The construction of the Egnatia highway through unstable slopes in northern Greece

P. Marinos and E. Hoek

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P. Marinos
School of Civil Engineering, Geotechnical Department, National technical University of Athens, marinos@central.ntua.gr

E. Hoek
Evert Hoek Consulting Engineer Inc., Vancouver, ehoek@mailas.com

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1. Introduction

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 four lane highway, which is currently under construction in northern Greece, is named after that old roman road and follows a similar route. Forming part of the Trans-European highway network, the Egnatia highway stretches from the west coast of Greece to the Turkish border. The principal axis is 680km long, and is 50% co-funded by the European Union.

The Egnatia highway runs across the entire width of Greece traversing the Pindos mountain range which is the southern most extension of the Alps, with a morphology of great beauty. The highway runs almost perpendicular to the main geotectonic units of the country (figure 1 & 2).

Thus, there are a great variety of geological situations and this fact imposes the need for different approaches in designing the highway. Each geotectonic unit exhibits different particularities in terms of weak rock masses and of the potential for instability. The availability of such knowledge led to a selection of alignment so as to avoid areas of slope instability or areas affected by old large landslides as much as the restraints of the highway allow and for the chosen alignment to follow a route dictated by the rock/ground model as accurately as possible.

As members of the Egnatia Odos Panel of Experts the authors are responsible for reviewing route selection, design and construction issues as well as safety and cost implications of the designs

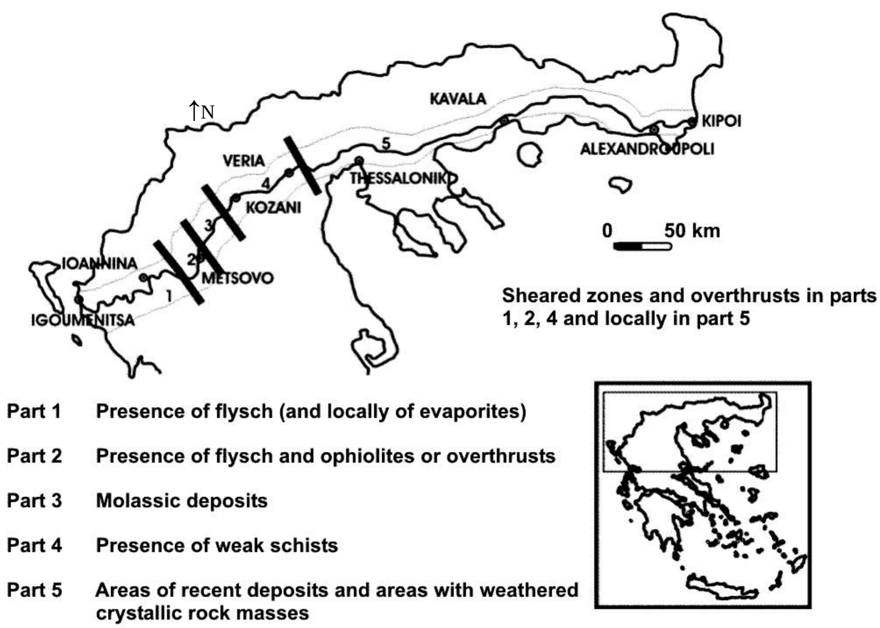


Figure 1: Alignment of Egnatia Highway and spatial setting of the major geological sectors along with the geotechnically weak conditions.

2. General inputs from the geological structure

The highway can be subdivided into the following units (figure 1 and 2, Marinos and Hoek, 2000, Marinos et al, 2005):

1. From the west coast port of *Igoumenitsa* 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. Flysch is the main weak formation with large exposures on slopes. The geomorphological evolution has produced large scale landslides.

2. From the *Metsovitikos* river to the town of *Metsovo-Anilio* 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 already poor quality of the rock mass producing slopes with critical stability and large scale landslides. From *Anilio* 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. Instability may be produced if cuts are not adequately designed. Weak flysch, depressed by this ophiolitic nappe, is also present.

3. From *Panagia* to *Siatista* the molassic domain consists of 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. However, Instability issues may be present in the weathered mantle with landslides of reduced size.

4. From *Siatista* to *Lefkopetra* the Pelagonian geotectonic unit is characterised by the predominance of hard rocks such as marbles and gneisses. The presence of tectonically weakened zones through faulting is very localised. From *Lefkopetra* to the city of *Veria,* the Axios to Almopia geotectonic units consist of phyllites, limestones and ophiolites while overthrusts and sheared zones are the main tectonic structures. Old instability may be present in slopes in weak phylites due to the geomorphological deepening of valley floor of the area.

5. From the *Aliakmon* river to the *Axios* river flood plane and *Thessaloniki* region the entire area consists of recent alluvial fill. 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. Large instability is rare but shear zones may produce slides in cuts.

