PREFACE

The First and Second Symposia on Uranium Mill Tailings Management were held in Fort Collins in November, 1978 and November, 1979. During these recent years considerable effort has been devoted to research into the needs of both industry and regulatory agencies. In addition, a number of developments have occurred with regard to regulations. The Third Symposium is intended to disseminate the results of the research conducted to date and to provide a forum for the discussion of issues relating to regulation. It is expected that these Proceedings will represent the current State-of-the-Art in Uranium Mill Tailings Management. We wish to thank the authors of these papers and registrants at the Symposium for their interest and participation.

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FAILURE OF THE CHURCH ROCK TAILINGS DAM

by
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INTRODUCTION

On July 16, 1979, the Church Rock Tailings Dam failed. After the failure, United Nuclear Corporation retained two consultants to conduct independent investigations to determine the cause of the failure. At the same time, several Federal and State agencies reviewed the data and the reports by the owner's consultants. The authors of this paper were responsible for the technical review on behalf of the U.S. Nuclear Regulatory Commission.

The general nature of the failure as reported by the owner's consultants is generally agreed upon. The probable cause of failure as reported by them is presented in the following section of this paper. During the review, a number of factors became evident that may have significant influence on the design of earth dams for the retention of uranium mill tailings.

PROBABLE CAUSE OF THE BREACH

The probable cause of failure presented by each of the owner's consultants (Ref. 1, 2) was nearly the same and will be summarized below.

The embankment was about 35 ft. high and was constructed on a relatively deep deposit of clayey, silty sand. Certain soils were collapsible and some laboratory tests indicated collapse in excess of 10% upon wetting. The impoundment was not lined, and seepage into the foundation soils could occur readily. In addition, the fact that borrow for embankment fill was excavated just upstream of the tailings dam would have increased the seepage flow and rate of wetting of the foundation soils. A wetting front would have advanced downward from the impoundment with lateral spreading into the area beneath the embankment. Along the southern half of the embankment, approximately 3 feet of settlement had been observed since the beginning of operations in 1977.

As a result of the large settlement, differential movement of the embankment would be expected. Both longitudinal and transverse cracks had been observed in the embankment prior to the failure, and were

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attributed to the differential movement occurring in the embankment. In order to protect against internal erosion of the embankment in the cracks, it was recommended that a sand beach be maintained against the face of the embankment. The purpose of this sand beach was to provide for sand to be carried into any cracks that developed, thereby preventing internal erosion from occurring. The fact that sand was carried into the cracks was confirmed by the observation of sand in cracks on the walls of the breach after the failure.

Just prior to the failure, freeboard on the embankment had been decreased by pond filling operations to the point where tailings solution came into direct contact with the embankment and a sand beach was not maintained. In that configuration, the sand beach, because of its level below the tailings fluid, was ineffective and the cracks within the embankment could become filled with tailings solution. The owner's consultants then proposed a probable failure mechanism consisting of internal erosion of the embankment under the action of the fluid in the cracks.

FOUNDATION ASPECTS

The embankment was constructed on soil deposits having depths extending up to about 100 feet that included collapsible soils. Some of these soils exhibited collapse in excess of 10% upon wetting.

As the impoundment was filled with water (tailings solution) a seepage front would progress downward through the foundation soils. In addition, capillary pressure in the water in the foundation soils would cause some lateral migration of moisture from the zone of high water content above the wetting front.

The amount of water required to collapse such soils is generally small and collapse could be accomplished by capillary water. As the wetting front moved laterally, the upstream toe of the embankment would settle first, followed by settlement of the downstream toe when the wetting front had reached that point. Furthermore, the lateral migration of the wetting front would increase in depth as time continued.

After the failure, piezometers were installed along the northern portion of the embankment before operations were resumed in the northern pond area. Water levels in these piezometers have indicated that over a monitoring period of approximately six months, ten to fifteen feet of rise in piezometric level could occur. This piezometric behavior indicates that a groundwater mound has been established in that region within a short period of time after resumption of operations. Consequently, it is apparent that wetting of the entire depth of foundation soils beneath the impoundment could have occurred in the breach area from the beginning of operations until the time at which the breach occurred. Due to lateral migration of the wetting front, the upstream side of the embankment would settle first thereby causing cracking. Longitudinal cracks had been observed in the embankment and had been plugged with a bentonite and kerosene slurry on two occassions prior to the failure.

