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Diversion Tunnel of a Hydroelectric Plant on Nestos River (Greece). A Case Study

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SYNOPSIS A complete description of a geological and geotechnical investigation for the construction of a diversion tunnel in Platanovrisi dam area, East Macedonia, Greece is given. The numerical stress analysis and stress failure analysis results are presented, along with results of structurally controlled failure modes. The requirements for the application of the method are described. The conclusions from the application of the failure criterion, as well as the comparison between the analysis and the actual construction results have been discussed.

INTRODUCTION

The determination of the possibly unstable zones surrounding underground excavations is a crucial issue for the designing as well as the support estimation for underground constructions. The present paper deals with the analysis that was carried out prior to the excavation of a diversion tunnel on Nestos river in the eastern part of Macedonia, Northern Greece (Figure 1).

The tunnel is circular with a 12 m diameter and 495 m total length, yielding a maximum water discharge of 2000 cub.m/sec. It is a part of the underground constructions of the Platanovrisi dam that will be finished by 1994. The analysis was carried out for a semi-circular cross-section (12 m span, 6 m high) due to the excavation procedure and a two-dimensional boundary element method was used for the determination of the stress distribution around the different cross-sections. The six elaborated cross-sections were selected in order to meet two requirements: The geological conditions i.e. the incorporation of the complete range of geological formations, along with the ubiquity of each formation in every cross-section. The determination of the extent of the stable and unstable zones was accomplished by the application of the Hoek-Brown rock mass failure criterion.

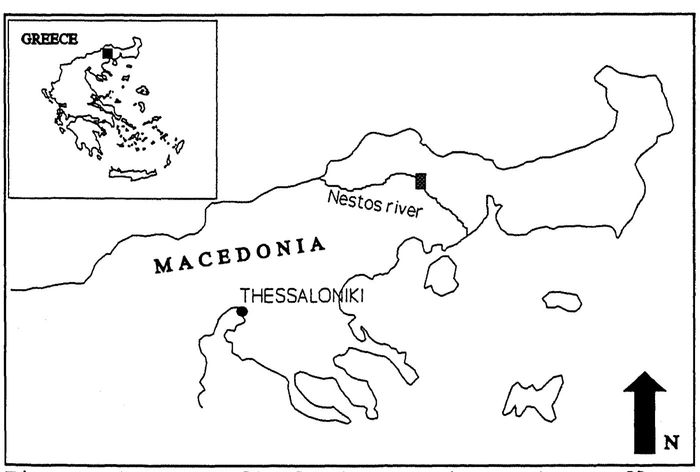

Figure 1. Map displaying Nestos river flow. Square includes dam area.

GEOLOGY AND TECTONICS

Nestos river flows through a big crystalline massif consisting of metamorphic rocks i.e. gneisses, mica-schists, marbles and amphibolites interrupted by granitic, granodioritic formations as well as volcanic intrusions. Following Dimitrov (1959) the massif is known as Rhila-Rhodope geotectonical zone and the age of the metamorphic rocks is Palaeozoic, where as that of the granitic rocks ranges from Eocene to Oligocene. The aerial photography and detailed mapping of the dam area, along with petrographic analysis produced the classification of the rocks into granite, gneiss (mainly biotitic) and granite-gneiss (intermediate type with absence of distinct schistosity). There was also calculation of the degree of fracturing (surface and underground), the degree of weathering and the water conditions in the rock mass. The underground investigations included data collected by the first of the authors from the investigation adits that had been excavated in both slopes of the dam, as well as from investigation drillings throughout the area. The adits and the boreholes were part of geotechnical investigation projects completed by various constructors for the Hellenic P.P.C. (Public Power Corporation).

The main results of the tectonic investigation of the area are the observed absence of major fault zones, the one predominant orientation of the schistosity planes, the existence of more than four joint sets (both surface and underground). Figure 2 includes the stereographic projections of the distinct underground and surface joint sets along with the schistosity planes. Table I contains a summary of the main characteristics of the measured discontinuities.

The observed weathering reached an average of 1-2 m in depth while the permeability of the rocks (calculated from Water-Pressure tests in

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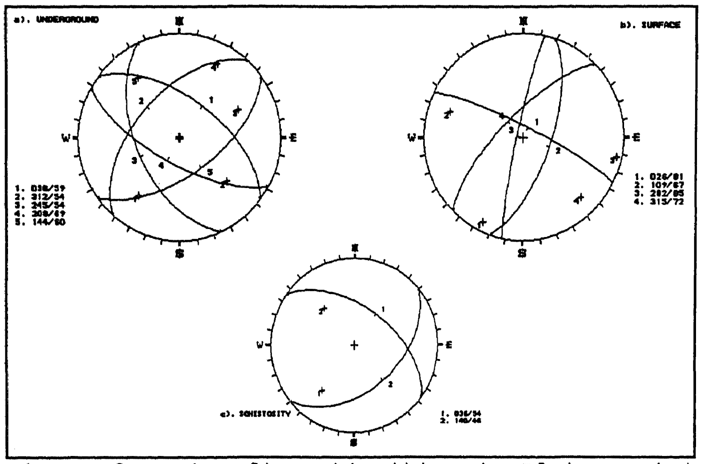

Figure 2. Major discontinuities in Platanovrisi area: a) Underground joint sets
b) Surface joint sets
c) Schistosity planes.

Table I. Ranges of values of degree of fracturing and joint spacing for the different rock types of Platanovrisi area.

Rock type	Degree of fracturing (m^{-1})		Joint spacing (m)	
	min	max	min	max
Granite	0.525	0.875	1.11	1.74
Gneiss	0.425	0.875	1.11	2.1
Granite-gneiss	0.225	0.775	1.21	3.63

the boreholes) was insignificant (very low discharge to practically impermeable).

