# Stabilization of a Tailings Dam by De-Watering

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SYNOPSIS The failure of the outer slope of one of the paddocks of a tailings dam, resulted in a significant decrease in the area available for tailings deposition. Overloading of the remaining five paddocks resulted. To maintain the stability of the existing dam whilst construction of new disposal facilities was in progress, three techniques were investigated; horizontal, push-in drains at the dam toe, a rock buttress around the operational paddocks, and vertical de-watering wells installed on the first terrace, or 'step-back' of the dam. De-watering wells proved most effective, and 165 wells were installed. A monitoring programme included regular determination of the water level in each well, and the installation of piezometers at selected locations around the dam. These observations were used to optimise the use of the five remaining paddocks, thus ensuring stability of the dam during the transfer of deposition operations to two new tailings dams.

## INTRODUCTION

### Background

The subject of this paper is a tailings dam that was operated in the Transvaal province of South Africa. The dam was used for the disposal of platinum tailings, and made use of the ring-dyke method of tailings deposition. In this technique a low, compacted earth wall is built around the perimeter of the area to be used for storing the tailings. Tailings thickened to 55% solids (by mass) was deposited onto the dam via a spigot pipeline that was also located around the dam perimeter. Spigot outlets were spaced at 2.4m centres, and during tailings deposition sufficient outlets (usually 10-16) were opened at any time in order to allow the full tailings tonnage to be deposited.

The platinum tailings had a well-graded particle size distribution, which resulted in segregation of particles during hydraulic deposition. As a consequence, coarser particles were deposited adjacent to the dam perimeter, whilst the finer, clay-sized particles were transported to the centre of the dam. This generally leads to an outer shell that is relatively free-draining, and does not retain water. The thickened tailings has a specific gravity of 1.7, and therefore a large volume of excess water must be disposed of after each deposition cycle. It also means that a rate of rise of the dam of 2m per year has been adopted as a maximum upper limit, in order to ensure sufficient consolidation of the tailings and the underlying foundation soil.

The soil below the dam was derived from the weathering of the norite bedrock. The resulting soil was a high plasticity clay with low shear strength properties (see later).

### History of problems.

In early 1977, when the dam was approximately 25m high, a shallow surface slip occurred on the north face of the dam, immediately after a trench had been excavated at the toe of the dam by the mine operators in order to drain seepage water that was accumulating at the toe of the dam. Considerable rain had fallen in the days just prior to the failure, which had exacerbated an already precarious situation. The trench excavation at the toe of the dam was stopped immediately, and no further movement of the slope was observed.

Approximately a year later, a particularly wet zone was noticed at about midway along the western side of the dam, at up to five metres above the toe of the slope. This situation was attributed to the pool of water that was stored on top of the dam, with the edge of the pool being only some 50m from the dam crest.

In the light of the above problems, it was recommended that the outer spigot delivery line be moved inwards by 30m around the entire dam, effectively forming a terrace, or stepback, that would push the water pool further away from the outer slope of the dam. In addition, three lines of two standpipe piezometers were installed at problematic locations around the dam.

### Main failure.

Whilst stability analyses were being undertaken using the water levels measured in these piezometers, a major slip of the North face of the dam occurred in September 1978. At the location of the slip, the dam was approximately 32m high, and as can be seen from Figure 1, the slip affected the entire outer height of the dam. It was about 110m long at crest level, and presented as a classic infinite-slope geometry. Fortuitously, the line of two piezometers that was located in the area of the slip had been read only two days before the slope failure occurred. Furthermore, the slope had been surveyed a few days previously, so the geometry of the slope and the location of the phreatic surface at the time of failure were extremely well defined.

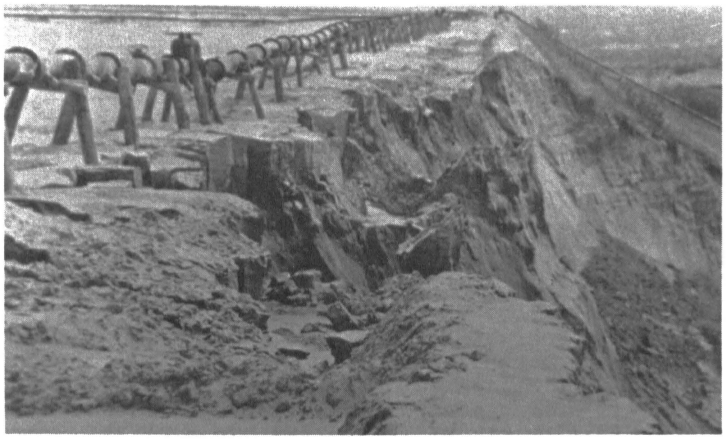


Figure 1. Slope failure on north-eastern paddock of dam.

