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The Use of Limited Field Observation in Remedial Design

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SYNOPSIS As a consequence of a slope failure, an investigation was carried out to determine the present and future stability of a gold tailings dam in the Orange Free State, South Africa. Recomendations were also required concerning the type of remedial measure(s) necessary in order to permit continued deposition of the tailings waste product on the dam. The field and laboratory investigation involved sampling of the tailings and foundation soils and installation of piezometers at various locations around the dam. The paper describes how, using data obtained from a limited monitoring period, evaluation of in-situ parameters enabled prediction of future phreatic surface variations under differing operating and climatic conditions.

INTRODUCTION

In June 1981 a circular-type slope failure occurred in the outer daywall of a gold tailings dam in the Orange Free State, South Africa. The failure itself was small in size with only minor financial expenditure necessary for reinstatement. Its importance, however, lay in the implications it held for the present and future stability of the dam should continued deposition of the tailings product occur.

As in the case of the dam under consideration many of the older tailing dams were constructed without the installation of drainage. Consequently the position of the phreatic surface becomes a major factor in controlling stability as the dam height increases.

The tailings dam covers an area of approximately 160 hectares (1,6 square kilometers) having a total peripheral length of 6 kilometers. At the time of failure the dam height varied between 8,5 m and 16,0 m, depending on topography. Consequently, an accurate model of the aspects related to the stability condition was required in order to provide realistic recommendations for remedial measures, in terms of both engineering and cost effective considerations.

An assessment of the present stability of the tailings dam was carried out using in-situ measurements of the phreatic surface from standpipe piezometers and results from a laboratory testing programme. The results of piezometer measurements taken over a four month period were then used to calibrate a finite element program capable of determining the position of the phreatic surface position within a slope subject to specified boundary conditions. Once the model had been calibrated, the phreatic surface under conditions existing at the time of failure was obtained. This then allowed for a backanalysis of the failure and verification of strength parameters obtained from laboratory tests. It was also possible to investigate the effect of climatic variations on the phreatic surface position. This was neccesary since the period of monitoring was carried out during the dry season and no data existed for conditions where a higher phreatic surface may have occurred. The results of these analyses were then used to analyse the future stability of the tailings dam and for selection of the optimum type of remedial measure.

GEOLOGY OF THE AREA

The site of the tailings dam is on recent soils underlain by the Middle Ecca Shales of the Ecca Series, Karoo System. The Middle Ecca Shales were only encountered in a residual state during the field investigation.

The surface deposits around the site consist of moist to wet, loose to medium dense, brown, clayey silty fine sand (SM) of aeolian origin that are present to a maximum depth of approximately 3,0m below ground level. The sands exhibit a collapsible grain structure of low potential.

The aeolian sands are underlain by a residual soil of the Middle Ecca Shales which generally consists of a wet, medium dense, mottled, silty fine sand (SM) or sandy silt (ML), with varying proportions of clay-size material. The residual soil was proved to a maximum depth of 3,4m below ground level.

Ground water was encountered at depths between 1,1 m and 2,6 m below ground level depending on location around the dam; water levels being higher on the east side of the dam where the failure occured.

OCCURRENCE OF FAILURE

Figure 1a. shows a cross-section through the failure which was located between piezometer sections C and D (Fig. 1b). The shape of the failu

