Protection measures against geological failures, during the construction of Thessaloniki - Kavala section of Egnatia Highway in N. Greece

M. Chatziangelou & B. Christaras
Aristotle University of Thessaloniki,. mcha@geo.auth.gr *,* christar@geo.auth.gr

ABSTRACT: The present paper describes the support measures against geological failures during the construction of Thessaloniki - Kavala section of Egnatia Highway in N. Greece. For this purpose, the mechanisms of sliding and rock falling procedures were studied.

As far as slopes concern, the orientation of the discontinuities and the poor quality of the rock mass, that creates circling sliding, were responsible for the instabilities. Rainfall also helps landslides to be occurred. During the tunnelling excavation, the sliding along a plane, the décollement from the roof and the fall of wedges were the common failure causes.

1. INTRODUCTION

The present paper refers to the confronting of possible geological failures along Egnatia Highway, in a 100km part, which connects Thessaloniki and Kavala cities, in North Greece (Fig.1). Actually, it is divided in three parts: i)Nymphopetra-Asprovalta, about 40km long, ii) Asprovalta-Strymonas, about 20km long and iii) Strymonas-St. Andreas, about 40km long. The highway has been, recently, constructed.
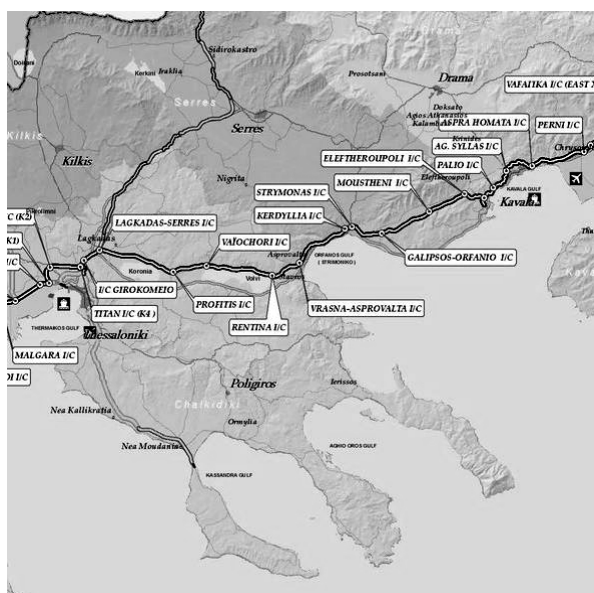

Figure1. Thessaloniki – Kavala section of Egnatia Highway

Driving from the west to the east, the highway, at the beginning of Nymphopetra-Strymonas part, passes near by Volvi Lake, at the foot of Vertiscos Mountain. Easterly, it passes through Kerdillia Mountain, Strymona's river and leads to Pangeo's Mountain, ending through Symbol Mountain.

The highway passes through five tunnels; i)Vrasna tunnel, which is located at Nymphopetra – Asprovalta's part, ii) Asprovalta's tunnels, which

are three tunnels locating at Asprovalta – Strymona's part and iii) Symbol Tunnel, which is located at the last Strymonas – St.Andrea's part.

The area is characterized by a variety of geological formations and numerous of faults, which are related to instability phenomena, which created difficulties to the excavations. Landslides have already occurred causing the delay of the use of highway. Landslides were due to circular sliding of soil or totally weathered material in addition to planar sliding along discontinuity planes, or intersection of two or three discontinuity planes, which formed potential wedges.

Tectonic data were collected and rock mass quality classification systems, RMR (*Bieniawski,1989*), SMR (*ROMANA, 1985*) and GSI (*Marinos et al, 2005*) were used in order to study the quality and estimate the stability along the slopes and into the tunnels. In stability analysis, test Markland (*Markland, 1972*) and Bishop Method (*Bishop, 1955*) were used.

2. GEOLOGICAL DESCRIPTION

The part "Nymphopetra – Strymonas" is geologically located into Serbomakedonian zone. At the beginning of the part, there are gneiss, amphibolite and thin layers of marble that belong to Vertiscos union. The above formations are placed under lake deposits, dated to Pleio-pleistokene. The lake deposits consist, generally, of gravel and sand layers. At the east part of the highway there is Keldilia union, which consists generally of gneiss and marble being crossed by pegmatitic veins. There are also alluvial deposits along the rivers.

At the eastern part of Strymon river, the highway passes through Symbol Mountain, which is geologically located into Pangeo's Union of Rodope Mass, consisting of gneiss, schists and

marble. The granite of Symbol union, which was created during the volcanism of Rodope Mass dated to Eocene – Oligocene, is placed deeper th

an the excavations, but it affects the above geological formations creating chloritization at the contact between granite and gneiss.

