The tunnels of Egnatia Highway. Design in a variety of rock masses under difficult geological conditions

Οι σήραγγες της Εγνατίας Οδού. Σχεδιασμός σε ποικιλία από βραχόμαζες κάτω από δύσκολες γεωλογικές συνθήκες

MARINOS P. Professor, N.T.U.A., School of Civil Engineering, Geotechnical Department
HOEK E. Consultant, Evert Hoek Consulting Engineer Inc., Canada
KAZILIS N. Former Tunnels' Disc. Head, Egnatia Odos S.A. Now Manager Geodata Greece
AGISTALIS G. Tunnels' Discipline, Design Dep., Egnatia Odos A.E., Thessaloniki
RAHANIOTIS N. Head of Tunnel Support Team, Egnatia Odos A.E., Thessaloniki
MARINOS V. Post Graduate Student, N.T.U.A., School of Civil Engineering, Geotechnical Dep.

ABSTRACT: The Egnatia highway with a total of 73 tunnels runs across the entire width of Greece. Thus, there is a great variety of geological situations that impose different approaches in the design. Such knowledge was used initially in the final choice of alignments to avoid, wherever possible, areas of old large landslides. In turn, for the chosen alignment, the emphasis was placed on defining the rock mass model as accurately as possible. Tunnelling conditions range from straightforward to extremely difficult. Design methods vary from simple, to complex, usually requiring numerical analysis. The method of excavation involves large faces (top heading and bench) with a temporary or a final invert in difficult ground. Face instability is faced mainly by a forepole umbrella and fibre-glass anchors.

ΠΕΡΙΛΗΨΗ: Η Εγνατία Οδός, με ένα σύνολο 73 σηράγγων διατρέχει όλο το εύρος της Β. Ελλάδος και συνεπώς μια μεγάλη ποικιλία γεωλογικών καταστάσεων που επιβάλουν διαφορετικούς κατά περίπτωση σχεδιασμούς. Η γνώση τους χρησίμευσε στην τελική επιλογή, όπου είναι δυνατόν, της χάραξης προκειμένου να αποφευχθούν περιοχές με μεγάλες κατολισθήσεις και, μετά, στον ακριβή προσδιορισμό του μοντέλου της βραχόμαζας στις θέσεις των τεχνικών έργων. Οι συνθήκες κατασκευής των σηράγγων ποικίλουν από απλές ως ιδιαίτερα δύσκολες στις περιοχές έντονου τεκτονισμού· ανάλογα και οι μέθοδοι σχεδιασμού. Η κατασκευή εφαρμόζει ευεξία μέτωπα με προσωρινό ή μόνιμο δάπεδο στις δύσκολες καταστάσεις. Η ευστάθεια του μετώπου εξασφαλίζεται συνήθως με ομπρέλα δοκών προπορείας ή αγκύρια μετώπου.

1. INTRODUCTION

In Roman times, during the 2nd century B.C, Via Egnatia was the first highway built by the Romans outside Italy (Gaius Ignatius) and the first to cross the Balkan peninsula from the Adriatic Sea to the west to Marmara and Black Sea to the east. “Egnatia Odos S.A.” is the government company to design, build and operate this Motorway.

Today Egnatia Motorway is a priority European Union project and a high standard road axis that runs with a West-East direction through northern Greece. Being 680Km long, it constitutes an essential development lever for all areas it crosses. Along with its 9 vertical axes, it opens up real prospects and changes the way of living in several areas in Greece and in the whole South East Europe. The motorway runs through an extremely diverse natural morphology of great beauty.

The main axis is 50% co-funded by the European Union. When complete the Egnatia Motorway will have a total of 73 road tunnels of an overall combined length of approximately 100Km. Their construction cost, including all E & M systems, amounts to 35% of the total estimated construction cost. The majority are twin bored tunnels, while few of them were constructed by the cut and cover method. Fifteen of these tunnels are classed as long with lengths from 800 to 4.600m. Most of them
have already been excavated and this has contributed to the development of a high level of tunnelling expertise.

Figure 1. Alignment of Egnatia Highway and spatial setting of the major geological sectors along with the geotechnically weak conditions.

Σχήμα 1. Η χάραξη της Εγνατίας Οδού και η παρουσία γεωτεχνικά ασθενών σχηματισμών.

