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Experiences from the construction of a twin tunnel in a marly formation

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ABSTRACT: The construction of the Mpozaitika tunnel allowed for the application of the N.T.M. in the design of the excavation procedure and the primary support measures in a soft rock formation. The classification according to the Q system was used for an initial appraisal and numerical modeling for the final design. The latter was based on laboratory and in-situ defined mechanical properties. Nevertheless, during the design of the final lining the mechanical parameters of the soft rock formations were updated from backanalysis results based on monitoring data measured during the excavation.

1 INTRODUCTION

The Patra deviation motorway includes several twin tunnels constructed by various contractors. Each of the tunnels comprises two lanes in each direction.

The geology of the region consists of formations, from the younger to the older: quaternary deposits, conglomerates of the plio-pleistocene, neogene sediments, flysch, limestone, schist and chert. The quaternary deposits are alluvial deposits, contemporary fluvial deposits and scree.

These geologic formations belong to three geotectonic zones, the Olonos, the Gavrovou-Tripolis and the Ionian. The first zone consists mainly of limestone, schist and chert, the second mainly of flysch, and the third mainly of flysch and limestone.

The area is very prone to landslides. This is due to the weak bond of the sediments, the intense relief of the local morphology, the tectonic structure, the orogenic movements, the proximity to the earthquake epicenters of the gulf, the weathering of the rocks, the superficial and underground water flows, and the man made excavations. Part of the tunnels may pass through or close to such regions.

The high standards of the alignment of the highway, that require small curvatures and inclinations, necessitate the construction of a number of tunnels and bridges to cross the intense local morphologic relief.

These tunnels had to cross through the younger hard soil-soft rock formations of the quaternary deposits, the conglomerates and the neogene sediments; the latter constituting the largest portion of the crossed rocks.

The Mpozaitika tunnel started first. It is a double tube tunnel situated in the northern part of the de-

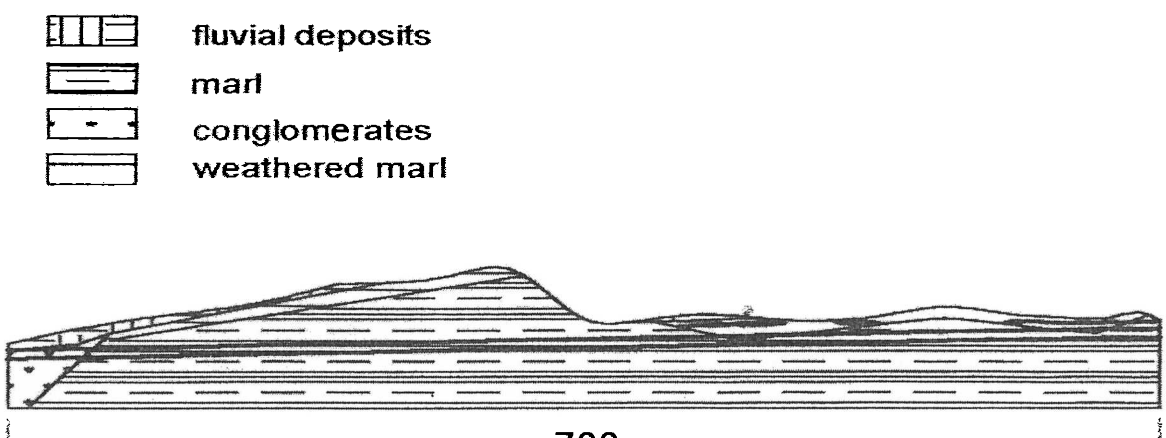

Figure 1. Geological longitudinal section

viation highway. The right branch is 615m and the left 686m.

The alignment is partly linear and partly curvilinear, with a maximum radius of curvature 990m. The maximum inclination is 2.3%. Each branch contains two lanes, with no enlargements in any part of the tunnel.