Immediate de-commissioning of the North-East paddock was implemented, thus decreasing the area available for tailings deposition by about 13%. As it happened, the above problem coincided with an increase in the amount of tailings produced, as a result of increased tonnages being mined and processed. Recognising that these factors would combine to result in the dam rapidly being loaded beyond its design capacity, and being mindful of the catastrophic failure of Bafokeng tailings dam in 1974 (Jennings, 1979), plans were immediately initiated for the design, construction and commissioning of alternative facilities for tailings deposition. A fuller understanding of the existing stability of the remainder of the dam, as well as a measure of how this would change as a consequence of the increased rate of rise was clearly necessary.

## EVALUATION OF EXISTING AND FUTURE STABILITY

### Installation of additional piezometers.

Recognising the sensitivity of the dam stability to an increase in pore water pressure along a potential failure surface, it was decided to install a further eight lines of standpipe piezometers around the dam perimeter, each consisting of three piezometers. The location of these sections is shown on Figure 2, which depicts the dam after decommissioning of the North-Eastern paddock. Figure 3 shows a section along one of these piezometer lines, which illustrates the location of the water pool atop the dam, and the elevated location of the phreatic surface.

### Backanalysis of main failure.

Observation of the failed slope indicated that the plane of failure was highly unlikely to have been circular, and was most likely a wedge-type failure, with a steep back slope and the horizontal portion confined to the underlying weak clay layer, (see Figure 3). The results of a number of undrained (with pore pressure

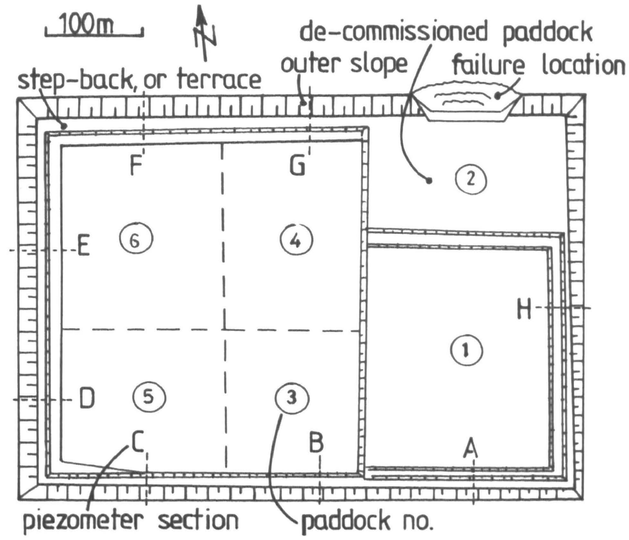



Figure 2. Plan view of tailings dam.

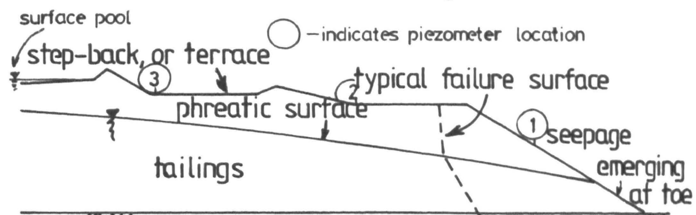



Figure 3. Typical section through tailings dam

measurement) and drained shear tests on undisturbed samples of the black clay are shown in Figure 4a. A total of 28 tests were carried out. A mean cohesion value of 25 kPa was obtained, with a mean peak friction angle of 15° and a mean residual friction angle of 6°. Triaxial tests on eighteen samples of the tailings material indicated that the cohesion was negligible, with a mean friction angle of 35°, (see Figure 4b).

The Morgenstern-Price (1965) approach, which utilised the simplified Janbu method of slices, was used for the backanalysis. A possible failure surface was chosen, and the factor of safety for the measured slope geometry and phreatic surface profile calculated, using the most likely value of the soil shear strength parameters. The geometry of the failure surface was varied until a minimum factor of safety was obtained, and since the friction angle of the clay had been defined with the least degree of accuracy, it was varied until a factor of safety of unity was obtained for the critical failure surface. This occurred when the friction angle of the clay was set to 10°. Although this is an extremely low friction angle, the clay had a plasticity content of 45%, and it is likely that sufficient movement had occurred to exceed the peak strength value during construction of the tailings dam.

