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Mined out open pits are presently under consideration as disposal sites for uranium mill tailings. In this method of tailings management, the escape of contaminated liquid into an adjacent aquifer is the principal environmental concern. The modified Bishop Method was used to analyze the structural stability of a clay liner along the highwall and fluid flow models were used to analyze the effect of tailings solutions on ground water under several operating conditions. The slope stability of a clay liner was analyzed at three stages of operation: 1) near the beginning of construction, 2) when the pit is partially filled with tailings, and 3) at the end of construction. Both clay lined and unlined pits were considered in the fluid flow modeling. Finally, the seepage of tailings solutions through the clay liner was analyzed.

Results of the slope stability analysis showed that it would be necessary to construct the clay liner as a modified form of engineered embankment. This embankment would be similar in construction to that of an earthfill dam. It could be constructed on a 1:1 slope provided the tailings

slurry were managed properly. It would be necessary to maintain the freeboard height between the embankment and tailings at less than 4 m. A partially dewatered sand beach would have to be located adjacent to the embankment.

Potential leakage and aquifer contamination was modeled for lined and unlined pits of various designs. Sulfate, and possibly U and Th, are the most likely contaminants. Results from the model showed the clay and soil cement lined pit to be most effective in containing the pollutants.

Analyse du stockage des résidus de traitement minerai d'uranium dans une carrière à ciel ouvert désaffectée

Les anciennes carrières à ciel ouvert sont actuellement considérées comme sites possibles pour le stockage des résidus de traitement du minerai d'uranium. Dans cette technique le stockage des résidus, le principal problème d'environnement est une fuite de liquide contaminé dans la couche aquifère adjacante. La méthode de Bishop modifiée a été utilisée pour analyser la stabilité structurale d'un revêtement d'argile le long d'un haut mur et des modèles d'écoulement de fluides ont été utilisés pour étudier l'effet des solutions de queue de traitement sur l'eau du sol dans différentes conditions opératoires. La pente de stabilité du revêtement d'argile a été analysée aux trois stades de l'opération: (1) au début de la construction, (2) quand la carrière est partiellement remplie de résidus, (3) à la fin de la construction. Des carrières avec ou sans revêtement ont été prises en considération dans le modèle d'écoulement des fluides. Enfin, le suintement des solutions de queue de traitement à travers le revêtement d'argile a été étudié.

Les résultats de l'analyse de la pente de stabilité ont montré qu'il serait necessaire de construire un revêtement d'argile suivant la forme modifiée d'une digue. La construction de cette digue serait identique à celle d'un barrage de terre. Elle pourrait être construite avec une pente de 1:1 en admettant que la pâte de résidus soit gérée correctement. Il serait necessaire de maintenir un dénivelé entre le haut du barrage et les résidus d'au plus de 4 m. Une plage de sable partiellement deshydraté devrait jouxter la digue.

La fuite potentielle et la contamination de la couche aquifère ont été modelées avec différents schémas de carrières avec ou sans revêtement. Les contaminants les plus probables sont les sulfates et les possibles: U et Th. Les résultats du modèle ont montré que la carrière avec revêtement d'argile et de ciment de sol est la plus efficace pour contenir les polluants.

Introduction

Disposal of tailings in depleted ore pits is an appealing alternative to other tailings management methods of the recent past. In the 1950s and 60s, tailings were often piled on the surface and left to dry.[1] Wind erosion and sheet runoff widely dispersed these tailings with their low concentrations of radioactive components. Another common method of containment was the construction of a ring dike made from the sandy portion of the tailings. Clay slime and contaminated water were impounded within the ring dikes. Often the dikes were poorly designed and constructed, they were located on the flood plains of major streams and their reservoirs were unlined. Failure of several of these dikes led to the uncontrolled surface discharge of tailings. Although dikes have remained intact at most tailings impoundments, groundwater contamination occurred by seepage through the floor of the reservoir. More recently, high earth fill embankments have been constructed across natural drainage basins to impound slurried tailings within lined reservoirs. While dams provide short-term (tens to perhaps hundreds of years) protection against erosion of tailings, the natural stream course will eventually breach the embankment and cut intricate and progressively deepening channels through the tailings. Even in the short-term there is the risk of catastrophic failure of a poorly constructed or earthquake damaged embankment. It is generally agreed that below grade disposal of tailings in depleted ore pits substantially reduces the impact of wind erosion, eliminates the possibility of catastrophic failure and reduces the possibility of stream erosion.