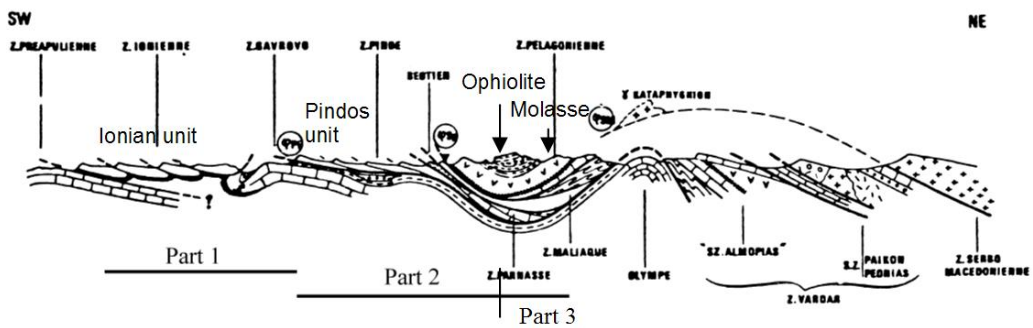


Figure 2: Schematic cross-section of the Hellenic Alps (Aubouin, 1979, in Papanikolaou 2003). Part 1, 2 and 3 from figure 1

3. A classification of cases

Many times the situations described 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 have 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 even justified a drastic change of the initial alignment, when the cost was not prohibitive and the operational safety did not inherit the uncertainties of the initial alignment.

Along the route of the Egnatia highway, the difficult geotechnical cases are classified as follows.

3.1 Mountain slopes with an overall critical stability

This case corresponds to old large landslides having affected entire mountain slopes. The morphology of the slopes is impacted by these movements and flat areas, due to the displacement of the ground, may entice the highway designers by their geometrical attractiveness. However, the weak sheared surface is still there, dangerously hidden and further weakened by creep. Thus instability can easily be triggered, even without the need of the impact of the highway. These are valley sides consisting of flysch, usually overthrusted by limestones. This configuration further downgrades the generally poor geotechnical quality of flysch and allows softening by preferential drainage.

In these cases any remedial or stability measures were impractical and a dramatic realignment was the solution. This realignment in most cases included long tunnels under the landslide surfaces or relocation of the highway to the opposite valley side.

3.2 Creeping mountain slopes

This case includes weathered or landslide materials and clayey debris from earlier flows, creeping at a rate of 1-2 cm per year. The creeping zone could be a few to several metres thick. Although this case has similarities to the previous one it is considered differently in those cases where the risk of a reactivation of an old landslide is very low and a design of the highway through them can be considered. In fact wherever realignment was not possible a residual risk could be accepted. The design however includes all measures to minimize the risk, mainly by engineering the alignment, providing drainage and selecting highway structures to accommodate deformations. An important component of the safe operation of the highway in these areas is the implementation of a carefully designed monitoring system.

3.3 Crossing landslides at the scale of the surrounding slope

This is the case where unstable masses associated with the highway alignment are of such size that the masses involved can be stabilized. The solutions that are adopted and implemented are selected following detailed analysis based on the geological model of each case. Most of these landslides are in flysch, mainly siltstone flysch sometime with a chaotic structure. The solutions were the placement of counterweights to buttress the slopes, cut and cover structures or, in smaller unstable areas, pile walls. In all cases drainage works, either through wells - draining towards an underlying permeable layer - horizontal holes or even drainage galleries are present.

3.4 Failures in cuts

This is the case in stable natural slopes consisting of weak rock masses or of strong masses with hidden weaknesses. Failures may occur if the geometry of the cuts of the highway is not compatible with the strength of the material. Instability occurred when there was a misunderstanding of the mode of failure and its mechanics and consequently of the selection of the appropriate geotechnical parameters. The ground is usually flysch, phyllites, heavily broken crystalline rock masses, weathered ophiolites or strong rocks containing very weak shear zones with clayey material in faults.

For the analysis of the stability of cuts against a first time failure, where the benefit of back analyzing the slope – always the best way to define parameters - is missing, the selection of the appropriate design parameters is always a challenge. Again, the knowledge of the geological model is of prime importance. This model will allow the determination of the mechanism of failure and thus the selection of the parameters, either of the rock mass or of discontinuities and the appropriate simulation in the numerical analysis of the design. In cases, for instance, of a globally weak rock mass, the Hoek and Brown criterion adequately describes the failure and the use of the GSI to characterize the rock mass and evaluate the strength parameters constitutes a valuable tool in the simulation procedure. The measures selected and optimized by the design depend on the type and the quality of the rock mass, the mechanism of failure and the size of the cut.

