The Maliakos–Kleidi Motorway Tunnels - Geotechnical conditions and Construction Experience

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ABSTRACT: The tunnels of the Motorway Maliakos – Kleidi (MMK) BOT project constitute the application of the NATM under demanding construction requirements as posed by: the nature of the project (BOT), the dimensions of tunnel section type (two traffic lanes and an emergency lane, per bore), the underground lengths (totaling 21.5km in length and including the longest highway tunnel in Balkans), in conjunction with the quality and spatial variability of the involved geomaterials and the anticipated hydro-geological and geotechnical hazards. The overall dimensions of the main tunnels (120÷180m^2) as well as the rest of the underground facilities designated to intersect the tunnels (ventilation galleries, shafts and escape adits), posed challenges both in the design and construction, at areas where adverse hydro-geological and geotechnical conditions were encountered. The geotechnical risks included tunnelling, under: high (>200m) but also extremely low (<10m) overburden heights, through long fault zones, featured with persisting, severe face instabilities, mixed face conditions in geomaterials of dramatically different geomechanical behavior, water ingress, high convergences rates and delayed deformations. Certain troublesome situations were identified during construction that required modifications in construction methodology and establishment of special methodological approaches to successfully cope with. Close collaboration between the Designers' Consortium and the Construction JV resulted in quick adaptations of the E&S methodology and processing of efficient solutions that significantly associated with modifications of the E&S design to address the increased geotechnical hazards arisen due to unpredictable geotechnical conditions.

Keywords: Bored Tunnels, NATM, Observational Method, Pilot Tunnel, Unreinforced Final Lining

1 Introduction

In the project alignment of the Motorway Maliakos – Kleidi 3 twin-tube tunnels are involved. They are designated to bypass the Tembi – Rapsani region (Tembi Valley) and the Platamonas – Skotina region. Since the general direction is South (Athens/Maliakos) to North (Thessaloniki/Kleidi), the branch going to Kleidi direction is denoted the North bound (NB) and the branch going to Maliakos direction is denoted the South Bound (SB). The portals, located in the Maliakos direction are named South Portals, whereas the portals located in the Kleidi direction are named North Portals.

According to the traffic & safety requirements the tunnels were designed with two traffic lanes of 3.75m width and an emergency lane of 2.50m width, in each traffic direction, while the crown of the final lining is at el +7.36 above the tunnel redline. The distance between the tunnel axes depends on the alignment, but it is not less than 25 m. Therefore the remaining rock mass pillar between the tubes has a minimum width of approx. 11.70m.

The safety concept for tunnels was the objective of a Risk Analysis study, which accounted for the current international Tunnel Safety Regulations and Project Specifications. In this concept the below listed requirements were considered:

- cross passages between both tubes in a general distance of ≤ 300m;

- cross overs between both tubes, designed as traffic bypasses each 900m (maximum distance) for service and rescue vehicles;
- emergency niches each 150 m on the right hand side;
- a closed road dewatering system with water intakes at the lower side of the carriageway connected with a main dewatering pipe and a separate basin outside the tunnels.
- All structural elements and components within the tunnel tubes have to have a fire resistance of 90 minutes.

A longitudinal ventilation system with jet fans is initiated for the operation phase, while for tunnel Tembi T2 three (3) smoke extraction points were also constructed to fit to the international regulations (requiring smoke extraction every 2km length, for tunnels with length > 3,000m).

Design & construction of the MMK tunnels follows the general principles of the NATM (New Austrian Tunnelling Method) whereby: the tunnels are designed as dual lining structures with an initial shotcrete lining, for the first and temporary support of the excavated tunnel and a permanent cast-in-situ concrete permanent lining for the final state (12.50m standard blocks of C30/37 either steel reinforced or un-reinforced). A geotextile fleece (600g/m^2) and waterproof HDPE membrane (>2mm) located at the vault, between primary lining and permanent, cast insitu inner lining, located in the vault and a drainage system at the bottom of both side walls protects the structure from water inflows.

In correspondence with the excavation and support types defined, generally three (3) different cross section types for the permanent lining were elaborated; one for the “open bottom” sections (the vault founded by isolated beam foundations, Type I) and two ones for sections equipped with the an inverted foundation (“closed bottom” sections, Types II, III). The geometry of the closed bottom section can be identified in Fig.1.

The Geotechnical Design of underground excavation & primary support was based on the following points: the Rock Mass Types, the Excavation and Support Classes and well defined Application Criteria, to link the anticipated Rock Mass Behaviour with the primary support classes.

