Impact of Ground Water on the Design and Exploitation of an Open Coal Mine in Northern Greece

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ABSTRACT

The design and construction as well as the long term stability of the slopes after exploitation for a large open coal mine at Northern Greece was the purpose of a complete geological, hydrogeological and geotechnical investigation. The aim of the project was the geotechnical design in conjuction with the underground water influence during the exploitation period so that the most cost-effective but engineering safe solution would be provided. The stability analysis involved three types of failure, deep circular failure, wedge rock type failure as well as planar sliding failure following the most unfavorable dip orientation. The final excavation of slopes at the design proposals was only feasible following the construction of a complete network of drainage system with deep trenches. Nevertheless, expected minor and localised landslides were also observed, as a result of the combined action of pore water pressure and surface water erosional effect.

INTRODUCTION

The present paper describes the principles of the method of modelling and design that was employed to assess the correct geotechnical characteristics and ground behaviour associated to the ground water influence for the openning and exploitation of a large coal mine at the area of Kozani in Northern Greece.

The site investigation works aimed at the assesment and correct evaluation of the physical and mechanical properties of the soil layers encountered during exploitation through two different fronts. The central front presented a length of 550 m and the maximal height of exploitation for the lignite layers was calculated to be 80 m. The southern front had a length of 460 m and a maximal exploitation height of 60 m approximately. The in-situ works comprised the geological mapping of the two fronts areas at scales 1:200 and 1:5000, the sampling of intact samples and the measurements of the discontinuities of the soft rock mass. Special attention was given during mapping to the superficial erroded soil cover, which presented intense creep behaviour, mainly due to the influence of the ground water at different sub-acquifers.

GEOLOGICAL - GEOTECHNICAL CONDITIONS

The area of interest is a part of a major tectonic graben created during Neogene and then filled up by neogene sediments and possibly by Lower Pleistocene sediments.

The elevation of the area varies between 840 and 880 a.s.l. and there are two torrents crossing from the west to eastwards (the first one separating the central front from the southern one).

The main geological formation consists of sandstone marls prevailing over a few marly sandstones, mainly diversified by the average grain diameter observed. According to the geological mapping, there are two types of marl formations: the superficial erroded cover, having a mechanical behaviour of a loose clayey sand with small gravels and the lower sane part.

The superficial erroded and altered part, subyellow colour, extends to a depth varying from 6 to 8 m approximately, presents some trace of the thin - bedded structure of the marl but has lost its initial cohesion and consistency and is easily foliated.

The lower, sane part of the marl has a thickness of approximately 100 m., light grey colour, and presents a very fine structure. Locally is intensively bedded to 08/85 (dip/orientation) and has intercalations of thin sublayers of yellow - green colour showing the difference in sedimentation. Local pelitic interfaces were also observed. In its totality, the soft rock mass is rather dense but where it stays unprotected, it is altered and rather highly foliated.

The lignite layers are encountered at the lower limit of the marly rock mass and present thicknesses varying from 7,00 to 12,00 m.

The fracturation in the considered neogene sediments is related to the vertical post - alpine tectonisme, thus the tension cracks were expected almost vertical. Measurements of the rock mass discontinuities corresponding to the exploitation of both the central and the southern front were taken and evaluated using the SCHMIDT diagrams that follow (Fig.1). Generally, the discontinuities are long, mainly closed and present the systems:

J_1 = 80/310
J_2 = 75/335
J_3 = 80/290
J_4 = 10/095 (Bedding Plane)
J_5 = 40/155
J_6 = 80/140

