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Uranium Mill Trailings Geotechnical Investigations - A Case History

V. Singh

J.P. Berry

D.F. Kapral

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V. Singh
Senior Staff Engineer-Geotechnical, Acres American Incorporated

J. B. Berry
Project Manager, Acres American Incorporated

D. F. Kapral
Manager of Project Engineering, Union Carbide Corporation

SYNOPSIS

Uranium mill tailings at Union Carbide Corporation's mining and mill complex at Uravan, Colorado, are deposited in two tailings piles along a steep hillside. The tailings are deposited in slurry form, allowed to decant, and the decant liquid removed for recycling in the milling operation. The impoundment dikes are raised using the coarser portion of the tailings in an upstream method of construction. At the time of the study, the height of the tailings piles was in excess of 100 feet. Continued use of these piles necessitated a detailed geotechnical stability evaluation and design of stabilizing measures in order to maintain safety factors and meet regulatory requirements. Any failure of these slopes could have serious consequences. This paper discusses the geotechnical evaluation of the tailings piles, design and construction of the stabilizing measures, and the performance of the tailings pile slopes. The work was performed to meet the requirements set by the Colorado Department of Health, a State of Colorado agency, and the Nuclear Regulatory Commission of the United States, which acted as consultant and reviewer to the Department of Health. These regulatory agencies conducted a detailed review of the design and construction activities. Since the construction of the stabilizing berms, a regular monitoring program has been in effect. The data collected to date indicate that the performance of the slopes has been satisfactory.

INTRODUCTION

The uranium mine and mill complex at Uravan, Colorado, is one of the oldest uranium extracting facilities in the United States. The facilities are owned and operated by Union Carbide Corporation (UCC). Approximately ten million tons of uranium tailings have been stored on the site, deposited along a steep hillside. Although the radioactivity level at the tailings is low, increasing height of relatively steep slopes raised some concern about their stability in view of the potential for seismic activity in the region.

Engineering studies were conducted in 1978 to evaluate the slope stability of these tailings. The studies indicated that a significant seismic event could cause failure of saturated slopes. The tailings ponds were still receiving additional tailings, and it was essential that these slopes remain stable under all possible design conditions. Therefore, in 1978 UCC contracted Acres American Incorporated (Acres) to perform a detailed geotechnical evaluation of the tailings slopes and design stabilizing measures as required. These studies were carried out during the period of May 1979 through November 1980. As a result of the studies, it was determined that stabilizing berms would be necessary to assure stability of slopes. The construction of the berms and associated drainage and collection system was completed in November 1980. Instruments were installed to check the validity of the design assumption and monitor the performance of the slopes. Three years of data collected to date confirm the tailings slopes are performing satisfactorily.

SITE LOCATION

Uravan is located on the banks of the San Miguel River, approximately 100 miles south of Grand Junction, Colorado. The mill complex and the tailings are located near the town on the San Miguel River, which flows into the Dolores River, a tributary to the Colorado River. An aerial view of the site and the town is shown in Figure 1.

TAILINGS DEPOSITION

During the early days of operation, the tailings were deposited in three separate ponds designated Ponds 1, 2, and 3. In recent years, Ponds 1 and 2 have been combined into a single pond; they are referred to as Pond 2 in this paper. Currently, the tailings are deposited alternatively in Ponds 2 and 3.