The excavated cross section area is 111m^2. The pillar between the tunnels is larger than 15m. The overburden ranges from 15 to 60m in the eastern part, while in the western part the overburden is much lower, from 6 to 17 meters. These lower overburden regions belong usually to a slope and in some places old landslides may have occurred.

The crossed geologic formations, shown in the cross section of figure 1, consist of conglomerates, sandy or clayey marls, and intercalations of sand and loose soils. The tunnel is constructed above the water table. However, some minor local ingress of water is encountered.

2 DESIGN

The Norwegian tunneling method was chosen for the construction of the Mpozaitika tunnel. The Q value of the rock mass and the size of the opening dictate the excavation and support system, proposed by this method. For the marl, which is the main formation crossed by the tunnel, Q is evaluated to lie between 0.1 and 0.3. Excavation phases and support are then given initially by the pertinent design chart of the method. In a second stage, numerical modeling is employed to evaluate the induced stresses and strains within the rock mass and the supporting structure.

Thus, excavation of the cross section was designed (Axon et al., 1995) mainly in three stages, as shown in Figures 2 and 3. Excavation is achieved with mechanical means in steps of less than 1m; the largest unsupported length was designed to be 1÷1.5m.

Primary support comprises steel reinforced ribs (RRS), shotcrete, rock bolts, and spiling (Sofianos & Aranitis, 1997) when required. For the lower overburden regions steel ribs were proposed instead of the RRS.

Excavation starts with the top heading. The excavation and mucking is followed by an initial spraying, on the walls of the tunnel, of a fiber reinforced layer of shotcrete 5cm thick. Rock bolts are then installed all over the exposed wall. Their most dense pattern is 1.3x1.3m. It is followed then with additional spraying of fiber reinforced shotcrete to form a total layer ~10cm thick. Finally, the thickness of the fiber reinforced shotcrete layer is increased to 15cm.

Stiffening of the shotcrete shell is achieved, for most of the tunnel length, with the RRS. Each RRS element, ~40cm wide, consists of 5 bars 20mm diameter. These elements are fixed in place with additional rock bolts with faceplates 10cm side. The distance of the rock bolts along the RRS is 1.5m. On the placed RRS, a 20cm thick layer of shotcrete without fibers is sprayed, covering a strip 80cm wide along the periphery of the top heading of the tunnel. The RRS are secured in their base with rock bolts 8m deep.

Any occurrence of micro fracturing in the shotcrete of the walls will necessitate an additional spraying of fiber reinforced shotcrete, 5cm thick.

The lowest part of the RRS is not covered with shotcrete in order to allow for a 0.8m free length of the bars to be overlapped with the RRS of the next excavation stage.

Next stage is the excavation of the bench. A lining of fiber reinforced shotcrete 15cm thick is sprayed in 5cm layers. Rock bolts are installed after the spraying of the first layer. This is followed by the installation of the RRS, which will have on both sidewalls an overlapping length with the RRS of the top heading of 0.80m.

The RRS will be anchored on the walls with rock bolts. The bases of the RRS are anchored with rock bolts 8m deep and shotcreted. The largest unsupported length is 1m.

The excavation of the cross section is completed with the excavation of the invert. This phase may be executed continuously over large portions of the tunnel, in sections where the excavation of the bench is stable.

Otherwise, if swelling is anticipated, it may be mandatory to install an invert arch with fiber-reinforced shotcrete and RRS. In the latter case advance is achieved in stages of vertical cuts all over the width of the section, followed by the completion of the support measures of one or two supporting rings. The RRS in this region is made in trenches, in order to achieve a smooth surface in the inside of the invert lining.

Especially vulnerable is the sidewall of the pillar. Therefore, the excavation of one branch lags the full excavation and the installation of the primary support of the other, by at least 50m.