To safely and efficiently address the variety of underground conditions from the most favourable to the most adverse ones, nine (9) support classes have been designed for the entire tunnel objective, named: 3A, 4A, 5A, 5B, 6A, 7A, 7B and 7C. These are grouped in open bottom sections (3A, 4A and 5A) and closed bottom ones (5B, 6A, 7A, 7B and 7C). For efficiency and better matching to the temporary as well as permanent stability requirements of the underground, two closed bottom sections were designated: one with deep invert foundation (7A, 7B and 7C) and one with a shallower invert geometry (to be employed in more favourable conditions 5B, 6A).

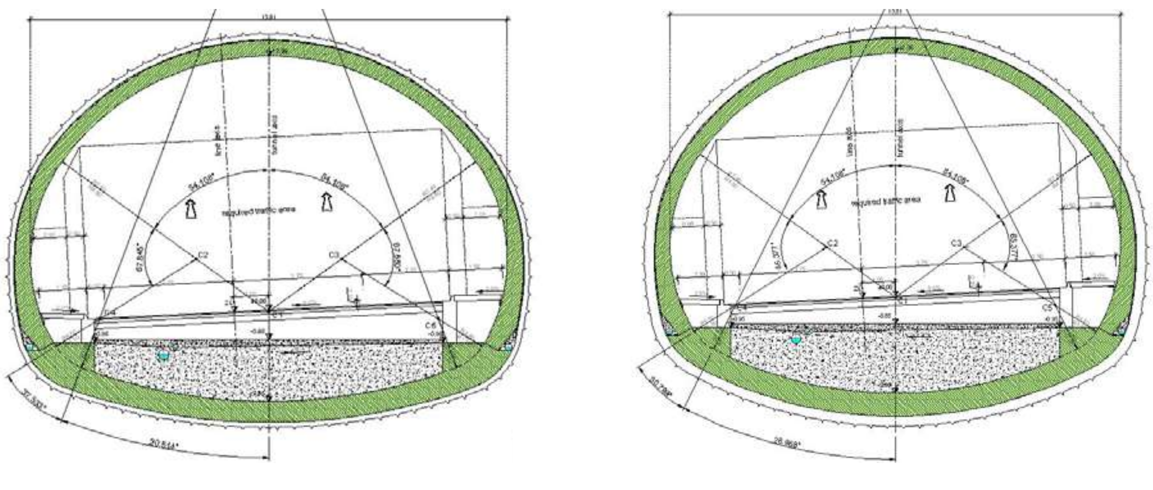

Figure 1. Typical Cross Sections (II and III)

The standard primary support elements consisted of: sprayed concrete, reinforced either with steel fibres or steel mesh, bolting (Swellex or fully grouted), steel sets (LGs, HEB160, HEB180). The suitability of the installed E&S class to the encountered underground conditions relies on the systematic instrumentation of the E&S sections in conjunction with established thresholds for the deformations and associated contingency actions.

To address the anticipated, increased likelihood of squeezing conditions, the key decision towards bringing the primary support into equilibrium was by allowance for adequate over-excavation to accommodate the predicted high convergence and use of dual primary support shells instead of finding recourse to a flexible initial support concept (such as by deformable members).

Special geotechnical conditions were encountered during the E&S of this 21.6km long underground project. The cases were related, among others to: mixed face conditions formed along the contacts of competent and the weak excavation geomaterials, need to accommodate significant straining phenomena over long stretches, very adverse stability conditions of the excavation face, boring through geomaterials of poor geomechanical properties either in shallow as well as deep overburden heights, squeezing conditions and delayed deformations, dealing with significant groundwater inflows. Construction also included the formation of complex underground spaces for the implementation of the tunnel ventilation facilities, in the long tunnel T2 (such as ventilation shafts, chambers and galleries) and application of cutting edge designs, such as the inner lining of unreinforced cast in-situ concrete. Unpredictable situations encountered led to modifications of the primary support system so that this can be adapted to the increased support requirements and successfully managing the geotechnical risks.

Certain key aspects of the NATM tunnelling to be presented herein regard: the methodology to effectively harness the systematic face instabilities of the wide excavation areas (by means of the “short pilot tunnel” concept), the variety of tunnel rehabilitation schemes developed and the application of the unreinforced final lining concept.