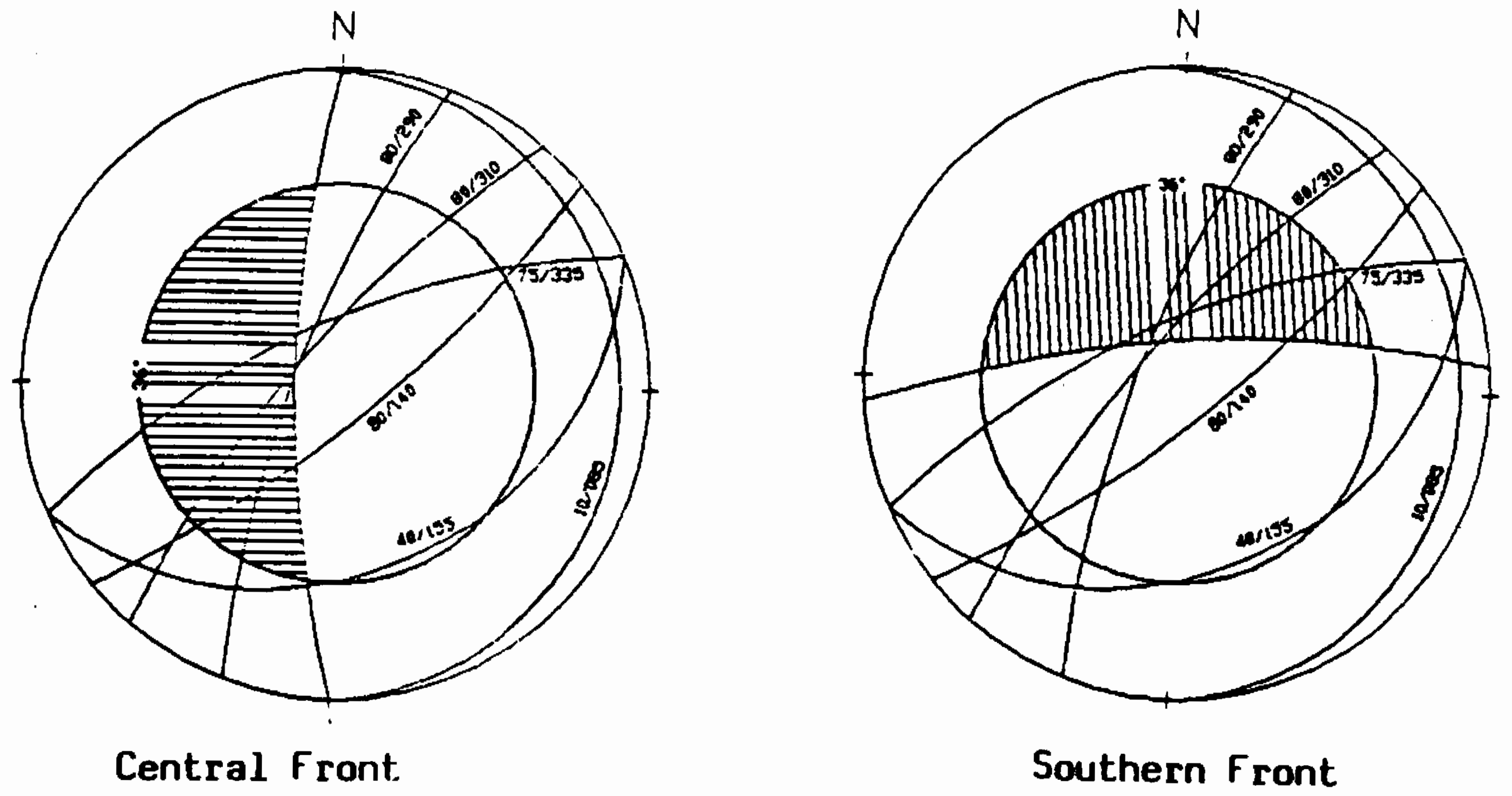


FIG.1. STATISTICAL EVALUATION OF DISCONTINUITIES

In effect, the discontinuities measured are almost vertical (75°-80°) except the bedding plane and the system J_5 (40°). This latter one presents probably a secondary dip or even a primary one, due to a certain lithological transition, which may form a weakness plane tranformed to a fracture surface. The plasticity of these marl formations and the surface alteration cause the partial or total disappearance of the fractures in surface and the progressive "healing" of the discontinuities in larger depths(2).

The hydrogeological behaviour of the marls depended directly upon its alteration. The lower sane part is generally impermeable while ground water circulation was observed in the upper altered and erroded part, and mainly at the interface. The presence of this water table activated the J_4 system of discontinuities and caused local landslides with traces extending to 80 - 90 m beyond the excavation limit. Some older fissures and cracks were also observed at a distance of 200 - 220 m beyond the excavation limit, while creep was evident in the boreholes limiting a superficial zone of 5,00 - 6,00 m thickness of altered marl.

The unfavorable combination of ground water to the bedding plane induced instability at the regions of excavation and had to be attentively taken into account during the final design stage.

The essential point is the foresight of drained or undrained conditions during the successive excavations.

LABORATORY TESTING - GEOTECHNICAL DESIGN PARAMETERS

An extensive laboratory tests program was carried out and permitted the evaluation of the necessary geotechnical design parameters. This program included grain and hydrometer size analysis, Atterberg limits, wet and dry bulk density, specific gravity, organic material and $CaCO_3$ content, petrographic analysis, direct shear and triaxial compression tests, uniaxial compression tests on both intact and remolded samples (clay sensitivity).