Although the above analyses provided a useful indication of the conditions pertaining at the time the failure occurred, it was clear that there was too great a degree of uncertainty in the shear strength parameter definition for too much reliance to be placed on a purely deterministic analysis of the stability of the remaining, operational paddocks.

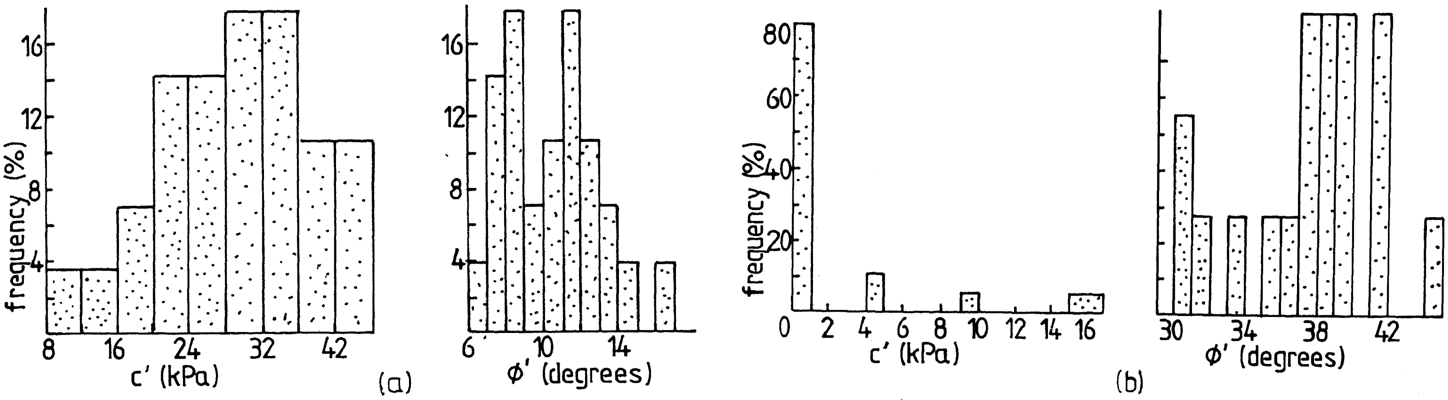



Figure 4. Triaxial test results for (a) foundation clay, and (b) tailings.

Probabilistic approach to evaluation of safety of dam.

The unrealistic assumption of assigning a point estimate to properties such as friction angle and cohesion can be appreciated by reference to Figure 4. An alternative approach is to accept the inevitable variability of the material properties, within certain limits, and to account for this in some way. A common approach is to make use of the probabilistic method of analysis, which has been discussed by Harr (1977), amongst others.

This approach was adopted in the evaluation of the existing dam by making use of the slope stability programme STABL, jointly developed by the Civil Engineering group at Purdue University, and the Indiana State Highway Commission, Siegel (1978). The analysis procedure made use of random generation techniques for:

(i) the selection of mutually exclusive material properties $c^1$ and $\phi^1$, from given distributions (Figure 4), and
(ii) the generation of potential failure surfaces.

Studies by Schultze (1972), and Lambe (1951) of frequency distributions and correlations with soil properties have shown that many material properties are normal-like variates. This was found to be true for the distributions shown in Figure 4, with the exception of the cohesion of the tailings, for which a log-normal distribution was found to be most appropriate (with a strong bias towards very small values of cohesion).

Using these distributions, the program randomly generated a different set of material strength parameters, from within the specified distributions, for each failure surface analysed. The failure surfaces were also randomly generated, although within given limits and subject to certain biases. A wedge failure similar to that observed in the field was obtained by confining the horizontal leg of the failure surface to within the weaker black clay layer. The active and passive legs of the failure surface were biased towards forty-five degrees and zero degrees respectively. A typical failure surface generated by the program is shown in Figure 3.

Each of the randomly generated failure surfaces was analysed by the simplified Janbu method of slices, and incorporated the phreatic surface obtained from the piezometers shown in Figure 2. A statistical distribution was fitted to the variation of calculated factors of safety. The area under this distribution curve, for a specified value of factor of safety, indicated the likelihood that the factor of safety of the slope being analysed was less than the specified value. This meant that if a factor of safety of unity was chosen, the area under the curve represented the probability of failure.

For each of the sections investigated, twenty different failure surfaces were analysed using 100 different sets of randomly generated strength parameters. The first section to be analysed using this approach was that where the failure of 1978 occurred. The probability of failure was calculated as 48%. The same procedure was used to analyse the eight sections along which piezometers had been installed. The results of these analyses are given in Table 1 below.