Little is known about the potential impacts of below grade disposal of tailings on groundwater. The purpose of this study is to analyze the stability of a clay liner on a steep slope and to determine its effectiveness in sealing off contaminated, acidic (pH <2) liquid waste from surrounding aquifers.

Slope Stability Analysis

Oak Ridge National Laboratory (ORNL) staff analyzed two simplified designs for clay lined pits (Figure 1). The design with a 1:2 highwall slope is similar to a tailings management alternative being considered by United Nuclear Corporation[2] for their proposed Morton Ranch uranium mill near Glenrock, Wyoming. ORNL proposed the 1:1 highwall slope which is more stable and may require a smaller volume of material for the clay liner.

Table I lists the engineering properties obtained from triaxial shear tests on undisturbed lithologic units in the Ft. Union Formation and on remolded (Proctor mold) soils. These data were supplied by Dames and Moore, consultants to United Nuclear Corporation. Saturated samples of the Ft. Union were tested under consolidated-drained conditions. The moisture content of remolded soils was on the wet side of optimum and samples were tested under consolidated-undrained conditions. Engineering properties for the tailings were assumed to be like that of Exxon's nearby Highland Mill.

The Modified Bishop method[3] was utilized for slope stability analysis. Table II lists the factors of safety for critical failure arcs under static and maximum probable earthquake loading conditions, (acceleration equal to 8% of gravity), with and without liquefaction of the tailings.