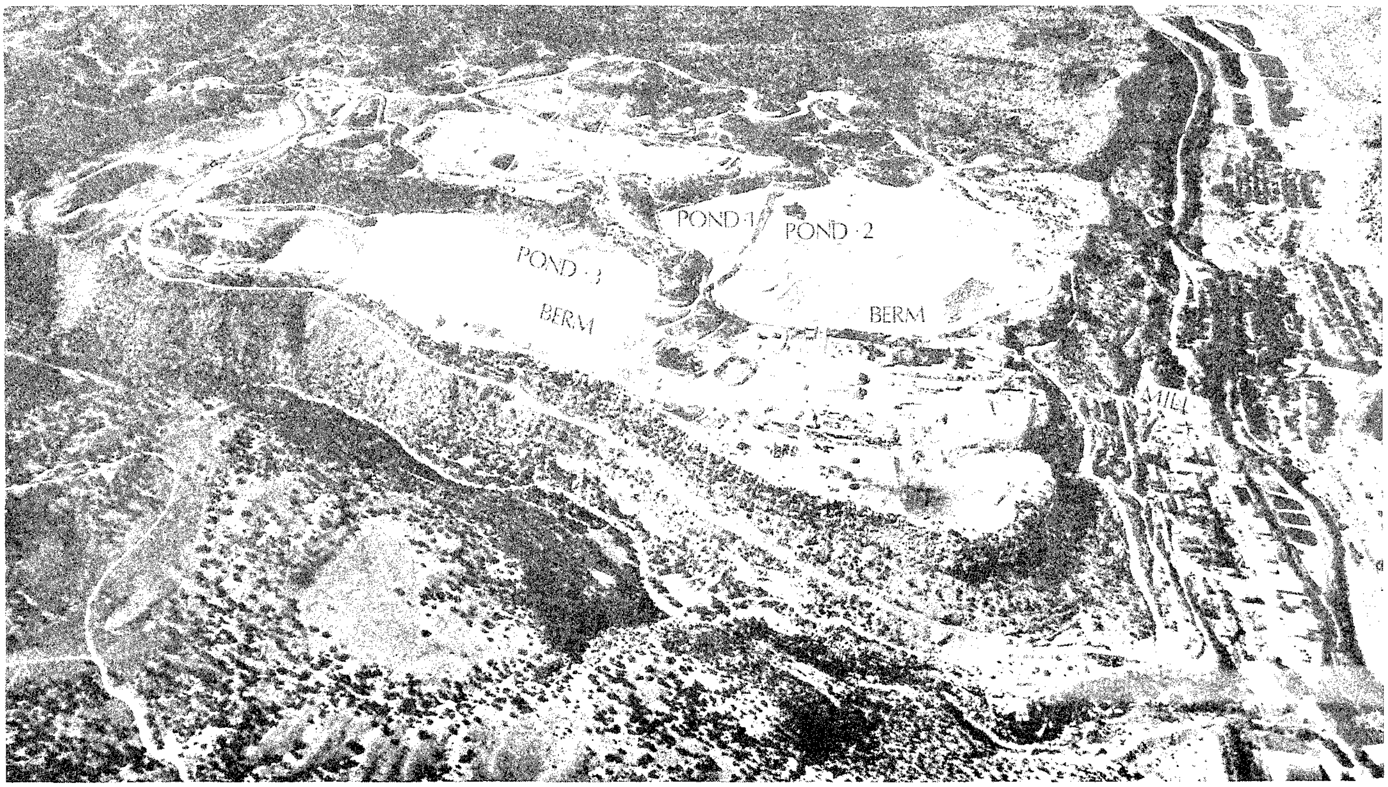
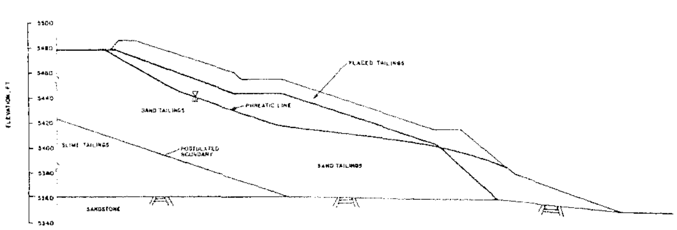


FIGURE 1 - URAVAN PROJECT SITE AERIAL VIEW



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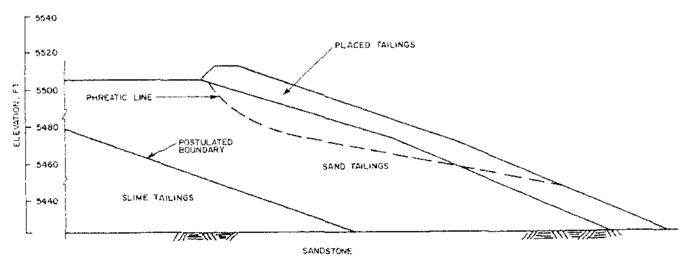


TAILINGS PILE 3

FIGURE 2 - TAILINGS PILE SLOPES BEFORE STABILIZATION

resent, a single point discharge method is used to sit the tailings slurry. Coarser particles in the ings slurry settle near the discharge point and r particles in suspension are carried towards the . interior where they settle under stagnant pool .itions. The decanted liquid is removed by .ting pumps for recycling in the milling opera- s. The coarser particles range from coarse to : sand size and are termed coarse tailings; finer .icles are in silt size and are termed slimes. The :harge point is moved along the perimeter of the l to raise the tailings level as evenly as ;ible.