2 ASPECTS OF THE NATM TUNNELLING IN MMK TUNNELS

2.1 SPECIAL CONDITIONS IN TEMBI TUNNELS T3

Platamon tunnels (T3) were bored through the rock complex of the East Olympos Mountain, which is situated between the villages of Platamonas and Skotina. The tunnel underpasses a mountain ridge with several incised valleys. The geological structure of the tunnel area comprises basement rocks as well as overlying Quaternary soft rock formations, i.e. landslide deposits, alluvial deposits, scree and alluvium. The bedrock comprises an ophiolitic rock complex with serpentinized peridotites. These rock formations have been overthrusted by limestones and limestone-ophiolite breccias. The contact between limestones and serpentinized peridotites is in places marked by thick tectonic breccias. The dip direction of the rock formations is highly variable which is mainly caused by the intense internal tectonic displacements of the rock mass. The bedding and schistosity planes dip towards the W, NW, NE and towards the E with medium steep to steep inclination. The incised valleys are filled with alluvial deposits, which in places reach down to the tunnel level.

The maximum overburden of the tunnel in respect to the red line’s elevation is as high as 150m (ch.13+150), while the minimum overburden areas of the tunnel apart from the portal areas is 15m for both bounds (ch.12+020 and ch.14+970) where the tunnel underpasses incised valleys.

Conditions at ch.12+400 – Tunnel Rehabilitation Case. An abrupt, very severe cave-in incident occurred around the low overburden area at ch.12+400, which resulted in full collapse and blockage of the primary support in a 40m long section of the NB tunnel. Initially, the violent tunnel instability phenomena in the NB tunnel manifested at significantly strained sections well behind the excavation face (at ch.12+420), and subsequently propagated uncontrolled in front of as well as backwards, thus affecting a 40m long tunnel section, in total. The cave-in progressed up to the ground level, creating a small crater at the ground surface along with a much wider subsidence area (with retrogressive surface ruptures at the ground surface) and disclosing a small part of the Natural Gas Pipeline running at small depth below the ground surface. The detrimental influence of the disturbance in the masses surrounding the NB had spread to the SB tunnel as well. However, the immediate contingency efforts which were effected, by the Contractor in the SB tunnel (by means of tunnel soil backfilling) saved the other bore from full collapse. At the moment of incident both tunnels have been advancing in top heading section through serpentinized peridotites featured with quite inferior geotechnical characteristics (GSI<15).

Design of the rehabilitation strategy for the blocked NB tunnel, relied on an extensive site investigation of the narrow area around the tunnels. Safe passage through the NB blocked with collapsed, loose peridotites presented a very demanding task, because of the restrictions additionally set by the involvement of the Nat. Gas Pipeline with the collapse area. The solution sought, rejected other options relevant to the improvement of the surrounding mass (because of the ground morphology as well as the intrinsic permeability properties of the affected masses).

Advancement of a pilot tunnel to pass all the way through the 40m collapsed materials was finally promoted to be first key phase of a two phases' construction. The second rehabilitation phase regarded the demolition of the pilot tunnel structure and the re-instatement of tunnel cross section. Demolition of the pilot structure was followed so that the space created proves wide enough to accommodate the induced convergences and an adequately thick final lining structure. Both phases were constructed under continuous protection of overlapping forepoling umbrellas (Φ114/140). Pilot tunnel design was supported by a very competent composite shell so dimensioned to deal with quite conservative assumptions for the anticipated dead weights. E&S for the pilot was designed to proceed in two phases, that is section excavation followed by an invert closure. It was formed by sprayed concrete (35cm thick) and embedded steel ribs HEB160, which were placed at 0.80m centres below the 12m long forepoling canopy. No bolting was adopted, while the invert was reinforced with LG140/30/200 and 2 layers of #12/15 steel grids. Representative section is displayed in Fig.2a.

Widening of the section to the normal tunnel dimensions with parallel demolition of the pilot tunnel was relied on the design of a stiff initial lining 50cm in thickness at the top heading (by implementation of a dual primary support lining), 30cm in the sidewalls and 40cm in the invert. Bench and invert closure were applied following successive 8m advances of the top heading drift.