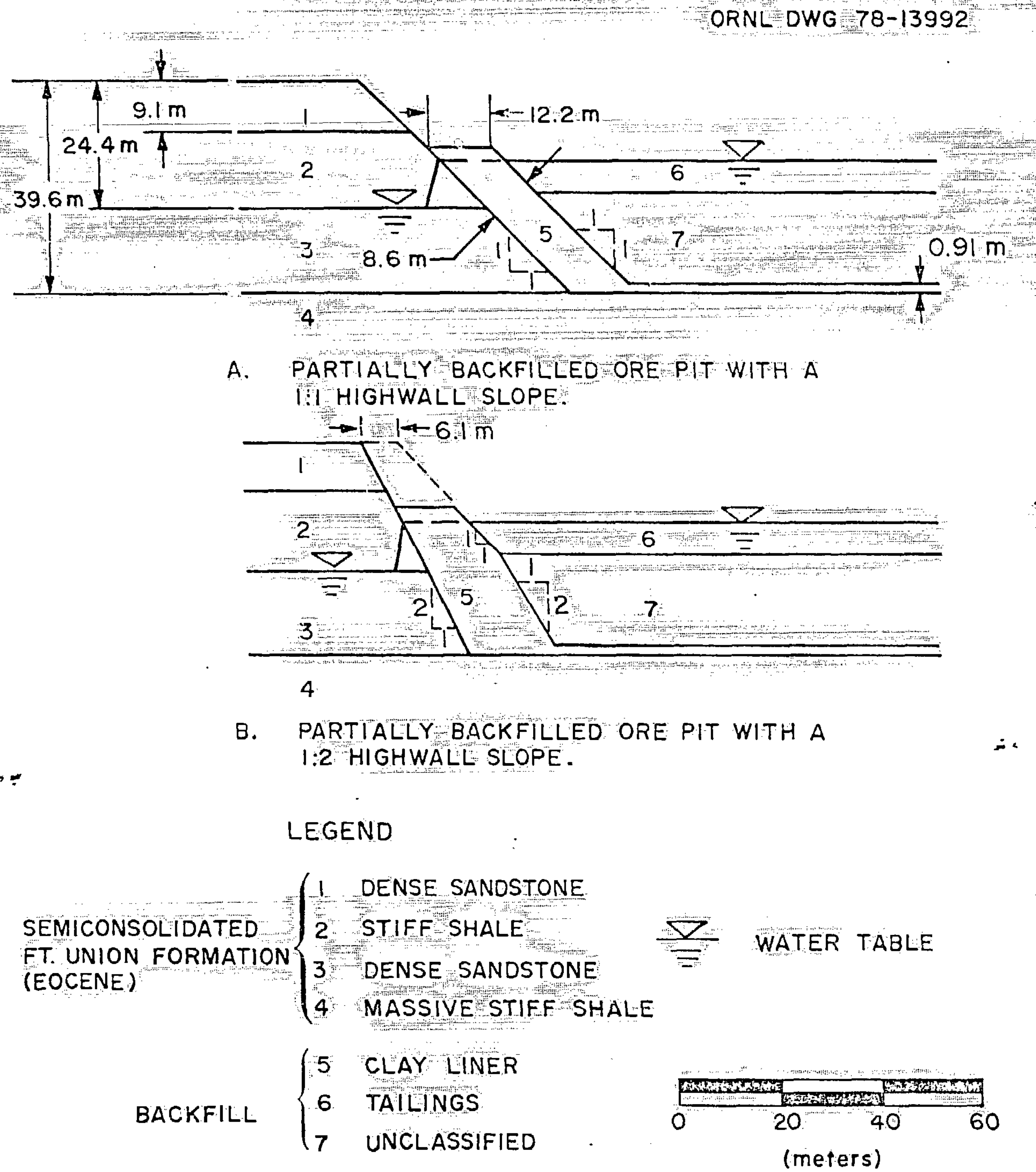

FIGURE 1: CLAY LINER DESIGNS USED IN COMPUTER CODE FOR SLOPE STABILITY ANALYSIS BY THE MODIFIED BISHOP METHOD

TABLE I: SHEAR STRENGTH PROPERTIES USED IN SLOPE STABILITY ANALYSIS

NO.	MATERIAL	TRIAXIAL SHEAR TEST	SPECIFIC WEIGHT GM/CM^3	SPECIFIC WEIGHT (PCF)	COHESION PASCALS X 10^3	COHESION (PSF)	FRICTION ANGLE (DEG)
1	DENSE SANDSTONE	CD[B]	2.0	(125)	47.9	(1000)	31
2	STIFF SHALE	CD	2.1	(130)	21.5	(450)	15
3	DENSE SANDSTONE	CD	2.0	(125)	47.9	(1000)	31
4	STIFF SHALE	CD	2.1	(130)	21.5	(450)	15
5	CLAY LINER	CU[C]	2.0	(125)	9.6	(200)	14
6	TAILINGS[A]	NONE	2.0	(125)	7.2	(150)	20
7	UNCLASSIFIED	CU	1.9	(120)	12.0	(250)	17

[A] SHEAR STRENGTH OBTAINED FOR TAILINGS AT EXXON'S NEARBY HIGHLAND MILL

[B] CONSOLIDATED-DRAINED SAMPLE

[C] CONSOLIDATED-UNDRAINED SAMPLE

TABLE II: RESULTS OF SLOPE STABILITY ANALYSIS FOR A TAILINGS PIT AT VARIOUS STAGES OF OPERATION

A. FAILURE SURFACES PASS THROUGH HIGHWALL AS WELL AS THROUGH LINER AND TAILINGS

SLOPE STEEPNESS		STAGE OF DEVELOPMENT	HORIZONTAL DIMENSION OF LINER (M)		HEIGHT OF FREEBOARD (M)	FACTORS OF SAFETY			
						STATIC		EARTHQUAKE[B]	
						LIQUEFACTION OF TAILINGS		LIQUEFACTION OF TAILINGS	
HWALL	CLAY LINER		TOP SURFACE	AT BASE OF TAILINGS		NO	YES	NO	YES
1	NA[A]	END OF EXCAVATION	NA	NA	NA	0.82	---	---	---
	1:1	START OF TAILINGS EMPLACEMENT	6.1	6.1	3.0	0.88	---	---	---
	1:1	PARTIAL POOL	6.1	6.1	6.1	1.09	---	---	---
	1:1	PARTIAL POOL	6.1	6.1	3.0	1.21	1.14	0.99	0.90
2	NA	END OF EXCAVATION	NA	NA	.NA	0.91	---	---	---
	1:1	START OF TAILINGS EMPLACEMENT	17.0	18.3	3.0	0.66	---	---	---
	1:1	PARTIAL POOL	12.1	18.3	3.0	1.14	1.07	0.92	0.93
MINIMUM ACCEPTABLE FACTORS OF SAFETY						1.30		1.00	

TABLE II (CONT.):

B. ASSUMED FAILURE SURFACES PASS THROUGH CLAY LINER AND TAILINGS BUT NOT THROUGH THE HIGHWALL.

SLOPE STEEPNESS		STAGE OF DEVELOPMENT	HORIZONTAL DIMENSION OF LINER (M)		HEIGHT OF FREEBOARD (M)	FACTORS OF SAFETY			
						STATIC		EARTHQUAKE	
						LIQUEFACTION OF TAILINGS		LIQUEFACTION OF TAILINGS	
HIGHWALL	CLAY LINER		AT TOP SURFACE	AT BASE OF TAILINGS		NO	YES	NO	YES
1:1[D]	1:1	ALL STAGES	6.1	6.1	3.0	0.86	---	---	---
			12.2	12.2	3.0	1.42	1.05	0.92	0.66
		FULL POOL	6.1	18.3	3.0	1.30	0.99	0.95	---
1:2[E]	1:1	PARTIAL POOL	12.2	18.3	6.1	1.04	---	---	---
		PARTIAL POOL	12.2	18.3	4.6	1.22	---	---	---
		PARTIAL POOL	12.2	18.3	3.7	1.44	1.19	1.22	0.94
		PARTIAL POOL	12.2	18.3	3.0	1.51	1.31	1.31	1.04