ROCK MECHANICS INVESTIGATIONS

The mechanical properties of the rocks had to be identified in detail, in order to provide the subsequent analysis with adequate data and thus produce realistic and useful results. The field investigations by the authors included classification of the rock mass according to the CSIR classification system given by Bieniawski (1979), which was carried out along the intended longitudinal profile of the diversion tunnel. The parameters Rock Quality Designation (RQD) and groundwater, for the Rock Mass Rating were extracted from the adit and borehole data, by selecting the data associated with the excavation depth of the tunnel (157-169 m) in terms of absolute altitudes. The parameter rock strength was rated according to the uniaxial compressive strength of intact rock samples calculated by laboratory tests performed in the frame of the geotechnical investigation projects. The mechanical properties described so far were essential for the application of the failure criterion as it will be mentioned later.

For the stress analysis however the values of Poisson's ratio (v) as well as the unit weight of the rock types had to be taken into account. A

Table II. Physical and mechanical properties of the different rock types in Platanovrisi area.

Rock Types	No	Uniaxial Compressive Strength (MPa)		Poisson's Ratio		Apparent Weight (MN/m3)	
		Min	Max	Min	Max	Min	Max
Granite	15	23.63	125.21	0.05	0.5	0.025	0.027
Gneiss	21	9.79	104.71	0.05	0.5	0.023	0.027
Granite-gneiss	3	47.65	64.71	0.04	0.28	0.026	0.027

summary of the mechanical properties that were defined by the laboratory tests is given in Table II. As shown in Table II there is a variation in the values of the different properties. For the uniaxial compressive strength both maximum and minimum values were taken into account in the stability analysis. This helped in establishing the limiting conditions of the rock mass strength.

The values of Poisson's ratio were selected considering the maximum concentration of values that were obtained by the tests. These values were 0.175 for granite and gneiss (maximum concentration in the range 0.15-0.20) and 0.2 for granite-gneiss.

The rock mass classification using the already mentioned parameters for the elaborated cross-sections, produced Rock Mass Rating (RMR) values 64 to 87 for the optimum geotechnical parameters, and 54 to 77 for the reduced (minimum) strength. Consequently the rock mass is designated as good to very good rock, and fair to good rock respectively.

FAILURE INVESTIGATION

The selected approach to the potential failure problems was that of the application of the rock mass failure criterion suggested by Hoek and Brown in the 1988 updated form. The criterion has a wide range of applications and direct connection with field geological data, it is therefore very useful in problems of engineering geology. In the case of Platanovrisi diversion tunnel all the requirements specified by the authors for the application of the criterion are met. More specifically, the rock mass displays more than five sets of discontinuities with similar characteristics and mechanical behaviour, thus solving the problem of rock anisotropy. In addition, the tunnel "span to joint spacing" and cross-section "span to length" ratios exceed five, which covers the applicability of the criterion and plain strain conditions as well.

Finally the excavation depths allow for investigation of stress-induced failure. The aim was to deal with shear failure and the stable and unstable zones around the different cross-sections are shown as contours of equal "Strength Factor" values. The "Strength Factor" is defined, following the suggestion by Curran and Corkum (1991), as the ratio of the maximum internal shear at failure for a given confining pressure at a point, to the maximum induced internal shear at the same point due to the excavation. This is the ratio of S_{max} to S as shown in Figure 3.

The value of S_{max} is extracted by the Hoek-Brown

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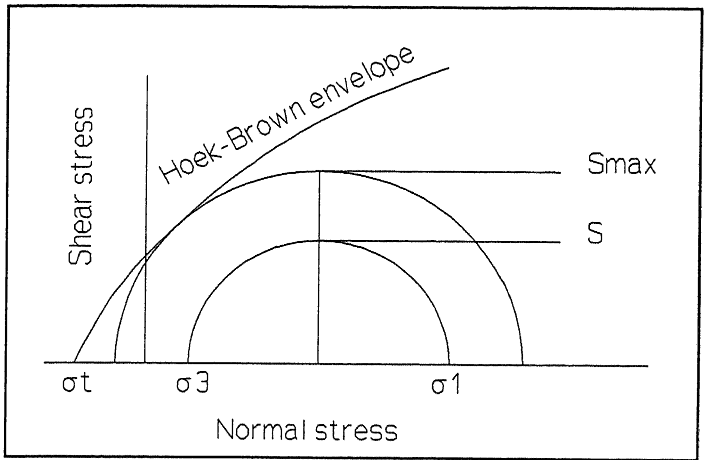

Figure 3. Hoek-Brown failure criterion and "strength factor" definition.

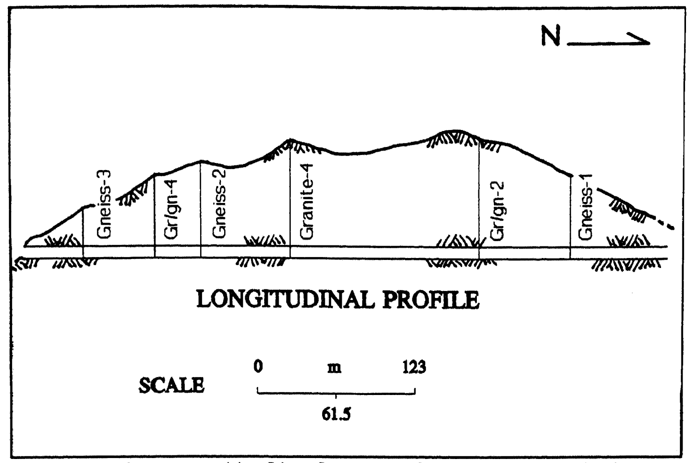

Figure 4. Longitudinal profile of the Platanovrisi diversion tunnel.

