Greece's Egnatia Highway Tunnels

Following the original Roman crossing of the Balkan Peninsula, the new Egnatia Highway has 73 tunnels with a total length of over 100km. The authors (see box) explain the intricacies involved in constructing this mega-project

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Built in the second century B.C, Via Egnatia was the first highway built by the Romans outside Italy. It crossed the Balkan peninsula from the Adriatic sea in the west to the Marmara and the Black Sea in the east. The Egnatia Highway, currently under construction, follows a similar route.

Forming part of the Trans-European highway network the Egnatia motorway stretches from the west coast of Greece to the Turkish border. The principal axis is 680km long and has 73 tunnels with a total tunnel length of approximately 100km. This component of the project is 50% co-funded by the European Union.

Above: an interchange and tunnel portals near the town of Metsovo in the Pindos mountains.

The geological environment

The highway traverses extremely diverse natural morphology of great beauty. It crosses the Pindos mountains which are the southern most extension of the Alps. The highway has been subdivided into the following units (figure 1):

1. From the west coast port of Igomenitsa to the Metsovitikos River the Ionian geotectonic unit consists of flysch and alternations of various carbonate formations, mainly limestones, with very limited occurrence of cherts and siliciferous shales. 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.

2. From the Metsovitikos River to the Metsovo tunnel the Pindos geotectonic unit consists mainly of flysch, characterised by intense folding, heavy shearing with numerous overthrusts. The tectonic deformation at some places drastically degrades the quality of the rock mass. From the Metsovo tunnel to Panagia region the tectonic Nappe of Pindos comprises ophiolites as the predominant rock mass. These ophiolites 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.

3. From Panagia to Siatista the molassic domain consists of molassic formations in the form of alternating thick-bedded conglomerates, sandstones and siltstones or claystones. From a tectonic point of view, the area is of low disturbance and although weak rock masses are present in places, there is no dramatic decrease of geotechnical quality due to the absence of significant tectonic shearing.

4. From Siatista to Lefkopetra the Pelagonian geotectonic unit is characterised by the predominance of hard rocks such as marbles, gneisses and granites. The presence of tectonically weakened zones through faulting is very localised. From Lefkopetra to Veria the Axios to Almopia geotectonic units consist of phyllites, limestones and ophiolites while overthrusts and sheared zones are the main tectonic structures.

5. From the Aliakmon River to the Axios

Above: Fig 1 - Location map of the Egnatia Highway

River flood plane and Thessaloniki region the entire area consists of recent alluvial fill which exhibit insufficient natural compaction. From Thessaloniki to the Turkish border the Serb-Macedonian massif and the Rhodope massif comprise a basement 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 also passes through areas of younger sediments such as marls and sandstones and areas of recent geological deposits with soft soils of loose or open structure,

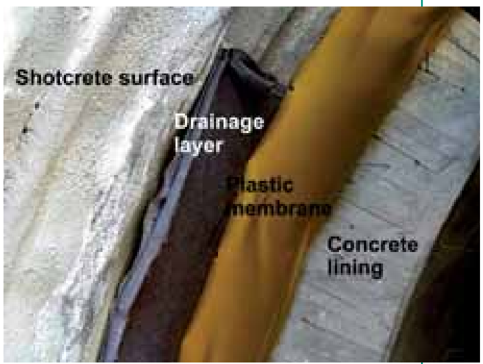
Responses to tunnelling through these different rock masses are shown in table 1.

Table 1

Rock type and conditions	Response to tunnelling and stability problems
Massive or bedded limestones. Marbles	Simple tunnnelling conditions. Structurally controlled failures, mainly controlled by rockbolts.
Filled karstic voids	Risk of collapses. Probing ahead essential and use of spiles or a forepole umbrella to cross void.
Sandstone flysch	Gravity driven structuraly dependent instability in low stress environments and occasionnally stress dependant instability when strength to stress ratio is low.
Siltstone flysch and shales	Stress dependent instability resulting in significant deformations and minor face instability. Control of deformation is essential and both temporary and permanent inverts may be required to form a load bearing shell.
Sheared and chaotic flysch	Squeezing conditions and face instability problems at depth (even as low as 100m in some cases, usually more than 200m). Control of deformations is essential and, to control extreme squeezing, yielding support may be required.
Sound ophiolites (peridotitees and gabbros)	Structurally dependent instability, more severe when discontinuities are serpentinised. Block size normally irregular and this requires a conservative excavation and support approach.
Sheared serpentinites and ophiolitic melanges	Squeezing conditions at depth (e.g. more than 200m). Control of deformations is essential and, to control extreme squeezing, yielding support may be required.
Molasses (tectonically undisturbed sedimentary sequence of rocks)	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.
Gneiss schists	Simple tunnelling conditions if not heavily tectonized and/or weathered. Structurally dependant instability.
Phyllites	Weak rock tunnelling. Deformation problems in cases of deep tunnels. Control of deformation is essential an both temporary and permanent inverts are generally required to form a load bearing shell.
Tectonics breccia in brittle rocks, kataclastes	Ravelling due to loss of interlocking as confinement is released at face. Maintaining confinement is important and this can generally be achieved by retaining a core at the face and the use of pre-reinforcement elements (e.g. face bolts, spiles)