3. SLOPE O11 (CH.20+500 – CH.20+800) ALONG THE NYMPHOPETRA TO ASPROVALTA'S SUBSECTION

The slope "O11" consists of lake deposits. A fault of 255/46 cuts these deposits.
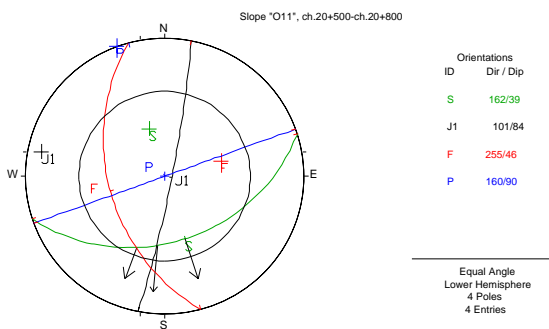

Figure 2. Potential sliding along the slope "O11"

The lake deposits may slide along the direction of the bedding 162/39. Two potential wedges were formed by; i) bedding 162/39 and fault 255/46 and ii) joints 101/84 and bedding 162/39. The sliding took place at SSW direction (Fig.2). Also, the critical slip cycle was estimated using slope geometry of 15m high and slope inclination of 90°. The radius of the critical circle was estimated 15m and the safety factor was calculated 3, for saturated conditions.

In order to protect the slope against sliding, a slope inclination of 34° was proposed. Using the proposed inclination, the radius of the critical circle was estimated at 29m and the safety factor was calculated 5 in saturated conditions.

4. SLOPE O19 (CH.25+215,89 AND CH.25+373,95) ALONG THE NYMPHOPETRA TO ASPROVALTA'S SUBSECTION

The slope is placed on the right hand of the road and consists of weathered gneiss with pegmatitic intercalations (Fig.3). The poor quality of the rock mass, together with the unfavorable orientation of the schistosity, affected the stability of the slope and caused extensive sliding.

Two types of probable sliding were determined, according to our investigation. The first one referred to slope sliding along the schistosity with direction of 221/36, and the second one referred to potential wedge sliding. There was a potential unstable wedge, which was formed by the surfaces of a fault with direction of 195/60, the schistosity with direction of 221/36 and the joints with

Figure 3. Slope placed on the right hand of the road

direction of 357/52.

According to *test Markland, 1972*, the schistosity of the gneiss with direction of 221/36 was parallel to the strike of the slope causing planar sliding conditions. Two approaches were considered, in order to protect the slope from sliding (Fig.4).

  i. Change of the slope inclination, or
  ii. Change of the highway direction.

Considering that the second solution was almost impossible, the slope geometry was studied using the limited inclination in order to ensure the stability without changing the road design. Using an inclination more than 36°, ex. 45° (1:1), we could not protect the slope from sliding. This was because the plunge of the schistosity was located inside the area, which was bounded from the slope plane and a friction angle equal to 21°. So, if the average inclination would be less than 34°, the safety was ensured.
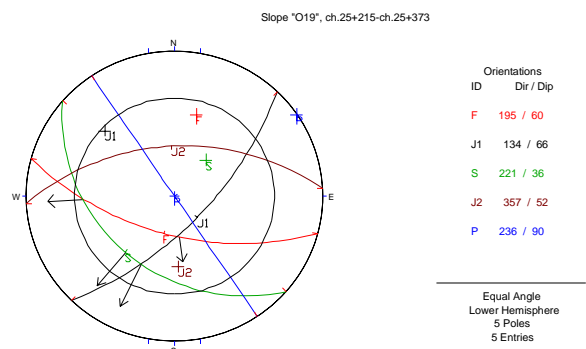

Figure 4. Potential sliding along the slope "O19"

In order to analyze the stability of the wedge, a friction angle for the fault surface of 21° was used, a friction angle for schistosity surfaces of 25° was used and a friction angle for joint surfaces of 35° was used. There was no cohesion between the discontinuity surfaces.

So, according to the above data, the safety factor of the wedge was calculated 4,18 determining high stability of the slope;

$$F = 4,18 \qquad (1)$$

F = safety factor

Critical Circle for all Sets:

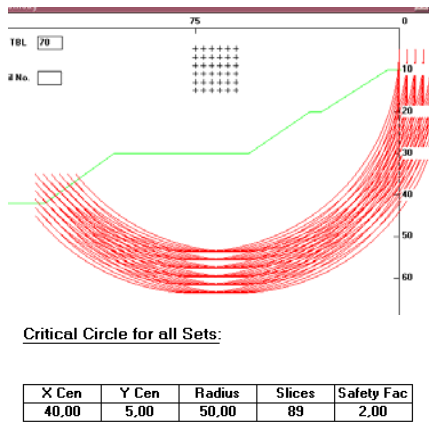| X Cen	Y Cen	Radius	Slices	Safety Fac
40,00	5,00	50,00	89	2,00

Figure 5. Circle sliding analysis and safety factor of slope "O19" after the embankment's construction , in unsaturated conditions

Gneiss was easily weathered, mainly by rainwater and salt spray coming from the sea, loosing important part of its cohesion behaving as a soil.

So, for the stability of the slope, a critical slip cycle was estimated for proposing the proper slope geometry.