criterion equation, considering maximum internal shear $S = (\sigma_1 - \sigma_3)/2$ and confining pressure $P = (\sigma_1 + \sigma_3)/2$, and is defined by Equation 1.

$$S_{max}=\sqrt{\left(\frac{m\sigma_c}{8}\right)^2+\frac{m\sigma_c P}{4}+\frac{s\sigma_c^2}{4}}-\frac{m\sigma_c}{8} \qquad (1)$$

In Eq.1 σ_c is the uniaxial compressive strength of the intact rock, while m and s are the material constants introduced by Hoek-Brown and are calculated using the authors's pertinent relations, following the Equations 2 presented by Priest and Brown (1983):

$$\frac{m}{m_i}=\exp\left(\frac{RMR-100}{14}\right)$$
$$s=\exp\left(\frac{RMR-100}{6}\right) \qquad (2)$$

The value of m_i is the value of m for intact rock sample and is taken equal to 25 following the approximate values given by Hoek (1990) for coarse-grained polymineralic igneous and metamorphic rocks (gneiss, granite, granodiorite etc.).

STABILITY ANALYSIS

The stress calculations were performed using two-dimensional indirect boundary element method, for plain strain conditions in an isotropic elastic material. The program used was EXAMINE 2d version 3.1 by Curran and Corkum (1991), applying symmetrical solution in a 496 point grid with a 49 linear element boundary discretization.

The problem of in situ original stress field was handled by tectonic data kinematic analysis, in order to determine the active (more recent) tectonic regime of the area, combined with data presented by Hoek and Brown (1980) and Goodman (1980), for K, i.e. the ratio of horizontal in situ stress to vertical stress. The maximum and minimum principal stress axes of the original stress field were calculated applying the P and T Axes and Right Dihedrons Area method according to Angelier and Mechler (1977) and were found to be vertical and horizontal respectively (extensional tectonic stress field), which simplifies the field stress conditions and agrees with the plain strain deformation assumption.

Consequently and taking also into account the shallow depth of the excavation, the field stress calculations were performed considering a gravitational stress field, i.e. linearly varying with depth. For the calculation of the horizontal stress, the values of K finally selected were K=1 and K=0.33, which in the authors' opinion cover the expected original in situ stress conditions.

Table III. Elaborated cross-sections' Rock Mass Ratings and material constants.

Cross-sections	Rock Mass Rating		m		s		Depth from surface
	max	min	max	min	max	min	(m)
Gn-1	75	70	4.192	2.052	0.015	0.0029	50.5
Gn-2	64	54	1.91	0.935	0.0025	0.0005	61.7
Gn-3	75	70	4.192	2.052	0.015	0.0029	26
Gr-4	87	77	9.878	4.836	0.114	0.022	79
Gr/gn-2	75	72	4.192	2.933	0.015	0.0094	80
Gr/gn-4	74	69	3.903	2.731	0.013	0.006	52

As already mentioned in the introduction six cross-sections were selected, by adjusting the results of the geological mapping to the proposed tunnel direction. The positions of the cross-sections are shown in Figure 3 which also shows in scale the depth of each cross-section below ground surface.

The complete series of calculated RMR (rock mass ratings) and values of m and s is included in Table III.

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STRESS AND STABILITY ANALYSIS

Stress analyses results are presented here briefly with examples of three cross-sections, one for each geological formation. The contoured cross-sections in Figures 5-7 display the distribution of the maximum principal stress around the excavation for K=1, while the ones in Figures 8-10 for K=0.33.

The contour labels show the magnitude of the

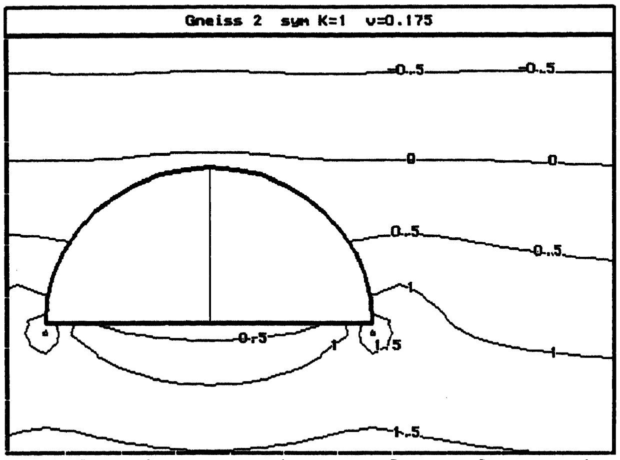

Figure 5. Stress contours of σ_1 for Gneiss-2 cross-section and K=1.

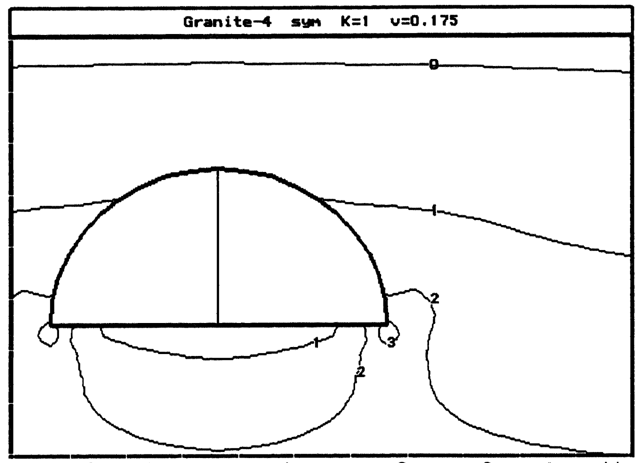

Figure 6. Stress contours of σ_1 for Granite-4 cross-section and K=1.