Above: a parking bay in a typical operating tunnel on the Egnatia highway

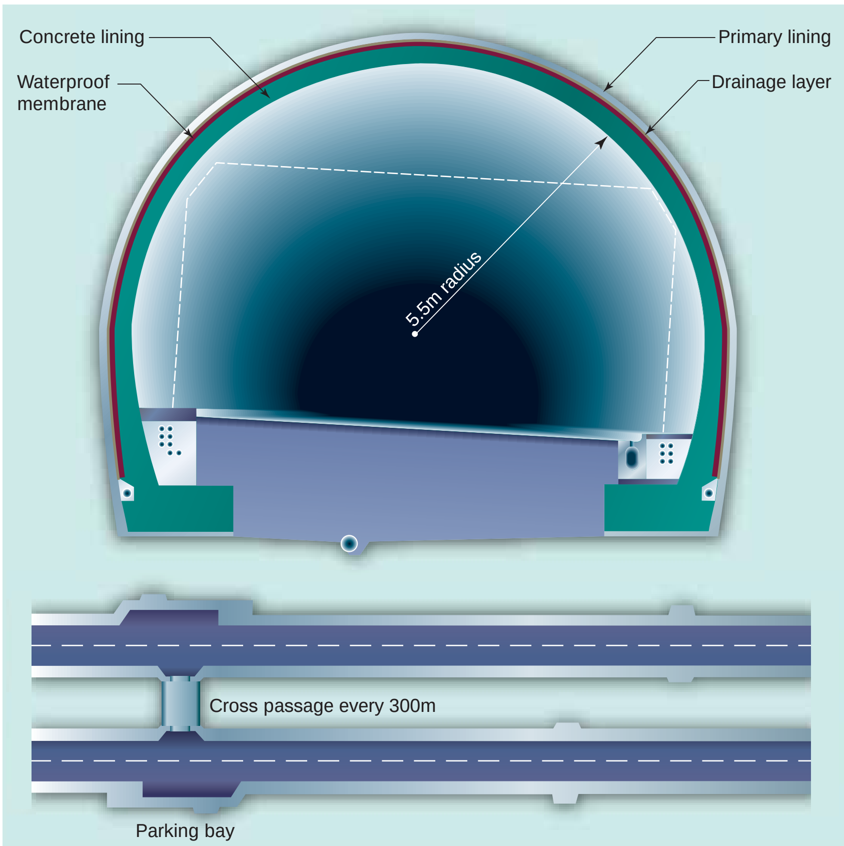
Below: drainage and waterproofing layers behind the concrete lining.



Tunnel geometry

The tunnels of the Egnatia Highway meet and, in many cases, exceed the minimum requirements for road tunnels recommended by the European Union[1]. All tunnels are twin two lane tunnels with unidirectional traffic in each tunnel. The tunnels have two 3.75m wide lanes, two 0.5m shoulders and two 1m wide pedestrian walkways. The traffic envelope is 8.5m wide and 5m high. The twin tunnels are linked by cross passages with fire doors spaced at 300 to 400m apart (figure 2). In tunnel bores longer than 2km every third cross passage is large enough to allow emergency vehicle access and is associated with parking bays to allow vehicles to be moved out of the way. Lighting, ventilation systems, signals and fireproof structural components are all features that have been found to contribute to the overall safety of road tunnels[2].

The interior of every tunnel is finished with a cast-in-place concrete lining which is backed by a waterproof membrane which is backed by a drainage layer. This is placed against the inner surface of the primary lining of generally shotcrete. Depending upon the difficulty of the conditions, the primary lining is supplemented by steel arches and rockbolts which are designed to support the rock mass surrounding the tunnel during excavation.