4. Examples

4.1 Case of areas of entire slope instability

This case, where a realignment was decided in the early stage of the design, is illustrated in two major sections of the highway in its western part.

4.1.1 In the Metsovitikos valley masses of competent formations (limestones) have overthrust the soft and "ductile" flysch. The former comprises the highest parts of the southern valley sides forming high cliffs which produce screes covering the flysch of the lower parts which are deformed and sheared due to the overthrusting. The screes allow water to percolate and moisten 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 by a realignment of the motorway to the north side of the valley where more competent sandstone flysch is present away from the thrust (fig. 3 & 4). The length of the relocated alignment is almost 5km (Kazilis and Georganopoulos 1999).

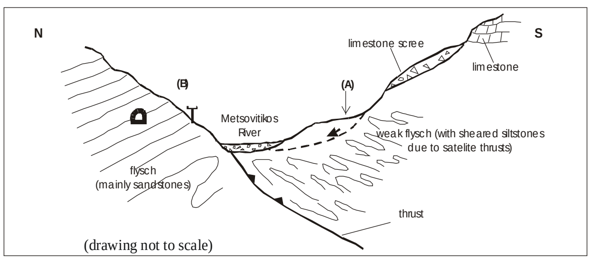


Figure 3: Drive through the section at the Metsovitikos valley. The initial alignment (A) was selected through a mild topography. However, this morphology was due to a series of old landslides in unstable flysch. Extensive drainage including drainage galleries of 5km of length were proposed. Solution: relocation of alignment at the northern, steep but stable, slopes of Metsovitikos River where sandstone flysch occurs (B) with tunnels alternating with bridges.

Figure 4 Construction works at the north side of Metsovitikos valley after the realignment of the highway in the Metsovo section in order to avoid globally unstable slopes at the south side of the valley

The comparison between the two alternatives which led to the realignment of the highway to the northern side of the valley is most conveniently presented in the following tabular form.

	Issues	*Southern alignment*	*Northern alignment*
1	Types of works imposed by morphology	Cuts and fills, bridges	Tunnels, bridges and fills
2	Topographic conditions	Smooth but diversified slopes in downhill areas and steeper slopes in higher parts in some cases.	Ridges with steep slopes, deep gullies
3	Complexity of geological structure	High	Normal
4	Quality of ground	Thick talus loose material and weak to moderate quality flysch	Practically no cover material. High proportion of good quality sandstone flysch
5	Tectonic structural disturbance	High	Normal for the Pindos rock masses
6	Geological uncertainties that can remain after the geological studies	Yes, to some extent	Very limited
7	Presence of old landslides	Yes, sometimes of considerable size	No
8	Risk for reactivation of old landslides	Moderate to high risk	Not applicable
9	Risk for new landslides	High. mainly in cut slopes and around bridge abutments	Very localised and of limited extent

10	Presence of groundwater as a contributing factor to slope instability	Yes	No
11	Remedial strengthening measures	Absolutely necessary. Of considerable size in extent, length and density.	Limited to very limited to some bridge foundation sites
12	Uncertainty in effectiveness of drainage systems	Some uncertainties may remain in local areas due to the low permeability of the poor quality flysch	Not applicable.
13	Maintenance of drainage works	Of utmost importance otherwise reliability of the system must be downgraded with time	Not applicable for drainage, minor for support measures in some bridge foundations
14	Monitoring of stability conditions	Of utmost importance	Not applicable for drainage, minor for behaviour of support measures in some bridge foundations
15	Delays in case of failures during construction	Long delays anticipated since any failures will probably be large	Short, generally due to local incidents
16	Risk from geotechnical hazards during operation of the highway	Uncertainty may exist over long sections of highway. This uncertainty is difficult to reduce to very low levels	May exist locally due to rockfalls from steep slopes. These risks can be reduced to very low levels by appropriate remedial measures
17	Foundations of bridges	Foundation sites close to unstable areas, often in weak flysch	Foundation sites in good quality flysch in most cases
18	Environmental issues associated with geotechnical remedial works	Cuts, embankments, large and high retaining structures, gabion river training measures. Possible encroachment on the river. Damage to forest from cuts. Cuts and embankments will re-vegetate.	Very small impact except for access roads during construction. Muck from tunnels can be used to construct embankments for interchanges

The northern alignment has an apparent disadvantage in terms of the increased length and the number of tunnels involved. However, the risks, both during construction and subsequent operation were considered to be lower than the problems associated with the unstable slopes of the southern alignment. The quality of the rock mass along the northern alignment is such that tunnelling problems were expected to be minimal. This was indeed the case as this section of the highway is already constructed in operation.

4.1.2 A similar case of a generous realignment of the Egnatia axis exists further to the west of the highway in the Peristeri area. The initial axis was selected due to the mild and friendly morphology (fig. 5). However this morphology was the result of a tongue of landslide debris of limestones over the flysch. These huge limestone blocks still move over their weak basement at depths of about 50m at rates of 1 to 2cm per year (fig. 6).