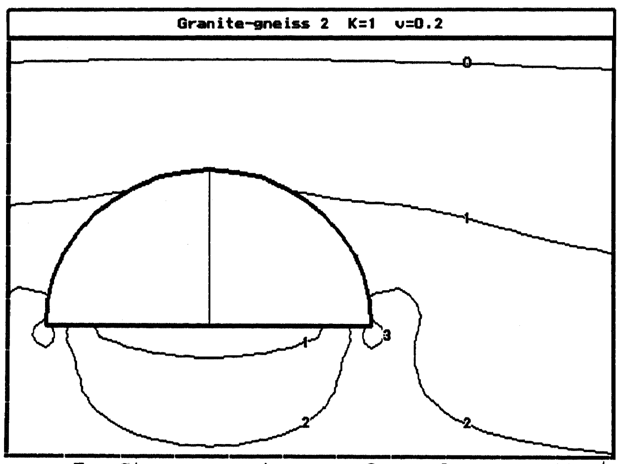

Figure 7. Stress contours of σ_1 for Granite/gneiss-2 cross-section and K=1.

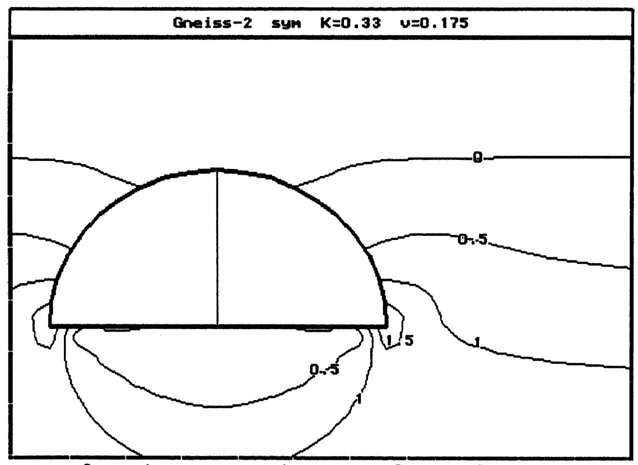

Figure 8. Stress contours of σ_1 for gneiss- 2 cross-section and k=0.33.

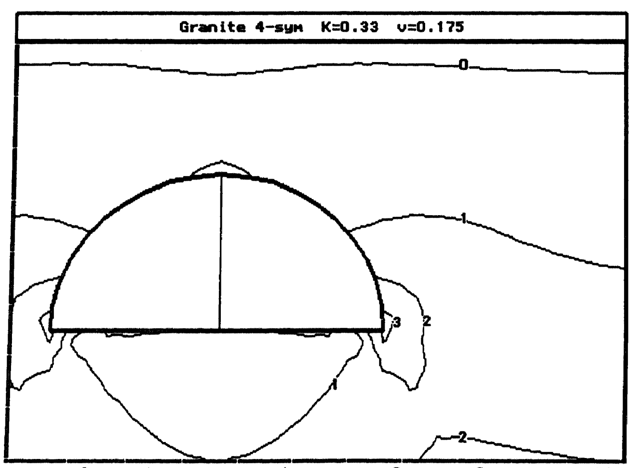

Figure 9. Stress contours of σ_1 for Granite-4 cross-section and K=0.33.

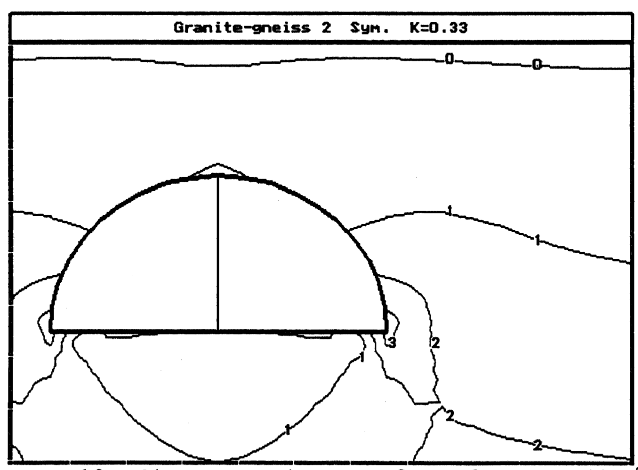

Figure 10. Stress contours of σ_1 for Granite/gneiss-2 cross-section and K=0.33.

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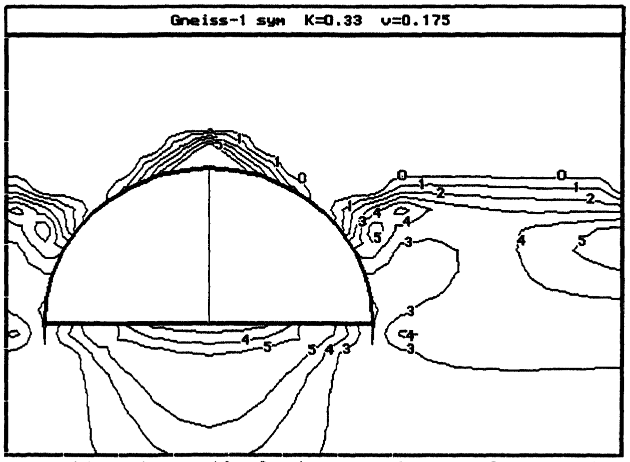

Figure 11. Strength factor contours for Gneiss-1 (maximum strength parameters for K=0.33).