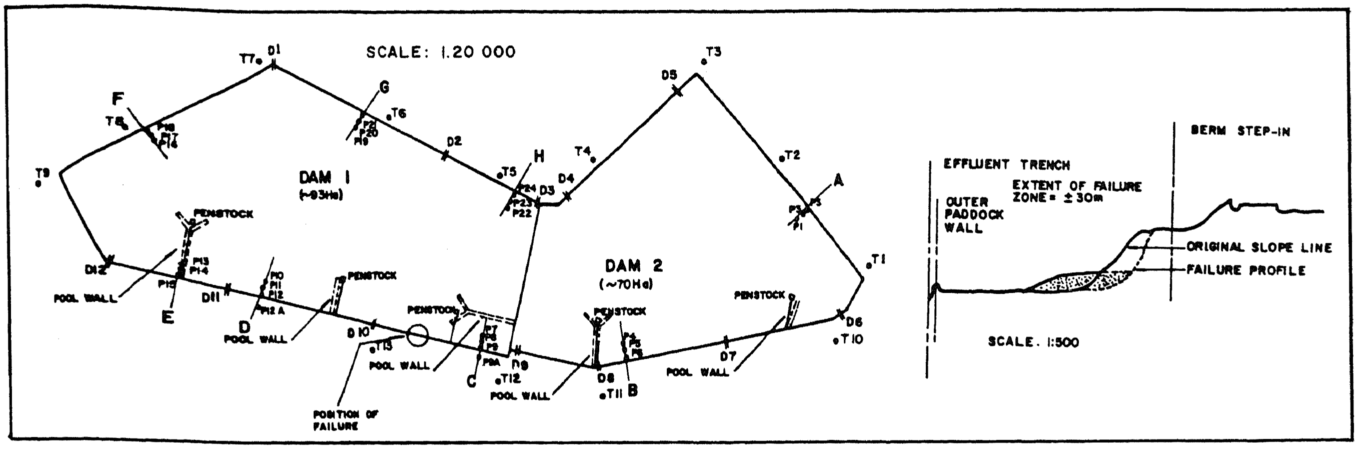

Fig. 1 (a) General Layout of Tailing Dam showing Piezometer Sections and Position of Failure; (b) Cross-Section through Failure Area. (D1-D12 Delivery points; P1-P24. Piezometer Positions; A-H Cross-sections used for stability analyses; T1-T13 Positions of Excavated Trial Pits).

re suggests a circular surface occurring mainly within the tailings material with only slight influence of the underlying aeolian sands. The extent of the failure parallel to the daywall was approximately 30 m.

At the time of failure, piezometer readings had not been taken and the slope survey was the only information available. However, subsequent discussions with Mine personnel working on the dam provided important data on conditions prior to failure. These conditions can be briefly summarised as:

- daywall slopes, constructed at high angles which had been steepened by erosion due to rainfall run-off. The average slope angles at sections C and D were 52° and 55° respectively.
- continual sliming in the area of the failure for nine days prior to failure.
- high groundwater conditions in the outer paddocks at the toe of the slope; water table being at ground level.
- evidence of seepage from the lower part of the slope.
- several days of heavy rainfall on the dam causing partial saturation and subsequent loss of apparent cohesion in the tailings material above the steady-state phreatic surface position.
- storage of large volumes of rainwater and slimes delivery water on the surface of the dam due to inadequate penstock decant capacity and poor operational procedures. The stored water was about 0.3m deep at the top of upper slope above the berm step-in.

The failure occurred during the night and was discovered the following day by Mine Personnel. The failure material had spread very little indicating the absence of liquefaction during shear as has been the case in many previous tailings dam failures. This can be accounted for since the failure was restricted to the outer daywall where the tailings material is of a coarser nature and more dense, and thus less likely to undergo liquefaction during shear. However, the occurence of a more catastrophic flow-type failure, after initial local failure followed by slumping of the steep back scarp, was a factor to be considered when evaluating the stability of the dam under present and future slope configurations.

FIELD AND LABORATORY INVESTIGATION

A site investigation programme was carried out which comprised:

- excavation of trial pits around the dam and undisturbed sampling of the foundation soils (Fig. 1a).
- disturbed an undisturbed sampling of tailings material at various locations around the dam.
- the installation of piezometers to supplement existing piezometers bringing the total number installed to 26, with a minimum of three piezometers at each section (Fig.1a).

General soil characteristics are shown in Table I.

TABLE I. Averaged Results of Foundation Indicator Tests.