A realignment to the north side was necessary. Old landslides occur also in the new position of the highway but these are of a size that can be stabilised effectively.

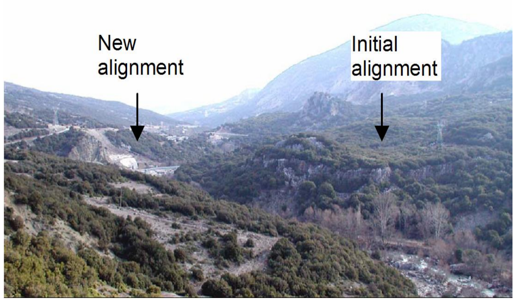


Figure 5 View of the Peristeri area. At the right of the picture the south side of valley showing the large limestone blocks in gentle slopes of landslide debris. The alignment of the highway initially selected at the south has been moved to the north side of the valley

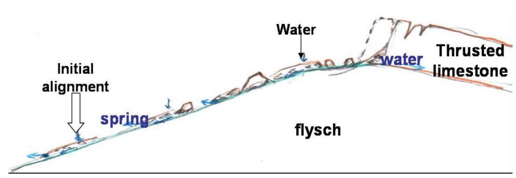


Figure 6 Simplified model at Peristeri area. Debris flows, landslides and earthflows at the south slope of the valley. A change of the alignment was imposed

4.2 Cases of crossing landslide masses

4.2.1 A landslide that was not possible to avoid is located in the Peristeri area at the new position of the axis of the highway after its realignment in order to avoid the huge unstable zone described in the previous paragraph.

This potential slide is within old landslide debris which overlies bedrock consisting of conglomerates and sandstone flysch. Inclinometers in the slope have indicated deeper movements than had been previously assumed and the magnitude of these movements has accelerated with the increase in construction activities associated with the creation of the portals and the mining of the adjacent tunnels (figure 7). The indications from these measurements are that the slide surface is probably coincident with the boundary between the overburden and the bedrock and that the slope is marginally stable. Groundwater conditions in the slope have been monitored by a number of piezometers which indicate that the present groundwater level is high.

The designer has proposed drainage of the slope as the primary remedial option and that this drainage would be achieved by one or more drainage tunnels. In order to evaluate these proposals and to examine other remedial options, we carried out a number of crude stability studies to explore the sensitivities of the proposed solutions rather than to offer a specific design. The results from one of these studies are presented in Figure 8.

Figure 7: Potential instability in area that will be crossed by the Egnatia highway under the national highway. The portals of tunnel indicate the alignment of the Egnatia highway.

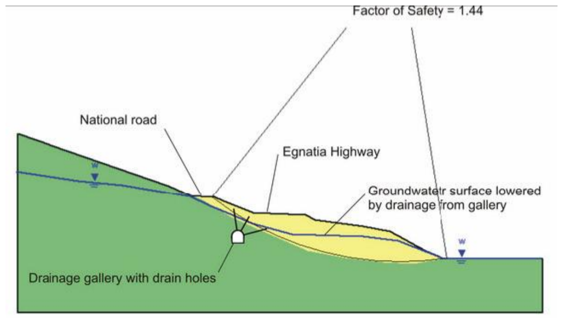


Figure 8: Stability analysis of the existing slope with partial drainage by means of a drainage gallery in bedrock with drainage holes penetrating the assumed failure surface. The factor of safety for the overburden/bedrock interface failure surface is shown as 1.44.

Options that were considered for stabilizing the slope in this area include the following:

1. Stripping material from the top in order to reduce the driving force at the top of the slope. This option is attractive because the limit of the upper boundary of the slide at the overburden/bedrock interface would prevent propagation of the slide into the rock mass above the National road. Unfortunately, the amount of material available for stripping is quite small and stability analyses indicate that an increase of only about 10% in the factor of safety can be achieved by this stripping. Consequently, this option cannot be considered a primary remedial option although it may be used in conjunction with drainage to improve the stability of the slope.

2. Buttressing the toe of the slope is not a viable option in this case because there is insufficient room to create an effective buttress on the bank of the Metsovitikos river.