Transverse cracks formed as a result of differential settlement

along the axis of the embankment. In addition, the convex alignment in the location of the failed section would have encouraged the development of transverse cracks. Sand was observed in the cracks at various locations on the face of the breach indicating that the tailings sand was carried into the transverse cracks prior to the failure. These cracks would provide a seepage path resulting in a high phreatic surface being developed in the embankment.

Thus, the cracking of the embankment as a result of the large settlement would have a two-fold effect. One would have been to provide a high phreatic surface in the embankment and the other would have been to provide a pathway for seepage allowing for potential piping of the embankment.

STABILITY ASPECTS

After the failure, a number of test pits and test borings were located in the area of the breach. Unconsolidated undrained (UU) and consolidated undrained (CU) triaxial tests with pore pressure measurement were performed on samples taken from the test pits and borings. These data were reported in late 1979 (Ref. 3). Shear strength parameters were determined from these data by the authors of the paper for use in stability analyses. Those shear strength parameters are presented in Table 1.

Stability analyses were conducted in an attempt to evaluate whether slope instability may have been a factor in the dam failure. These stability analyses were conducted for a variety of conditions including the following:

- Analyses that adopted the effective stress shear strength parameters listed in Table 1. As shown in Fig. 1, this analysis assumed essentially complete saturation of the downstream slope from the pond level that existed just prior to failure. This conservatively assumed level of saturation attempted to allow for the effect of extensive cracking. The critical failure circle as shown in Fig. 1 for these conditions reflected a factor of safety less than unity.

- On the basis of field investigation and the test pit logs completed after the breach, the cross-section of the breach and the actual final failure surface was located by the owner's consultants (Ref. 4) as shown in Fig. 2. Also shown in Fig. 2 are several water content readings taken from borings in the vicinity of the breach. The water content values marked by asterisk in Fig. 2 indicate that a zone of unusually high water content extended from the upstream side of the embankment down to the downstream toe. This zone of high water content exists in close proximity to the proposed actual failure surface. Analysis using the effective stress shear strength parameters listed in Table 1 and the failure surface shown in Fig. 2 indicated a factor of safety of approximately 2 for even a relatively high phreatic surface. It is not expected, therefore, that a general shear failure would have occurred along that entire plane.

- A limited number of unconsolidated, undrained triaxial tests (UU) were conducted on samples taken from the zone of high water content as shown on Fig. 2. The lowest shear strength that was measured indicated an undrained strength of about 300 lb/ft^2. Other tests indicated an undrained strength of about 900 lb/ft^2. In this third variation of stability analysis, it was considered that conditions represented by the UU tests were more appropriate for the collapsing type soils following the period of initial wetting. Wedge type stability analyses were conducted assuming that a longitudinal crack formed the upstream boundary of the active wedge and the crack was filled with water. For various assumed positions of longitudinal cracks, factors of safety less than unity were computed assuming an undrained shear strength of 300 lb/ft^2. Thus, it is evident that after cracking of the embankment and introduction of tailings water into the cracks, instability of the downstream toe of the embankment could result.

It was noted by the owner's consultants (Ref. 2) that the breach area was too narrow to have indicated a general stability failure extending far back into the embankment. Also, the stability analyses conducted in the preparation of this paper indicate that for a potential failure surface extending to the upstream face of the dam, the factors of safety are high. However, instability of the downstream toe as indicated by the stability analyses reported herein may have resulted in an initial shear failure of relatively small extent. The shear failure may then have progressed upstream by a combination of internal erosion (piping and dispersivity) and subsequent slope failures. This initial slope failure would have occurred, therefore, as a result of the high phreatic surface in the cracks of the embankment and a loss in shear strength due to collapse of the foundation soils during wetting.

TABLE 1. SHEAR STRENGTH PARAMETERS

Location	Triaxial Test	$\bar{\phi}$	ϕ (Degrees)	$\bar{c}$	c (psf)
Embankment	CU	25	-	500[1]	-
Foundation Soils	CU	19	-	200	-
Foundation Soils	UU	-	0	-	300-900

[1]Assumed to be zero after cracking.

DISPERSIVITY OF THE SOILS

Sergent, Hauskins and Beckwith (Ref. 1, Vol. 3) conducted a series of laboratory tests to investigate the dispersivity of the embankment soils. Pinhole tests were conducted using distilled water and tailings solution buffered to different values of pH. Using distilled water or tailings solution buffered to a pH of 2, 4 or 7, the embankment was indicated to be nondispersive. However, when the tailings solution was not buffered and was used at a pH of 1.2, the embankment material was found to be dispersive. (The solution in the impoundment has a pH of about 1.2.)

