Stability of an Erratic Tailings Deposit

P. C. Kotzias
Kotzias-Stamatopoulos Co. Ltd, Athens, Greece

A. C. Stamatopoulos
Kotzias-Stamatopoulos Co. Ltd, Athens, Greece

B. Karas
Public Power Corporation of Greece, Athens, Greece

Stability of an Erratic Tailings Deposit

P. C. Kotzias and A. C. Stamatopoulos
Partners, Kotzias-Stamatopoulos Co Ltd, Athens

B. Karas
Public Power Corporation of Greece, Athens

SYNOPSIS 150 million cubic meters of erratic colliery spoils were deposited between 1971 and 1983 close to the Kardia lignite fields, in Greece. The heap finally reached satisfactorily an escalating height of 73 m. Field reconnaissance, collation of local experience and geotechnical investigations were performed during midstage of deposition. They aimed at an optimal configuration of the fill in progress, within the possibilities allowed by extensive earth moving operations. Present paper outlines the approach and the methodology used to arrive at strength parameters and slope design of a highly inhomogeneous massive comprising variegated soils, marls and encaptured lignites.

Fig. 1. View of Fill at the Time of Investigations

INTRODUCTION

The mountainous deposit shown partially in Fig.1 was developed by colliery wastes generated from the nearby "Kardia" deep open lignite pit, in north-western Greece. Its final plan dimensions are about 3,000 m x 800 m, with an escalating height of 73 m (Figs. 2, 11), amounting today to approximately 150 million cubic meters of spoils. The main bulk about 100 million cubic meters was placed gradually between early 1971 and June 1982. The remaining fill was intermittently deposited up to October 1983. It rests now over a stable and moderately flat ground on a region of low seismicity. Late pliocene lacustrines - primarily marls and lignites, and the more recent alluvials, viz. clays with sands and gravels in various combinations, comprise a heavily overconsolidated terrain (Kotzias, 1982). Spoils extracted from this terrain resume a state of normal consolidation after being transported by conveyor belts, placed in heavy dumpers and simply spilled over. They form a mound of erratic composition with a slope-berm configuration, comprising diverse soils and marls, including encaptured lignites. A deep slide could harm people and expensive equipment working at the disposal, block a railroad line close to its western side, or encroach on the fringes of a northern village. It would have only minor repercussions on the adjacent southern lands.

METHODOLOGY OF APPROACH

Intensive field reconnaissance, review of local experience and geotechnical investigations, performed between September and December, 1978, when the fill was about 37 m high - aimed at designing the future slopes and berms in conformity with the vast and inflexible earth moving operations. The logs of mining explorations were reviewed to bracket the ranges of forthcoming soil wastes. The behavior of older and the present fill were collated. Numerous crossections were taken. 522 meters of borings with dense, sampling and standard penetration testing have been carried out primarily at the critical north and north-eastern side, and at the location of a corrected slide to be mentioned later (Fig. 2). Water table at the very deposit was monitored by piezometers. Laboratory test results will be dealt with in the sequel.