2. THE GEOLOGICAL ENVIRONMENT

2.1 Basic principles

From west to east, the Egnatia Highway can be subdivided into the following parts:

- From Igoumenitisa to Metsovitikos River. Ionian geotectonic unit (part 1 in Fig. 1 and 2)
  Flysch and alternations of various carbonate formations, mainly limestone, with very limited occurrence of cherts and siliciferous shales, are the typical rock masses. Local occurrences of gypsum in diapiric intrusions can be also encountered. The rocks are folded while large scale overthrusts; big faults and mylonitized zones are present in this region (Fig. 2).
  - From the Metsovitikos River to Metsovo tunnel. Pindos geotectonic unit (part 2 in Fig. 1, 2)
    The area crossed consists mainly of flysch in various forms, characterized by intense folding, heavily sheared with numerous overthrusts. The degree of the tectonic deformation at some places drastically degrades the quality of the rock mass.
  - From Metsovo tunnel to Panagia region. Nappe of Pindos ophiolites (part 2 in Fig. 1, 2)
    Ophiolites comprise the predominant rock mass in this area, however they exhibit great heterogeneity regarding their degree of serpentinisation and the occurrence of shear zones with tectonic melanges. Weak flysch depressed by this ophiolitic nappe, is also present.
  - From Panagia to Siatista. The molassic domain (part 3 in Fig. 1)
    This region consists of molassic formations in the form of alternating thick-bedded conglomerates, sandstones and siltstones-clay stones. From a tectonic point of view, the area is of low disturbance and although weak rock masses may be present in places, there is not any dramatic decrease of geotechnical qualities due to the absence of significant tectonic shearing.
  - From Siatista to Lefkopetra. Pelagonian geotectonic unit (part 4 in Fig. 1)
    The area is characterized by the predominance of hard rocks such as marbles, gneisses and granites. The presence of tectonically weakened zones through faulting is very localized.

Figure 2. Schematic cross-section of the Hellenic Alps (Aubouin, 1979, in Papanikolaou 2003).

Σχήμα 2. Σχηματική εγκάρσια τομή των Ελληνίδων (Aubouin, 1979, in Papanikolaou 2003).
From Lefkopetra to Veria. Axios to Almopia geotectonic units (part 4 in Fig.1)

Phyllites, limestones and ophiolites are the usual rock masses in this area, while overthrusts and sheared zones are the main tectonic structures.

From Aliakmon River to Axios River flood plane to Thessaloniki region.

The entire area consists of recent alluvial fill which often exhibit insufficient natural compaction.

Section east of Thessaloniki to the Turkish border.

The Serb-Macedonian massif and the Rhodope massif comprise the region. The basement consists of hard crystalline marbles, gneisses and granites. At some localities, the latter two appear weathered and are locally crosscut by faults with sheared zones within the rock mass. The Egnatia Highway passes also through areas of younger sediments such as marls and sandstones and areas of recent geological deposits with soft soils of loose or open structure.

2.2 Tectonically active areas

Although areas of tectonic activity do not exhibit the same dynamic characteristics in the northern part of the Hellenic region, as is the case for central and southern Greece, the Egnatia Highway crosscuts active tectonic depressions such as the Volvi – Lagkadas trench or the Grevena – Kozani trench as well as potentially active structures at the western part of the alignment (the area of Paramithia has potential for strike slip activity).

3. THE GEOTECHNICALLY DIFFICULT CASES

In many cases the situations mentioned above result in very difficult engineering conditions that necessitated specific investigations and the use of sophisticated design methods. The ignorance of situations yielding such conditions could lead, at least, to delays and also to failures. These failures could develop not only during the course of construction, when engineering solutions can generally be found, but also in the operational stage of the Highway. In many cases, the early detection of potential problems justified drastic changes of the initial alignment, when the cost was not prohibitive and the operational safety was not jeopardized.

Along the route of the Egnatia Highway, the difficult geotechnical cases are classified as follows:

A. Areas with entire slope scale instability (fig. 3). The spatial distribution of the various geologic formations and geotectonic units has yielded some geomorphologic features whose final arrangement creates conditions of generalized slope instability on mountain slope scale. This fact is particularly significant in western Greece and in the Pindos mountain range. For instance, masses of competent formations (limestones) have overthrust the soft and “ductile” flysch. The former comprise the highest parts of the valley sides forming high cliffs which produce scree covering the flysch of the lower parts which are deformed and sheared due to the overthrusting. Furthermore, the scree allows water to percolate and dampen the geotechnically poor mass of flysch. Earthflows as well as old and new landslides are typical phenomena, often on impressive scales on such slopes. Additionally, the development of this instability of the flysch undermines the limestone crests of the slopes provoking further falls of limestone masses which further weaken the flysch slopes downhill. Such unstable areas were avoided where possible by a realignment of the highway to the opposite valley side, often with the choice of tunnels and high bridges.