Site investigations, including boreholes along the axis of the tunnel and inspection and tests in an adyt close to the tunnel, provided the mechanical parameters of the surrounding the tunnel geomaterials. Particularly, the maximum shear stress along the mortar-rock interface of the rock bolts was calculated from the results of pull out tests to lie in the 0.6 to 1.1 MPa range. However, much lower values of the strength parameters were indicated by the laboratory tests on specimens of collected samples; the undrained shear strength c_u was measured to have

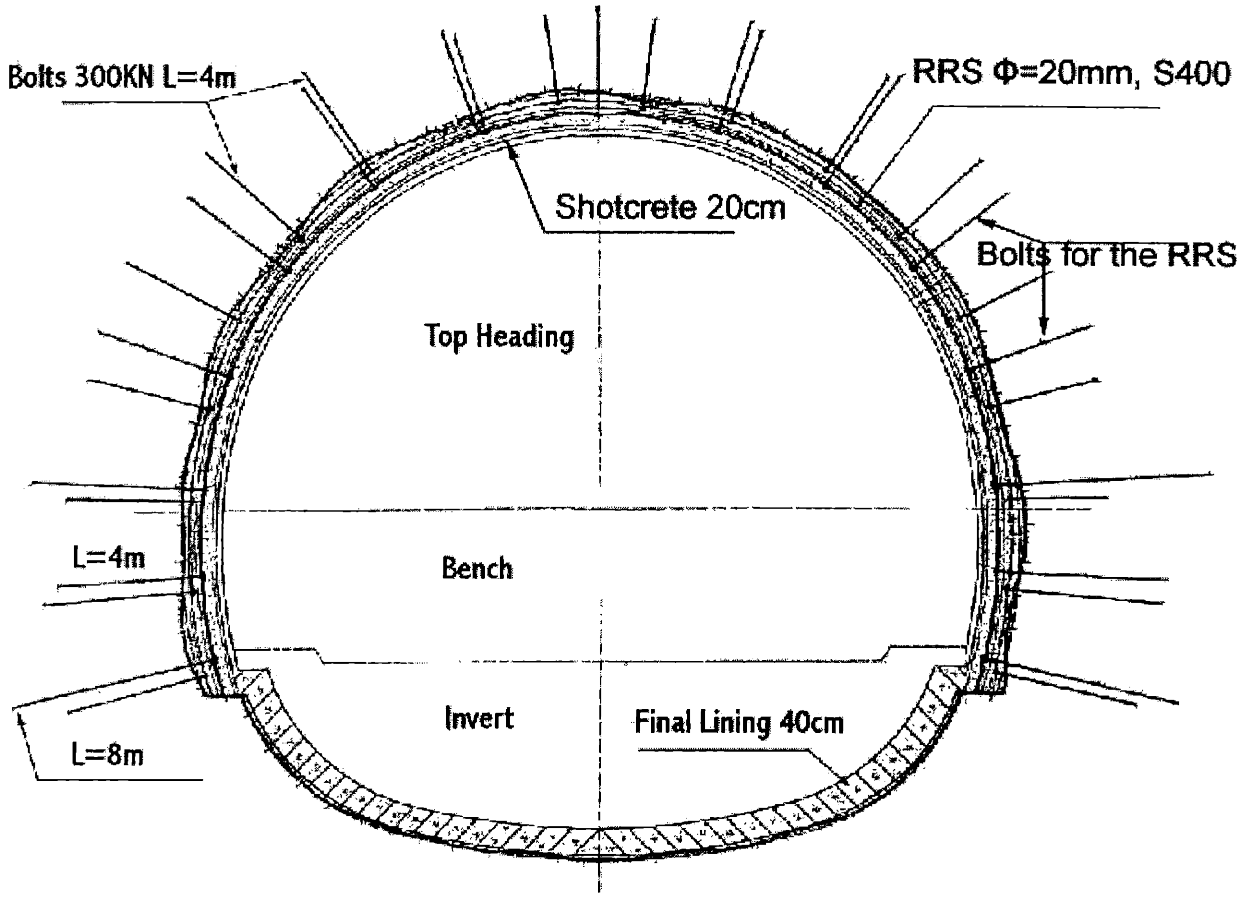

Figure 2. Excavation and primary support measures of the Mpozaitika tunnel; cross section.