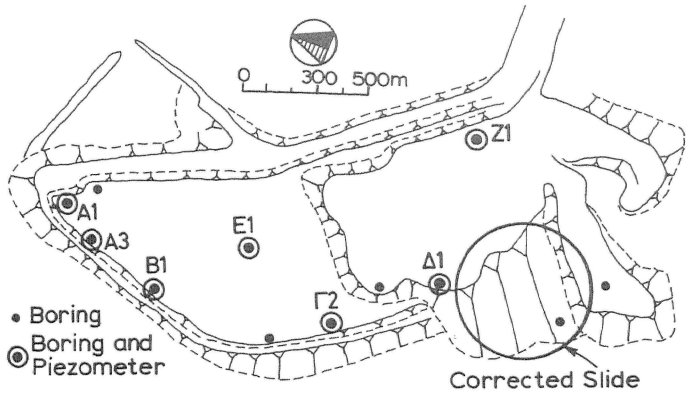

Fig. 2. Plan at the Time of Investigations

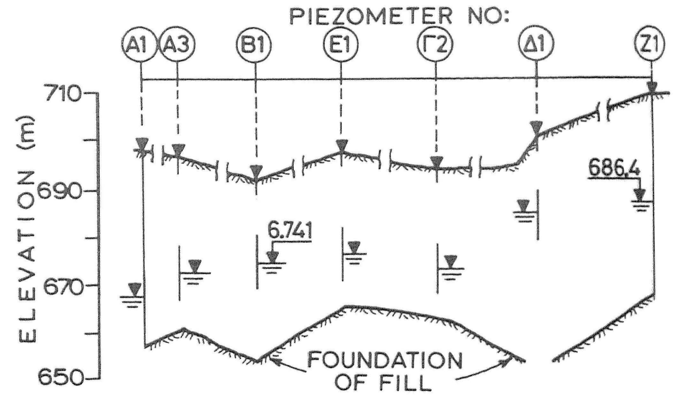

Fig. 3. Water Table in the Fill: September 1979

EXISTING DEPOSITS - SENSITIVE MARLS - LIGNITES

Two principal deposits : 90 and 80 meters high, at the "northern exterior" and the "main" lignite fields - completed in 1974 and 1975 respectively, have been extracted from neighboring terrains of same geology. The slope-berm geometry of the "northern exterior" deposit was taken into consideration as no major slides or earth movements (except early fringe cracks, described subsequently), were observed during the past nine years. Sensitive marls, flowing as slurry during transport and deposition, constitute a constant source of instability to the main deposit. They were observed only haphazardly in this project and are described extensively by Kotzias-Stamatopoulos, 1983. Combustion of lignites, dissociation of grains and other non mechanical processes (Bishop, 1973) have not been experienced in the two principal or the secondary deposits throughout the area.

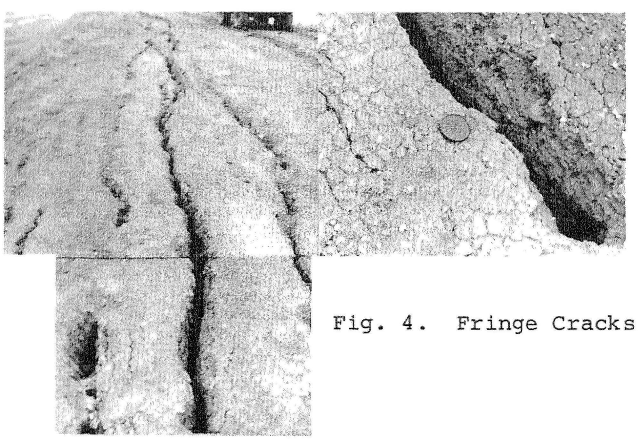

Fig. 4. Fringe Cracks

BEHAVIOR OF THE DEPOSIT PRIOR TO INVESTIGATIONS

Fringe movements - very common in this and the other mounds, as horseshoe cracks and even exterior slides (Fig. 4), do not necessarily forecast an avalance in spite of local inconveniences. They are, nevertheless, considered in the stability analysis (Fig. 9b).

A deep slide that occurred in 1974, when the tip was about 32 m heigh was not systematically investigated as to its causes. It was of a plain gravity type, slowly progressing, forecastable and conveniently readjusted with earth moving machinery.

WATER TABLE - TYPES OF SPOILS

Although ground water is low in the area (Kotzias, 1982), a perched water table within the fill was detected in all seven piezometers shown in Fig. 3.

TABLE I. Spoils: Description - Identification

Statistical Parameter	Liquid Limit	Plasticity Index	Gravel* (%)	Sand* (%)	Fines* (%)	<0.002 mm (%)	Water Content	Dry Unit Wt T/m^3	Degree of Saturation
Mean Value	54	22	7.5	20.5	72	27	40	1.2	9.6
St.Deviation	12	7	10	8.5	13	7	17	0.26	3.6
Number of Results	78	78	78	78	78	43	144	43	43

* Gravel: > No.4 Sand No.4-No.200 Fines < No.200

In Fig. 5 and in Table I the numerous descriptive data are compiled statistically (Kotzias-Stamatopoulos, 1983). A strong correlation of specific gravity and dry unit weight is worth noticing (Fig. 5c). Characteristic features of this extremely inhomogeneous spoils massive are: 1) A very high to complete saturation below and above water table and 2) A constant variation with depth of all properties shown in Table I. Soil variability as related to geologic origin, is further discussed by Kotzias, 1982.