3. Drainage of the slope, as illustrated in Figure 8, results in a significant improvement in the factor of safety[1]. In the case illustrated, partial drainage of the slope is achieved from a single drainage gallery in the bedrock with inclined drainage holes drilled into the overlying overburden. It has been assumed that the groundwater table is drawn down by a rather modest amount at the head of the slide but left largely unchanged at the toe of the slope. The increase of factor of safety from 1.00 to 1.44 for partially drained conditions suggests that there is good potential for optimizing the drainage measures to achieve the required improvement in stability.
When construction schedule demands that drainage measures to improve stability are required before a drainage gallery can be completed, consideration should be given to the drilling of a series of horizontal

[1] The analysis was carried out using the program SLIDE, details of which are available from www.rocscience.com. The shear strength of the overburden material is characterized by c' = 0 and ϕ= 20º while the bedrock is characterized by c' = 0.2 MPa and ϕ= 40º.

drain holes near the toe of the slope. These drains can usually be installed very quickly and they will have an immediate beneficial effect. Because of ongoing movements in the slope some of these drainage holes would be damaged over time but by then the primary drainage function would have been taken over by the drainage gallery.

On the basis of experience in other projects in which drainage galleries have been used successfully it is suggested that the drainage tunnel must be located in bedrock. The cost of constructing a gallery (5m diameter) within the slide material is approximately three times the cost of mining it in the bedrock and, in addition, high ongoing maintenance costs are associated with a tunnel in deforming ground. The drain holes have to be filtered in order to prevent erosion of fines from the overburden material and it has to be recognized that cleaning and/or re-drilling of these holes will be required periodically in order to ensure that the drainage system remains effective.

An essential requirement for a successful drainage program is careful monitoring of the groundwater surface trough a good number of piezometres both before and after the implementation of the drainage scheme.

4.2.2 One other case where the highway alignment cannot avoid a significant and deep landslide is shown in figure 9 (Krystallopigi area in the westernmost part of the highway). An embankment will be used for this crossing.

Figure 9 Frontal view of the ancient Krystallopigi landslide which will be crossed by the highway

In this case the base of the mountain slopes consist of flysch and, in some areas, of the underlying series of the Ionian limestones. Over these formations a comprehensive series of the Ionian limestones has been thrusted. These limestones are heavily fractured or even brecciated.

The exposed part of the flysch on the slopes, weakened by water, suffered in the past from landslides which undermined the overlying limestones and generating rock falls in a similar manner to that described in paragraph 4.1. Debris from these falls cover the flysch lying on the lower parts of the slope, adding to their disturbance and assisting the water penetration into the slope.

The area is thus undergoing a dynamic geomorphological process that has formed the amphitheatric scarp delineating the actual limits of the landslide and this process is still evolving. The most likely interpretation of the fact that the underlying weak flysch improves in depth into the slope is that it may not allow the development of huge slides at the mountain slope scale but creep movement and smaller slides cannot be excluded. Indeed creep is present in the slope and some inclinometers inside the main mass have recorded rates of movement of few tens of millimetres since 1998. The question that arises is whether these ongoing geomorphological processes can affect the highway. As mentioned, the landslide will be crossed by an embankment and remedial measures will include a long drainage gallery. Consideration also is given to assist drainage by drilling of vertical wells into the permeable underlying limestone.

4.2.3 A most particular case consist the crossing of an unstable slope with a tunnel. The tunnel, of a length of 260m, named S3, is located in the central section of the highway (figure 10).

The slope is of phyllites (figure 11) and no signs of instability were found when selecting this route for the highway. However, driving of the was suspended because cracking inside the tunnel and on surface gave rise to a concern that the slope in the vicinity of the entrance portal was unstable and that these conditions did not permit safe advance of the tunnel. The final invert cracked for a distance of approximately 60 m inwards from the entrance portal (figure 12).

Figure 10. The tunnel S3 in the central part of the highway from the entrance portal, crossing an unstable slope stabilized during the construction

Figure 11: Appearance of the phyllitic formation in the face of the remaining bench of tunnel S3. The well preserved structure of the rock mass is clearly visible in this face.

Figure 12: Severe invert cracking near the entrance portal of the S3 tunnel.

Ongoing downslope movements were recorded in the tunnels, on surface and by means of inclinometers installed in the slope. These measurements, together with observations of the cracks, suggest that a shallow slide had occurred in the vicinity of the Entrance Portal and that the tunnels and portal structure were moved downhill. The maximum recorded downslope movement was of the order of 270 mm and the movements were ongoing (fig. 13 & 14).