Table 1: Computed stability of existing dam.

| Section | Probability that factor of safety is less than (%) | |
|---|---|---|
| | 1,0 | 1,2 |
| A | 17,7 | 53,2 |
| B | 10,0 | 39,0 |
| C | 8,2 | 36,6 |
| D | 20,9 | 66,9 |
| E | 17,8 | 61,7 |
| F | 75,1 | 96,2 |
| G | 83,1 | 97,2 |
| H | 19,9 | 63,0 |

From these results it can be seen that the north face of the dam in particular was in a critical condition. It is apparent that the dam could be expected to have already failed, and the fact that it was still standing is probably attributable to the inherent conservatism of the Janbu method, as well as the possibility that sampling disturbances produced lower estimates of shear strength parameters for the clay than existed in the field. These points aside, it was clear that the dam was very unstable, and immediate steps to improve the stability were necessary. To avoid complete cessation of deposition on the dam, which would have implied major production losses, three alternative systems for improving the dam stability were considered. These were the installation of horizontal drains at the toe of the dam, construction of a rock buttress against the outer slope of the dam, and the installation of vertical dewatering wells around the dam perimeter on the first stepback. These are discussed in detail below.

Alternatives for stabilisation of dam.

i)Horizontal drains installed at the toe of the dam.
The principle of this measure was to insert a pre-formed drain into the toe of the dam, for a distance of at least 20m, resulting in the phreatic surface being drawn down at the toe of the dam and preventing water emerging on the outer slope. Figure 5 shows the anticipated effect of these drains, which were assumed to be effective over only half their length. The use of intruded horizontal drains had been used with success in the then-Rhodesia, and the proposed procedure was to drive the drain sleeves into the slope using a bulldozer blade whilst flushing with water at the advancing end of the drain.

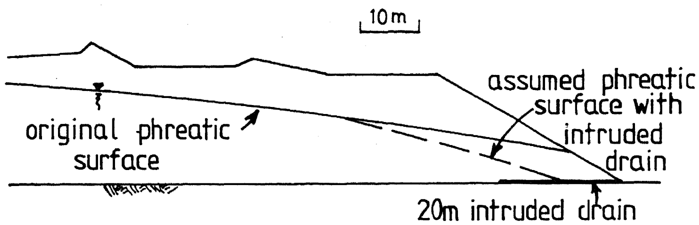



Figure 5. Section showing horizontal drain and expected phreatic surface.

The improvement in stability that could be obtained using this technique was evaluated by carrying out probabilistic slope stability analyses of four of the sections previously analysed. The results are shown in Table 2 below.

Table 2: Predicted stability improvement using horizontal drains.

| Section | Probability that factor of safety is less than 1,0 (%) | |
|---|---|---|
| | Without push-in drains | With push-in drains |
| A | 17,7 | 5,5 |
| D | 20,1 | 18,2 |
| E | 17,8 | 13,9 |
| F | 75,1 | 44,7 |

Estimates of the required spacing of these drains was based on the work of Kenney et al (1977), which gave a spacing of 20m for the highly stressed North wall, and 25-30m for the remainder of the dam.

ii)Construction of a rock buttress around the dam perimeter.
A rock buttress is a passive stabilisation technique, which improves slope stability by providing extra weight at the toe of the slope and increasing the length of potential failure surfaces. In addition to the buttress itself, measures such as a blanket drain on the slope face and drainage collection trenches would have been required. Problems with construction were expected where the toe area was extremely wet, and concern existed about exacerbating an already precarious situation.

In sizing the rock buttress for calculations o: potential improvement in stability, a heuristic o: making the buttress one third the height of th slope, and 15m wide at the top was used. Thre sections were analysed for improvements i stability to be gained by using a rock buttress once again using the probabilistic approach. Th results are summarised in Table 3.

Table 3: Predicted stabilit improvement using rock buttress.

| Secti on | Probability that factor of safety is less than 1,0 | |
|---|---|---|
| | Without Rock Buttress | With Rock Buttress |
| F | 75,1 | 25,0 |
| G | 83,1 | 35,4 |
| H | 19,9 | 4,0 |

Despite the potentially substantial improvement in stability, the cost of this option, together with operational difficulties made it necessary to consider a third option.

iii) Vertical de-watering wells installed on first step-back.
The proposed procedure was to install cased, slotted vertical wells on the first step-back, and then draw down the phreatic surface by pumping from these wells. The envisaged effect on the phreatic surface is shown in Figure 6 below. At the time of the investigation, the technique had not been used before in South Africa for dewatering a tailings dam but it was thought that the potential benefits of using the technique made it worth pursuing.