B. Areas of tunnel constructions consisting of tectonised and generally weak rock masses or rock masses crosscut by sheared zones. Design, after selection of the driving parameters, required the use of advanced methods. It has been found that a reliable initial estimate of potential problems can be given by the ratio of rock mass strength to in situ stress (Hoek and Marinos, 2000). This is usually followed by a detailed numerical analysis of the tunnel’s response to sequential excavation and support stages. However, in some cases of extremely weak rock masses, the capability for theoretical analysis is limited and here sound technical judgment and experience from similar cases are valuable aids in design. In order to cope with face stability and high squeezing deformation, the use of heavy support with application of forepole umbrellas were applied while the use of yielding shells were an alternative solution.
Figure 3. Drive of the highway through the Anthohori - Anilio section in Metsovo area: initial alignment (A) was selected through a mild topography in the Southern slopes. However, this morphology is due to a series of unstable old landslides in flysch; and extensive drainage including drainage galleries of 5km of length was proposed. Solution: relocation of alignment at the northern steep but stable slopes of Metsovitikos River (B) with tunnels alternating with bridges. (drawing not to scale).

Σχήμα 3. Πέρασμα του αυτοκινητοδρόμου από τη περιοχή του Μετσόβου (Ανθοχώρι - Ανήλιο). Αρχική χάραξη (A): επιλογή λόγω ήπιας μορφολογίας. Εντούτοις η μορφολογία αυτή οφείλεται σε σειρά μη σταθεροποιημένων παλαιών κατολισθήσεων σε φλύσχη. Λύση: μετατόπιση της χάραξης στις βόρειες, απότομες, σταθερές, κλιτές του Μετσοβιτικού (B) με σήραγγες και γέφυρες κλιτών (σκίτσο χωρίς κλίμακα).

4. TUNNELLING THROUGH THE VARIOUS GEOLOGICAL ROCKMASSES

The ground conditions met by the Egnatia Motorway tunnels and the consequent response in tunnelling are summarised in table 1.

Wide tectonic zones produced by big thrusts and satellite shears within the thrusted body are very difficult to assess as far as the conceptual model is concerned (fig. 4).

Due to the geometrical complexity and heterogeneity it is difficult precisely to specify the occurrence of zones of weak rock masses and a conservative support section together with a strict construction schedule is the proper solution to apply.

Table 1. Ground types and conditions and their response in tunnelling

<table>
<thead>
<tr>
<th>Rock Type and conditions</th>
<th>Response in tunnelling and stability concepts</th>
</tr>
</thead>
<tbody>
<tr>
<td>Massive or bedded limestones. Marbles</td>
<td>Simple tunnelling conditions. Few stability problems, gravity driven</td>
</tr>
<tr>
<td>Filled karstic voids</td>
<td>Risk of collapses. Probing ahead and umbrella arch crossing.</td>
</tr>
<tr>
<td>Sandstone flysch</td>
<td>Gravity driven structurally dependent instability in low stress environment. Stress dependant when strength to stress ratio is low.</td>
</tr>
<tr>
<td>Siltstone flysch and shales</td>
<td>Stress dependent instability. Control of deformation is the key action. Closed shell is necessary. Minor face instability</td>
</tr>
<tr>
<td>Sheared and chaotic flysch</td>
<td>Squeezing conditions and face instability problems in depth (e.g more than 100m). Control of deformations is the key action. Yielding support could be considered at great depth</td>
</tr>
<tr>
<td>Sound ophiolites (peridotites and gabbros)</td>
<td>Structurally dependant instability, more severe when discontinuities are serpentinised. Block size normally irregular needs conservative support approach</td>
</tr>
<tr>
<td>Sheared serpentinites and ophiolitic melanges</td>
<td>Squeezing conditions at depth. Control of deformations is the key action</td>
</tr>
<tr>
<td>Molasses</td>
<td>Simple tunnelling conditions. Gravity driven instability under low stress. Under confined conditions brittle failure can occur in high stress environments. Weak geotechnical conditions in the weathered surface layers, slope stability issues in portals</td>
</tr>
<tr>
<td>Gneiss, schists</td>
<td>Simple tunnelling conditions if not heavily tectonized and/or weathered. Structurally dependant instability.</td>
</tr>
<tr>
<td>Phyllites</td>
<td>Weak rock tunnelling. Deformation problems in cases of deep tunnels.</td>
</tr>
<tr>
<td>Tectonic breccia in brittle rocks kataclasites</td>
<td>Ravelling. Maintaining confinement is the key action in low stress environment</td>
</tr>
</tbody>
</table>
5. THE DESIGN