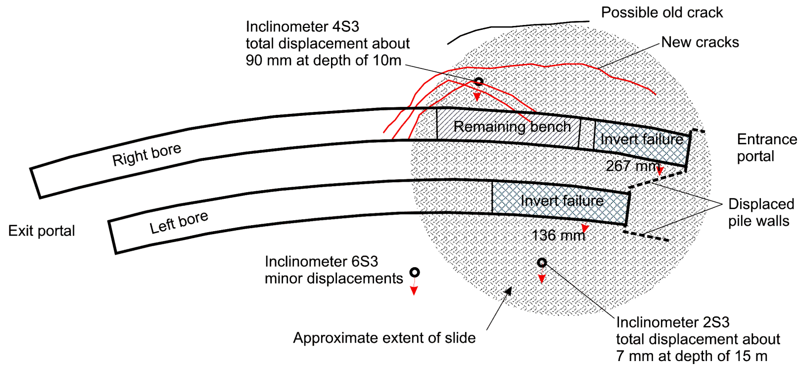


Figure 13: Simplified plan showing most significant features related to slope instability in the area of the entrance portal of tunnel S3. (scale: length of right bore: 260m)

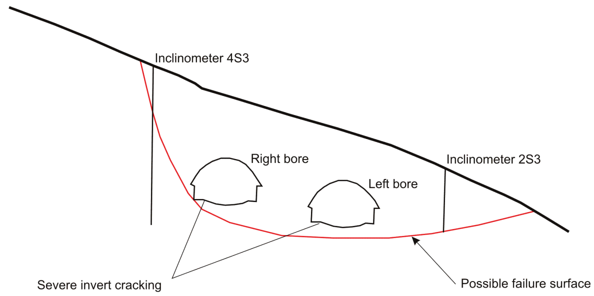


Figure 14: Possible slide geometry through inclinometers 4S3 and 2S3 including the tunnel S3. (scale: failure in 4S3 at a depth of 10m and in 2S3 of 15m)

An additional site investigation revealed signs of old instability uphill and detailed analyses were performed in order to decide on the stabilization measures to be applied. These measures included:

- A pile wall uphill to prevent any propagation of the slide and protect a gas pipe crossing the site. This pile wall was tied back with long stressed anchors.
-
- Stressed anchors from inside the tunnels. These are the most important component of the slope stabilization process.
-
- Grouting. The purpose was to strengthen the pillar and the surrounding mass of the tunnels and to improve the foundations for the anchors.
-

The designer performed detailed analyses but figure 15 present our own crude analyses of the remedial measure to check their effectiveness. Although this analysis is not an exact simulation of the FLAC studies performed by the Contractor's Designer, it illustrates that the solution is effective and that the factor of safety for the tunnels is approximately 1.3. The low 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 and higher shear strength values were assumed for this area in subsequent analyses.

These measures have been implemented and the tunnel is already in operation (figure 10).
Figure 16 gives a simple analysis for an alternative possibility with removal of a part of the slide and buttressing the valley against any potential deeper failure. The analyses illustrated in figs 15 and 16 were performed using the program Slide (www.rocscience.com)

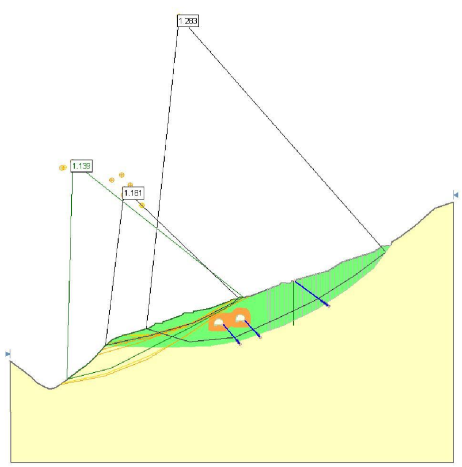


Figure 15 Approximate analysis of remedial proposals by Contractor's Designer with a grouted rock mass surrounding the tunnels, 1000 kN anchors installed from the tunnels and an anchored pile wall to protect the gas pipeline on the upper road.

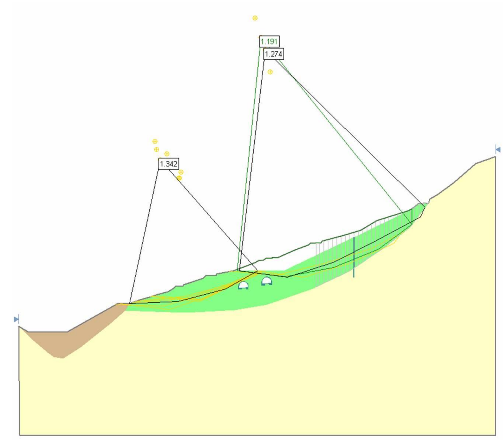


Figure 16. Alternative solution with a significant unloading of the upper part of the slope. It was suggested that the excavated material be used to buttress the slope against a potential deep slide.

4.3 Case of failures in highway cuts

A number of examples of failures in cuts during the construction of the highway in various rock masses illustrate this case.

4.3.1 Cut in granites

The highway cuts a granitic mass near the city of Kavala in eastern Greece. The granite is crossed by a sequence of faults almost normal to the road axis and dipping generally 50^o to 70^o to the south west. Many of these faults are associated with mylonites and sheared zones of clayey material. These weak zones, associated with other discontinuities, may form wedge like rock slides. Such a slide occurred during construction and is shown in figure 17 with a total vertical height of approximately 47m.