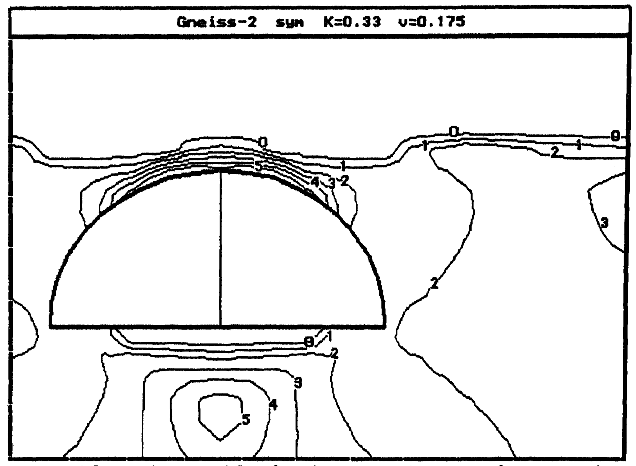

Figure 12. Strength factor contours for Gneiss-2 (maximum strength parameters for K=0.33).

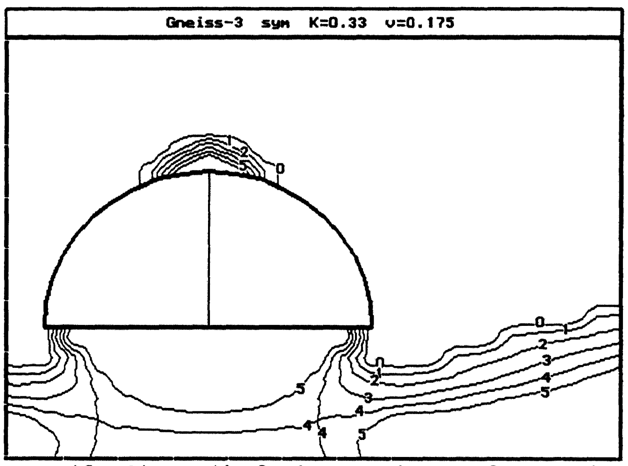

Figure 13. Strength factor contours for Gneiss-3 (maximum strength parameters for K=0.33).

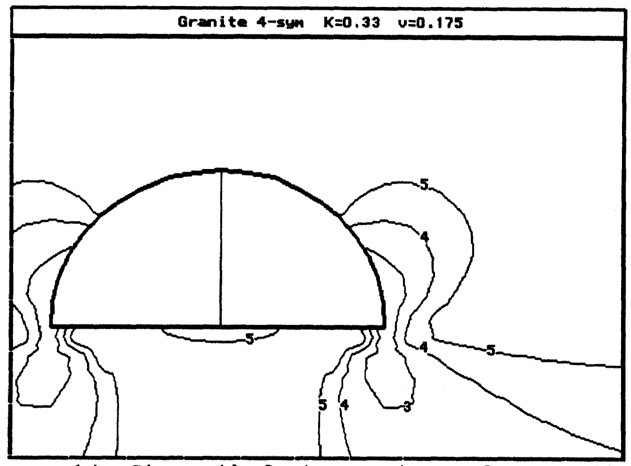

Figure 14. Strength factor contours for Granite-4 (maximum strength parameters for K=0.33).

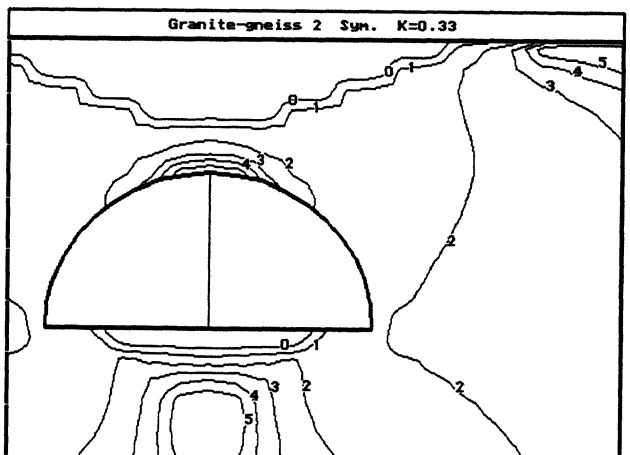

Figure 15. Strength factor contours for Gr/gneiss-2 (max. strength parameterers for K=0.33).

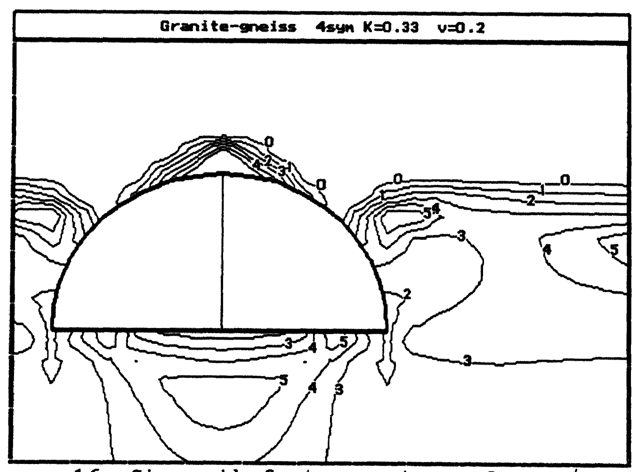

Figure 16. Strength factor contours for Gr/gneiss (maximum strength parameters for K=0.33).

maximum principal stress σ_1 in MPa. It must be mentioned that no additional support was used regarding the boundary conditions during the stress calculations. The grid density was adjusted in order to produce sufficiently detailed contouring around the excavation, combined with acceptable elaboration time. The same assumptions were made for the total number and size of the elements.