(A) NOT APPLICABLE

(B) HORIZONTAL ACCELERATION IS 8% OF GRAVITY (MAXIMUM CREDIBLE EARTHQUAKE FOR THE POWDER RIVER BASIN).

(C) ASSUMED COHESIVE STRENGTHS WERE GREATER BY A FACTOR OF 2 THAN AS INDICATED IN TABLE I.

(D) CRITICAL FAILURE SURFACES INCLUDE SEPARATION AT THE BOUNDARY BETWEEN HIGHWALL AND LINER.

(E) CRITICAL FAILURE SURFACES LIE ENTIRELY WITHIN CLAY LINER AND TAILINGS.

If the shear strengths of rock in the highwall (Table I) are accepted at face value, the slope stability analyses show the designs to be inadequate under all conditions ranging from an empty to partially filled pit based on Nuclear Regulatory Commission's minimum acceptable factors of safety of 1.3 and 1.0 for static and earthquake loading, respectively.[4] However, the results of the analyses conflict with practical experience at the Morton Ranch site where an open pit highwall stands on a 1:2 slope with no apparent evidence of massive slope failure. Murdock[5] states that this perplexing result is not uncommon when standard methods of slope stability analysis are applied to cut slopes. For example, the shear strength of Ft. Union strata in the highwall may have been underestimated. The factor of safety approaches unity for a 1:2 slope at the end of excavation, if cohesive strengths are assumed to be greater by a factor of two than those listed in Table I. Partially saturated strata above the water table may have considerably more strength (due to surface tension) than their saturated counterparts.[6] Such strength is difficult to quantify because it varies with the degree of saturation as well as particle size distribution and clay content. Furthermore, if shale units dry out too much, shrinkage cracks will weaken the strata along the highwall.

Lee, et al.[7] have studied the stability of highwalls in open pit coal mines in the western part of the Powder River Basin. They report much higher shear strengths for the sandstones and shales of the Ft. Union in their study area. They concluded, however, that fracture patterns (whether by drying out, stress release, or pre-existing) play a significant role in reducing the slope stability. Futhermore, they conclude that slope stability is time dependent.

Another possibility is that the analyses are correct but that the presence of massive slope failure is not always readily apparent. Slope failure may be of an insidious nature. Telltale bulges that appear in the floor of a pit may be removed or covered by earthmoving equipment while small escarpments remain unobserved a hundred meters or more behind the highwall.

The results of this analysis and the conclusions of Lee, et al.[7] cast some doubt on the stability of an open pit highwall over a period of years. If the pit can be backfilled over a reasonably short time frame, it is less likely that massive slope failure will become a problem.

Table II also lists the results of slope stability analysis for the clay liner based on the assumption that the critical failure surface may include the liner-highwall interface without passing through the highwall. Because of differential settlement of the clay liner with respect to the adjacent steep highwall, there will be very little shear strength developed at the liner-highwall interface (Robertson[8]). In the interest of producing conservative results, it was assumed that the shearing strength was zero along this interface.

The low shearing strength at the liner-highwall interface is the controlling factor in the design of a clay liner with an outer surface parallel to a 1:1 highwall slope. The minimum practical horizontal width (6.1 m, to allow for freedom of movement of compaction equipment) considered for the clay liner, combined with the minimum practical freeboard (3 m) provides an unacceptably low factor of safety (0.86 under a static load) against slope failure. An acceptable factor of safety can be obtained by doubling the width of the liner to 12.2m. Even then the factor of safety is slightly less than the lower limit for a maximum credible earthquake. The critical failure surface is a combination of plane failure along the liner-highwall interface and circular failure arc through the liner and tailings.

For the clay liner with a 1:1 slope constructed adjacent to a 1:2 highwall slope the critical failure surface does not pass near the liner-highwall interface. The steeper slope angle for the interface together with greater clay liner thickness results in acceptably high factors of safety for combinations of plane and circular arc failure surfaces.