Based upon the interpretation of the core drilling results, the orientation of surface outcrops, the records of observations and measurements and the overall appearance of the slide, it was possible to construct a simplified slope failure model such as that illustrated in Figure 18. This figure can give a reliable indication of changes in factor of safety for various remedial actions. The trend of the results provided an adequate basis for the final decision regarding the measures to apply (figure 19).

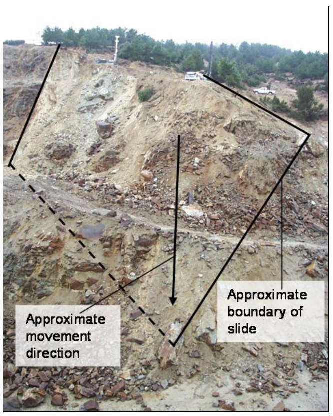


Figure 17: Photograph of landslide in Kavala section in a granitic mass showing approximate slide boundaries and movement direction.

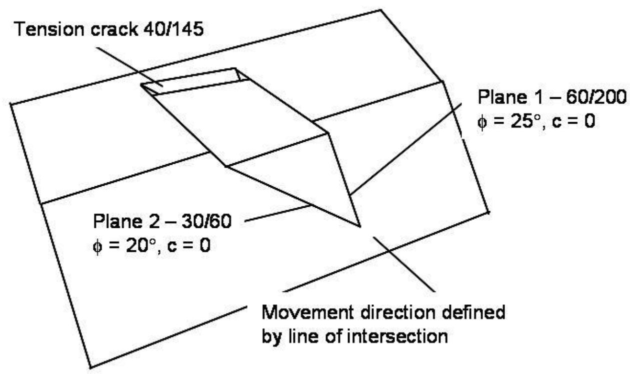
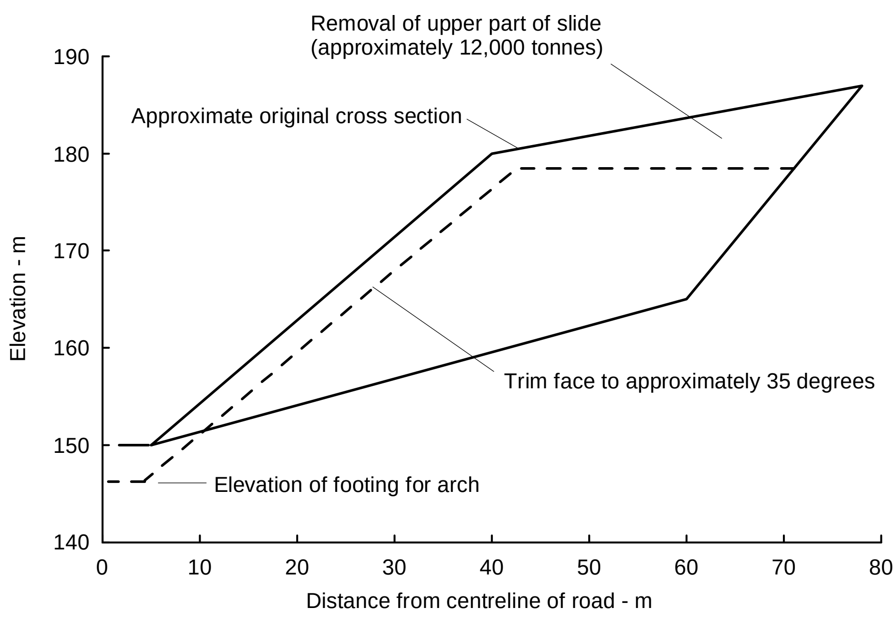


Figure 18. Model of landslide of figure 17 created using the program Swedge (www.rocscience.com)



Factors of safety (with water present in slope)	
Geometry	Factor of safety
Original slope	1.02
Original slope with upper part of slide removed	1.33
Slope toe excavated to footing level and slope trimmed back to 35 degrees	1.53
Backfill placed against slope	Approximately 4

Figure 19. Original cross-section of slope and possible revision to slope geometry (shown as a dashed line). Approximate factors of safety for various conditions are given in the table. These are based on indicative analyses to demonstrate the change of factor of safety with changes in slope geometry.

4.3.2 Cut in gneiss

Although these crystalline rock masses exhibit good behaviour, shear zones can be combined with other discontinuities of various orientations when the mass is quite jointed and produce rotational slides. Such a slide is show in figure 20 and 21 in the eastern part of the highway. Unloading of the slope was the applied solution.