The slope stability was performed using unit weight of gneiss equal to 2,65gr/m³, friction angle equal to 22° and no cohesion.

Two benches, 10m high were proposed, with inclination of 34° (V : H = 2:3). Low vertical benches, having width equal to 5m and height of 2,5m – 3,25m, designed to be created at the site of the embankment, so as the friction between the embankment and the slope to be increased. Using the geometry above, the slope in study was stable in dry conditions (S.F. = 2) and almost was unstable (or limited stable) in saturated conditions (S.F.=1) (Fig.5,6).

After the construction of the embankment, the safety factor, in dry conditions, was estimated equal to 2 and in saturated conditions equal to 1 (instability).

5. SLOPE O22 (CH. 26+057 - CH. 27+162) ALONG THE NYMPHOPETRA TO ASPROVALTA'S SUBSECTION

The slope "O22" consists of very cracked and totally weathered gneiss (RQD = 5%), covered by sand formation of Pleistocene.

Critical Circle for all Sets:

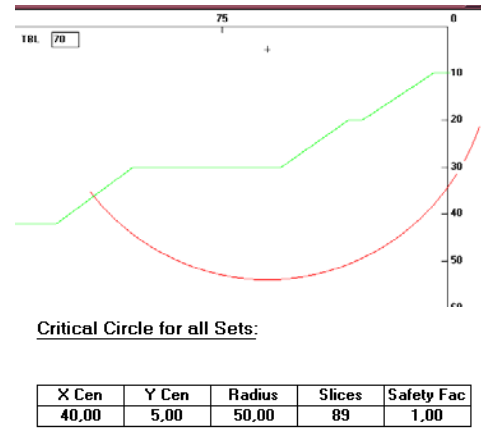| X Cen	Y Cen	Radius	Slices	Safety Fac
40,00	5,00	50,00	89	1,00

Figure 6. Critical slip circle and safety factor of slope "O19" after the embankment's construction , in saturated conditions

The quality of gneiss of the slope was poor as the rock mass was very weathered and absolutely cracked (RMRbas=28). For this reason, rock mass behaved as a soil. According to the SMR classification system, the slope was unstable.

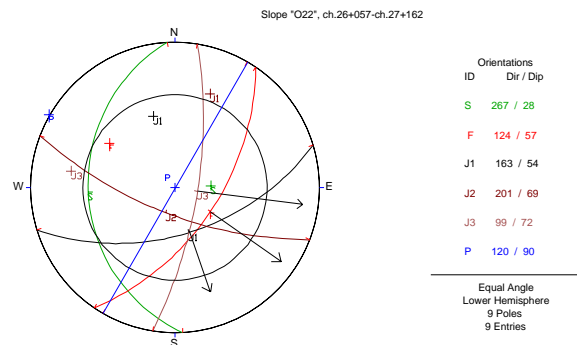Studying the stability of the slope, the gneiss

Figure 7. Possible sliding along the slope "O22"

rock mass slipped along; i) the fault 124/57 and ii) the joints of 99/72. Potential wedges were also formed by joints 201/69, 99/72 and joints 124/57 parallel to fault. Sliding took place at SSE direction.

Terraces inclined at 45° were proposed to form the "O22" slope in order to be protected from planar sliding. The maximum height of the slope was about 50m. Taking into account that the slope consists of totally weathered gneiss (γ=2,65gr/cm3,

φ=26°, c=O), there was a potential cycled sliding (Fig.7). But according to slope stability analysis in saturated conditions, the slope was stable as the calculated safety factor of the critic circle was 7.

6. SLOPE O23R (CH. 26+057 - CH. 27+162) AND O23L (CH.27+242 – CH.27+422) ALONG THE NYMPHOPETRA TO ASPROVALTA'S SUBSECTION

The slope "O23r" consists of marble and gneiss with pegmatite veins. The rock mass quality was estimated medium (RMRbas = 53, GSI = 53-

Slope "O23r", ch.27+242-ch.27+422, Gneiss formation

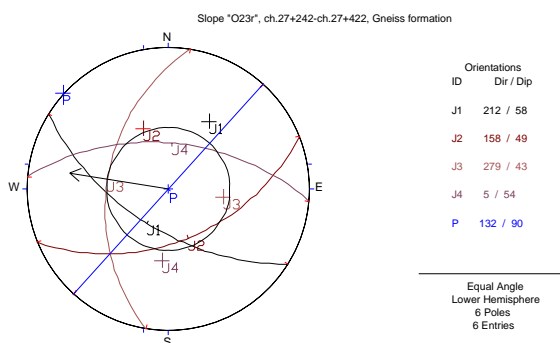| Orientations	
ID	Dir / Dip
J1	212 / 58
J2	158 / 49
J3	279 / 43
J4	5 / 54
P	132 / 90

Equal Angle
Lower Hemisphere
6 Poles
6 Entries

Figure 8. Possible sliding on gneiss at slope "O23r"

63) in the marbles, and good (RMRbas = 68, GSI=33-37) in the gneiss. The slope "O23l" is located at the left side of the road between ch.27+242 and ch.27+422. It consists of medium quality gneiss (RMRbas = 57, GSI=33-37).