In the 1:2 highwall slope case, the critical failure surfaces are arcs that lie completely within the clay liner and the tailings. Acceptable factors of safety are functions of construction and operational stages of development, the height of the freeboard between liner and tailings, and the physical state of the tailings. A partially filled pool (6.1 m of tailings) with a 3 m freeboard is stable against all failure modes including maximum credible earthquake and liquefaction of tailings. On the otherhand, if the freeboard in the above case is increased to 3.7 m liquefaction of the tailings will induce instability under both static and earthquake loading conditions. Maintenance of a partially dewatered sandy beach around the entire inside perimeter of the tailings impoundment would be a safeguard against liquefaction. At full pool, the clay liner is marginally stable (because of decreasing thickness) under a static load without liquefaction and an earthquake or liquefaction would destablize the liner. At full pool, however, the critical failure surface would pass through only the upper 1 to 2 meters of tailings. Thus slope failure of the clay liner at full pool would result in the exposure of a small percentage of the tailings fluid along the upper few meters of the highwall.

The above slope stability analysis suggests that a clay liner can be constructed on a steep highwall slope. Design details would vary depending on the highwall slope angle. Nevertheless, it is clear that: 1) the slope of the clay liner should not be greater than 1:1, 2) the thickness of the liner (or combination of liner and supporting shell with similar or greater strength) should exceed 6 m at the top of a 1:2 highwall slope or 12 m for a 1:1 highwall slope, 3) the height of the freeboard should be less than 4 m, and 4) a partially dewatered sand beach should encircle the inside surface of the clay liner. Furthermore, soil cement amy be required along the inside surface of the liner to add strength along the unconfined slope where it will be difficult to compact clay to design specifications. Specifically designed equipment may be required for compacting clay against the steep highwall and it may also be necessary to construct a filter between the liner and highwall in order to prevent piping failure (erosion of the clay liner by seepage). From the standpoint of stability it should not be necessary to install a drain along the outside of the embankment because sandstone exposed in the highwall will permit any seepage through the liner to drain away without building up pore pressure.

Modeling of Aquifer Contamination

In addition to assessing the structural stability of various waste pit designs, the effect of such designs on the possible contamination of an aquifer was analyzed. The mathematical model of the Morton Ranch waste pit is primarily based on hydraulic behavior of the pit and aquifer and is composed of two sections. The first assesses fluid transport from the waste pit to the aquifer, while the second section models dispersion of contaminants in the aquifer itself. The mathematical assumptions correlate with results from other studies of groundwater contamination.[8]

Waste Pit Drainage

The Morton Ranch waste pit may be approximated as a long rectangular box, its long axis parallel to the direction of groundwater flow. In the unlined pit design the wastes are to be placed above the water table. Drainage is modeled using a modified Huggins and Monke[8,9] equation for surface infiltration of ponded water:

$$DR = ER - R + f\,[1 - P/G]^3$$

where

DR = drainage rate

ER = average potential evaporation rate

R = average rainfall rate

f = steady state infiltration capacity

P = unsaturated pore volume

G = maximum gravitational water minus field capacity.

The value of f is assumed to be 1.25 cm/hr (0.5 inches/hr.), corresponding to a drainage rate typical of sandy soils.[10,11]

The clay lined pit model assumes that the hydraulic head in the pit is the main driving mechanism for drainage. The leakage rate decreases exponentially with the decline in head. The mass flux q(t) from the pit at time t is given by:

$$q(t) = C_i \, D_c \, H_i \, \exp\left[-\lambda_c t\right]$$

where C_i = initial waste concentration

$$D_c = \sum_{j=1}^{n} \frac{K_j A_j}{\ell_j}$$

n = number of sections of clay liner

K_j = hydraulic conductivity of section j

A_j = area of section j

ℓ_j = fluid path length through section j

H_i = initial hydraulic head

λ_c = D_c/(area over which head is dissipated)

If failure occurs in the clay liner, the model predicts that the level of resulting contamination would be between the model results for the lined and unlined pits.

Dispersion in the Aquifer

Fluid movement in the aquifer is assumed to conform to Darcy's law. Vertical averaging of the concentration allows simplification of the dispersion calculations by modeling the movement only in the x and y directions. The initial volume of the waste plume is set equal to the aquifer volume directly under the waste pit. This plume, assumed to be of uniform concentration, moves downgradient with time and disperses horizontally. The amount of dispersion, (calculated in both the x and y directions), is given by:

$$\sigma = (2\ ay)^{1/2}$$

where a is the dispersivity and y, the distance from the waste source centerline parallel to flow. The final concentration is calculated considering the amount of dilution occurring due to the increased plume size.