STRENGTH

Standard Penetration: Two trends in SPT strength are explicit from 479 test results: THE DOMINANT, i.e., a linear increase with depth (Fig. 6), and THE SPORADIC, i.e., a constant strength with depth. Former was observed in 10 out of 12 borings (Table II).

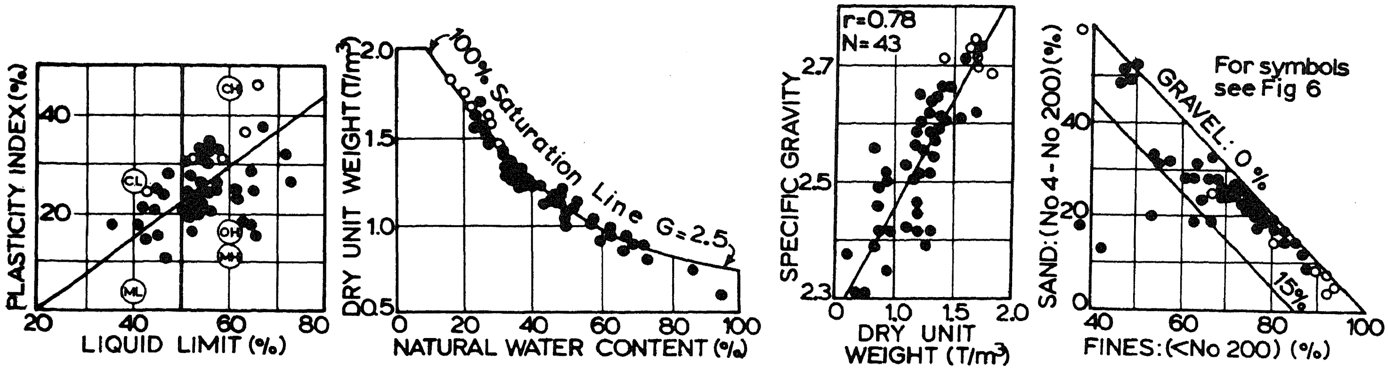

Fig. 5. Spoils: Description - Identification

TABLE II. Standard Penetration Strength Vs Depth

BORE-HOLE NO	A1*	A2*	A3*	Γ1*	Γ2*	Γ3*	Δ1*	Δ3*	Z1*	E1*	B1**	Δ2**
r	0.62	0.41	0.86	0.75	0.75	0.82	0.68	0.53	0.72	0.89	0.04	0.11
n	39	35	28	26	26	32	35	30	21	26	32	20

r = Coefficient of Correlation n = Number of Pairs

* = Strong Correlation ** = Weak to No Correlation

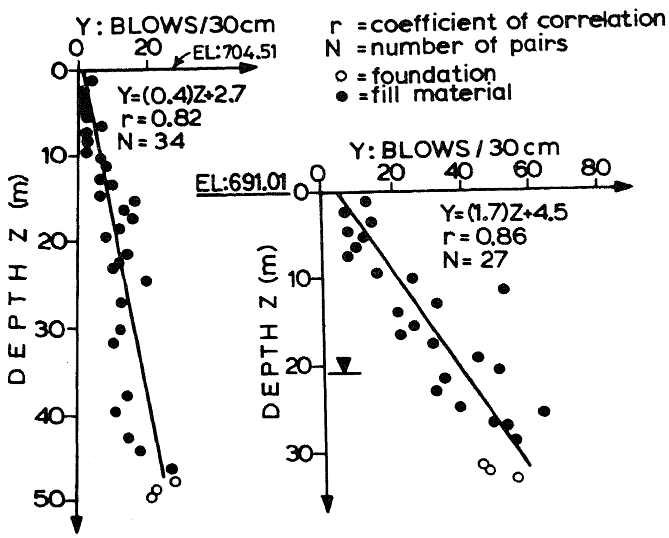

Fig. 6. Standard Penetration Strength Vs Depth: Characteristic Examples

Unconsolidated Undrained: 43 triaxial (UU) strength results, covering all the ranges of overburden, density and plasticity (Fig. 5), are linearly correlated with depth in Fig. 7 by the linear equation:

$C_u = (0.0152)Z + 0.28$ C_u=Shear Strength at Overburden Stress (kg/cm²)
Z =Depth (m) (I)

No worthwhile information is gained by multiple correlation (SPT, γ_d, L.L) nor by exponential or polynomial fits. Equation (I) is used in the stability analysis after being checked by back analysis

Constant strength is assumed to be

$$C_u = 0.38 \text{ (kg/cm}^2\text{)} \qquad \text{(II)}$$

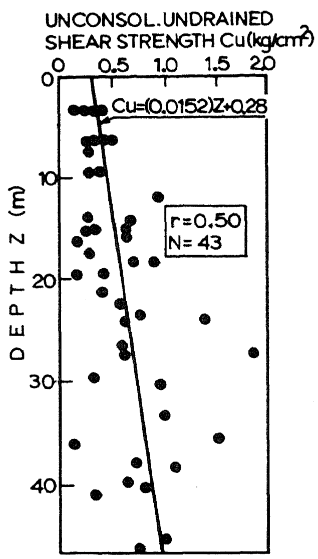

Fig. 7. UU Strength Vs Depth

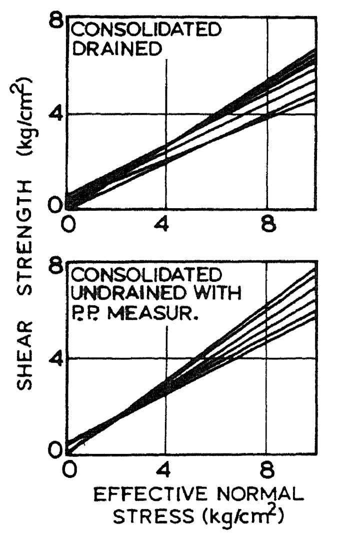

Fig. 8. Drained Strength: Summary

from average values of tests at constant strength with depth.

Consolidated Drained and Consolidated Undrained with Pore Pressure Measurements: Drained strengths (CD) and (CUPP) do not present any variability with depth or within samples as seen in Fig. 8.

Following simplified approximation to drained conditions is used:

$$c = 0 \quad \varphi = 32° \qquad \text{(III)}$$

Stress-Strain: The 140 single triaxial test results, viz. 98 (UU), 24 (CD) and 18 (CUPP), cover the wide spectrum of properties and depths. In all individual stress-strain curves the difference between peak and residual shear strength is not discernable. The "Brittleness Index" (Bishop, 1973) can be considered as minimal for all samples tested, possibly pointing at a "non brittle" potential sliding.

STABILITY

Method-Factor of Safety: The SARCMP program, originally developed by Dr S.K. Sarma, 1973, was chosen. It considers all forces on each slice and is applicable to any rupture configuration. The Factor of Safety is defined in terms of shear as:

$$F.S. = \frac{\tau(\text{ultimate})}{\tau(\text{mobilized})} \qquad (IV)$$

Back Analysis: Not shown here; it was carried out for the typical, the steepest and the modified slice section. Results indicated that assumptions of (UU) strength were slightly on the conservative side. Equation (I) was finally chosen with origin at each berm elevation, as (II) holds only at a few locations without contributing additional knowledge.

Assumptions: Lack of stratification in the massive delimits a cylindrical or nearly cylindrical critical toe rupture; base failure being unlikely due to favorable ground. (UU) strength determines immediate stability irrespective of water table. Any incipient slide will start from a 5 m surface crack with zero strength (Fig. 9b). Effects of marl liquefaction, combustion of lignites or dissociation of grains are considered minor. CD stability is seen in Fig. 9c.

Geometry - Limitations: The average repose slope of all dumped materials was measured and found consistently: 2(H)/1(V). Any step is limited by equipment and operations to a height of about 12 m or 24 m. Alteration of the given initial geometry or drainage of perched water (Fig. 3) entailed prohibitive costs. Only two degrees of freedom are given in stability analysis: Width of any future berms and a binary choice of step heights as shown in Fig. 9a.