Soil Property	Aeolian Sand*	Residual Soil**
Liquid Limit (LL)	18	26
Plastic Limit (PL)	11	13
Plasticity Index (PI)	7	13
Water content	6,0	17,6
Linear Shrinkage	2,8	5,5
Clay Fraction ($<$ 0.002mm)	14	21
Specific Gravity	2,68	2,65

* Results averaged from 3 tests
** Results averaged from 2 tests

The types of laboratory tests carried out on retrieved samples were selected in order to increase existing data* rather than provide a complete set of new data. For this reason, only limited shear strength testing was performed. This was solely to demonstrate the conformity, or otherwise, of soil strength parameters with existing data. Strength parameters were obtained from saturated drained shear box tests, the results of which are summarised in Table II.

TABLE II. Strength Parameters for Soils Encountered.

Soil Type	Friction Angle (degrees)		Cohesion (k/Pa)	
	Mean	S.Dev	Mean	S.Dev
Tailings[1]	33,04	3,14	4,92	1,44
Aeolian Sand[2]	32,84	4,86	9,8	3,0
Residual Soil[3]	32,60	4,43	8,0	2,0

1 Results from 8 tests
2 Results from 10 tests
3 Results from 5 tests

The emphasis of the laboratory testing was to provide information on soil type permeabilities for use in the finite element program described below. The tests were carried out in the triaxial apparatus and also in an Anteus Consolidometer (Lowe et al, 1964). Permeability values were obtained at various stress levels, selected to represent in-situ values corresponding to existing and possible future heights of the dam. The results of the tests carried out on the foundation soils and tailings material are summarised in Table III.

*Two previous investigations for the tailings dam had been carried out.

TABLE III. Results of Permeability and Consolidation Tests.

Soil Type	Effective Stress of sample (KPa)	Coeff.of Consol. Cv (m^2/yr)	Coeff.of Permeab. K (m/sec.)
Tailings* (coarse grading)	50	112	9,77 x 10-8
	100	67	7,14 x 10-8
	150	82	7,40 x 10-8
	200	68	6,78 x 10-8
	300	55	6,72 x 10-8
	400	81	-
Aeolian Sand	150	47	3,77 x 10-7
	250	79	3,02 x 10-7
	450	79	2,15 x 10-7
Residual Soil	150	63	5,39 x 10-8
	250	62	5,31 x 10-8
	450	-	3,25 x 10-8

* Permeability values of fine tailings were approximately half the value of coarse gradings at each stress level.

PHREATIC SURFACE POSITION

Method of Analysis

The analyses were carried out using the computer program FPM500-AXISYMMETRIC AND PLANE FLOW IN POROUS MEDIA developed in 1968 by R. L. Taylor at the University of California. The version