ıpstream method of construction is used to raise perimeter dikes of the ponds. Coarse tailings · the perimeter are excavated, moved to the dike ıtion, and compacted with dozers while the tailings ·ry is discharged to the alternative pond. The ›action is achieved with the dozer in as uniform a ıer as possible. The cycling between Ponds 2 and 3 nits an uninterrupted operation of the mill.

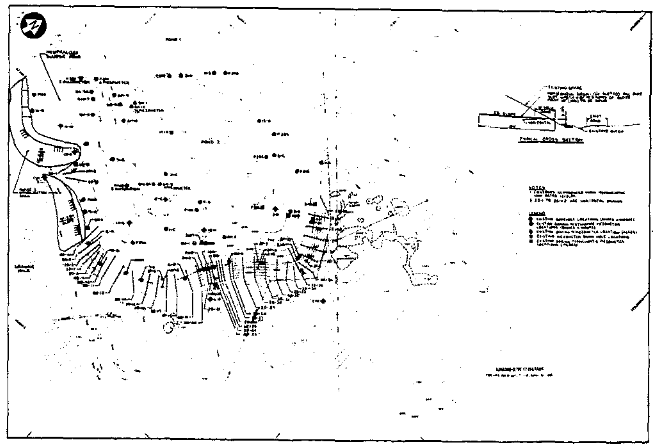
.INGS SLOPES

tailings piles were over 100 ft high and located inst a steep hillside. Before stabilization, the pes were steep and irregular, broken by berms used access roads. Average slope angle varied from 20 rees to 42 degrees for Pond 2 slopes and from 20 to degrees for Pond 3 slopes. Typical slopes for se ponds, before the construction of the berms, are wn on Figure 2. Investigations completed prior to s study indicated that the outer portions of the pe consisted of coarse tailings with some silt size ticle layers. The interior of the pond contained dominantly slimes with some pockets of coarse 'lings.

TECHNICAL INVESTIGATIONS

otal of 35 shallow and deep borings had been lled at the site at different times prior to this :dy. However, the information was not complete, and :omprehensive subsurface and laboratory investiga- ıns program was initiated in 1979 to more completely :ermine the characteristics and distribution of lings and collect engineering data for the analyses l design. A total of 68 additional borings were lled in the tailings pond, some of which penetrated : entire depth of the tailings and several feet into : underlying bedrock. Thirty-nine of these borings ·e converted to piezometers to define the hydro- ›logic regime within the tailings. In addition, a ıld pumpout test was performed in tailings Pond 2 to :ermine the mass permeability of the outer tailings i the flow pattern within the complex matrix. The :ation of these borings, piezometers, and the nping wells is shown on Figure 3. All field work ; performed under the full-time supervision of geo- :hnical engineers who carefully logged all the nples and kept detailed records of drilling tivity, including any unusual conditions.

ɐ investigations were carried out in four stages, ch subsequent stage planned on the basis of the sults of the previous stage. Part of the reason for is staged investigation was to obtain certain



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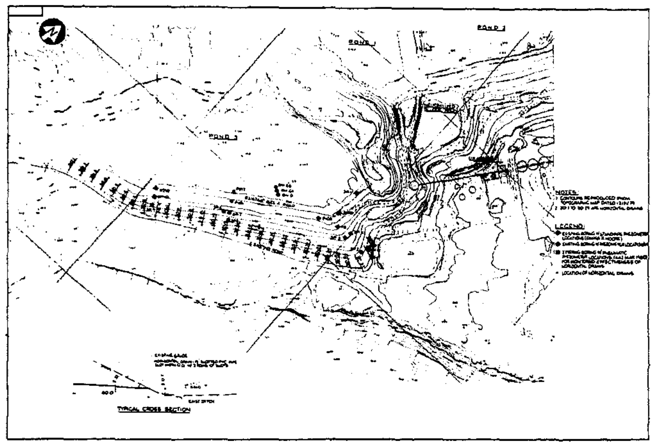


TAILINGS PILE 3

FIGURE 3 - BORING LOCATION PLAN

specific data within a predetermined time period to avoid interruption of pond use as discussed later.

Stage I investigations were carried out during July 1979. Seven boreholes (P-101 through P-107) were drilled by Basin Range and Drilling Company using rotary drilling techniques. A steel casing was used to stabilize the holes. Standard penetration tests were performed at 5-ft intervals and split spoon samples obtained for soil classification. A casa- grande type piezometer was installed at the bottom of each hole upon completion. The results of these investigations indicated that the tailings are a com- plex matrix, and it was decided to drill additional holes and take continuous samples for the entire depth.