Existing compressibility tests on calcite marls showed different results related to the environment of the test (air, capillary saturation, partially saturated, totally saturated).[4]

Because of the importance of the shear geotechnical parameters to be used in the final design, a statistical analysis was carried out on both direct shear and triaxial compression (C.U.P.P.) tests results with the assumption that :"minimal values of the angle of friction and cohesion are considered those that even lower values could be present in samples amount not exceeding 10% of the total". Following the exception of those values, the least squares best fitting curves were considered and average values used in the design. The modelling system involved two main types of soil formations with different mechanical behaviour:

<u>(a) SUPERFICIAL ERRODED MARL</u>

		FRICTION ANGLE (°)	COHESION (KN/m^2)	MINIMUM REQUIRED SAFETY FACTOR
WITHOUT PORE PRESSURE	Mean Values	24	79,0	min F_1 = 2,00
	Most Unfavorable Combination	17	13,0	min F_2 = 1,50
WITH PORE PRESSURE UP TO U= 50%	Mean Values Effective Stresses	22	75,0	min F_3 = 1,10
Bulk Density γ = 16,2 KN/m^3				

<u>(b) LOWER SANE MARL</u>

		FRICTION ANGLE (°)	COHESION (KN/m^2)	MINIMUM REQUIRED SAFETY FACTOR
WITHOUT PORE PRESSURE	Mean Values	41	149,0	min F_1 = 2,00
	Most Unfavorable Combination	24	60,0	min F_2 = 1,50
WITH PORE PRESSURE UP TO U= 50%	Mean Values Effective Stresses	45	262,0	min F_3 = 1,10
Bulk Density γ = 16,6 KN/m^3				

For the few cases of design where the excavation was realised through the first lignite horizon, the shear parameters employed were identical to the ones corresponding to the lower marl.

SLOPE DESIGN

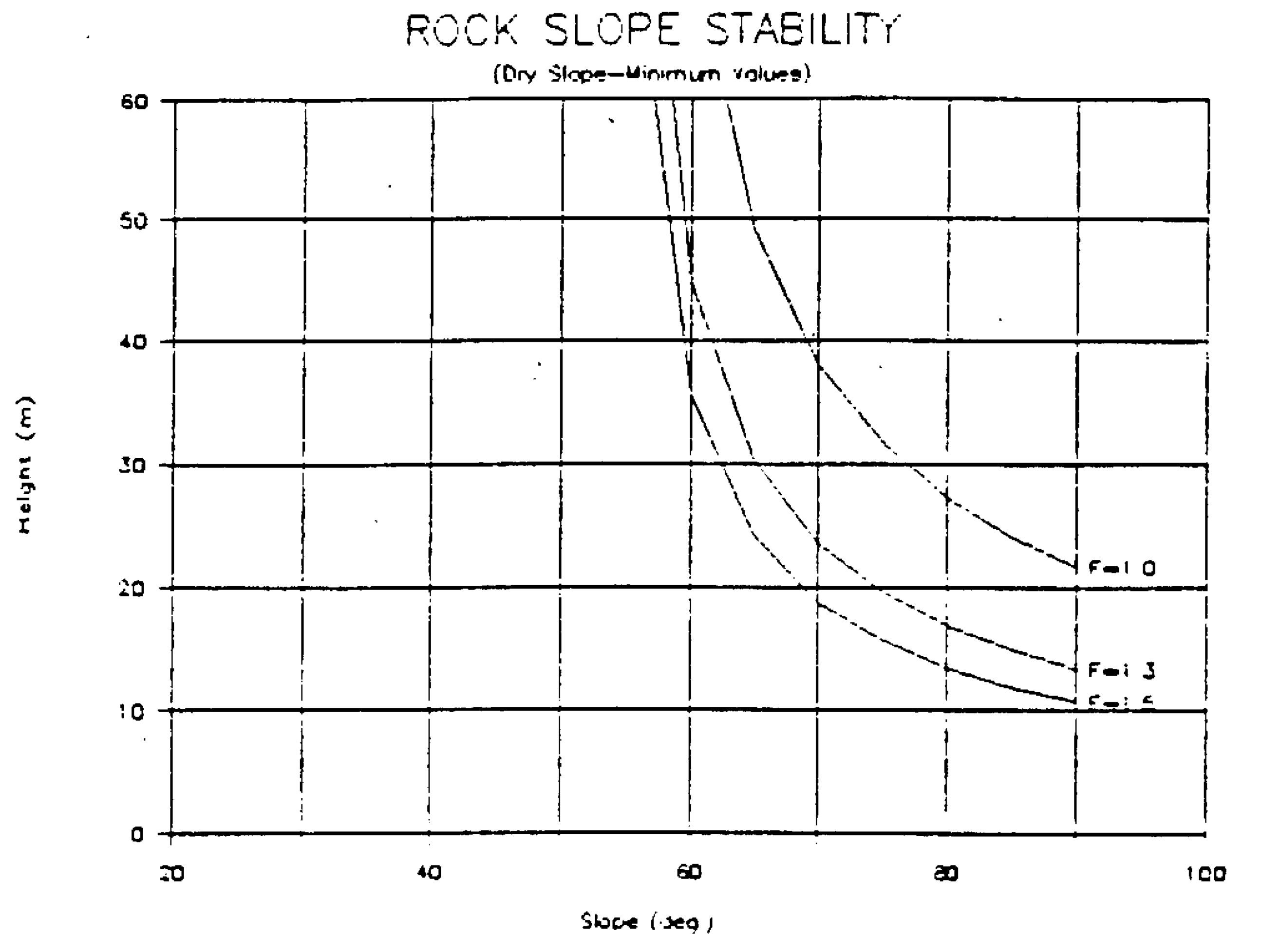
The success of the slope stability design depends directly upon the realistic and correct evaluation of the geotechnical and hydrogeological parameters of the formations encountered during an excavation (a difference of 3° in the average slope corresponds to some 100.000 m^3 of waste excavated material). The geotechnical behaviour of the marly formations and their natural slopes at the area mostly favorise (from the cinematic point of view) the deep circular failure mode as most probable to occur at the scale of the total height of excavation. Nevertheless, rock wedge type failure and planar sliding along the bedding plane J_4 could not be excluded, especially at the scale of each isolated excavation step. Therefore, the strict definition of only one failure mode might be adequate for one location but very conservative or even dangerous at another. The slope design procedure was the following:

(a) Design against deep circular sliding using BISHOP'S method of analysis for the final excavation height for both fronts. Four cross-sections of the central front and two cross-sections of the southern one were checked using the average shear parameters (F_1 = 2,00), the minimal shear parameters corresponding to the most unfavorable combination (F_2 = 1,50) as well as the influence of the increase of pore pressure in the discontinuities. This increase corresponded to partial up to full saturation degree (U=25-50%, 50% = saturation) with the use of effective stresses and a minimum required safety factor of F_3 = 1,10. Due to construction reasons and mainly the available equipment, the excavation should proceed by using a maximum height of each step of 10 m. Safety factors calculated for an average slope of 40° varied as follows:

FACTORS OF SAFETY

	MEAN VALUES WITHOUT PORE PRESSURE	MOST UNFAVORABLE COMBINATION φ,c	SATURATION
CENTRAL FRONT	F_1 = 1,65 ÷ 4,60	F_2 = 1,33 ÷ 3,12	F_3 =1,20 ÷ 1,81
SOUTHERN FRONT	F_1 = 2,40 ÷ 3,38	F_2 = 1,55 ÷ 2,29	F_3 =1,44 ÷ 2,08

(b) Design against rock wedge type failure[3] was performed at the scale of each separate excavation step, considering two different heights of most possible geometrical form of the wedge : 12 m and 20 m. The increase of the pore pressure was controlled and the safety factors observed for this case allowed the design to adopt that the height of 10 m for the excavation step and its nearly vertical face (i=80-90°) do not create any istability for the lower sane marl formation (Fig.2).