The subsequent stage as already mentioned was

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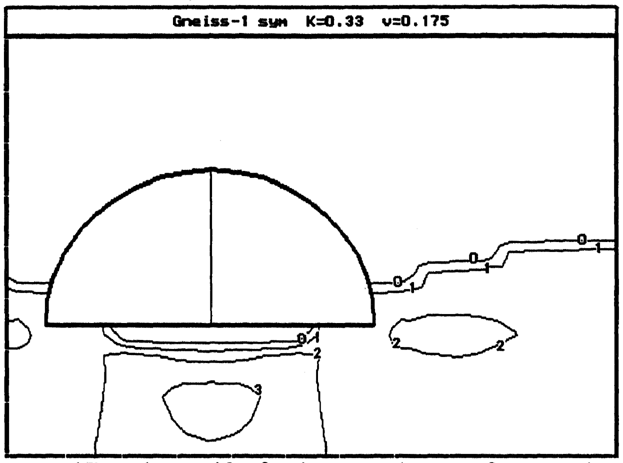

Figure 17. Strength factor contours for Gneiss-1 (minimum strength parameters for K=0.33).

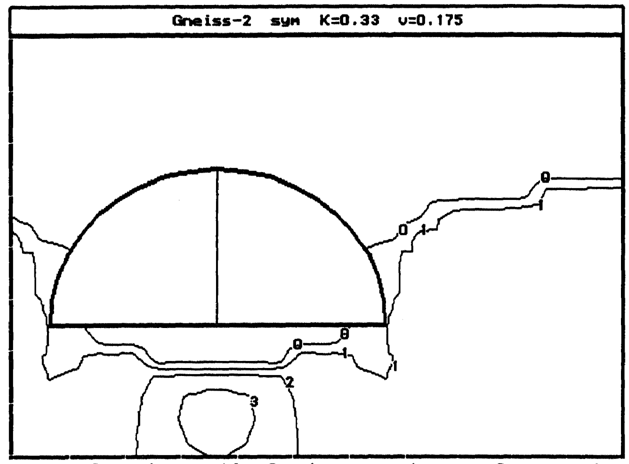

Figure 18. Strength factor contours for Gneiss-2 (minimum strength parameters for K=0.33).

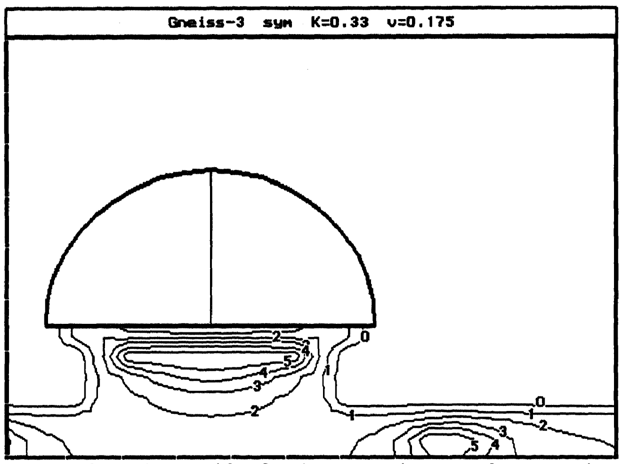

Figure 19. Strength factor contours for Gneiss-3 (minimum strength parameters for K=0.33).

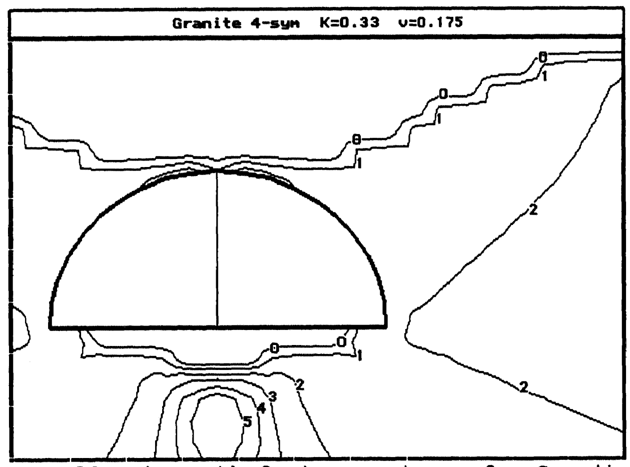

Figure 20. Strength factor contours for Granite-4 (minimum strength parametrs for K=0.33).

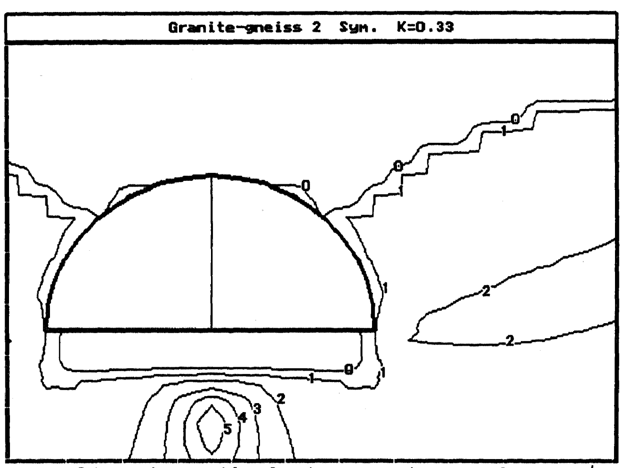

Figure 21. Strength factor contours for Gr/gneiss-2 (minimum strength parameters for K=0.33).

