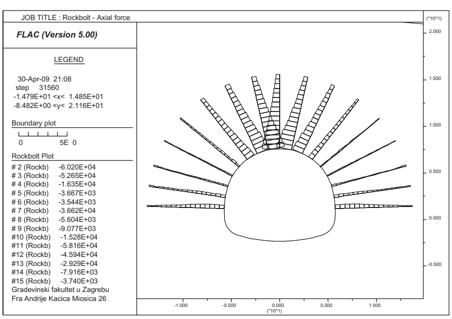

Figure 14. Forces in rock bolt anchors.

the design presupposed a quasi-homogenous soil section, which is unlikely to occur in reality.

5 CONCLUSION

The back numerical analyses combined with geotechnical measurements provide for a safer and a more rational approach to desinging and execution of underground structures. Moreover, such analyses contribute to the development of knowledge on rock mass behavior and defining its physical and mechanical parameters by relating them with the rock classification results.

Although the design considers a relatively low deformability module for marl, the back numerical analyses of the "Konjsko" tunnel show the real deformability module to be approximately two and a half times smaller than in the design, making the internal forces in the support system larger. Despite the fact, the design support system is endowed with sufficient safety reserves, thus making any additional stability safety measures unnecessary.

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