According to the SMR classification system, the marble formation on "O23r" slope was moderately stable, although the gneiss formation on the same slope was stable. The "O23l" slope was moderately stable.

According to stability analysis of the "O23r" slope, the type of probable sliding was; i) a planar sliding of gneiss rock mass along the joints

Slope "O23r", ch.27+242-ch.27+422, Marble formation

Orientations	
ID	Dir / Dip
J1	83 / 53
J2	232 / 54
J3	287 / 71
P	132 / 56

Equal Angle
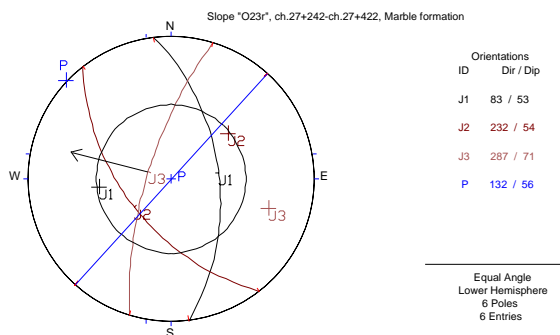Lower Hemisphere
6 Poles
6 Entries

Figure 9. Possible sliding on marble at slope "O23r"

directed to 279/43 and ii) a potential wedge sliding at west direction. The planes of joints 212/58 and 279/43 formed the wedge (Fig.8). The marble rock mass sliced along the joints of 287/71 (Fig.9). As the rock mass was slightly weathered, there was no possibility of circled sliding.

The gneiss rock mass of the "O23l" slope sliced

Slope "O23l", ch.27+242-ch.27+422

Orientations	
ID	Dir / Dip
F	143 / 72
J1	210 / 64
J2	295 / 70
P	182 / 90

Equal Angle
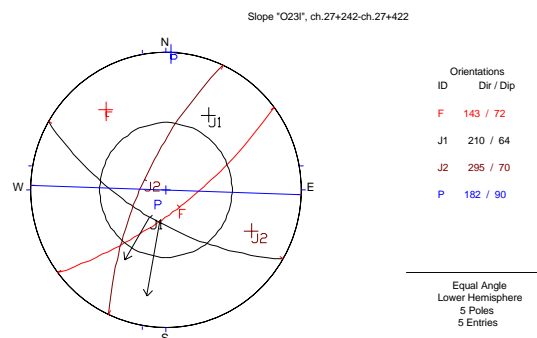Lower Hemisphere
5 Poles
5 Entries

Figure 10. Possible sliding along the slope "O23l"

along the planes of joints 210/64. A potential wedge, being formed by the joints of 210/64 and joints parallel to fault 143/72, sliced at South direction (Fig.10). As the rock mass was slightly weathered there was no possibility of circled sliding.

In order to protect the slope, against sliding, an inclination of slope equal to 45° was proposed.

7. VRASNA TUNNEL (CH.28+240 – CH.28+380) AT THE REDINA TO ASPROVALTA'S SUBSECTION

Vrasna tunnel belongs to Redina – Asprovalta part of Egnatia highway. The tunnel, which is about12 m high, consists of two parallel bores, 140 m long each, being oriented from the west to the east. A cavern is located at the northern part of the tunnel.

The quality of gneiss, which is closely jointed, was generally characterized as poor (IV), changing to very poor (V), near tectonic contacts. The quality of marble, which is widely jointed and less weathered than gneiss, was characterized as good (III) and near tectonic surfaces as poor (IV). The presence of karst phenomena, which were observed in marbles, during the excavation, was also taken into account on the estimations.

As it is well known, the failure of a rock mass around an underground opening depends upon the in situ stress level and the geotechnical characteristics of the rock mass. In highly stressed rock masses the failure, around the opening, progresses from brittle spalling and slabbing, in the case of massif rocks with few joints, to a more

ductile type of failure for heavily jointed rock masses. The presence of many discontinuities provides considerable freedom for individual rock pieces to slide or rotate within the rock mass (*Hoek et al, 1995*). Failure, involving slip along intersecting discontinuities in a heavily jointed rock mass, is assumed to occur with zero plastic volume change. For this purpose, in shallow tunnels, as the Vrasna tunnel is, the geometry of the discontinuities is considered to be the main instability cause (*Christaras et al, 2002*), taking also into account that no groundwater is present higher than the construction floor.

Thirty-seven unstable wedges, heavier than 5tns, were estimated. In order to stabilize the wedges, the thickness of shotcrete was considered 10cm and the length of rock bolts was considered 6m. Taking into account the orientation, the spacing of discontinuities and the overall ground conditions, the rock bolt spacing was considered to be varied from 1.5mx1.5m to 1.5mx1m.