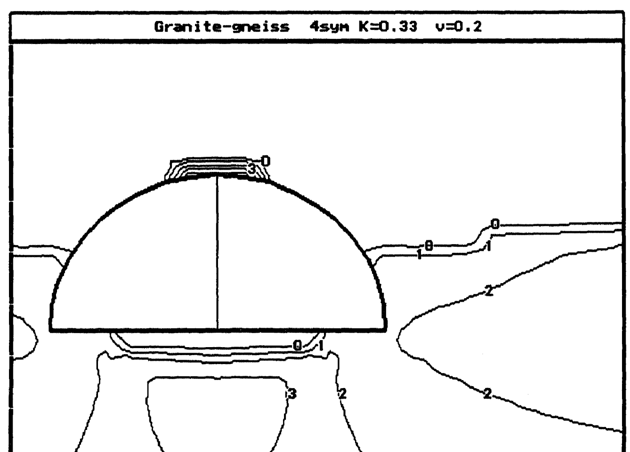

Figure 22. Strength factor contours for Gr/gneiss-4 (minimum strength parameters for K=0.33).

the stability analysis by means of failure criterion application. The strength factor contours are plotted with values between 0 and 6. Figures 11–16 show the output for elaboration with maximum strength parameters and ratio of horizontal stress to vertical stress K=0.33. The elaborated cross-sections for K=1 have not been included to abbreviate the paper size.

The calculated strength factor values which exceed 6 are contained in the same contour inter–

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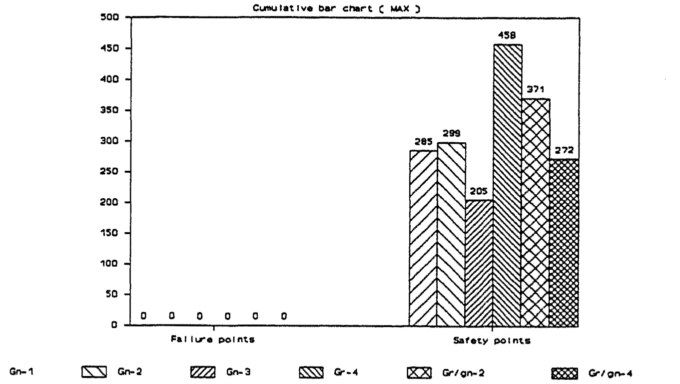

Figure 23. Strength factor distribution for all cross-sections (max. strength parameters, K=1).

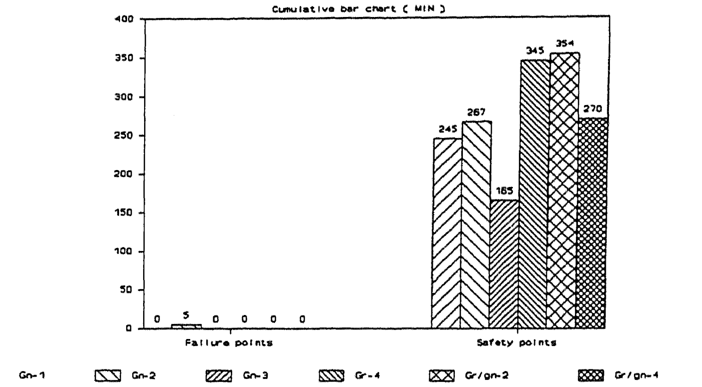

Figure 24. Strength factor distribution for all cross-sections (min. strength parameters, K=1).

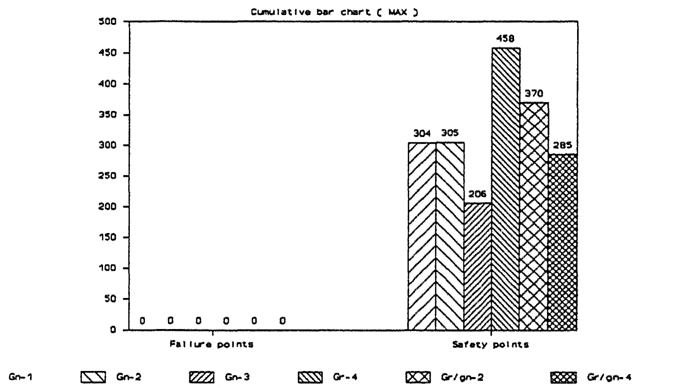

Figure 25. Strength factor distribution for all cross-sections (max. strength parameters, K=0.33)

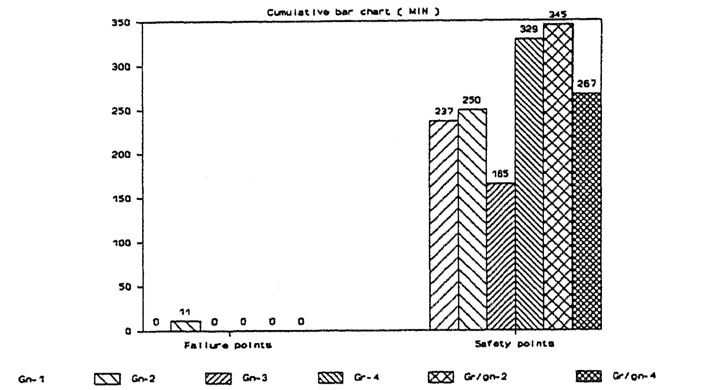

Figure 26. Strength factor distribution for all sross-sections (min. strength parameters, K=0.33)

val as those of 6. The value of 0 strength factor is the limit for values of σ_3 (minimum principal stress) less than the calculated tension (negative stress) cutoff for the Hoek-Brown criterion.