Several assumptions were used in modeling the drainage and dispersion of wastes: (1) The aquifer is homogeneous in its fluid flow properties. (2) Chemical interactions are not incorporated due to lack of quantitative data on the chemistry of the aquifer. (3) Seepage takes place only through 25% of the pit floor, where the sandstone aquifer is exposed. Numerical values of parameters assumed in this study are listed in Table III.

TABLE III: FLUID FLOW MODEL PARAMETERS[12,13]

PARAMETER	MAGNITUDE	
AVERAGE HORIZONTAL AREA OF PIT	$239 \times 10^3 M^2$	$(2.57 \times 10^6 FT^2)$
INITIAL HEIGHT OF TAILINGS ABOVE WATER TABLE	18 M	(60 FT)
INITIAL WEIGHT % WATER IN TAILINGS	34%	
ESTIMATED POROSITY OF TAILINGS AFTER EVAPORATION	30%	
HYDRAULIC CONDUCTIVITY		
WALL ABOVE WATER TABLE	$.421 \times 10^{-2}$ M/YR	$(.138 \times 10^{-1}$ FT/YR)
WALL BELOW WATER TABLE	$.2 \times 10^{-2}$ M/YR	$(.7 \times 10^{-2}$ FT/YR)
FLOOR OF PIT (CLAY ONLY)	$.2 \times 10^{-2}$ M/YR	$(.7 \times 10^{-2}$ FT/YR)
LONGITUDINAL DISPERSIVITY	21 M	(70 FT)
TRANSVERSE DISPERSIVITY	4.3 M	(14 FT)
POROSITY OF AQUIFER	23%	
AQUIFER THICKNESS	12 M	(40 FT)
POTENTIAL GROUNDWATER VELOCITY	3 M/YR	(9.8 FT/YR)
EVAPORATION RATE	1.07 M/YR	(42 IN/YR)
RAINFALL RATE	0.30 M/YR	(12 IN/YR)

In general, the numerical assumptions used in the model are in the conservative range of values. Since chemical interactions are not considered quantitatively in the fluid flow model, the results are conservative and must be considered in terms of the probable behavior of the ions in the aquifer. Orientation of the pit at some angle to the principal direction of groundwater flow would shorten the effect travel path across the pit, producing results similar to those shown in Figure 2. However, the assumption that the aquifer is homogeneous is more likely to produce results which are too low. The presence of structural features, such as buried stream channels, would act to confine the flow, reduce the total dispersion and result in less dilution.

Fluid Dispersion Results

Results were calculated for various clay liner thicknesses across the floor of the pit and permeabilities, and for the more generalized cases of flow and pit design. Table IV gives the calculated concentration of those waste constituents which at some point exceed drinking water standards.

Figures 2 and 3 show the effect of clay liner thickness on the ionic concentration in the aquifer, under various conditions of pit orientation and groundwater velocity.