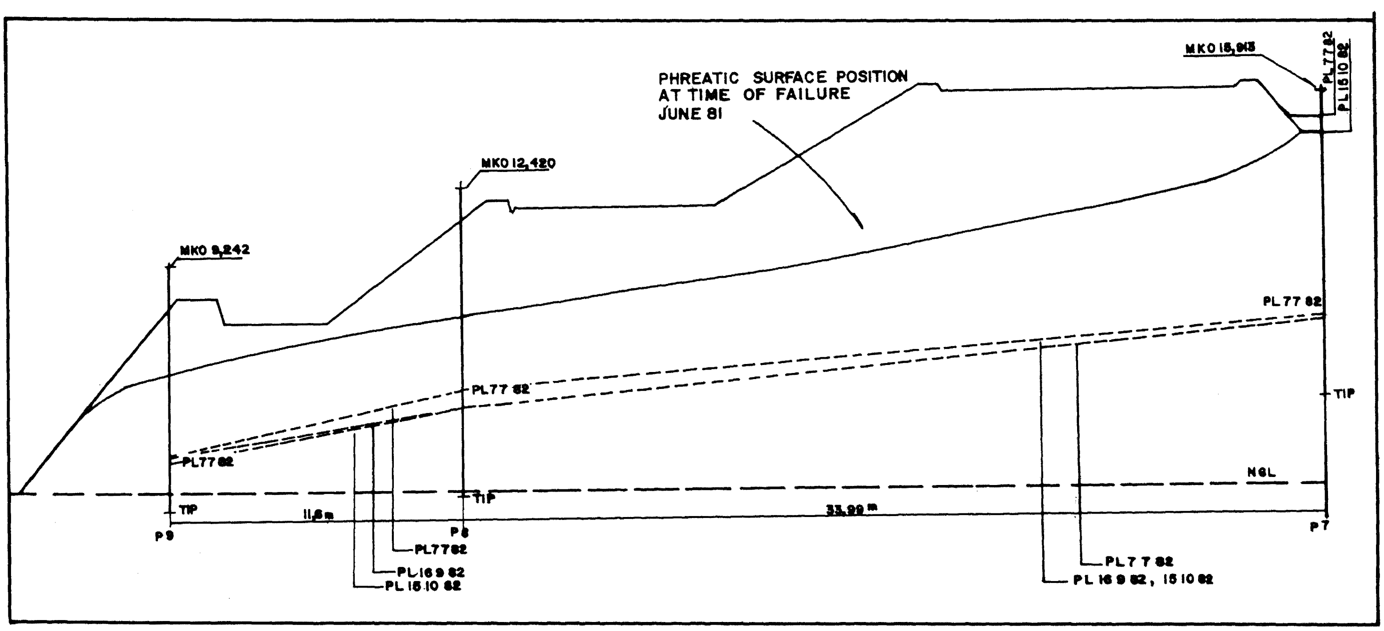

Fig. 2 Phreatic Surface Data For Cross-Section C

used was as modified by Kealy and Busch (1971). The program uses finite element methods to solve, in two dimensions, Laplace's and Richard's equations describing the location of the phreatic surface. It is able to take account of differences in vertical and horizontal permeability within a medium. The program locates the phreatic surface, solving for closure conditions, by iterative means. A mesh is generated over the section and described by nodes and elements. The nodes are allowed to move in specified directions as the phreatic surface position is adjusted in the iteration process.

TABLE IV Comparison of Permeability Values

MATERIAL TYPE	Vertical Permeability (m/s)		Horizontal Permeability (m/s)		Permeability Ratios	
	Laboratory Result	Backanalysis Result	Laboratory Result	Backanalysis Result	Laboratory Result	Back-analysis
Tailings	$5,29 \times 10^{-8}$	$4,2 \times 10^{-7}$	$9,11 \times 10^{-8}$	$1,05 \times 10^{-6}$	1,72	2,5
Aeolian Sand	$2,98 \times 10^{-7}$	$6,5 \times 10^{-6}$	$2,98 \times 10^{-7}$	$1,8 \times 10^{-6}$	1,0	0,3
Residual Soil	$6,64 \times 10^{-8}$	7×10^{-9}	9×10^{-8}	$1,0 \times 10^{-8}$	1,94	1,43

Calibration of Program against Piezometer Readings.

Due to considerable variation in pool geometry from month to month, only one section was analysed to provide calibration of the computer program. Section C was used for this purpose since:

- a complete record of phreatic surface position across all installed piezometers existed.
- it had a relatively high phreatic surface
- the slopes in this area had high angles thus requiring accurate definition of phreatic surface conditions for stability analyses.
- the section is close to the failure area thus permitting a backanalysis of the failure to be carried out after prediction of conditions existing at the time of failure. The results of the backanalysis could then be used to determine the accuracy of the prediction method.

The results of piezometers for Section C during the dry season are shown in Fig. 2.

Table IV. compares average permeability values from laboratory testing with those obtained from the computer model to provide a match with piezometer measurements. As can be seen, the ratio between the two sets of results varies between 6 and 21. The laboratory values can be considered correct for the intact material since tests were carried out on cut block samples which had undergone very little disturbance or stress relief. The generally higher model-derived permeability values can be ascribed to macrostructural differences not considered in the small-scale laboratory tests i.e. permeability variations in tailings due to horizontal layering and lateral segregation during deposition.