The tension cutoff value is the boundary of the Hoek-Brown failure envelope in the tensile area (σ_t in Fig. 3). The important areas for the stability of the excavation are the areas of strength factor less than 1 i.e. the shear failure areas. No shear failure can occur in the case of σ_3 less than the tension cutoff value.

Figures 17-22 show the same cross-sections but this time the values inserted in the calculation are the minimum strength parameters determined by the investigation prior to the stability analysis.

The areas of probable instabilities can be well distinguished from the contoured cross-sections. It is obvious that in the event of such areas bordering the excavation boundary, problems are likely to emanate in the course of construction. The strength factor contours are plotted on the basis of the individual grid point values of the factor. Therefore the amount of shear failure (instability) for a constant 496-point grid, is pertinent to the percentage of grid points displaying a strength factor value less than 1. A special statistical elaboration of all the grid point values, for the total number of cross-sections, should produce a more complete view of the rock mass.

The results of such elaboration are summed up in the bar charts of Figures 23-26.

The "safety points" bars show the total number of grid points for which $_3$ is greater than the tension cutoff value and at the same time the value of strength factor is greater than 1. The results display a fluctuation of the total safety points, along with a lack of failure points, except for the single case of Gneiss-2 which still constitutes a mere 0.13% of the total number of calculated point values.

The small amount of stress induced calculated failure led to the supplementary assumption that possible instability risks could arise from tectonically (structurally) controlled failure. Hence a wedge analysis was carried out for the three more frequent joint sets of the underground (1991). The shear strength parameters (cohesion and friction angle) of the joint sets used were the ones determined by the in situ direct shear tests performed during the investigation projects.

The program calculates the largest wedges that can be formed for the given intersecting joint sets and the excavation surface, along with the weight of each wedge. A safety factor is assigned to each wedge. It is obvious that this stage of analysis overlooks the stress field, which was used in the previous stress induced failure analysis. Representative results of the elaboration of two groups of joint sets, are shown in Figure 27 . The joint sets used are:

a) 036/59 b) 036/59
312/54 312/54
144/60 245/54

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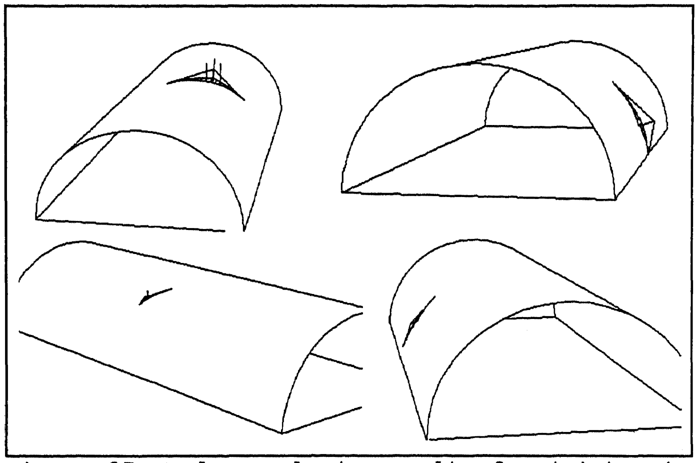

Figure 27. Wedge analysis results for joint sets of group a (top row) and group b (bottom row). Safety Factors (from top left to bottom right): 1.02 (stabilized), 3.63, 1.13 (stabilized), 6.24.

The size of the wedges shown in Fig. 27 is relative. The postulated conditions show that the wedges likely to be formed can be stabilized with rock bolts as shown in the first case, with bolt loadings depending on the weight of the wedge. More specifically the weights of the four cases of rock wedges presented here are:
Top left: 637 tonnes - Top right: 401 tonnes
Bottom left: 4.4 tonnes - Bottom right: 55 tonnes.

CONCLUSIONS, DISCUSSION

The primary conclusion for the construction of the Platanovrisi diversion tunnel was that it will encounter optimum geological and tectonic conditions, in good quality rock mass, with low mechanical disturbance.

The stress induced failure analysis pointed out that no problems should occur under the rock mass strength parameters that had been determined by the geotechnical investigation.

The structural failure analysis showed that rock wedge formation is likely and must be taken into account, though no major problems should originate. More specifically the investigation results lead to the following:

- The minimum strength parameters for the rock mass produced insignificant stress failure problems.
- Potential mechanically weak zones i.e. shear and mylonitic zones could cause stability problems, but can be dealt with using the same methodology, in order to determine the means of overcoming the problems.
- The rock types show stress behaviour analogous to their mechanical properties, thus being classified in ascending order of supporting capacity as gneiss, granite-gneiss, granite.
- The boundary element stress analysis for very shallow excavation points produces extended tensile zones with stress values less than the Hoek-Brown criterion tension cutoff. The possible tensile failure points must be examined separately.
- The rock wedge stability can be adequately controlled.
- Besides rock bolting, in the event of wedge formation, additional support measures will not be needed.

The excavation of the diversion tunnel of Platanovrisi was performed in two stages. The first stage comprised the semi-circular boring of the total length by rock blasting mode, while the second the excavation of the remaining cross-section area. This was the reason for the selection of a semi-circular geometry stability analysis, since no problems are expected in the second stage of the excavation.

The completion of the excavation of the tunnel verified the results and conclusions of the analysis. No stress failure problems were detected in the rock mass. Pre-existing single shear zones crossing the excavation at some points were encountered. The stability problems caused by these zones, which were usually filled with mylonite with very low compressive strength, are foreseeable and easy to integrate in the failure analysis, and relevant work by the authors is under publication. The support measures used were shotcrete and rock bolting of potentially unstable wedges. The size of the rock-wedges was small, usually of the type shown on the bottom right and left corners of Figure 27.

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