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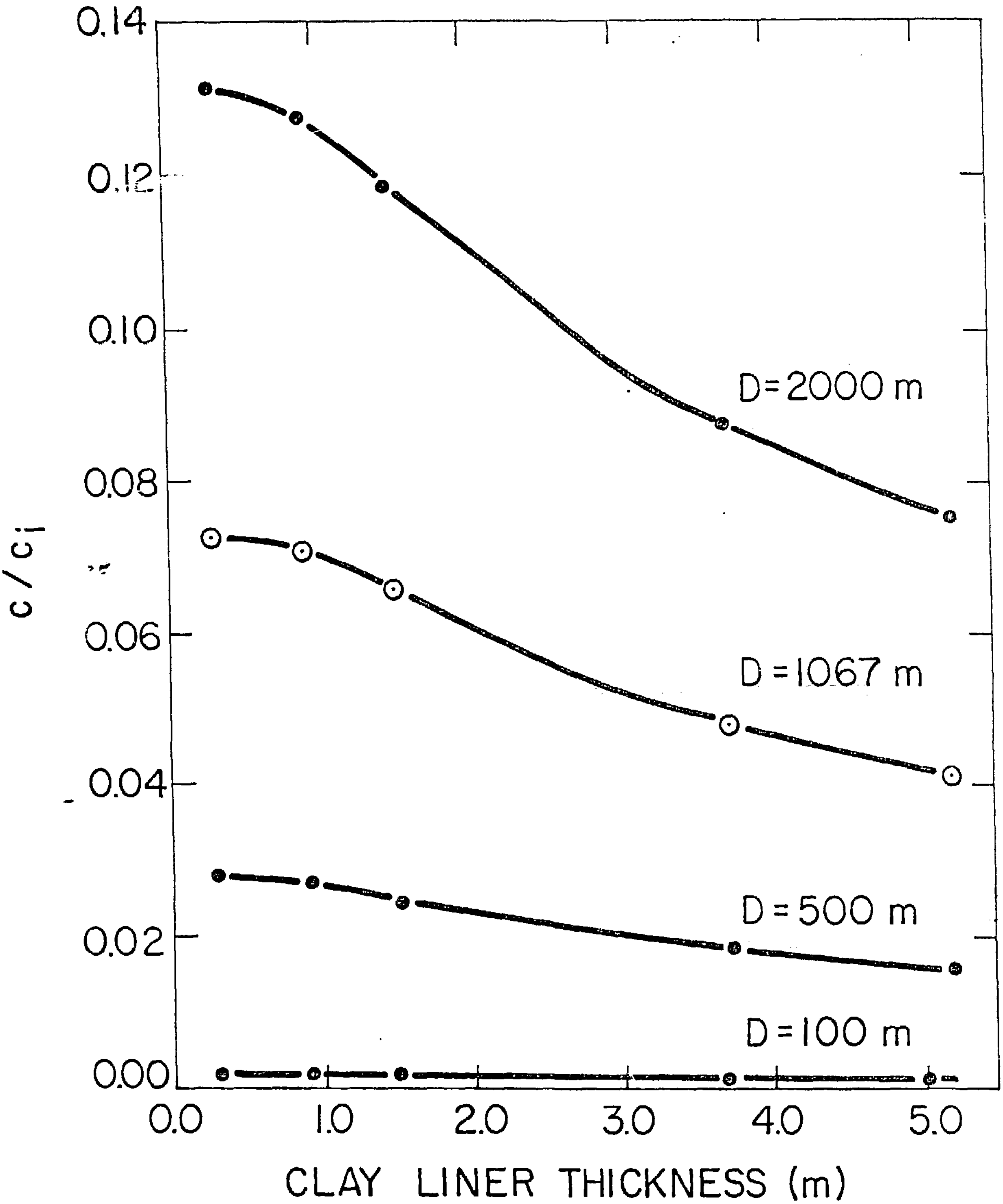

FIGURE 2: EFFECT OF LINER THICKNESS ON CONCENTRATION, ASSUMING VARIOUS ORIENTATIONS OF THE WASTE PIT RELATIVE TO GROUNDWATER FLOW (VOLUME = CONSTANT). D = LENGTH OF PIT PARALLEL TO FLOW, GROUNDWATER VELOCITY = 10 M/YR.

TABLE IV: CONCENTRATIONS OF SELECTED IONS UNDER VARIOUS WASTE PIT DESIGNS ASSUMING ONLY HYDRAULIC DISPERSION

DESIGN	INITIAL WASTE CONCENTRATION	UNLINED PIT	.9M THICK LINER	.9M LINER WITH SOIL CEMENT	9M LINER	EPA DRINKING WATER STANDARDS
DISTANCE FROM PIT	---	10,000 M	10,000 M	10,000 M	1000M	--
GENERAL CASE (C/C_I)	1	.127	.087	.043	.257	---
SO_4 (MG/L)	7350	936	638	316	1889	250
SE (MG/L)	.02	.003	.002	.0004	.005	.01
U (MG/L)	5.1	0.6	0.4	0.2	1.3	4.4
α (PCI/L)	84,300	10,731	7317	3625	21,665	15
β (PCI/L)	73,650	9,376	6393	3167	18,928	1000