BACKANALYSIS OF FAILURE

The calibrated program was then used to predict the phreatic surface position at the time of failure with simulation of applicable boundary conditions described earlier. The predicted surface is shown on Fig. 2 with results from post-failure monitoring for comparison. (The difference

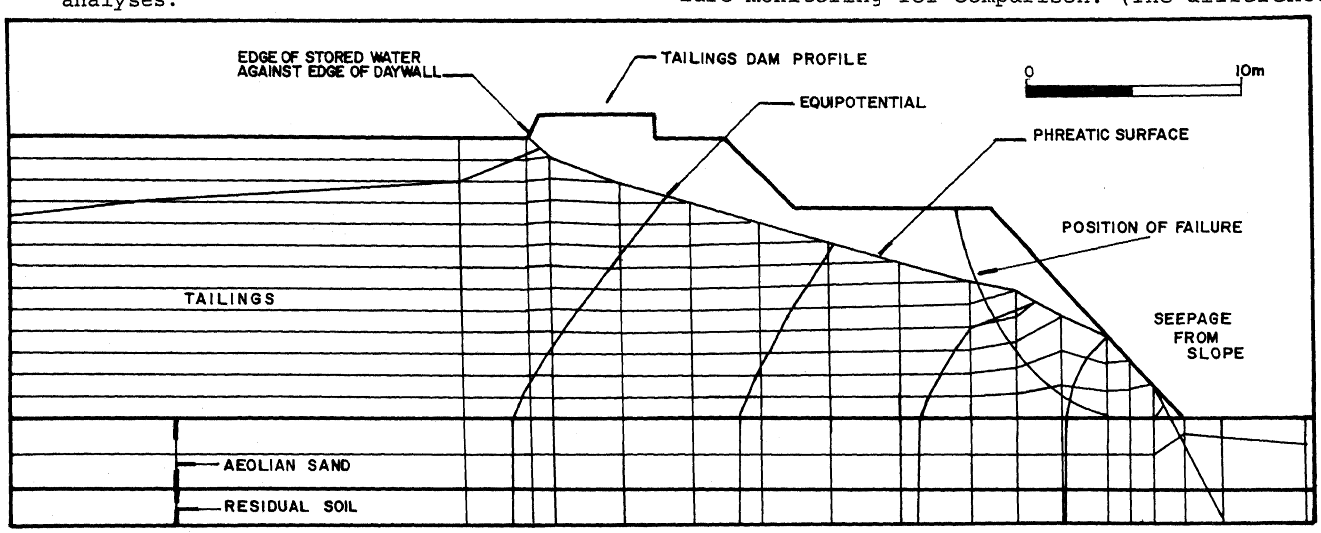

Fig. 3 Predicted Surface Position for Backanalysis of Failure

between the pre-failure and post-failure phreatic surface positions is due to improved operating conditions instigated as a result of the failure.)

The position of the failure surface was obtained from the survey carried out by the Mine just after failure had occurred.

The backcalculated factor of safety, using the slope stability program STABL (Siegel 1975) was found the be 0,97 with a probability of failure of 68%. The analysed cross section showing position of the phreatic surface and failure surface are shown in Fig. 3.
The following points were verified from the backanalysis:

- that the phreatic surface prediction was realistic albeit somewhat slightly conservative.
- that the soil parameters obtained from laboratory testing were representative of actual in-situ strengths.
- that the method of analysis could be realied upon to predict meaningful values of factor of safety and probability of failure for a slope.

PREDICTION OF FUTURE POSITIONS OF PHREATIC SURFACE

The stability of a slope is directy related to amongst other variables, the angle of the slope and the position of the phreatic surface. Present day slopes of the dam were analysed for stability using piezometer data. As shown in Fig. 2 the phreatic surface is very low which may be attributed to the underdrainage effect of the aeolian sands and the post-failure practise of decanting water from the top of the dam. The factors of safety for present slope configurations are shown in Table V. In these analyses no acount was made for apparent cohesion affects resulting from partial saturation which can significantly increase the effective safety factor (McPhail, 1982).

In order to recommend effective and realistic remedial measures, it was necessary that prediction of the future phreatic surface positions be made as the dam height increased under the following conditions:

- 'steady-state' conditions which exist during average weather conditions and normal periods of sliming. The 'steady-state', phreatic surface for Section C after a 20m rise above present level is shown in Fig. 4.