In accordance to our estimations, shotcrete, up to 3cm thick, could support the majority of the wedges, increasing the safety factor up to 9,88. The maximum thickness of shotcrete, which could support successfully the wedges, without using other support measures, was 8cm, although in the most cases, shotcrete 1cm thick could effectively support the tunnel. Rockbolts, up to 3m long, could also support the most wedges, increasing the safety factor up to 9,43. Rock bolts 1m long, could support the most of these wedges. Five wedges could not be effectively supported by rockbolts, although they were effectively supported by shotcrete. Consequently, shotcrete could support with efficacy the unstable wedges better than rock bolts. As it was observed, according to the linear relation between he safety factor of the wedges being supported by shotcrete of 10cm thick and the safety factor of the wedges being supported by shotcrete with the minimum required thickness, the safety, provided by the installation of the proposed by RMR system shotcrete thick, was about ten times the safety provided by the shotcrete with the minimum required thickness installation;

$$SF_{shot.=10cm}= 9.6604*SF_{shotcrete}-4.1394 \qquad (2)$$

$$R^2 = 0,97.$$

$SF_{shot.=10cm}=$ safety factor of the wedges being supported by shotcrete of 10cm thick

$SF_{shotcrete}=$ safety factor of the wedges being supported by shotcrete with the minimum required thickness

Furthermore, as it was observed, according to the linear relation between the safety factor of the wedges being supported by bolts of 6m long and the safety factor of the wedges being supported by

bolts, with the minimum required length, the increase of bolts length more than 3m, did not increase the safety;

$$SF_{bolts=6m}= 0.988*SF_{bolts}-0.5776 \qquad (3)$$

$$R^2 = 0,91,.$$

$SF_{bolts=6m}=$ safety factor of the wedges being supported by bolts of 6m long

$SF_{bolts}=$ safety factor of the wedges being supported by bolts, with the minimum required length

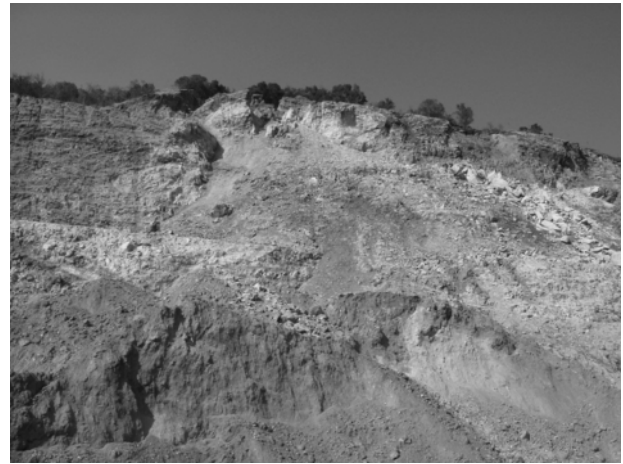## 8. THE HIGH SLOPE AT CHAINAGE 9+700 ALONG THE ASPROVALTA TO STRYMONA'S SUBSECTION

Figure 11. Landslide at ch.9+700 along the road

The slope, 60m high, consists of medium to high weathered, cracked biotic gneiss. The water of a short rainfall decreased the safety of slope and created landslide. The landslide was bounded at the half-formed part of the slope (Fig.11). Rock mass quality was pour (category IV). Tension
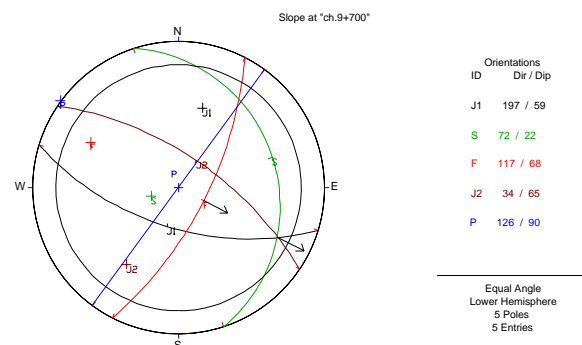

Figure 12. Possible sliding along the slope at "ch.9+700"

crack caused circular soil sliding. The critical circle in unsaturated conditions, was 50m deep and the safety factor was calculated 1,66. In saturate conditions, caused by rainfall, the safety factor was 1,16. A short rainfall, which had taken place on