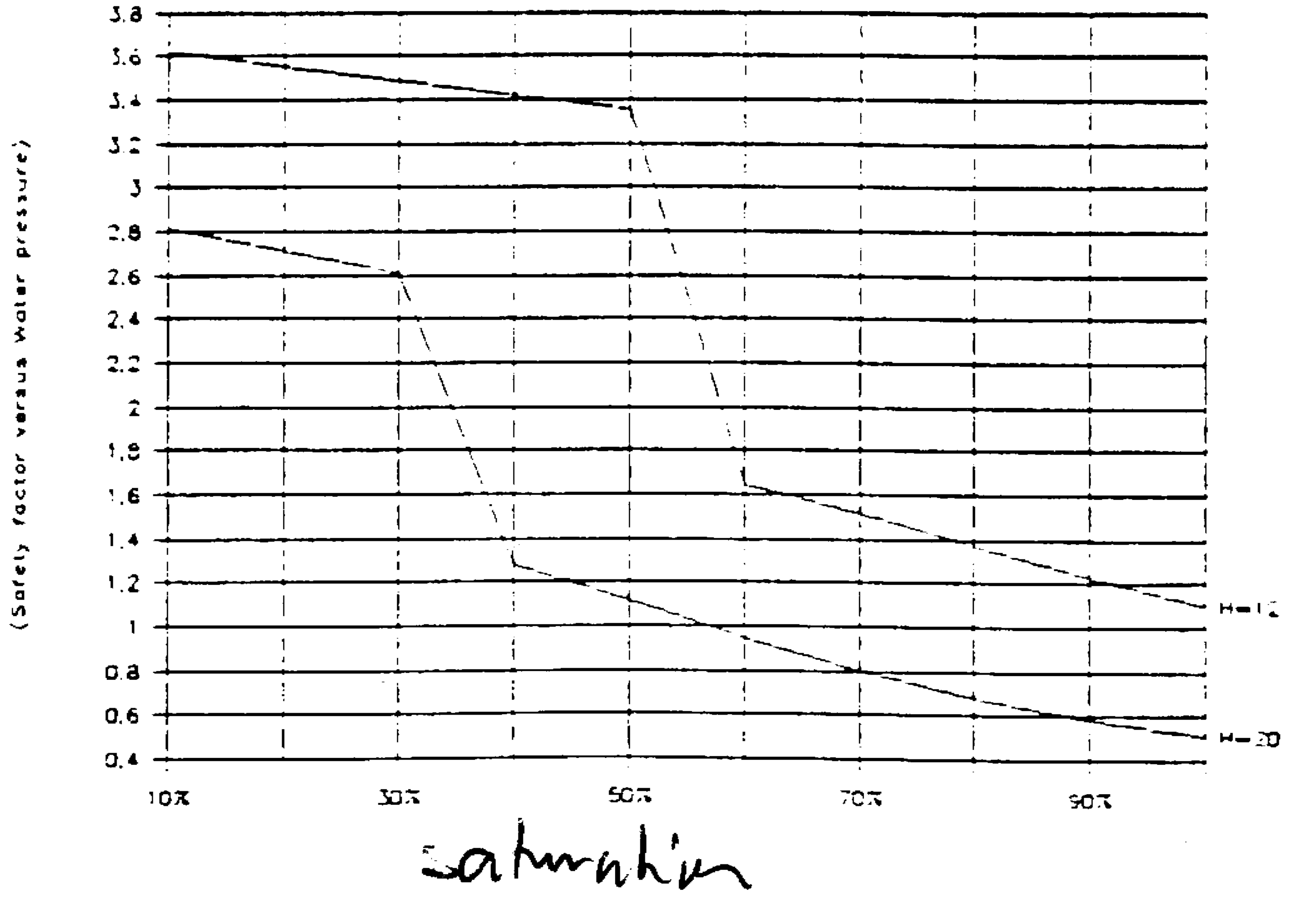


FIG.2. WEDGE TYPE ROCK MASS STABILITY DESIGN

(c) Design against planar sliding was performed at the scale of each separate excavation step (H=10 m) for the most unfavorable bedding plane J_4. The safety factors[6] calculated for the dry slope vary between 1,98 and 2,83 and the influence of pore water pressure obtained safety factors varying from 0,77 to 1,25 (Fig.3).