Above: Fig 2 - Cross section and plan of a typical tunnel

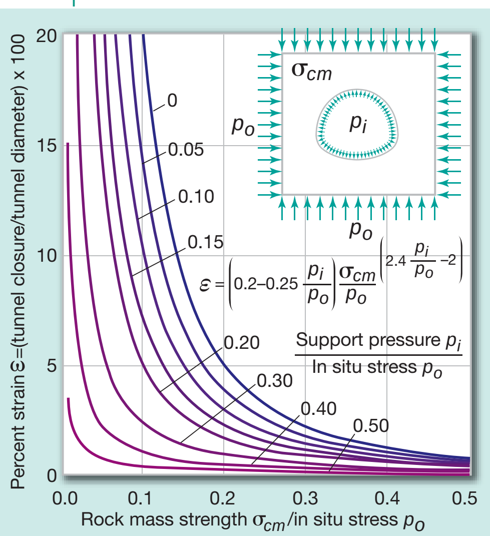
Tunnel design

Egnatia Odos S.A. has developed tunnel design guidelines that cover all the aspects of tunnel design. For the design of the excavation and temporary support, general design steps are shown in Table 2 (Kazilis and Angistalis[4]).

Many of the potential problems identified during the preliminary design stages can be easily dealt with by the timely installation of the appropriate combinations of shotcrete, rockbolts, lattice girders and steel sets with the occasional use of spiles as pre-reinforcement elements over and ahead of the face. However, in the case of the sheared flysch, weak serpentenites and ophiolitic melanges, the risk of severe deformations of both the tunnel and the face had to be recognised and dealt with.

Figure 3 shows that, when the ratio of rock mass strength to in situ stress falls below 0.2, deformation of the tunnel increases exponentially and can develop into severe squeezing problems if not recognised and dealt with appropriately. Consequently, when the geological model indicates that materials with low rock mass strength are present and preliminary checks (such as those described by Hoek and Marinos[5]) indicate a potential for squeezing is present, a design involving the use of numerical models is required. The sequential excavation and support installation is modelled and the progressive failure and deformation of the rock mass surrounding the tunnel face is observed in detail. This permits the excavation sequence, support types and capacities and installation sequence to be optimised. In many of these cases a temporary invert is required in the top heading excavation in order to maintain a closed structural shell. Most of this numerical modelling can be carried out using two-dimensional models but, in some cases, three-dimensional models are used to study particular details or to check the results of two dimensional models.

In dealing with the relatively large displacements discussed above, the tunnel has to be over-excavated to allow the deformation to occur and still to provide sufficient room to accommodate the final concrete lining. Even the best geological models and the most sophisticated numerical analyses cannot predict the over-excavation required with sufficient accuracy



Above: Figure 3 - Approximate relationships between tunnel strain and the ratio of rock mass strength to in situ strength for different levels of installed support, after Hoek[6]

Table 2

Step	Design stages
1	Assessment of the strength and deformability of the ground along the tunnel. Use of acceptable systems of rock mass classification and characterisation.
2	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 rock mass strength. Structurally dependent instability analysis is considered in cases of strong rock masses.
3	Premliminary selection of support classes based on experience and empirical methods.
4	Calculation of deformations and plastic zones along the tunnel considering support measures. Use of analytical equations taking into consideration the stress field, the rock mass strength, and the support pressure provided by the chosen support class.
5	Identification of problems. High deformations, face instabilities, floor heave etc.
6	Where potential stability problems are identified use of numerical analyses to check, confirm and finalize the support classes based on the anticipated failure mode

and it is essential that the actual tunnel deformations be monitored and used to calibrate and correct the models. With experience, this process has proved to be highly effective in the 59 Egnatia tunnels completed to date (June, 2006) and relatively few situations have occurred where "tights" have required trimming before the installation of the final lining

Fortunately, in most cases, it has proved possible to arrive at a combination of support types with sufficient capacity to limit the tunnel deformations to acceptable levels. Generally this has required the installation of overlapping forepole umbrellas consisting of 12m long grouted, 114mm diameter pipes, 14m long grouted fibreglass dowels in the face and temporary invert closure in the top heading excavation.