Figure 13. Critical slip circle and safety factor of slope at "ch.9+700" using the proposed geometry in unsaturated conditions
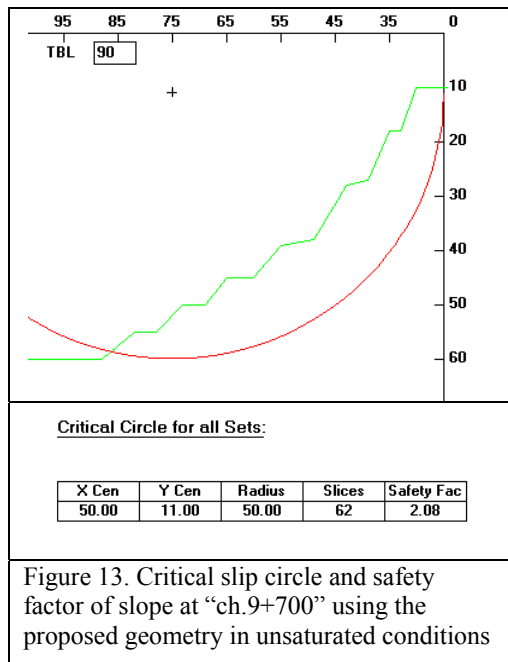
March 2002, created a landslide of the half part of the slope, which was just excavated using a beginning geometry. The slope consisted of small rock blocks. A cyclical sliding of loose material was bounded at east side of slope, where was a fault surface (115/65) and at west side of slope,
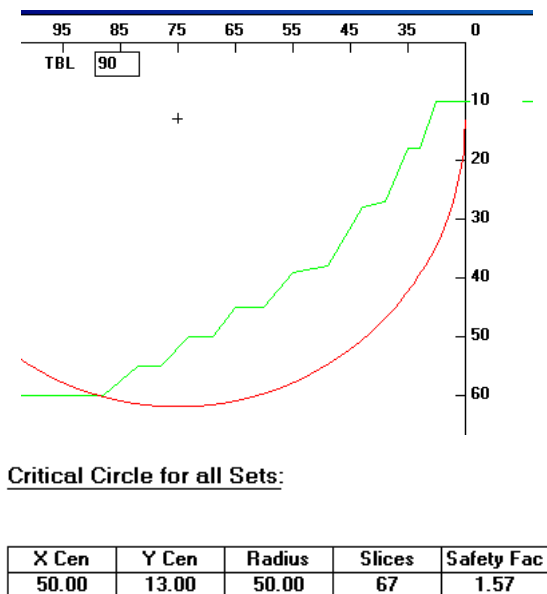

Figure 14. Critical slip circle and safety factor of slope at "ch.9+700" using the begging geometry in saturated conditions

where was a long sliding surface (120/76) (Fig12). Rainfall was the main cause of landslide at the safety factor (1,16) in saturated conditions was ultimate (*Chatziangelou & Christaras, 2005*).

For landslide restoration and support, dips of downstairs were proposed to be smaller (height/width=1/1.5 and 1/1) than dips of upstairs (height / width = 2 / 1). Using this new geometry, the safety factor was 25% higher (SF = 2.08) at unsaturated conditions and 35% higher (SF = 1,57) at saturated conditions than using the beginning geometry. The benches of the slope were designed about the same length so as the length did not affect the safety factor and pore pressure. Thus, safety factor depended only on inclination of benches. According to the new geometry, big dips were formed downstairs and small ones were formed upstairs. When the height was also increased, restrained powers were increased and sliding powers were decreased. So, the safety factor of every part was progressively increased downstairs to upstairs making slope stable (Fig.13,14).

9. LANDSLIDE IN AN OPEN TRENCH AT CH.10+500 TO CH.10+700 ALONG THE ASPROVALTA TO STRYMONA'S SUBSECTION

An open trench was planed to be constructed at the southern part of the Asprovalta tunnel II in order to replace the right bore of the tunnel (Fig.15). The excavation was performed from the upper parts to down slope. The geological formations, which were analyzed during the excavation were folded gneiss with joints of north and southeast directions, folded marble with joints of west and southeast directions, folded amphibolite, whose schistosity was oriented to northeast and southeast and the joints were of south and west directions. The gneiss and marble were co-folded and cut by three faults with NW, NE and SE directions.

Figure 15. The landslide in study

According to RMR classification system, the rockmass quality was characterized as poor (RMR_{bas} = 22-40, RMR = 3-17). There was a part

of slope between ch.10+570 and ch.10+640 consisting of amphibolite and marble formations with better rock mass quality (RMR_{bas} = 42-49, RMR=24-35) than the rest part of the slope. According to SMR classification system, the excavation of the slope was very unstable and extensive failures were expected.

Considering the stability of the slope, gneiss formation, being placed between ch.10+505 and ch.10+556 of the 3rd bench, might slide along the direction of schistosity 213/35. The same formation, being placed between ch.10+510 and ch.10+520 of the 2nd bench, might slide along the direction of schistosity 246/42 (Fig.16). There was a part of completely weathered gneiss, being placed between ch.10+670 and ch.10+680 of the 2nd bench, which might slide along the direction of the section being formed by schistosity 229/48 and joints 169/53 or 125/85. Marble formation, being placed between ch.10+600 and ch.10+640 of the 2nd bench, might slide along the direction of the section being formed by schistosity 195/39 and joints 278/76 (Fig.17, 18) .