Figure 20. Landslide at the Asprovalta area, controlled by the presence of joints and shear faults producing rotational slides. Notice the intact structure of sound gneiss inside the sliding mass

Figure 21. The landslide of figure 20. the rotational failure is clear

4.3.3 Cuts in molassic formations

Molasse, being vulnerable to weathering, is much weaker close to the surface. In a number of portals of tunnel of the highway in these rocks the solution of cover and cut construction was adopted instead of open cuts (figure 22).

Figure 22. Kiloma tunnel exit portal in molassic rocks showing cover and cut construction and small surficial slide on the left hand side of the photograph. The cover and cut structure protected the portal from further development of this slide.

4.3.4 Cut in ophiolites

The ophiolitic mass crossed by the Egnatia highway in the Katara area presents irregular serpentinisation of the good quality peridotite that is the persistent rock. This process has produced weak rock masses which have been further degraded by the intense tectonic shearing. This shearing is quite severe in this area in which the whole ophiolitic mass, part of an ancient ocean floor, was thrusted over the Pindos formations. Even when peridotites are sound, smooth serpentinised films coat the joints of the rocks facilitating structural instability.

A big cut in these formations was constructed in the early 90's before the main construction procedures were undertaken by "Egnatia Odos S.A.", the government company established to design build and operate this highway.

The slope has suffered from a series of slides, either in the weathered mass or in the ophiolitic rock mass which contains weak zones. It is clear that the slope, shown in figure 23, is not stable.

The remedial measures that were applied initially involved a series of piles along the toe of the slope at the level of the highway. A drainage layer was installed at the same level. These piles were intended to provide "shear pins" across the interface between the fill and the underlying rock mass. Given the conditions of the whole mass, concern with this system is that it depends upon the location of the potential failure surface. Weak zones were detected at depth in borehole cores (fig 24), during later site investigations. If such deeper weak layers fail then the piles will only serve to tie the fill together to create a larger failure (figure 25). Indeed the structure of the mass suggests that it is entirely possible that a failure surface can find its way along the weakest path through this rock mass which, in terms of slope stability analysis, can be treated as "homogeneous" due to the erratic distribution of weak zones.
Under such conditions and in order to accommodate the second carriageway under the "Big Cut", a stability and sensitivity analysis study of different alternatives was performed. Stabilisation of the toe of the slope by a buttressing counterweight, with additional drainage, was found as the most effective solution.

Another risk associated with this cut is from the possibility of generation of rockfalls. Several rockfalls have occurred since the slope was excavated and it has to be anticipated that such rockfalls will continue. In order to deal with this hazard a rockfall shelter was initially considered. However, this proved to be impractical because it was not considered feasible to design a structure wide enough to protect both carriageways. Finally it was decided to move the highway alignment onto a high fill on the valley floor.

Figure 23 General appearance of Big Cut with the Egnatia Highway platform running across the centre of the picture. The slope is a peridotite with disordered serpentinisation and tectonic shearing

Figure 24 Core of weak ophiolites recovered from borehole NO6 behind the Big Cut at depths from 60 to 65 m below surface. Pieces of clayey like transformed serpentine are present.

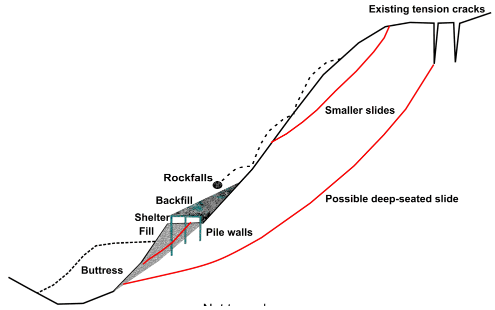


Figure 25: Potential failure modes in the "Big Cut" (not to scale). The sketch illustrates a number of possible alternative solutions. None of these solutions have been adopted and the highway has been moved onto a high fill in the bottom of the valley.

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6. References

Kazilis N., Georganopoulos Chr. 1999. The change of alignment in Metsovo area, *Conference on Egnatia Tunnels,* Ioannina, Greece (in Greek)

Marinos P., Hoek, E. 2000. From The Geological to the Rock Mass Model: Driving the Egnatia Highway through difficult geological conditions, Northern Greece, *10th International Conference of Italian National Council of Geologists: The fragile territory, Research and application on Hydrogeoloical disarray in the world, forecasting-prevention-mitigation,* 325-334, Rome

Marinos P., Hoek E., Kazilis N., Agistalis G., Marinos V. 2005. The tunnels of Egnatia highway. Design and construction in a variety of rock masses under difficult geological conditions, *Proceedings of Geoline 2005*, in CD

Papanikolaou D. 2003. Geology of Greece, *University of Athens publications*, Athens, Greece

Hoek, E., Marinos, P., Kazilis, N., Angistalis, G., Rahaniotis, N., Marinos, V. 2006. Greece's Egnatia Highway Tunnels**.** *Tunnels & Tunnelling International,* September 2006, 32-35