Therefore, during November 1979 Stage II investiga- tions were begun wherein four additional holes (BH-1 through BH-4) were drilled by Raymond International using a rotary wash method. Continuous standard pene- tration tests were performed in three of the borings and 3-inch Shelby tube samples were taken in BH-1 for its entire depth. Those samples were used for controlled laboratory tests and soil profile deter- mination. In addition, a casagrande type piezometer was installed in BH-2. This completed Stage II of the investigations.

As the data evaluation and engineering analyses progressed, it became increasingly evident that a wider coverage of the tailings perimeter was necessary to achieve the desired degree of confidence and meet regulatory requirements. Therefore, a third stage of investigations was carried out during the period of January 1980 through March 1980. A total of 44 additional borings were drilled by Armstrong Engineers. Sixteen of these borings (2P1 through 2P6 and 3P1 through 3P10) were drilled for the installation of pneumatic type piezometers at shallow depth to monitor the effectiveness of horizontal drains installed during February and March 1980 for the purpose of lowering the phreatic surface. (Berry & Velarde, 1981). Eleven borings (BH-5 through BH-11) were drilled using hollow stem augers. Six of the borings were sampled continuously for their entire depth using split spoon sampler and four borings were located adjacent to these split spoon sample holes to recover intact Shelby tube samples at selected depths. Both split spoon and Shelby tube samples were collected in Boring BH-9.

The investigations program was essentially complete at this stage, however, one area of concern expressed by the regulatory agencies still existed. A continuous horizontal layer of slime material was incorporated in the design calculations and this markedly reduced the calculated safety factors for the slopes as discussed later in this paper. Therefore, Stage IV investigations were carried out in June 1980 to specifically investigate the probability of the existence of a continuous layer and the engineering properties of the layer. Thirteen additional borings (BH-20 through BH-32) were drilled. Nine of these borings were continuously sampled with split spoon and 3-inch Shelby tube samplers to develop a complete soil profile. For the remaining four borings, the upper 40 feet was drilled without sampling and split spoon samples at 5-foot intervals were taken below that down to a depth of 10 to 25 feet above the top of the rock. This remaining depth of holes, which was suspected to contain the greatest concentration of slime layers, was continuously sampled with Shelby tubes.

During these investigations, no such continuous layer of slime was encountered. However, numerous pockets of low density fine-grained slime-like material were encountered scattered within the coarse tailings.

LABORATORY TESTS

A comprehensive laboratory testing program was undertaken to complement the drilling program and to determine the engineering properties of tailings. On the basis of index tests, the tailings were classified in two major soil classifications: coarse tailings or sand tailings, and fine tailings or slime tailings. Sand tailings consisted of coarse to fine sand size material within the outer portions of the slopes. Shear strength of this material was generally higher than slime tailings. Slime tailings consisted of fine grained, nonplastic soil in silt size range. The shear strength of these soils was generally lower. The range of grain size distribution for the sand tailings is presented in Figure 4.

Triaxial shear strength tests were performed on both the isotropically and anistropically consolidated samples with pore pressure measurements. In addition, cyclic triaxial and resonant column tests were performed under isotropic and anisotropic conditions to determine the dynamic characteristics of the tailings. A list of laboratory tests is presented in Tables I, II, and III. The following observations were made during the interpretation of the test results:

TABLE I

SUMMARY OF INDEX TESTS

Test Type	No. of Tests	Range of Values	Average Value
Unit Weight (pcf)	27	82.5 - 126.0	114.8
Water Content (%)	37	13.9 - 39.2	26.8
Grain Size (% passing 200):			
Sand Tailings (Wet Sieving)	47	12.0 - 47.0	28.0
Sand Tailings (Dry Sieving)	4	8.0 - 20.0	14.0
Slime Tailings	36	50.0 - 98.0	75.0
Atterberg Limits (%):			
L.L.	26	21.0 - 71.2	43.3
P.L.	26	NP - 43.0	24.2

TABLE II

SUMMARY OF SHEAR STRENGTH TESTS

Test	Number of Tests	
	Sand Tailings	Slime Tailings
Isotropically Consolidated Triaxial Test	12	13
Anisotropically Consolidated Triaxial Test	8	12
Direct Shear Test	--	2

TABLE III

SUMMARY OF CYCLIC TESTS

	Sand Tailings	Slime Tailings
Isotropically Consolidated Triaxial	21	--
Anisotropically Consolidated Triaxial	12	3
Resonant Column Anisotropically Consolidated ..	5	--

(a) A large variation was observed in both the index and the engineering properties of the tailings within each group. This variation could be attributed to the change in the host rock at different locations, slight variations in milling operations, depositional process within the pond, and possible sample disturbance.