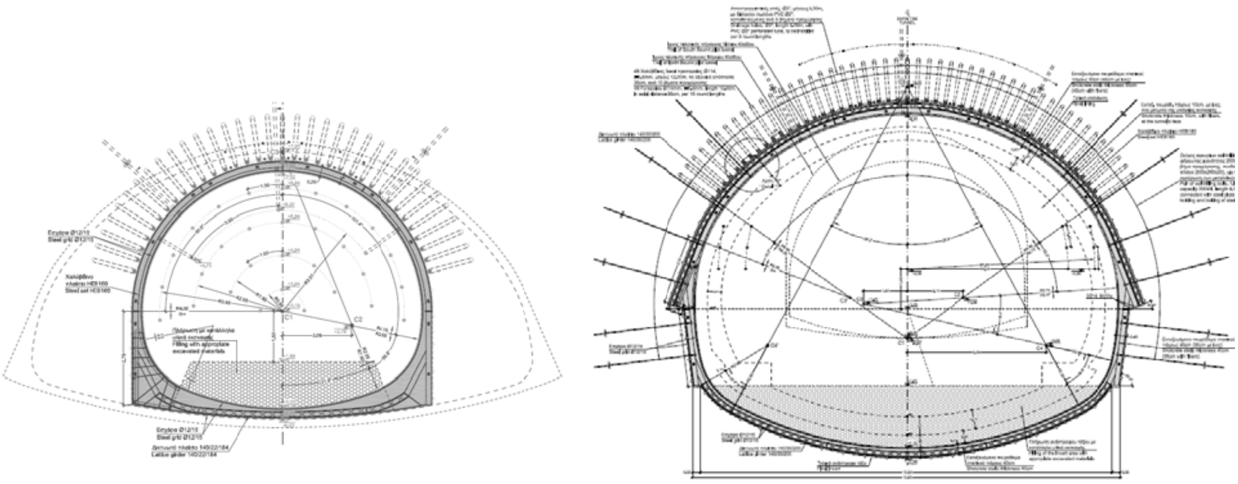

Figure 2. Tunnel T3 – Rehabilitation sections at ch.12+400 – a. pilot tunnel phase; b. widening phase

Driving of the pilot tunnel (Fig.3a) started 18 months after the tunnel collapse incident and took 3 working months to get through the collapsed zone.

Figure 3. Tunnel T3 – Rehabilitation Works at ch.12+400 – a. pilot tunnel phase; b. widening phase

The rehabilitation objective proceeded in close cooperation with the Nat. Gas Pipe Authority (DESFA) and had as prerequisite the disclosure of the pipeline for the length, under consideration to facilitate inspection of the facility over the entire rehabilitation phase. Very strict thresholds for the deformations monitored in the underground as well as at the ground surface completed the framework of the design.

Successful application of the demolition of the pilot tunnel and building the widening of the tunnel cross section (Fig.2b and Fig.3b) have been put on track after the installation of the inner concrete lining at the adjacent sections in the SB tunnel and was completed within a 3 months time period, in November 2013.

Design anticipations were fully verified by the recorded very limited deformations in the underground (4.5cm) as well as on the ground surface (2cm).

Conditions at ch.13+500. The case regards the rehabilitation along a 45m section in the NB tunnel (ch.13+510 to ch.13+465), which so severely converged that the tolerance for placement of the Final Lining was overly eliminated. It is an underground stretch under medium overburden heights (H≈70m), excavated in serpentinized peridotites of quite inferior geomechanical properties (resembling soil-like materials, GSI~15) with erratically floating megablocks of limestones and peridotites.

Strikingly different modes of deformation were recorded along the stretch in question, as evidenced from the recorded vertical deformation vs. time histories in Fig.4. The left hand side plot, representing uniform sinking of the sections is associated with homogeneous excavation conditions of the tunnel through the soil-like serpentines, while in the plot next to this the influence of the strong non-homogeneities in of top heading foundation is reflected; in the former case no overstressing was caused to the shell in contrast to the latter one, in which the initial support was severely impacted. Also interesting to note is the time for the evolution of deformation in the representative, different sections, in the two plots (early deformation versus delayed straining response).