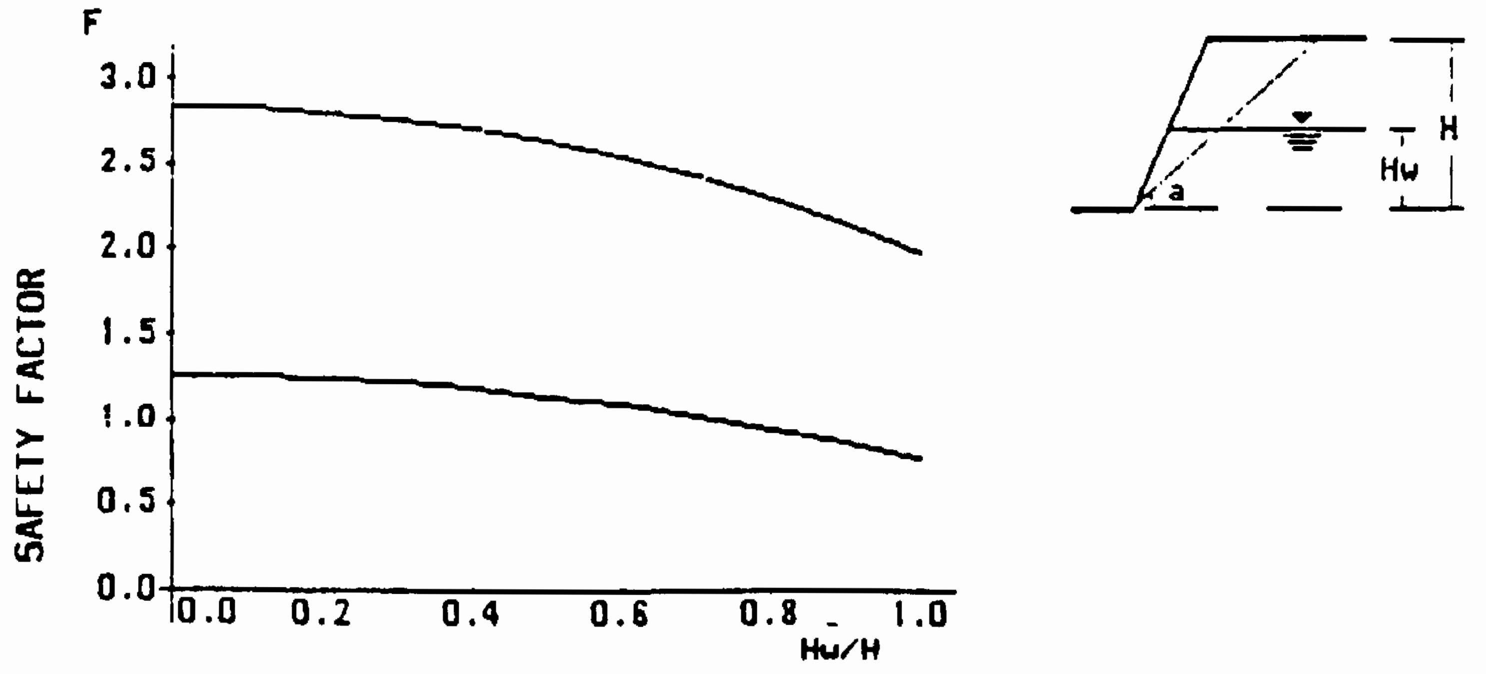


FIG.3. PLANAR SLIDING

CONCLUSIONS

The slope design concluded that the short term stability of the excavations for both the central and the southern front is satisfactory proven that average slopes during construction would not exceed 40°. The influence of ground water in the interface between the altered superficial marl and the lower sane layers is determining in the stability of each separate excavation step (vertical front for a height of 10 m). Therefore, a complete network of deep drainage trenches was constructed beyond the limit of excavations within the sane marl layers. Those drains were equipped with perforated 200 mm tubes, gravel filter and geotextile wrapping to assure the correct function of the draining procedure. Due to the superficial creep of the altered marls, large excavations had to be performed to remove the creeping zone away from the limit of the excavations.Nevertheless,local failures were expected at the scale of each excavation step (safety factor in planar sliding lower than F_3 = 1,00) and indeed, occurred periodically during the excavations. But those failures were not of an important scale and did not influence the rate of excavation, since continuous geotechnical and geological observations are assured throughout the excavation period.

REFERENCES

1. BJERRUM, L. Progressive Failure in Slopes of Overconsolidated Plastic Clays and Clay Shales. Journal of Soil Mech.and Found. Div., ASCE, Vol.93 No SM5,1967.

2. EDAFOMICHANIKI L.T.D. Stability Analysis for the Coal Mine in Servia - Kozani. ATHENS 1989.

3. E.HOEK - J.W.BRAY. Rock Slope Engineering. Third Edition. London 1981, p.199-224.

4. KOWALSKI, W.C. Influence of Capillary Water on Compressive Strength of Marls. Bull IAEG, No 5 p55-56, 1972.

5. SKEMPTON, A.W. Long Term Stability in Clay Slopes. Geotechnique Vo.14, No 2, 1964.

6. B.STIMPSON. Factor of Safety of Planar Wedge. Eng. Geol. 1979, Vo.12