(b) A marked difference was noticed in grain size distribution determined from dry sieve analysis and wet sieve analysis (ASTM D421). The curve from the wet sieving was on the finer side of the curve from dry sieving for the same sample. It is believed that the salts act as a binder in the dry state but dissolve upon wetting.

(c) The tailings exhibited greater shear strength and resistance to dynamic loading than expected of most natural soils with similar grain size and density. Presence of salts and other chemicals may have contributed to this cementation or apparent cohesion in tailings. Further, the angularity of crushed rock would contribute to the increase in strength.

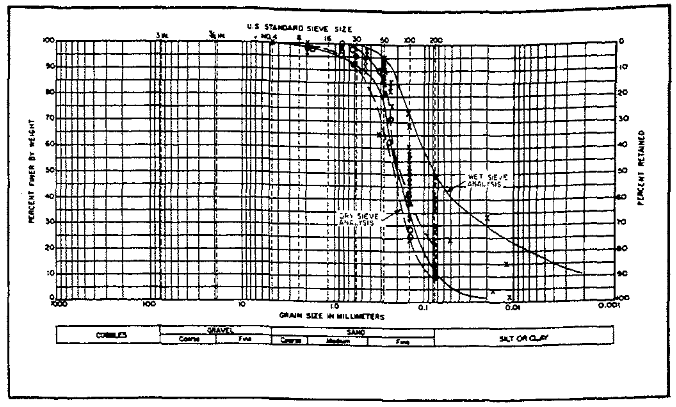
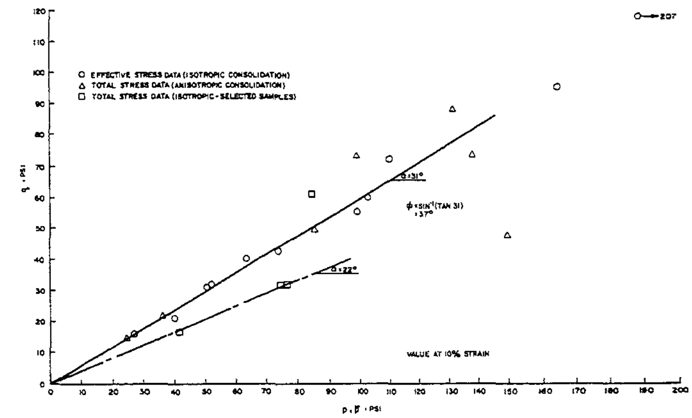


FIGURE 4 - GRAIN SIZE CURVES - TAILINGS

DESIGN PARAMETERS

The data from triaxial tests were analyzed using the Mohr circle plots and stress path method, the latter generally providing more consistent results. The design parameters are presented in Figures 5 and 6 and were selected conservatively to negate the effect of variability in tailings properties. The effective shear strength of Uravan tailings is presented in Table IV along with shear strength for selected tailings reported in various publications.



TRIAXIAL TEST RESULTS-SAND TAILINGS

FIGURE 5

DESIGN CONSIDERATIONS

Stabilization of the tailings slopes was carried out in several stages to meet regulatory and operational requirements. The interim stabilizing measures are discussed by others (Berry & Velarde, 1980); only the final stage stabilization is discussed here. Because of scheduling, portions of the design and construction progressed concurrently. A value engineering approach was used to provide a cost-effective design in order to meet budgetary constraints.

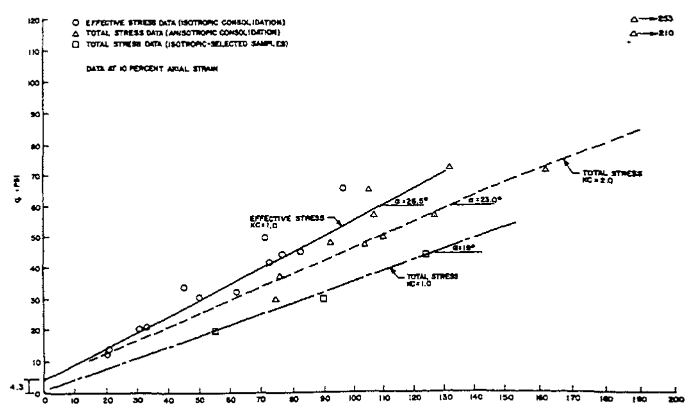
The following guidelines were used to establish design criteria:

(a) The tailings slopes and hydrologic parameters had to meet the requirements of NRC Regulatory Guide 3.11, Revision 2, December 1977.