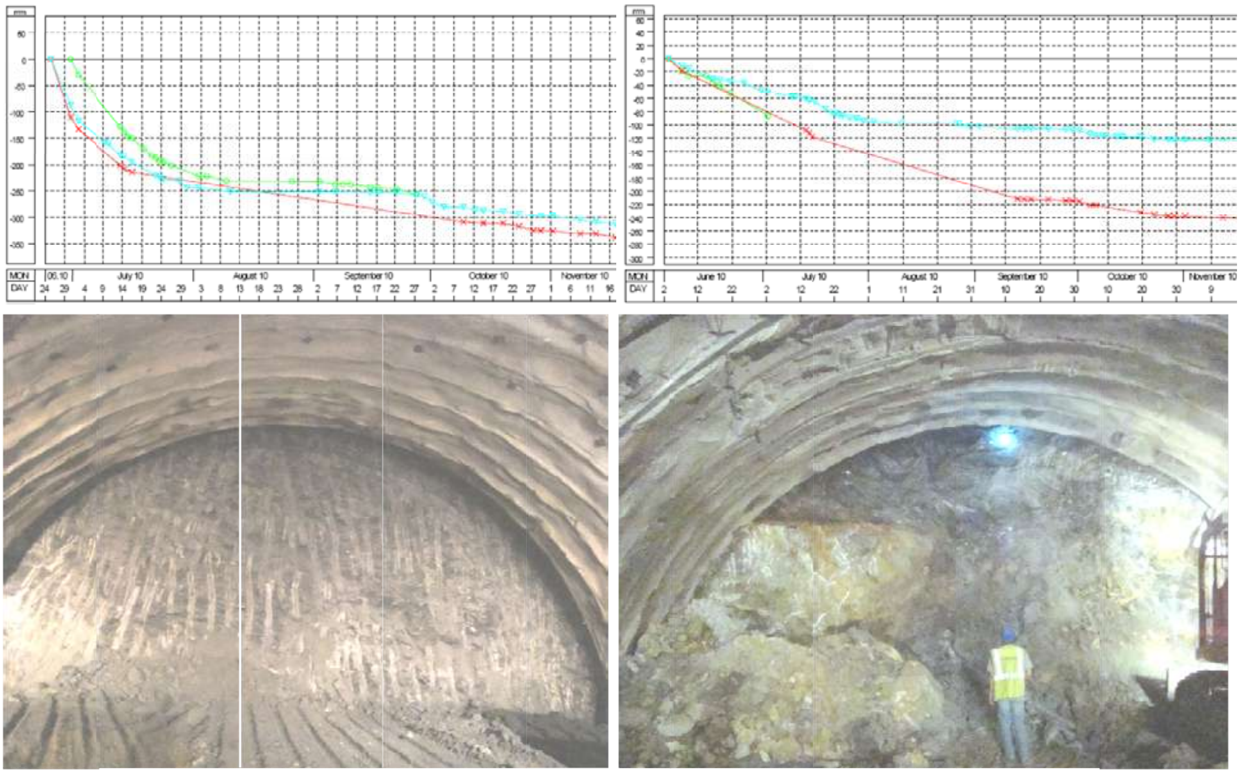

Figure 4. Tunnel T3 (ch.13+500). Different vertical displacement vs time histories in weak serpentines

This hazardous convergence tendency turned out impossible to deter by effecting conventional contingency actions (such as the installation of temporary invert, additional stiffening shell of reinforced sprayed concrete, bolting or micropiles etc). The cavity was kept operational by use of inner stiffening rings (sprayed concrete with HEB160), in combination with installation of pre-stressed anchors (18m long, 40tons pre-stress load), which were proved the most effective measure, in the particular case. Eventually, the unfavourable interaction between the shell and the surrounding masses has led the 45m long section top heading section to deform as much as 50cm.

Rehabilitation was carried out by designing for a special E&S class to fit to the particularities of the situation. The essentials of the rehabilitation scheme regarded the staged demolition of the original primary support shell, in the affected top heading tunnel sections, re-mining to re-instate the necessary tolerance for the permanent lining structure and the implementation of a significantly robust, dual primary support shell. The double primary support shell concept comprised:

- an outer primary shell (a composite sprayed concrete lining, 20cm in thickness with rectangular LG14/30/200 per 0.50m, which is completed 3 rounds behind the face of excavation), which is supplemented by
- an inner primary shell (a composite sprayed concrete lining 30cm in thickness with HEB180 arches per 1.0m rounds);

Actually, the double support was configured to be applied in the top heading section, aiming to obtain a robust shell (50cm) for the most vulnerable part of the initial support section, reliant on the findings from the structural design calculations and the conclusions drawn from actual behaviour of the original excavation. In this configuration, the inner component of the initial lining is continuous, i.e. it is applied all around the opening (bench and invert) with average thickness of 30cm. In this special class, completion of the initial support section by bench and invert closure (reinforced with LG70/22/32) was foreseen so as to close the heading drift of the rehabilitation works as quick as possible, and result in more favourable distribution of the stress field around the cavity. Apart from the regular bolting, stability of the top heading was enhanced by installation of four pre-stressed anchors (18m long, 40tons pre-stress load), per section in sections spaced 2m apart.