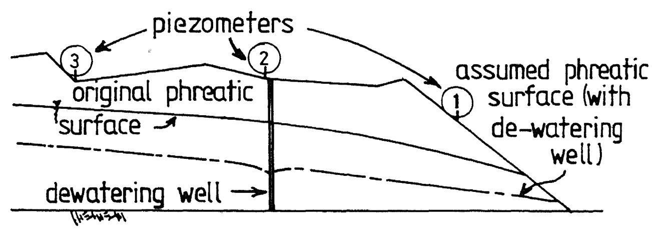



Figure 6. Section showing de-watering well, and expected phreatic surface.

The improvement in stability that was predicted for the envisaged de-watering is given below.

Table 4. Predicted stability improvement using vertical de-watering wells.

| Section | Probability that the factor of safety is less than 1.0(%) | |
|---|---|---|
| F | Without de-watering wells | With de-watering wells |
| | 75.1 | 41.9 |

The predicted improvement in stability is not as pronounced as for the rock buttress. However, as discussed in the following section, cost and other considerations outweighed these differences. Furthermore, field monitoring showed that the drawdown that was achieved was significantly better than that assumed in Figure 6.

EVALUATION OF ALTERNATIVE DE-WATERING TECHNIQUES.

Horizontal drains at the toe of the dam.

The technique was initially tried on the step-back terrace on the south side of the dam. Not only did this area provide easy access, but the terrace was dry, being well above the phreatic surface. Potential problems of drawing liquid tailings while installing the outer casing would therefore be avoided, enabling the technique to be refined before attempting drain installations on the very wet bottom slopes. Two drains were installed at this location, but it was only possible to achieve a penetration of 12m. Major difficulties were experienced withdrawing the outer casing whilst leaving the drain behind. The most success was achieved by withdrawing the casing completely before installing the drain. This was of course only possible if the tailings was sufficiently dry for the circular hole to remain open long enough for the drain to be installed, which largely negated the benefit of this system. Nevertheless, it was decided to proceed with additional trials on the lower slopes, with the intention of improving the installation technique at the same time. Two trials were carried out, both of which were even less successful than those mentioned above. Penetration depths of 14m and 7m were achieved, but the trials had to be aborted in view of the risk of liquid tailings rapidly discharging from the open end of the casing. This was controlled by plugging the casing with one of the finger drains, and building a small buttress around the exit point.

Although it may have been possible to improve the technique to an acceptable level, time was a critical consideration in the implementation of remedial measures, and furthermore it was found necessary to build a small working platform at each drain location, which would have increased the cost per drain by more than 300%. Further trials with this technique were accordingly suspended.

Construction of a rock buttress.

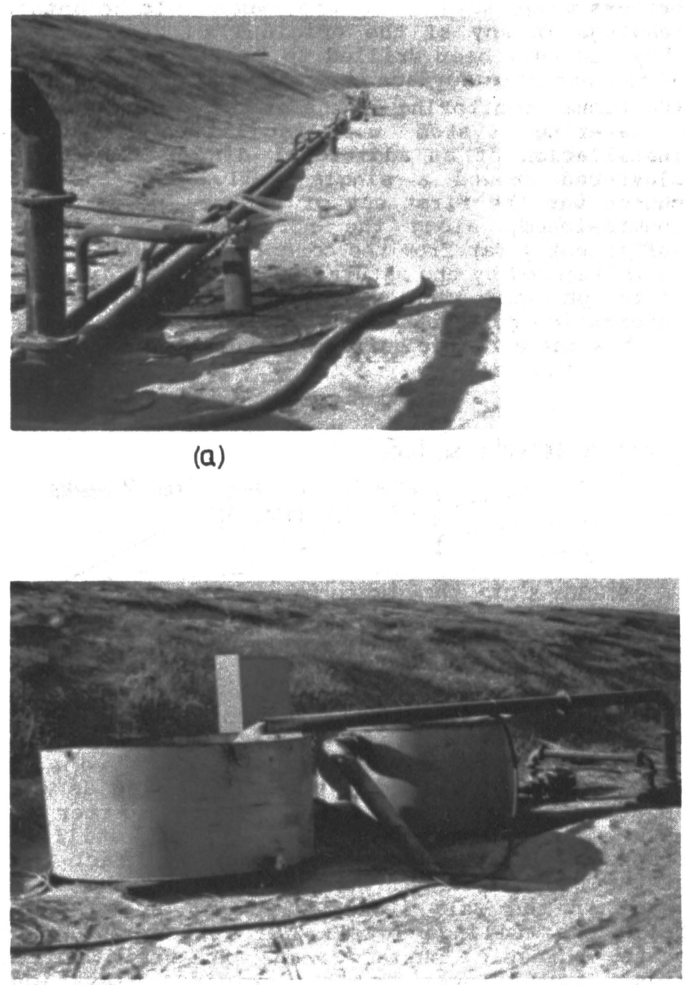
It was decided to construct a 35m long trial rock buttress along the western toe of the dam at a point where a great deal of seepage was occurring. During removal of the vegetative cover in this area, severe sloughing began developing, and it was necessary to extend the buttress over a length of 300m to stabilise the slope. Prior to placing and compacting the rock, granulated slag material was placed against the slope and connected into horizontal outlet drains. These measures proved successful in stabilising the west face (albeit not necessarily permanently), but the cost proved about 20% more than originally envisaged. This resulted in the rock buttress option working out to be more than 10% more expensive than the use of vertical de-watering wells, and it was accordingly decided to pursue the latter technique.