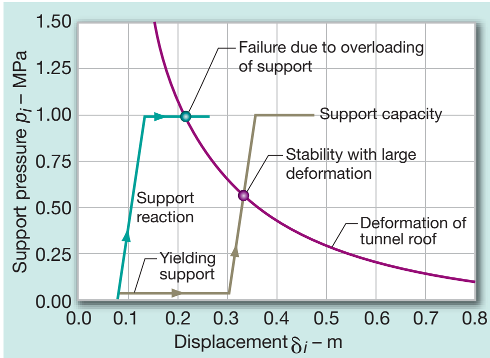
The two exceptions to the application of heavy support systems to control deformation are the second tube of the existing Metsovo and the twin Panagia tunnels in weak rock masses at depth of up to 600m. In this case the capacity of available support systems that can be installed at the face was found to be insufficient and yielding support has been required to control the deformations. The principle of yielding support is illustrated in Figure 4 which shows that the activation of the support is delayed by the yielding elements. Obviously the tunnel has to be over excavated to allow for the much larger deformations that occur in these cases. The yielding elements used in the cross passages and the second bore of the Metsovo tunnel are "stress controllers", as described by Schubert[7]. This system has been very effective and the severe problems encountered during the driving of the first tube of the Metsovo tunnel more than 20 years ago[8] (during an earlier project) have been avoided.

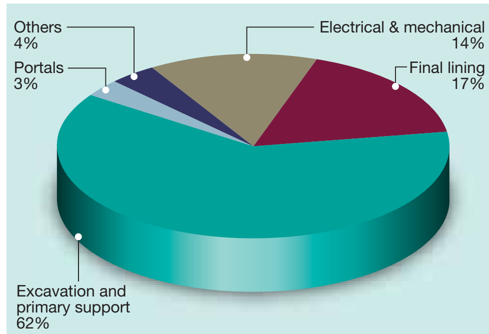
Final lining design

All tunnels have a final lining of cast-in-place concrete. The design of these linings follows the same procedure and is integrated into the design process used for the design of the primary support systems described earlier. In general a detailed numerical

Below: Fig 4 - Principle of delayed support activation by the use of yielding support.

Bottom: Fig 5 - Breakdown of the average total cost of tunnel construction on the Egnatia highway up to the end of 2003, after Lambropoulos[3]





analysis of the entire excavation sequence, primary support installation, completion of the tunnel excavation and installation of the final lining is performed. Current European Union practice require that all of the primary support be discounted in the design of the final lining and hence, as a final stage in the numerical modelling process, the primary support is removed and the loads carried by this support are then automatically transferred onto the final lining.

Tunnel costs

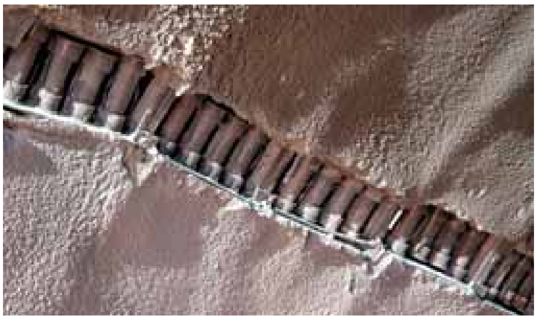
The average total cost for the 32 Egnatia tunnels completed by the end of 2003 was US$26.9M per tunnel/km (figure 5).

A more detailed analysis of the total tunnel costs, up to 2003, shows that, as would be expected, the difficulty of tunnelling has a major impact on this cost. For tunnels in good quality rock masses, where simple tunnelling methods can be applied, the total cost is in the order of US$16M per tunnel/km. On the other hand, difficult tunnelling conditions which occur in fault zones or heavily broken and deformed rock masses can result in total tunnel costs of up to US$35.9M per tunnel per km.

A database for the future

With technical and funding assistance from Egnatia Odos S.A., the School of Civil Engineering of the National Technical University of Athens has compiled a data base of information collected during the site investigations, designs and construction of all of the Egnatia tunnels completed to date. This data base, which operates on an SQL server contains structured digital data of all geological models, rock mass classifications and characterisations, tunnel support and lining designs, contractual details, excavation performance, results of convergence and other monitoring information and costs of all components of the tunnels.

Users of this data base will be able to examine correlations between any number of related parameters and, with such a large body of information covering a wide range of rock mass types, it is hoped that some of these correlations will provide useful guidance for future tunnelling projects. T&T



Above: a line of stress controller installed in one of the cross passages between the first and second tubes of the Metsovo tunnel

ACKNOWLEDGMENTS

The authors would like to thank Egnatia Odos SA for its support in the preparation of this paper. We would also like to acknowledge the leadership and encouragement provided by Dr Sergios Lambpropoulos during his tenure as general manager of Egnatia Odos SA from 1995 to 2006.

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