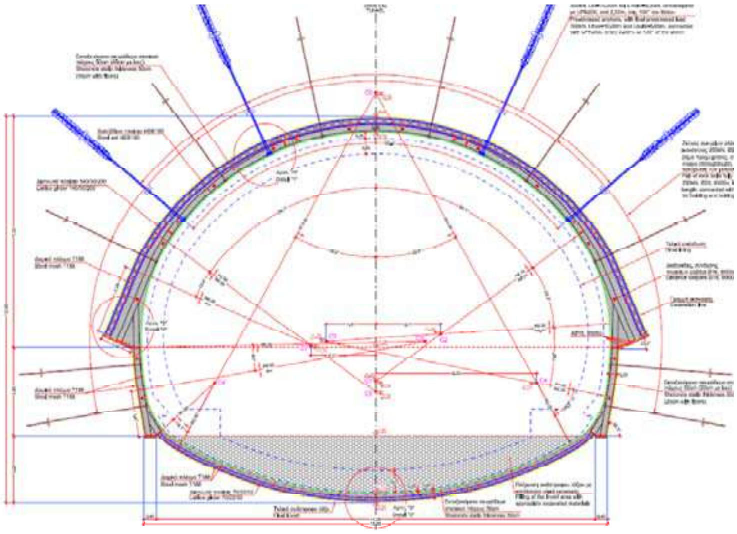

Figure 5. Tunnel T3 – Rehabilitation Section at ch.13+500

Systematic application of the design requirements of the special E&S support class resulted in successful rehabilitation of the damaged sections and establishment of stable conditions for application of the appropriate, permanent concrete lining.

2.2 SPECIAL CONDITIONS IN TEMBI TUNNELS T2

The two-branch tunnel Tembi 2 was constructed in the metamorphic rock complex of the Ossa mountain formation. The tunnel underpasses the main mountainous mass at the South side of the gorge, featuring steeply inclined rock flanks towards the Tembi Valley. Numerous deeply incised valleys and gorges cut the mountain in high angles to the Pinios river. The gorges are mainly fault controlled. The geological structure of the tunnel area comprises the geological basement and the overlying Quaternary soil formations (i.e. scree and talus cone deposits). The geological bedrock comprises the low-grade metamorphic rocks of the Ossa Unit consisting of crystalline limestones, with intercalated phyllites together with thin limestone intercalations. The crystalline limestones and phyllites are generally dipping towards the W to NNW, with dip angles between 10° and 30°.In places, the metamorphic rocks are overlain by soil-like rocks of Quaternary age. In the middle section of the tunnel stretch, cemented talus cone deposits (limestone breccias) overlying the crystalline limestones reach thicknesses of approx. 150m were encountered. The tunnel stretch meets two incised valleys filled with rock debris and talus cone deposits.

The maximum overburden of the tunnel in respect to the red line's elevation is 295m for both bounds (ch.8+400), while the minimum overburden of the tunnel alignment, apart from the portals entrance areas is 22m (ch.7+470).

Conditions at ch.10+730 – The "short pilot tunnel" method. Hazardous conditions evolved on tunnels boring through a 50m long zone of shattered phyllites (GSI~10) from the North Portals, under medium overburden heights (70m). The zone was represented by grey coloured geomaterials, heavily tectonized (completely fractured and sheared), exhibiting soil behaviour under an adverse stress regime in conjunction with a fully unfavourable orientation to the excavation, of the quasi-schistosity planes.

In essence, the triggered enormous face instabilities on advancing the tunnels excavation (both in top heading drift) caused inability to timely implement the support measures (bolting and temporary invert) as foreseen in the design. This delay turned out responsible for the poor performance of the initial support shell, by means of early development of high convergences of the shell, immediately behind the face (ranged from 20 to 40cm), strong evidence of interaction between the two bores, which were associated with evolution of overstress phenomena in the shell had led to cracking of the temporary invert. The case demonstrated the need to seek for more effective methods to control the stability of the wide excavation faces of the three lane tunnels, rather than the standard dense fibreglass rock bolting.

The management of the uncomfortable situation opted for: a) the immediate stabilization of the significantly stressed cavity along with b) the improvement of face stability conditions that would minimize the disturbance of the mass during excavation and ensure implementation of the support measures as close as possible to the face.

Accomplishment of the former (a) objective, that is the stabilization of the deforming top heading cavity in both bores was designed by adopting an inner stiffening shell 35cm in thickness (of sprayed concrete, heavily reinforced with 2 layers of #12mm/15cm steel grid) which was gradually applied all around the affected top heading section from the invert upwards. The contingency, additional shotcrete shell maintained stability of the top heading section for more than one (1) year time, by the time the section was scheduled to be rehabilitated; whereby the stressed shell was replaced by a new, competent initial support.