(b) Pond 2 slopes were projected to be raised by an additional 25 feet and Pond 3 by 30 feet from their current elevations.

(c) The slopes were to maintain a reasonable margin of safety for the most severe earthquake event with a 1,000 year return period. The peak horizontal free field acceleration for this event was estimated to be 0.12g at the site (Dames & Moore, 1978).

(d) A 200-foot wide continuous beach was to be maintained along the perimeter for Pond 2 and a 150-foot wide beach for Pond 3.



TRIAXIAL TEST RESULTS-SLIME TAILINGS

FIGURE 6

(e) A minimum factor of safety of 1.5 was considered acceptable for the normal operating conditions and 1.0 for the most severe seismic event against mass slope failure.

(f) A minimum freeboard was set to contain maximum probable precipitation without overtopping of the perimeter dikes.

SOIL CONDITIONS WITHIN THE SLOPE

As stated earlier, the slope geometry varied considerably along the perimeter of the pond, both in shape and height above the bedrock. Therefore, a relatively large number of cross sections were analyzed to assure the safety along the entire perimeter while maintaining the most economical size berm. This approach differed from a conventional design practice where the design is based on the results of the most critical section. Figure 7 shows two typical cross sections (one for each pond) that were analyzed. The geometry of these slopes was established from the borings and field survey data and is a good representation of actual conditions. The following key features that affected the design are worth noting:

(a) A layer of low shear strength slime tailings was postulated to be present within the coarse tailings. The critical, hypothetical location of this layer was established to yield the lowest factor of safety by trial and error. Due to its marked influence on the stability of slope and thus the cost of stabilization, a special investigation program (Stage IV) was undertaken to confirm or negate the existence of such a layer within the coarse tailings. Although no such continuous layer was encountered, numerous pockets of soft slime tailings were discovered within the coarse tailings. A decision, therefore, was made to postulate the existence of such a weak layer in order to provide a safer design.

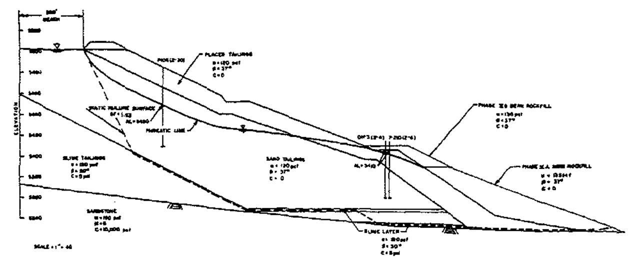
(b) Data from piezometers and pump out tests indicated the hydrogeologic regime of the tailings to be very complex with several perched water tables. For the design a conservative upperbound phreatic surface was selected. This surface was then projected for the planned tailings height using numerical analyses and judgement. The safety factors for the slopes, therefore, are somewhat higher than the calculated safety factors.

(c) Interface between the coarse tailings and slime tailings was established on the basis of drilling results and the records on tailings deposition history.

The conservative assumptions included in the design have more than compensated for the potential uncertainties related to the subsurface conditions and therefore increased the overall confidence level in the design.

DESIGN OF BERMS

The design of the berm was based on the results of static slope stability analyses. The analyses were performed using a computer program developed from the Janbu method of slope stability under limiting equilibrium conditions. First, the slope stability was evaluated using effective stress analysis and long-term loading conditions. Several trial berm sizes were considered to arrive at an "optimum" size that yielded a safety factor close to 1.5. Once an acceptable berm size was determined, the section was analyzed for mass stability during a seismic event using a pseudostatic coefficient of 0.12g. This analysis was performed using consolidated undrained shear strength parameters to simulate the quick loading during a seismic event. The undrained shear strength was determined for anisotropic stress conditions present within the slope. A static stress analysis, using finite element techniques, was performed to estimate the state of anisotropy within the slope, and results of this analysis were used in selecting anisotropic consolidation stresses for the laboratory tests. In most cases, the safety factor for the pseudostatic analysis was very close to 1.0. In a few instances, a safety factor of slightly less