Installation of vertical de-watering wells on first step-back.

Since there was no experience with installing large diameter cased wells in an application of the type envisaged, three different contractors were initially engaged to each instal 20 wells. Three different techniques were use, namely mud flush diamond drilling, percussion drilling, and reverse circulation hammer drilling.

By far the most successful technique was the reverse circulation hammer drilling method. A drilling rig manufactured by Drill Systems Ltd of Calgary, Canada was used. After installing the 200mm diameter casing to a depth such that it penetrated the underlying clay foundation by at least 1m, slotted plastic casing wrapped in a plastic grid and a non-woven geotextile was installed into the hole whilst simultaneously withdrawing the casing. Once all the casing had been removed, the annulus around the drain was backfilled with slag. The amount of slag required was consistently 0.7 $m^3$. Once initial procedural problems had been ironed out, it was possible to install two wells per day. A total of 165 wells were installed around the perimeter of the operational paddocks. All but 50 of these wells used reverse circulation hammer drilling.

A venturi pump was installed in each well, and each group of twenty wells was connected to a supply and overflow reservoir. An idea of how the system works can be gained by reference to Figure 7. Figure 7a illustrates the drive line and return water line (which is of course a larger diameter), connected to the venturi pump systems. Figure 7b shows the supply and overflow tanks.



(a)

(b)

Figure 7. Arrangement of de-watering system.

MONITORING OF DE-WATERING

The spacing of the wells was 15m, which was based on a permeability value of $4x10^{-4}$ cm/sec, and assuming the water pool on top of the dam constituted an infinite water source. The yield of each venturi was tested individually, and flowrates ranging between 400 and 1800 litres/hour were measured. Since a high phreatic surface at even only one location could have been detrimental to the dam's stability, it was essential to ensure that all the venturis continued to function satisfactorily during the period of de-commissioning the dam. A small diameter (10mm) plastic pipe was installed into every well, to the bottom of the well, and these pipes were dipped every second day with a piezometer dipper. As soon as a rising water level was encountered in a venturi well, the venturi was isolated, removed and either repaired, or in one instance replaced, and then returned to the well.

The effectiveness of the venturi well system in drawing down the phreatic surface was initially monitored with the existing eight lines of piezometers (see Figure 2). Daily readings for a 4 month period on the section E piezometers indicated that the phreatic surface dropped at a rate of 14-40mm per day whilst no deposition was occurring, and 2-3mm per day during or shortly after a deposition cycle. As the dewatering process progressed, it became impossible to obtain readings in any of the original piezometers as they had only been drilled to 2m below the level of the phreatic surface that existed at that time. Additional monitoring of the effectiveness of the de-watering system was facilitated by the installation of an additional nine piezometers, clustered around a single well. The location chosen was the first set of venturi wells to be commissioned, along the north-west face, and sufficiently far from the corner of the dam not to be influenced by the natural preferential drawdown of the phreatic surface in this area. The primary information to be obtained from these piezometers was how the phreatic surface was drawn down midway between two wells.