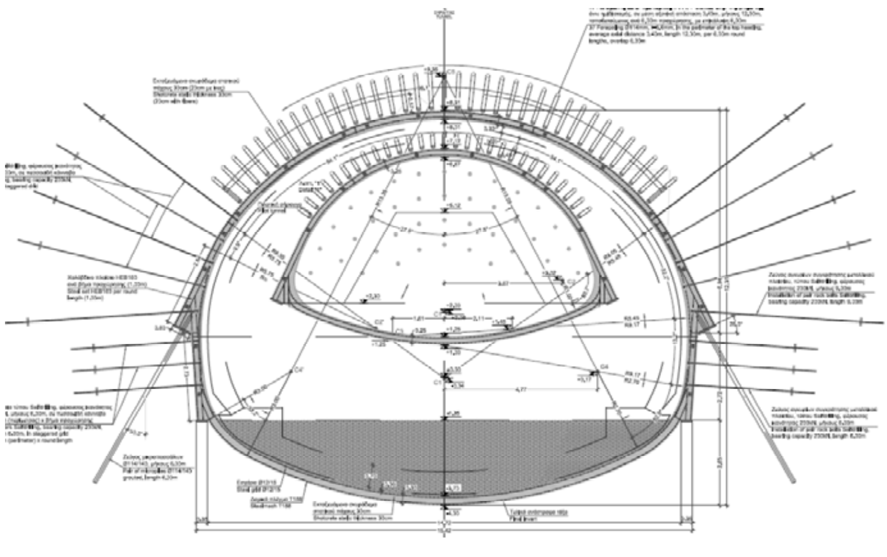

Figure 6. Layout Configuration and geometry of the "short pilot tunnel".

In addition, a novel method was devised so as to provide pre-reinforcement and increased confinement conditions of the extremely weak and unstable masses ahead of the face, called the "short pilot tunnel" method. With this method, in lieu of dividing the excavation by means of side drifts, the face was stabilized by a forepoling (Φ114/140) protected pilot tunnel (dimensions 6.12m x 10.80m), excavated at the core of the face and presenting a standard, maximum 5m advance ahead of the main tunnel.

The "short pilot tunnel" method proved a very efficient way to negotiate the hazard of face instabilities by deploying the same equipment used for the main tunnel excavation and support. And thereby, it was properly compiled to a new E&S class, which was appointed to address similar geotechnical hazards which were evolved very frequently along the 21.6km tunnel driving of the MMK tunnels.

Figure 7. Tunnel T2 - Stiffening Inner shotcrete shell in top heading – "Short pilot tunnel" application

2.3 Concept of Unreinforced Final Lining

The implementation of non-reinforced final lining possesses the state of the art to the latest structural safety standards of tunnelling in Europe (Directive 2004/54).

The contractual documents of the MMK project (SCC, TCC) provided the contractual basis for the design of the unreinforced concrete permanent lining for the three tunnels. Of the three main types designated for the permanent lining, the foundation (either as foundation beams or invert) was designed to be standard steel reinforced, while the vault could be either reinforced or non-reinforced.

Concrete quality for the permanent lining is C30/37, while for fire protection an increase in cover of the steel reinforcement (6cm) was considered for the steel reinforced sections. The principles for decision making on the appropriate Final Lining type were based on proper determination of the rockmass conditions in combination with the response of the initial lining (convergence monitoring). Re-confirmation of the appropriate final lining type or elaboration of special concrete final lining sections both proved necessary, during the progress of the project so as to cover the special underground conditions encountered.

In principle, the intersection blocks of the main tunnel with the escape crossings, the crossings as well as the complex underground spaces were designed as steel reinforced, while the standard blocks of the main tunnel bores were designed either reinforced or unreinforced. Of the unreinforced blocks of the main tunnels, in those with a niche (Emergency or Drainage) the reinforcement was placed only at the niche area. Of the three main final lining types, the concept of unreinforced vault was designated for the open section type (I) and under certain conditions for the shallow invert section (II), with thickness 0.45m at the crown. In principle, application was adopted in strong and adequately competent rock-masses (rockmass E>1GPa considered to be the design threshold value for application of an unreinforced permanent lining). Instead, the main tunnel sections surrounded by questionable as well as poor rock and / or sections with acknowledged vulnerability to seismic forces (such the portals and the low overburden areas) were virtually excluded from application. At certain cases dilatometer testing was conducted for the validation of the prevailing Young modulus of the rockmass. Design basis for the adequacy checks of the concrete section was based on the requirement for maximum crack width of 1.0mm, while the calculated crack width evolution should be restricted to less than one half of the section.

Formwork removal of the vault was allowed at minimum compressive strength of 2MPa, alongside construction anticipations towards specific curing process of the fresh concrete upon the formwork removal (and avoidance of undesirable cracking).