The state-of-the-art concerning the physico-chemical aspects affecting the dispersivity of soils is somewhat lacking at the present time. It appears, however, that conventional laboratory test procedures using distilled water will not necessarily predict the accurate performance of embankment and foundation materials which may be exposed directly to tailings solution. Laboratory test data to be utilized in the design of the embankment should be performed with representative tailings solutions.

DISCUSSION AND CONCLUSIONS

Stability analyses utilizing laboratory obtained shear strength values and conventional methods of stability analysis have indicated that general slope instability as represented by sliding of the embankment was most likely not responsible for the breach. Also, as noted by the owner's consultants (Ref. 2), the breach was too narrow to have been caused by a shear failure extending to the upstream face. However, if a decrease in shear strength due to collapse of the foundation soils is taken into account and if cracking of the embankment is considered, instability of the downstream toe of the embankment can occur. It is likely, therefore, that when the tailings solution was introduced into the cracks in the embankment, an initial stability failure of the toe occurred. Subsequent to that failure, internal erosion (piping) and progression of the slope failure may have occurred in an upstream direction leading to failure. This condition probably was aggravated by the dispersivity of the embankment soil under the low pH.

The existence of a zone of relatively high water content in the embankment and the foundation soils has not been explained to date. It is expected that this zone may have considerable importance relating to the failure and would merit further investigation.

ACKNOWLEDGEMENTS

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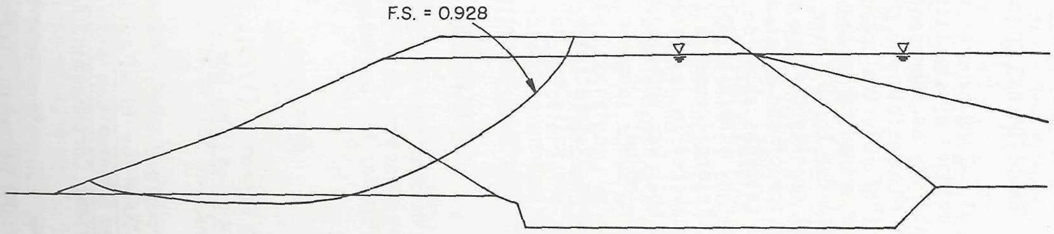

Figure 1. Critical Circle for High Phreatic Surface, Church Rock Tailings Dam.

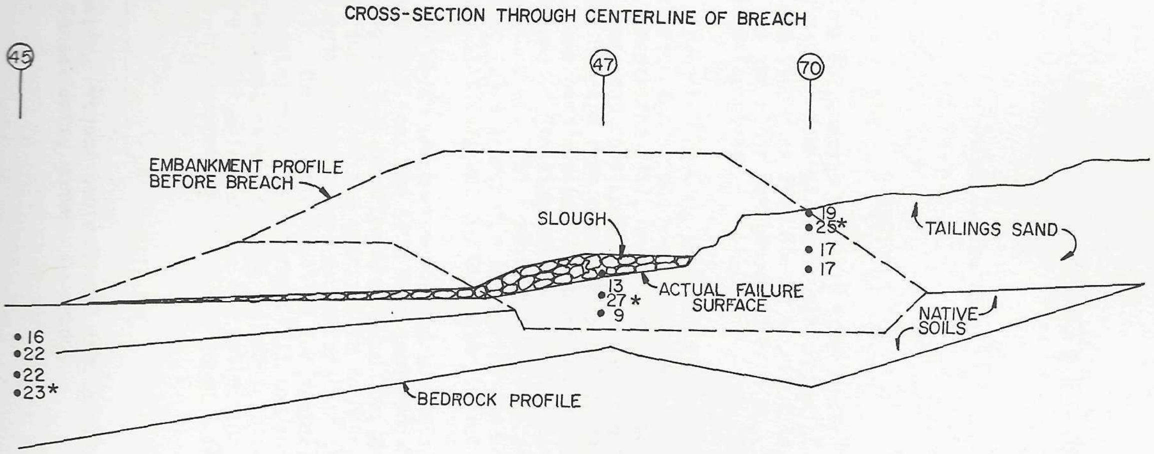

Figure 2. Actual Failure Surface after Breach (after Sergent, Hauskins and Beckwith).