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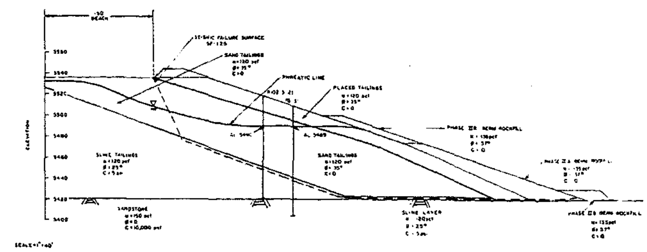

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FIGURE 7 - TYPICAL CROSS SECTIONS USED IN SLOPE STABILITY ANALYSIS

than 1.0 was calculated and the berm size was increased to bring this to 1.0 or higher. In most cases, a balanced design was obtained that provided a safety factor close to 1.5 for the static conditions and 1.0 for the pseudostatic conditions while utilizing the conservative soil and hydrologic assumptions stated earlier.

TABLE IV(a)

SUMMARY OF DESIGN PARAMETERS

Tailings	Unit Weight lbs/ft^3	Shear Strength Undrained c psi	Undrained Ø deg	Drained c psi	Drained Ø deg	Comments
Sand	120	-	-	-	37°	Anisotropic
		-	35	-	-	K_c = 2
Slimes	120	-	-	5	30	Anisotropic
		5	25	-	-	K_c = 2
Rockfill (Berm)	135	-	-	-	37	Based on Judgment

TABLE IV(b)

COMPARISON OF SHEAR STRENGTH WITH SELECTED PUBLISHED TAILINGS DATA

Tailings	c psi	Ø deg	c psi	Ø deg	Comments
Uravan					
Sands	-	35	-	37	28% passing #200
Slimes	5	25	5	30	75% passing #200
Copper Tailings (Volpe, 1979)	14.4	14-17	-	34-38	3-98% passing #200
Iron Tailings (Guerra, 1972					
Carrol Lake	-	-	-	32	16% passing #200
Knub Lake	-	-	-	37.5	90% passing #200
Copper & Molybdenum (Klohn & Maartman)	-	-	-	33-34	double cycloned 5% passing #200

An independent evaluation by a consultant for the Nuclear Regulatory Commission, assisting the Colorado Department of Health review team, confirmed these findings. Further, their report stated that the slopes have a reasonable margin of safety under post earthquake loading conditions.

Once the analytical design was complete, the minimum dimensions for the berm were established by considering factors such as constructibility, transition from one section to another, phased construction, and existing slope geometry.

SEISMIC STABILITY

Since the site is located in a potentially active seismic region, the stability of tailings slopes during a significant seismic event was to be assured. The slime tailings were found to have significant resistance to liquefaction and therefore their stability was of no concern as long as the outer tailings were stable.

The stability of the outer tailings was investigated for liquefaction and slope displacement. First, the liquefaction evaluation was performed using Seed's method (Seed, 1979). Standard penetration test results (blow counts) were compared with lower bound curves reported for sites where liquefaction is known to have occurred under comparable earthquakes. The analysis showed that the tailings would remain stable provided the phreatic line was ten feet or more below the surface.

To further confirm the results of this simple analysis, a dynamic response analysis (Quad 4) was performed for a representative section for Pond 2 (Idriss et. al, 1973). The time history used in the analysis was scaled from the Kern County earthquake, Taft record. Induced cyclic shear stresses were compared with the laboratory determined cyclic shear strength. This evaluation confirmed that the coarse tailings within the slopes will remain stable for a water table ten feet or more below the surface.

Further, a preliminary deformation analysis was performed for the same slope using the Makdisi and Seed approach (Makdisi & Seed, 1978). The analysis indicated a very low potential for any significant deformation and it was considered appropriate not to perform a more detailed deformation analysis.

The top of the berm elevation was adjusted accordingly to provide a minimum of ten feet of cover above the projected upperbound phreatic line.

CONSTRUCTION OF BERM

Interim remedial stabilization measures were implemented in late 1979 to eliminate local sloughing of slopes and to lower the phreatic surface within the slope. These measures are described elsewhere.