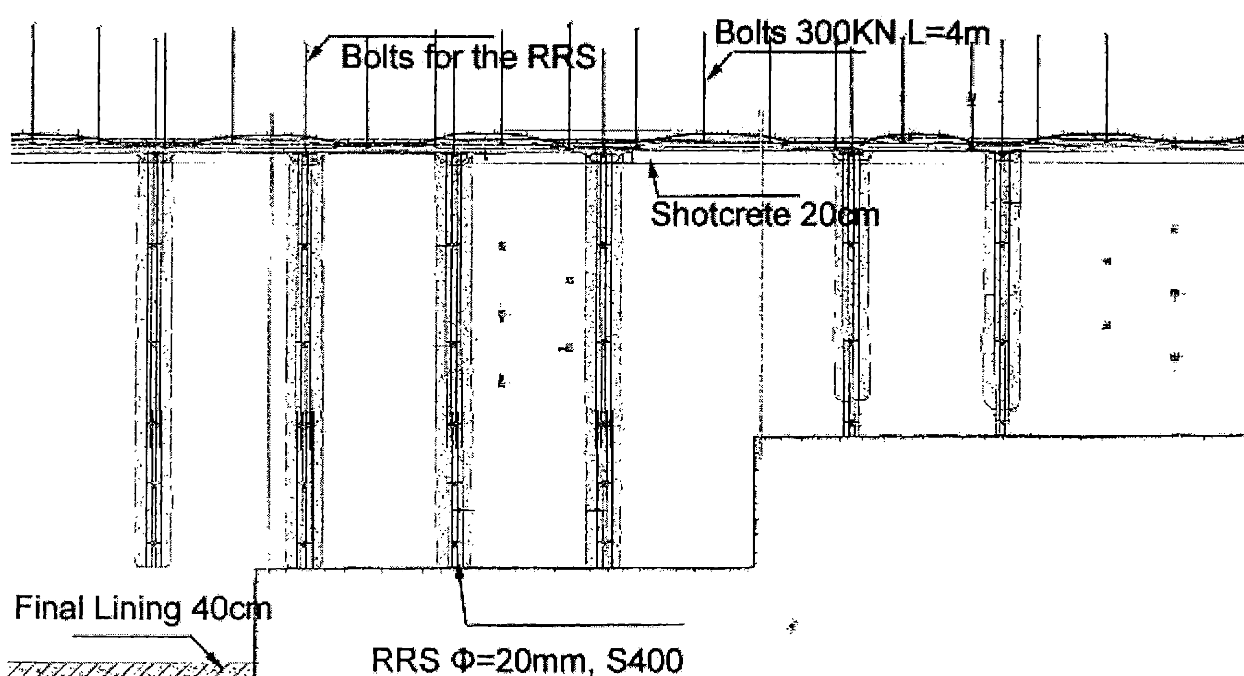

Figure 3. Excavation and primary support measures of the Mpozaitika tunnel; longitudinal section.

values less than 0.1MPa. Design, prior to the construction was based on these properties. Modeling of the excavation procedure employed the finite difference code UDEC™, which allows for the modeling of any sandy layers distributed spatially. In table 1 are given the parameters used and the expected maximum displacements of the walls of the tunnel (Chryssanthakis et al., 1998).

Table 1. Analyses based on the site investigation.

Overburden	E MPa	c kPa	φ °	v	Pillar m	max δ mm
40	100				11	210
40	100	40	22	0.33	16	190
16-22*	400				16	80

*excavation of the tunnel had already started.

3 MONITORING

During construction, a measurement program was established. This consisted of level, convergence, extensometer, inclinometer and rock bolt pull out load measurements.

The displacements of the walls and the roof of the tunnel were measured on optical censors. The displacements within the rock were measured with extensometers 5, 10 and 15 m deep.