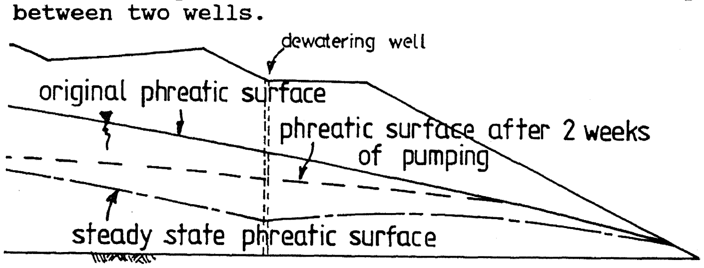



Figure 8. Section showing draw-down effect of de-watering system.

Results that were obtained using these piezometers are shown in Figure 8, which shows the drawdown for the section midway between two wells that was achieved after two weeks pumping as well as after 15 months continuous pumping. After two weeks of pumping, the drawdown was still in progress, which is attributable to the fact that at the time these readings were taken, the water level in the well itself was only about 18.5m below the tailings level. The long-term steady state phreatic surface can be seen to be significantly lower than the original elevation. The drawdown that occurred in the wells surrounded by the nest of nine piezometers is shown in Figure 9 for a nine-month period. It can be seen that it

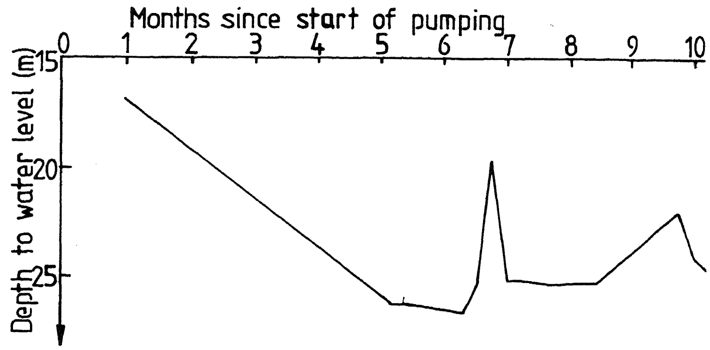



Figure 9. Water level measured in de-watering well.

took fully four months for the water levels to be drawn down to the base of the wells, possibly because deposition continued on this paddock throughout this period. These levels then remained depressed until the pumping system was stopped for routine maintenance. This resulted in a very rapid rise in the water level, which emphasized the effective role of the pumping system in maintaining stability of the dam. As can be seen from Figure 8, the phreatic surface midway between two wells was drawn down to within about 6.5m of the underlying clay.

Having implemented an effective dewatering system, and a means of monitoring the performance of this system, it was then necessary to implement a strategy for using this information to optimise the use of the tailings dam whilst it was being de-commissioned.

MANAGEMENT STRATEGY

The results of the probabilistic stability analyses showed that merely preventing the phreatic surface from rising was inadequate, and it was essential to actively decrease this level. Due to the unsatisfactory condition of the two northern paddocks illustrated by the probabilistic analyses, additional deposition cycles on the three southern paddocks were instituted. However, a rise of between 5 and 9mm per day was noted in the two lines of piezometers located on the southern side of the dam. Even though the commissioning of the dewatering system would clearly improve conditions, it was obvious from early on that it was going to be necessary to switch paddocks fairly often, and there had to be a rational basis for deciding when to make a switch.

Improvement in stability

A second series of probabilistic analyses were carried out for the eight sections previously analysed. In these analyses the phreatic surface was decreased in increments of 1m, with the shape of the surface being based on measurements similar to those shown in Figure 8. A plot such as that shown in Figure 10 was constructed for each of these sections, showing the depth of water level (in the well nearest to the section being analysed, and accounting for the approximate 6.5m difference between the water level in the well and the phreatic surface midway between two wells) versus the probability that the factor of safety was less than a given value. A probability that

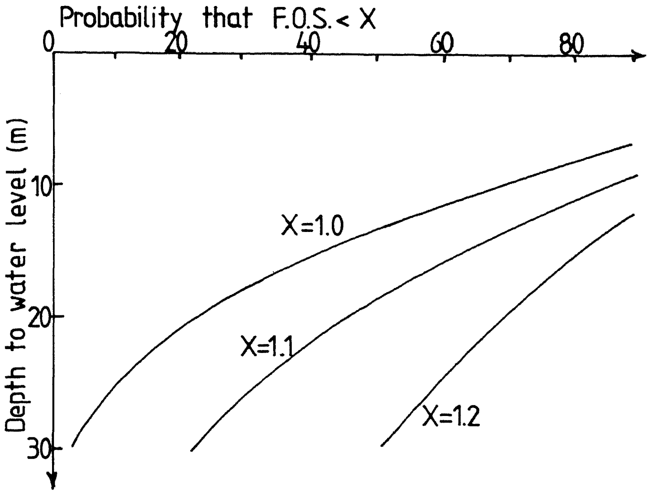



Figure 10. Typical figure used for optimisation of use of individual dam paddocks.