The application of the concept included mainly tunnels T1 (1.46km out of the 3.85km) and T2 (3.37km out of 11.94km), with a broader application generally restricted by the inherent geometry of the vault (to accommodate the three lanes' traffic requirements) as well as the unfavourable geotechnical conditions (weak masses and prevalent mixed face conditions for long stretches). Notably, none section in the 5.58 km of twin tunnels T3 was casted as non-reinforced.

Below displayed are the layout of unreinforced Final Lining section for Type II and the application of Type I for the drainage block.

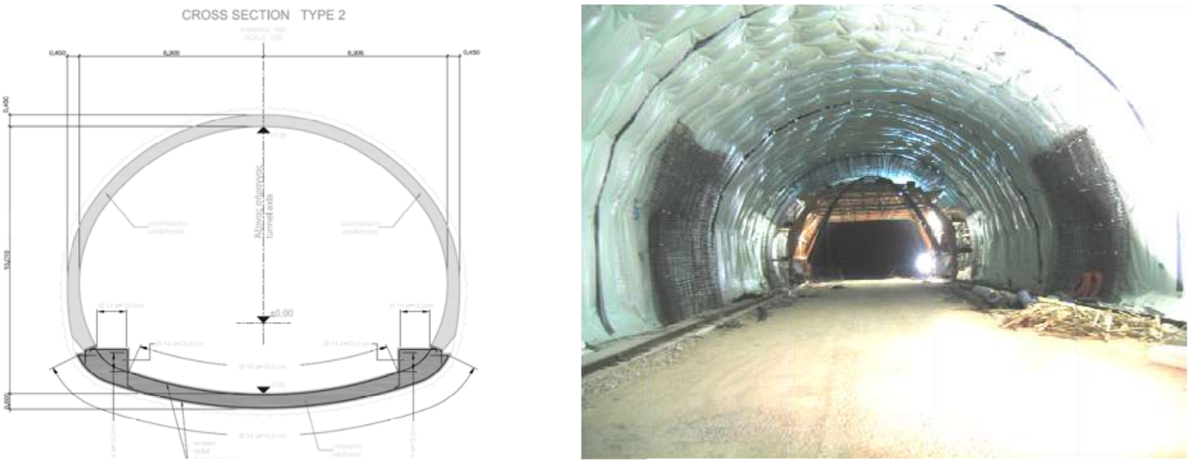

Figure 8. Unreinforced Final Lining for Type II and reinforcement at drainage niche for unreinforced type I

3 Conclusions

The tunnels of the Motorway Maliakos-Kleidi (MMK) constitute a challenging application of the NATM under demanding construction requirements as posed by the nature of the project (BOT), the tunnels section dimensions (three lanes section) and the total length of underground construction, in conjunction with the involved geomaterials and the anticipated geotechnical hazards.

Very characteristic aspects of the underground construction are presented herein, so as to give an insight into the associated geotechnical implications in the excavation and support (E&S) construction set by the unpredictable geotechnical situations and involved solutions adopted. The cases presented, involve tunnelling in very weak geomaterials, where the Designer had to tackle with unprecedented instabilities in the cavity as well as at the excavation face, significantly strained sections, and demanding rehabilitation tasks. In addition, a special section is devoted to the first broad application of the concept of the unreinforced permanent lining, in modern motorway tunnelling, in Greece.

With regard to the NATM underground E&S of such wide underground sections in weak and sheared geomaterials, the very significant experience from the MMK tunnels demonstrated that:

- Tunnelling through weak ground masses is linked with significant face instabilities for which the “short pilot tunnel” method is a viable solution to control compared with dense face bolting;
- By virtue, the wide top heading sections prove quite unfavourable in terms of the attracted rock loading, which results in advent of early deformations; evolution of these deformations can be hardly addressed by the standard techniques designated for the normal tunnel sections (such as temporary invert, use of micropiles etc);
- Implementation of independent drifts (top heading, bench and invert) for section excavation does not prove very effective in the aim to result in favourable loads redistribution around the cavity, unless the bench and invert drifts are kept as close as possible to the advancing top heading.
- Stabilization of the cavity in relatively high overburden employed the “stiff initial support concept” which was implemented by anticipation of proper over-excavation in conjunction with application of successive initial linings to secure stability of the cavity; Application of this concept was based on safe prediction of the required tolerance and relied on the evaluation of results of systematic convergence instrumentation (principles of the observational method).

The close collaboration between the Designers’ Consortium and the Construction JV resulted in quick design adaptations of the E&S methodology and processing of efficient solutions to successfully address the increased geotechnical hazards arisen due to unpredictable geotechnical conditions.

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