Wall displacement and extensometer measurements started at each measuring station at a distance from the face of 2 to 5m. This distance accounted for losing the measurement, of a major part of the short-term elastoplastic displacement, due to the excavation of the top heading. The measured vertical displacement values varied from very small to almost 120mm. The horizontal displacements were less than the vertical ones.

In the lower overburden slopy regions, the danger of a landslide necessitated the continuous measurement of any horizontal movements, with inclinometers, starting before the passing of the tunnel. Such inclinometers were installed in 8 boreholes either in between or uphill of the tunnel tubes. No landslide movements were measured. However, maximum horizontal movements of almost 20mm were measured during the passing of the tunnel.

Finally, it was considered necessary to evaluate the bearing capacity of the rock bolts; the success of the tunneling method was based on their efficiency. These, evaluated a shear strength of the marl larger than 0.36MPa.

4 REEVALUATION

After the complete excavation of the tunnel and the installation of the primary support, in order to dimension the final lining, the mechanical parameters of the surrounding the tunnel rock mass were reevaluated; these were based on the backanalysis of measured displacements. In figure 4 the tunnel portal may be seen ready for the installation of the final lining. Modeling of the final lining structure was then performed on a series of geological cross sections, along the tunnel axis, characteristic for the region close to each section. Five types of geomaterials are encountered in these sections; in some of them all types exist. These are denoted as clayey marl (ml), loose or landslide formations (Am, mwl, LS), weathered sandy marl (mws), sandy layers (sd, sl, sb) and silty marl with intercalations of sandy layers (ms). Of particular importance was the region with the lower overburden. In this region the parameters used were grouped into the three types given in the following table:

Table 2. Parameters based on the observed tunnel response.

Geomaterial	E MPa	c kPa	φ °	v
ml	280	40	23	0.45
Am, mwl, LS, mws	30	10	25	0.30
sd, sl, sb	60	0	36	0.30

The silty marl is taking values ranging from those of geomaterial ml to those of sd.

Implementation of a finite element code with these parameters applied to the chosen cross sections (Efpalinos, 1998), evaluated the response of the tunnel and allowed for the dimensioning of the final lining. The calculated rock bolt forces, after the excavation and primary support of the tunnel in this region, may be seen in the following table.

Table 3. Calculated rock bolt forces.

Section	Overburden m	Maximum rock bolt force kN	
		Roof	Sidewall
A	35	10	40
B	7-17	8	74
C	4-12	1	30
D1	12-18	2	40
D2	12-18	7	50

5 DISCUSSION AND CONCLUSIONS

The Norwegian tunneling method, applied to the Mpozaitika tunnel, employed a multi-face excavation procedure and a very flexible support system. Its innovation was the application of the Q system in order to design the construction of a tunnel in soft rocks-hard soils. Further, particularly successful was the application of the rock bolting system, which provided significant support in these low

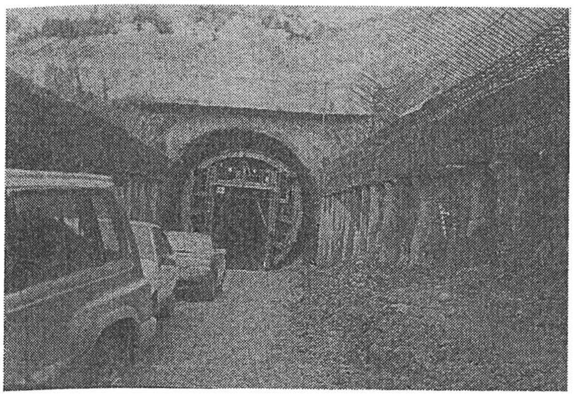

Figure 4. Formwork for the construction of the final lining entering the southern portal.

strength geomaterials and enhanced the effectiveness of the RRS supporting system.

The excavation design was based on parameters provided by in-situ investigation probing and laboratory testing. Monitoring of displacements and pull out tests on rock bolts during excavation allowed, through backanalysis, for a more realistic evaluation of the soft rock mechanical parameters. These updated parameters were then used in the final lining design.

ACKNOWLEDGEMENTS

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