5.1 Design parameters and methodology

Egnatia Odos A.E. has developed tunnel design guidelines that cover all the aspects of tunnel design. Particularly, for the design of the excavation and temporary support, the design steps described in table 2 are proposed (Kazilis N & Aggistalis G., 2005)

The appropriate use of rock mass characterization systems, notably the Geological Strength Index (for details Marinos P. and Hoek E., 2000 and Marinos V. et al. 2004), allowed the quantification of difficult ground for the evaluation of the geotechnical properties and the selection of the design parameters. It needs to be noted that the application of the mass properties from the GSI values essentially assumes the rock mass to behave isotropically which is appropriate for weak sheared masses. It is not applied where there is a clear anisotropic behaviour e.g. clearly defined preferred failure orientations guided by persisting discontinuities.

Table 2 refers only to the design methodology for the dimensioning of the support measures.

Before this stage, the preparation of a detailed joint geological/ geotechnical interpretative report, based on the ground investigation program, is required.

<table>
<thead>
<tr>
<th>Stages</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Assessment of the strength and deformability of the geomaterials along the tunnel. Use of acceptable systems of rockmass characterisation.</td>
</tr>
<tr>
<td>2</td>
<td>Calculation of deformations and plastic zones along the tunnel with no support measures. Use of analytical equations taking into consideration the stress field and the rockmass strength.</td>
</tr>
<tr>
<td>3</td>
<td>Preliminary selection of support classes based on experience and empirical methods.</td>
</tr>
<tr>
<td>4</td>
<td>Calculation of deformations and plastic zones along the tunnel considering support measures. Use of analytical equations taking into consideration the stress field, the rockmass strength, and the support pressure provided per support class.</td>
</tr>
<tr>
<td>5</td>
<td>Identification of problems. High deformations, face instabilities, etc.</td>
</tr>
<tr>
<td>6</td>
<td>Use of numerical analyses to check, confirm and finalize the support classes based on the anticipated failure mode. Use of numerical software e.g. UDEC, SOFISTIC, FLAC, PHASE2.</td>
</tr>
</tbody>
</table>

Structurally dependent instability analysis is considered in cases of strong rock masses at shallow depth.
Part of this report is the identification and description of the engineering geological units along the tunnel. Very important is the identification of engineering geological difficulties and of the specific geological problems that may be present and of those geological characteristics along the tunnel that will play an important role, or control the excavation of the tunnel.

Sometimes the description of these particular geological characteristics of the geological formations are beyond the scope of the well known rock mass classification systems, but they play the most important role on the tunnel design philosophy and on the tunnel construction.

For example, probing ahead and pre-reinforcement elements were implemented to excavate through filled karstic cavities in Dodoni tunnel (figure 5). Also, in S3 tunnel, at Lefkopetra, west of Veria, in the case of a loose phyllitic rock mass with numerous open voids, grouting was adopted to increase its strength around the tunnel (figure 6, 7, 8). In these two cases, the key engineering geological characteristics of the ground that significantly influences the design and construction of the tunnels, were the presence of the karst voids (or the cave-in potential) in the first case, and the open loose structure of the rock mass in the other case.

Another example is loose brecciated rockmasses with RQD=0. However, these rockmasses exhibit a good frictional strength and can be efficiently faced in tunneling if confinement is maintained.

The tunnels are large, with spans of 12 m or face areas of almost 110m$^2$. Except in areas of good rock masses, most of these tunnels are in difficult geological conditions and have been excavated using top heading and bench methods for security. Even with the reduced face areas of top heading excavation, additional measures have had to be taken to stabilize the face. Forepoling or spiling, installation of long grouted fibreglass dowels in the face, utilization of face buttresses and immediate shotcreting of the face have all been used in different combinations for face stabilization. Once the face has been stabilized, the installation of the primary support system, consisting of rockbolts, steel sets or lattice girders and shotcrete in various combinations was necessary to ensure the stability of the tunnel. Micropiles were used in few cases to assist the foundation of the shell (figure 9). The sequence of support installation and of the excavation and support of the bench were also critical factors for tunnel stability. In the case of very weak rock masses under high overburden (up to 250m in the Egnatia cases), the stability of the pillar between twin tunnels bores requires careful evaluation.