The formation of the upper three benches

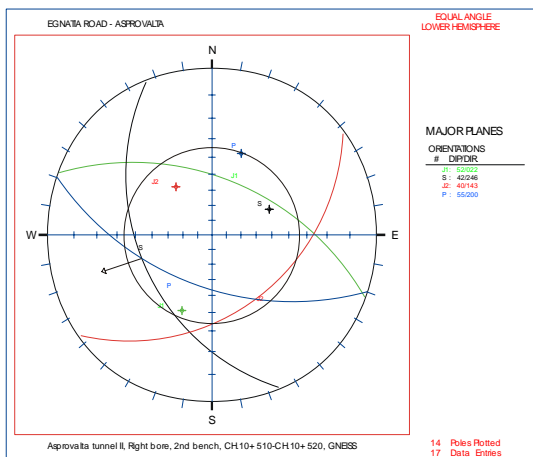

Figure16: Potential sliding in gneiss along the 2nd bench, between ch.10+510 and ch.10+520

(4th, 3rd and 2nd) was just completed when an extensive landslide took place. The landslide was placed between ch.10+533 and ch.10+620. The sliding of the trench took place. The sliding of wedge between ch.10+600 and ch.10+620, located in the marble of the 2nd bench, caused the beginning of landslide. The sliding occurred along the schistosity plane 195/39 and joints 278/76. The sliding of the wedge activated new sliding in the neighbor part between ch.10+533 and ch.10+600. The new sliding took place along the direction of gneiss schistosity 213/35. The excavation was interrupted for a long time and a new design of "cover and cut" was chosen to restore the failure (*Chatziangelou & Christaras, 2005*).
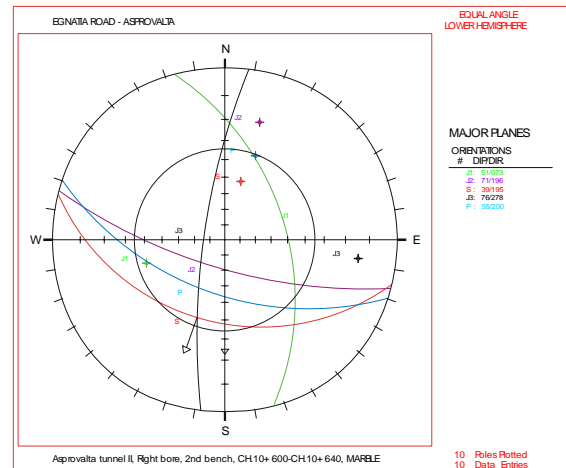

Figure 17. Potential sliding in marble, along the 2nd bench, between ch.10+600 and ch.10+640

10. ASPROVALTA TUNNELS (CH.10+022 – CH.10+708) ALOGN THE ASPROVALTA TO STRYMONA'S SUBSECTION

Asprovalta tunnels are two tunnels which are lengthen about 250m. The tunnel I consists with two bores. On the other hand the tunnel II consists with one left bore. The right bore of the tunnel II has been replaced by the "cover and cut" tunnel, which mentioned on the above paragraph.

The geological formations which were crossed by the tunnels were jointed gneiss, from slight weathered to abrasive rock mass, slight and medium weathered jointed marbles, good qualified amphibolite, pegmatitic and aplitic veins (*Christaras et al, 2002*)

The deformations, which were measured during the excavation, were less than 8,71cm along the X-axis and less than 4,4cm along the Y-axis. That is because the crack of rock mass helped at the detonation of tensions around the excavation. Although extensive failures were not possible to take place, big pieces of rock mass were failed down from the roof or sliced from the walls. The above observation showed that the deformations were not connected with the failures.

The failures were related to cracked wedges. The cracked wedges might fall down or slide along a sliding surface, as the connection of the rock mass, between the discontinuities, was not enough to balance the power of sliding, which was due to weight of wedges and the weight of the overlying formations.

Actually, the failure of wedges was created in stages. At the beginning, the sliding and falling of the small pieces of wedges, which uncovered during the excavation, took place. Those small

pieces caused the failure of the wedges behind themselves.

Comparing different support measures, shotcrete created the most effective support. Shotcrete some centimetres thick could effectively support the potential cracked wedges. We concluded to a relationship between the thickness of shotcrete and the face area of the wedges;

$$F_{(m2)} = 0,3489*(h_{(cm)})^2+16,654*h_{(cm)}+14,049 \quad (4)$$

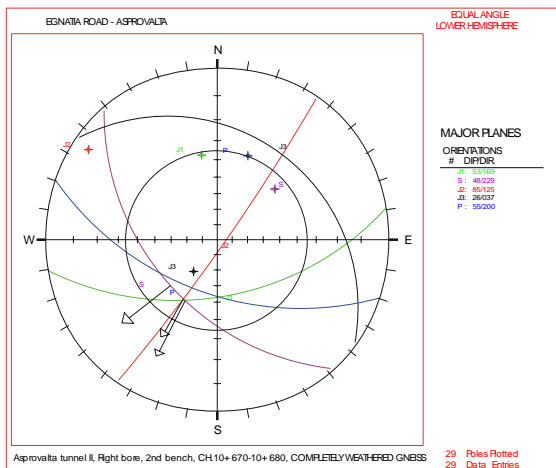Figure18. Potential sliding in complete weathered gneiss, along the 2th bench, between ch.10+670 and ch.10+680.

h = the thickness of shotcrete
F = the face area of the wedge

The correlation coefficient of the above relationship was 0,882.