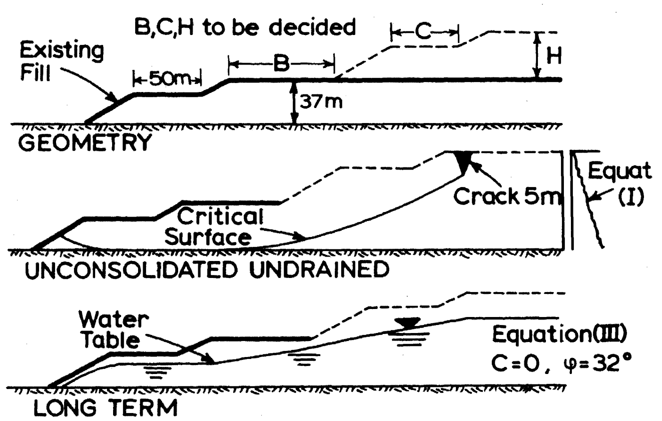

Fig. 9. Assumptions: Geometry - Strength

Limit Equilibrium: Fig. 10 sums up all results based on (UU) strength analysis for various combinations of future step heights and berm widths. Case (2) was selected with two berms 100 m and 50 m wide and a F.S.=1.06. Resulting figure is conveniently implemented encompassing the programmed amount of spoils. A slightly higher factor of safety would result in undesirable reductions of volumes. Long term stability analysis considering perched water table and equation (III) gives a F.S.=2.5 which is high. For the transition period of pore pressure dissipation, monitoring of earth movement was recommended.

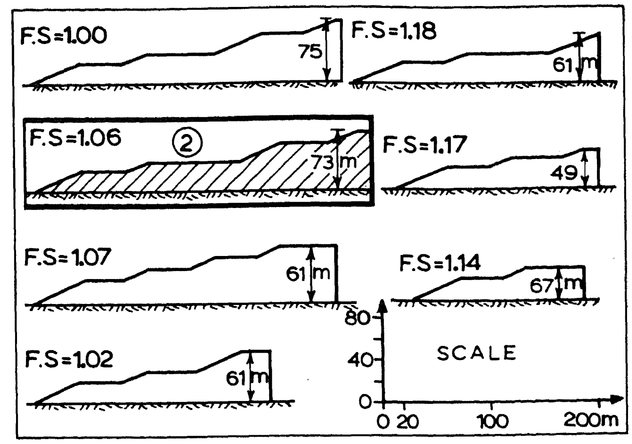

Fig. 10. Results of Limit Equilibrium Analysis: (UU) Strength

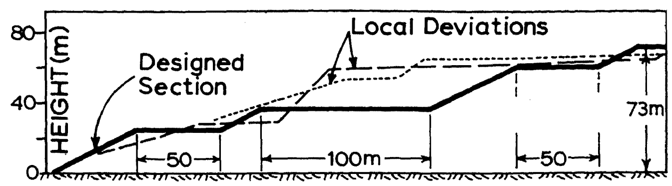

Fig. 11. As Built Fill with Local Deviations

CONCLUDING REMARKS

The main bulk of the fill was gradually completed within three years after the geotechnical studies. No stability problems have been experienced either during construction, after completion, or even locally in slopes inevitably steeper than recommended (Fig. 11).

ACKNOWLEDGEMENTS

Present work was assigned by the Public Power Corporation of Greece, Directorate of Mines. All stability computations were performed by Dr P. Croney, M.I.C.E.

REFERENCES

Bishop, A.W. (1973), "The Stability of Tips and Spoil Heaps", The Quarterly Journal of Engrg Geology, Vol. 6, Nos. 3 and 4.

Kotzias, P. (1982), "Soil Formations in Greece", General Report:Proc.Sympos. on Greek Soils. Hellenic Society of S.M.F.E., pp. 103-109.

Kotzias, P.C. and A.C. Stamatopoulos (1983),"Sensitivity of a Very Hard Pliocene Marl", Jrnl of Geotech. Engrg Div., A.S.C.E. Forthcoming December.

Kotzias, P.C. and A.C. Stamatopoulos (1983), "Statistical Graphics for Multiple Geotechnical Data", Proc. 4th I.C.A.S.P., Florence.

McKecknie-Thomson, G. and S. Rodin (1972),"Colliery Spoil Tips-After Aberfan", The Institution of Civil Engineers, London.

Sarma, S.K. (1973), "Stability Analysis of Embankments and Slopes", Geotechnique 23, No. 3, pp. 423-433.