TABLE V. Results of Stability Analyses for Piezometer Sections.

Cross-section	Present Factor of Safety	Slope Angle (degrees)
A	1,64	39°
B	2,04	30°
C	1,31	52°
D	1,11	55°
E	1,18	48°
F	1,29	46°
G	1,94	39°
H	1,29	49°

- 'transient' conditions which exist for short periods during adverse periods of deposition and rainfall. The 'transient' surface for section C after a 20m rise above present level is shown in Fig. 5.

The difference between the two conditions was modelled by allowing ancroachment of the pool to the daywall slope. The distances used, i.e. 110 m from crest and 10m from crest for 'steady-state' and 'transient' conditions respectively, are considered realistic since the gold tailings material beaches at a very low gradient ($\pm 2\%$) and small rises in pool level can cause significant lateral movement of pool boundaries.

CONCLUSIONS

The paper has described how a limited period of monitoring coupled with finite element modelling allowed predictions of phreatic surface conditions under circumstances controllable by variation of input parameters, namely:

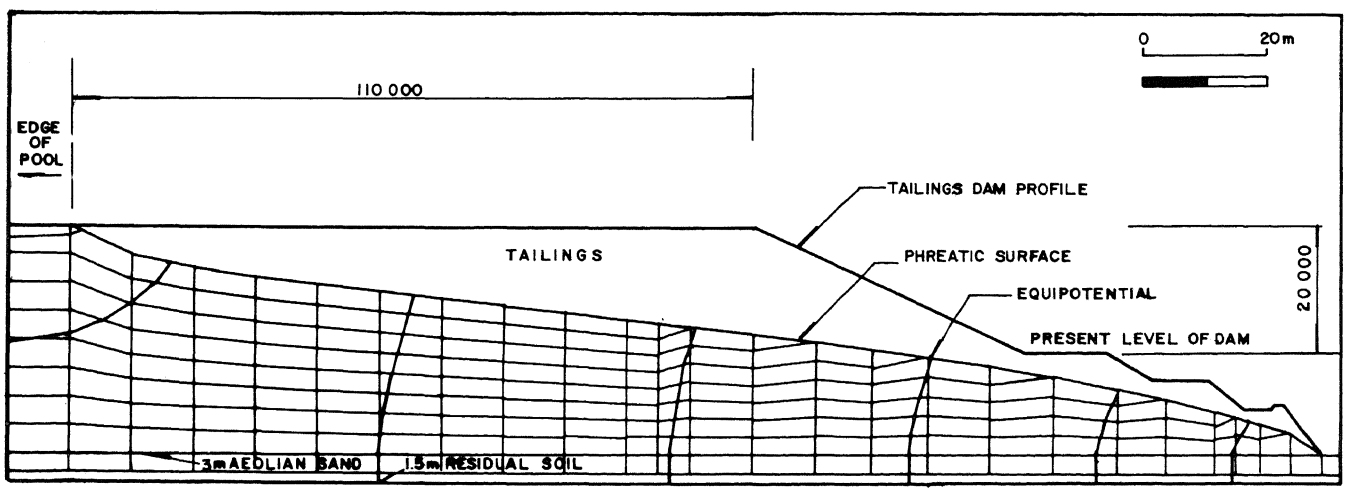

Fig. 4 Predicted Steady-State Phreatic Surface Position after 20m Rise

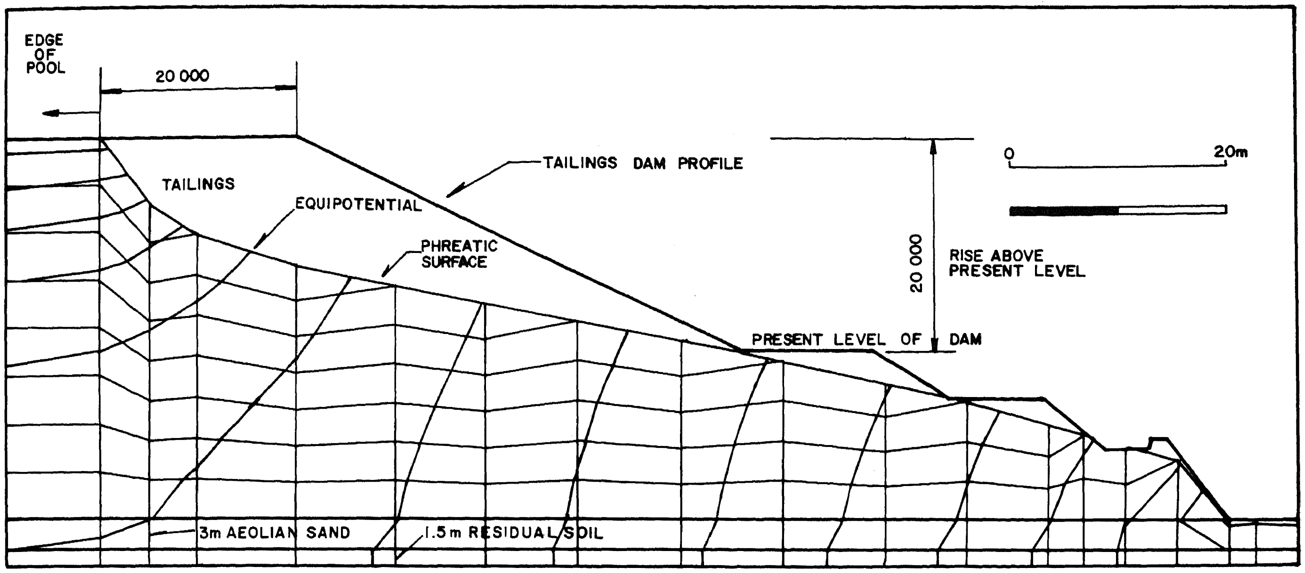

Fig. 5 Predicted Transient Phreatic Surface Position after 20m Rise

- the extent of the tailings pool
- drainage boundary positions
- varied deposition rates of tailings
- infiltration of rain falling on slimes dam surface.

By backanalysis of the failure, made possible by computer prediction of phreatic surface condition, the method was shown to be realistic although slightly conservative. Due to poor definition of the failure in three-dimensions only a two dimensional analysis was performed. However, transforming the failure to a planar from and using results obtained by Hovland (1977) the ratio between the 3D and 2D factors of safety lies in the range 1,03 to 1,06 for this particular failure. Although this figure can only be used as an indication, it does remphasise the validity of the model used.