Empirical tunnel design methods based on rock mass classification or simplified methods such as the convergence-confinement method...
were of limited use in the design of these tunnels because they cannot deal adequately with issues such as the sequential excavation and installation of support measures or face stability. Consequently, as mentioned earlier, the design of almost all of the Egnatia tunnels has involved the use of numerical methods.

In some critical cases three dimensional numerical models such as FLAC3D\(^1\) or SOFISTIK3D\(^2\) have been used.

Generally these models have been applied to study specific issues, such as the effectiveness of forepoling, and to derive simplifications that can be applied in two-dimensional modelling. In most cases, two-dimensional models such a PHASE2\(^3\) or SOFISTIK\(^2\) have been used to carry out the detailed design work.

In all cases the results of the model studies have been validated by the interpretation of convergence measurements and by observation of the tunnel and installed support performance.

Figure 7. Tunnel S3. Approximate analysis of the remedial proposals by Contractor’s Designer (Omicron Kappa) with a grouted rock mass surrounding the tunnels, 1000 kN anchors installed from the tunnels and an anchored pile wall to protect a gas pipeline uphill. It illustrates that the solution is effective and that the factor of safety for the tunnels is approximately 1.3. The lower factors of safety for the lower part of the slope are questionable since the material properties have been crudely estimated in this area. The steep lower slope suggests that these properties are probably too low. The placement of a toe buttress against a deep failure can be considered.

Figure 8. A typical heavy support design for weak rock masses using top heading and bench. The necessity, the amount and the combination of the various elements of this typical section is a result of numerical analysis and optimisation by monitoring. For highly squeezing ground yielding support was recommended. The excavation step rarely exceed 1m per day.

Σχήμα 7. Σήραγγα Σ3. Προσεγγιστική ανάλυση των μέτρων αντιμετώπισης που πρότεινε ο μελετητής του κατασκευαστή (Omicron Kappa) με ισχυροποίηση της βραχώματος γύρω από τη σήραγγα, προεντεταμένες αγκυρώσεις και πασαλοστοιχία, που δείχνουν την εφικτότητά τους.

Σχήμα 8. Τύπικη διατομή προσωρινής υποστήριξης για ασθενή βραχώματα με κατασκευή σε 2 οριζόντιες ημιδιατομές. Η ανάγκη, τα μεγέθη, οι ποσότητες και ο συνδυασμός των διαφόρων στοιχείων προκύπτει από αριθμητική ανάλυση και από βελτιστοποιήσεις με ενόργανη, αξιόπιστη, παρακολούθηση. Σε ισχυρώς συνθλίβοντα στρώματα συνιστάται η εφαρμογή των αρχών ολισθαίνουσας (ενδίδουσας) υποστήριξης. Το βήμα προχώρησης εδώ δύσκολα ξεπερνά το 1m την ημέρα.

1 Details from www.itascacg.com
2 Details from www.sofistik.com
3 Details from www.rocscience.com
5.3 Final lining

The design of the final lining assumes that the primary support degrades with time and that it offers no permanent ground support. Hence the final lining is designed to carry full ground loading. A waterproof membrane and drainage layer is incorporated between the primary and final lining.

Drainage of the rock mass surrounding the tunnel is assumed in the design but checks are carried out on the adequacy of the lining to withstand a nominal water load of 5 m above the crown. The load cases for the final lining design take into account the topographic, geological, geotechnical and hydrogeological information on the site. Also considered are influences of earthquakes, self weight, thermal stresses, vehicle impact and explosions, incorporation of electrical and mechanical components, construction sequences and other factors identified by the designer for specific cases. In general the design of the final lining is carried out using plane strain modeling with combinations of the above load cases. Stresses applied to the lining are obtained from numerical analyses of the tunnel excavation and primary support installation sequence. The thickness of the final lining of the Egnatia Motorway tunnels ranges from 0.30m to 0.60m and B25 concrete type is generally used for these linings.

6. CONCLUDING: TUNNEL COSTS

The average construction cost of a long road tunnel, with final lining and electromechanical systems installed, calculated on the basis of the pricelist policy implemented by “Egnatia Odos A.E.”, varies from €8,000 per linear metre in good geotechnical conditions to €35,000 per linear metre per bore in extremely poor geotechnical conditions. For shorter tunnels the average construction cost is in the neighbourhood of €10,200 per linear metre per bore.

7. ACKNOWLEDGEMENTS

We would like to thank Egnatia Odos S.A. for its support for the preparation of this paper.

8. REFERENCES