the factor of safety was less than unity of 10% was considered acceptable, although wherever possible it was attempted to reduce this probability to 5%. This level of risk was regarded acceptable, as it was a short-term risk whilst new dams were built and commissioned. Implementation of this strategy, together with regular inspections of the condition of the toe of the dam, were used to decide on which paddocks it was acceptable to deposit tailings. In addition, as strict a control as possible was exercised on water storage on top of the dam, to limit the proximity of the water pool to the crest of the dam.

## Schedule of decommissioning

Commissioning of the first of the two new tailings disposal facilities began in early 1980. However, the start up phase of a new dam is always a slow process, unless the entire perimeter is initially at the same level, and accordingly a reduction in tonnage being deposited on the old dam of only about 15% was envisaged by the end of 1980. The planned deposition volume, as well as the actual values for a nine month period, are shown in Figure 11. After about September 1980 the decrease in usage of the tailings dam went approximately according to the plan shown in Figure 11. Once the second of the new facilities had been operational for three months, usage of the old dam had dropped to about 50% of the maximum value.

The venturi pumps were kept going throughout the de-commissioning phase as well as for 9 months after de-commissioning. When repairs occasionally had to be made to a series of venturis, deposition was switched away from the paddock in question. Were it not for the continued operation of the venturi system, this could have resulted in a very rapid rise of the phreatic surface in the paddock taking the additional load.

## EVALUATION OF ACTION TAKEN

The use of vertical de-watering wells, together with a management programme that controlled very closely the operation of the tailings dam, resulted in successful de-commissioning of the dam, and transfer of deposition operations to the

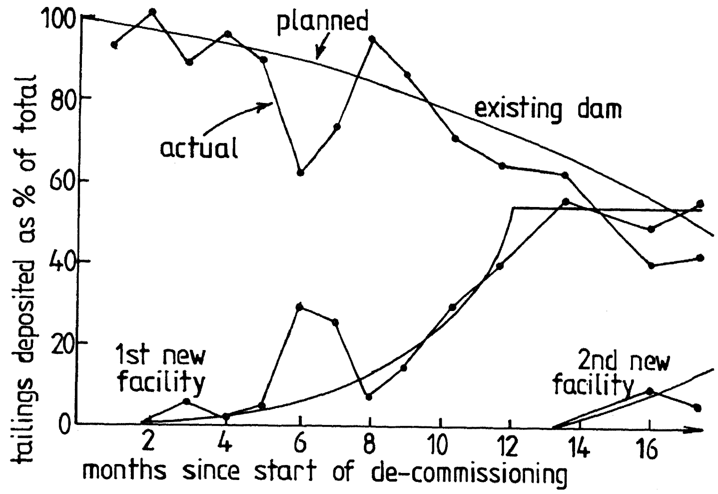



Figure 11. De-commissioning plan for old dam, showing planned and actual tonnages deposited.

new disposal facilities. It was a very effective short-term solution, that ensured the mine remained in production, whilst optimising the use of an overstressed facility. The success of the operation was in large measure due to the enlightened approach of the mine management. They were prepared to allow field trials of a largely unproven technique in South Africa, and were not averse to incorporation of a fairly comprehensive monitoring programme into the de-watering operation. The benefits of the monitoring instrumentation in this job far outweighed their cost, by enabling the optimum use of the various paddocks on the dam.

Although in retrospect the vertical wells may have been somewhat too closely spaced, the fact that total drawdown between wells was not achieved perhaps vindicates the choice of spacing. Being an active stabilisation technique was a major advantage, as changes in phreatic surface could be responded to immediately. The use of push-in drains proved singularly unsuccessful, primarily because of the very wet nature of the bottom of the dam slopes. To be able to utilise this technique it would have been necessary to cease deposition operations on a particular paddock many weeks before installation of the drains, in order to allow drying of the slope. This was not possible in view of the limited depositional area available. Construction of a 300m long section of rock buttressing showed that this approach was too costly, and would take too long to construct in view of the urgency of improving the dam's stability. Furthermore, the very wet conditions at the base of the dam made access and construction very difficult. Despite the difficulties encountered during testing and evaluation of the various stabilisation options, no further problems of instability of the dam itself were encountered. No surface sloughing of the outer slope occurred, and the base of the dam became noticeably drier.

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