Using the above method it was possible to predict future stability of the dam and assess the affect of various types of remedial measure (Sully & McPhail, 1983).

The conclusions drawn from the investigation were that limited in-situ measurement can provide a sufficient base for design provided that:

- the data is evaluated realistically with due consideration of possible variations.
- the necessary computational expertise exists to translate the limited data into a representative condition.
- continued monitoring is carried out to verify the assumptions used in the analysis, especially where predictions of future conditions are made.

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REFERENCES

Hovland, J., (1977); Three-Dimensional Slope Stability Analysis Method, Jour. Geot. Engrg. Div., ASCE, Vol.103,GT9, Sept.pp 971-986.

Kealey C.D., & Busch,R.A.(1971); Determining Seepage Characteristics of Mill-Tailings Dams by Finite Element Method, US Bureau of Mines, Report No. 7477.

Lowe, J., et al,(1964);Consolidation Testing with Back Pressure,Jour.Soil Mech. & Found Div., Proc., ASCE, Vol 90, No. SMS, September, pp 73-90.

McPhail, G.I.,(1982); The Influence of Capillary Forces in Slope Stability Analyses, Master of Engineering Thesis Submitted to University of Witswatersrand, Johannesburg, South Africa.

Siegel, R.A.,(1975); Computer Analysis of General Slope Stability Problems, Joint Highway Research Reports 75-8 & 75-9. Purdue University, West Lafayette, Indiana, June.

Sully, J.P., & McPhail, G.I.,(1983); Internal Report No. MI2738, Steffen Robertson & Kristen (Mining) Inc. Johannesburg, South Africa, January.