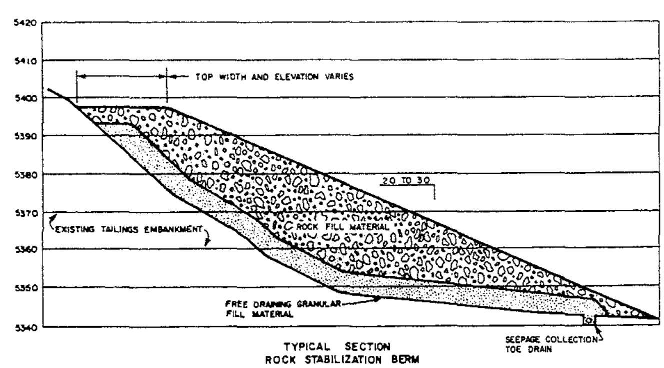

FIGURE 8

The construction of the main berm started in May 1980 and was completed in November 1980. The berm was constructed in two phases because certain minimum dimensions had to be maintained to improve the stability of the tailings slope by a predetermined mandated date in order to continue the use of tailings ponds. While the design studies were still going on, a conservatively designed berm was constructed. Later, the final berm was constructed by widening and raising the previously constructed berm.

A typical cross section of the berm is shown in Figure 8. A layer of properly graded gravel with a minimum thickness of three feet normal to the slope was placed to within three feet of the top of the berm and covering the entire tailings slope under the berm where it terminated in a seepage collection system at the toe. A select size sandstone rock layer with a minimum horizontal dimension of ten feet was constructed over the gravel layer for the entire height, extending beyond the toe of the berm to cover the seepage collection drain. The remainder of the berm above the select size sandstone rock layer was constructed with quarried rockfill.

The seepage collection system was included in the design to safely collect and carry the seepage liquid to several pump stations for pumping to a pond for reuse. An 8-inch diameter high-density polyethylene perforated pipe was placed in an excavated French drain along the entire length of the berm. Clean out pipes were installed every 200 feet along the drain to permit sampling, aid the observation of seeping liquid, and facilitate flushing if blockage were to develop in a particular section.

Approximately 200,000 tons of gravel were used for the filter and 700,000 tons of rock for the berm. Gravel was obtained from a bank-run gravel pit located approximately ten miles from the site. Processing was required to obtain specified gradation. Sandstone rock was obtained from an open pit quarry located about a mile east of the Pond 3; selective quarrying was necessary to secure good quality rock. Adequate compaction tests were performed to assure a uniform compaction.

POST CONSTRUCTION PERFORMANCE

A comprehensive monitoring program was established to monitor the performance of the piles. Piezometers and surface monuments were installed to monitor changes in the phreatic surface within the slope and any slope movements. In addition, flows from the seepage collection drain system are monitored and recorded for each pond. The frequency of each measurement is shown on Table V. The data are evaluated every month and summarized quarterly.

TABLE IV

MONITORING PROGRAM

Frequency	Measurement
Weekly	Piezometers
Daily	Beach width and freeboard
Daily	Seepage flows
Daily	Visual inspection of embankment and berm surfaces
Daily	Decant and slurry transport systems inspection
Quarterly	Movements of settlement monuments
Annual	Site Inspection by a consultant.

Since the completion of the stabilization berms, the ponds have only been used intermittently. Therefore, the fluctuations in piezometer levels and seepage quantities have been inconsistent due to constantly changing hydraulic conditions. Nevertheless, on the basis of the data collected through June 1983, the following conclusions are drawn:

(a) Water levels in piezometers fluctuated randomly.

(b) Generally, piezometric levels have been less than projected in the analysis. This is a favorable condition from stability considerations. This could mean that the ultimate tailings height may be allowed to rise higher than projected in design, while still maintaining required safety factors.

(c) Changes in average flow rates from the drainage system are not consistent with changes in the piezometric levels.

(d) An insignificant change in the piezometric levels was supported by the water balance on a yearly cycle basis.

(e) The maximum vertical movement of the slope was 6.28 inches followed by another monument with a movement of 5.74 inches. The maximum horizontal movement for the same monuments was 1.53 and 2.85 inches, respectively.

(f) No distress or other problems have been oberved regarding the performance of the slopes.

SUMMARY AND CONCLUSIONS

The stability of the tailings slopes was improved by constructing a rockfill berm. The size and dimension of the berm were established on the basis of comprehensive geotechnical investigations and studies. A value engineering approach in the design resulted in a cost effective construction with minimal overbuilding of the berm. Ongoing monitoring has shown that the piles are performing satisfactorily.

ACKNOWLEDGEMENT

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