The presence of rock bolts increased the safety factor 8%, but they could not support the rock mass using them alone. The rock bolts supported a point of the rock mass, so the pieces that were not in the influence radius of that point were sliding.

11. TUNNEL OF SYMBOL MOUNTAIN ALONG THE STRYMONAS TO ST.ANDREA'S SUBSECTION

Symbol's mountain tunnel belongs to Strymonas River – New Peramos subsection of Egnatia highway. The tunnel, which is oriented from West to East, is about13 m high and consists of two parallel bores, about 1150 m long each. The area is geologically located in mass of Rila - Rodope, generally consisting of gneiss, schist, amphibolites, marbles and plutonic rocks.

The tunnel was excavated dangerously because of the difficult geological status with unexpected failure conditions. The sliding along a plane, the décollement from the roof and the fall of wedges were the common failure causes.

Different methods were used in order to excavate the tunnel safely. The NATM method of excavation was used near the outlets and where the rock mass was very poor. On poor and medium quality of hard rock mass the explosive measures were the most effective. Also, light explosion was used in order to crack the hard rock mass helping the excavation. Chloritic schist formation and the places, where the loose weathered material flowed from the walls and the face, were excavated by the SCL method. The sudden change of rock mass quality created the necessity of fore polling.

One hundred and eleven wedges were measured along the right bore of the tunnel. All the wedges were to be collapsed, so the calculated safety factor, before the application of support was zero.

Twenty five wedges were observed to be supported with mechanical anchors with length of 6m. Five wedges were supported with swellex bolts. So, the mechanical anchors can support more wedges than the swellex bolts can. Also, there is no difference when the bolts are grouted at 50% of their length and are totally not grouted. The safety becomes bigger when the bolts are totally grouted. Forty seven wedges are supported sufficiently. Also, the grouted anchors, with 100% bond length, give higher safety factors than the grouted anchors with 50% bond length. The percentage of safety increases two times with the use of grouted anchors with 100% bond length. As far as shotcrete concern, seventy four wedges are supported effectively with shotcrete 5cm thick.

As the excavation of tunnels and the application of the support measures are dangerous, the quick calculation of shotcrete thickness during the excavation is useful. Comparing the apparent face of the wedges (the surface which is appeared at the inner surface of the tunnel) with the demanded shotcrete thickness (thinner than 40cm), in order the unstable wedges to be supported, a relationship was resulted;

$$F\ (m^2)\ = 0,0061 * [h\ (cm)]^2 + 0,7484 * h(cm) + 1,4068 \quad (1)$$

where h = shotcrete thickness (cm)
F = apparent face of the wedge (m2)

The coefficient of the above relationship was calculated 0,877.

12. CONCLUSIONS

The present paper describes the main places, along Thessaloniki - Kavala section of Egnatia Highway in N. Greece, with characteristic

instability phenomena, and studies the mechanism of sliding and rock falls, so as support measures and methods of excavation will be estimated.

The orientation of the discontinuities and the poor quality of the rock mass, were responsible for the instability. Rainfall also helps landslide to be occurred. Probable failures and landslides can be prevented if the inclinations of a) the low slopes (lower than 50m), are reduced to lower than 45°, and b) the high slopes (higher than 50m), decrease down-slope. Furthermore, the following solutions are also proposed: a) the construction of embankments in order to restrain the sliding of the slopes, b) the construction drainage systems in order to increase retaining forces, c) wire mesh in order to restrain small wedges and d) rock bolts in order to restrain wire mesh and wedges.

Concerning the support of shallow tunnels, where the water is placed down the floor of the tunnels, the measured deformations during the excavations were not connected with the failures. The cracked wedges were created the main failures, as they failed down or slide along a sliding surface, as the connection of the rock mass, between the discontinuities, was not enough to balance the power of sliding, which was due to weight of wedges and the weight of the overlying formations.

Comparing different support measures, shotcrete could support with efficacy the unstable wedges better than rock bolts. In addition to that, if we increase the thick of shotcrete, the safety factor will also be increased. On the other hand, the increase of the rock bolts length, does not affect so much the safety factor.

Examining the effectiveness of different types of anchors, on suddenly changed rock mass quality, we concluded that the mechanical anchors can support more wedges than the swellex bolts can. Also, there is no difference when the bolts are grouted at 50% of their length and are totally not grouted. The safety becomes bigger when the bolts are totally grouted.

Finally, comparing the apparent face of the wedges with the demanded shotcrete thickness, in order the unsteady wedges be supported, a relation being formed was resulted;

$$y = a * x^2 + b * x + c$$

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