*HYDRAULIC CONDUCTIVITY OF CLAY + SOIL CEMENT = $\frac{1}{100}$ CONDUCTIVITY OF CLAY

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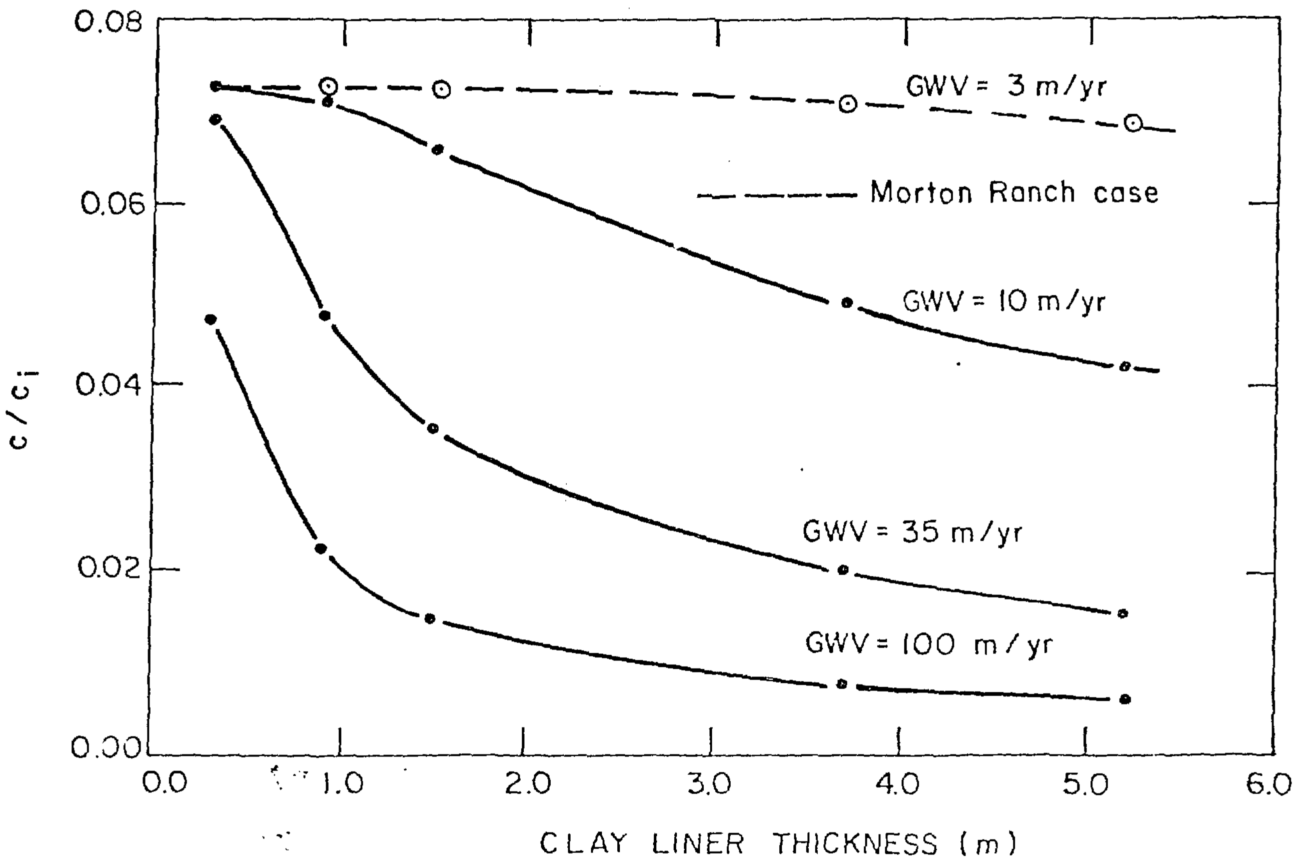

FIGURE 3: EFFECT OF CLAY LINER THICKNESS ON IONIC CONCENTRATION (C/C_I) IN THE AQUIFER WITH VARIOUS GROUNDWATER VELOCITIES (GWV). WASTE PIT = 1067 M X 224 M X 18 M. DISTANCE FROM PIT = 10000 M.

Since the drainage rate decreases exponentially, extending the draining time by increasing liner thickness has less effect at long time intervals. At Morton Ranch, the time span is long due to the low groundwater velocity and large pit, hence the increase in clay liner thickness has less effect than in the other theoretical cases studied.

The results indicated in the tables and figures must be considered in terms of the geochemistry of the aquifer-waste system. The aquifer at Morton Ranch is a clayey sandstone with low concentrations of carbonates and metal oxides. The initial pH of the wastes is 1.8, which would increase to an aquifer pH of approximately 6 to 8.[14] Characteristic values of Eh for shallow aquifers range from +200 to +400 mV[8] (oxidizing environment). Under these conditions, the following conclusions may be drawn:[15,16,17] (1) sulfate is likely to remain mobile due to the high solubility of its salts,[18] (2) Se will precipitate out as metallic Se^{o}, (3) U will be soluble in the +6 oxidation state, and (4) α and β emissions are likely to be reduced through removal of both ^{226}Ra (by precipitation as a sulfate) and ^{230}Th by precipitation (as hydroxide). Fixation by reaction with clay constituents may also occur. However, the ^{230}Th may not undergo fixation if complex formation occurs.[19] More field data is necessary to determine the extent of ionic migration and the magnitude of the Th problem.

Conclusions

Slope stability of the clay liner along the highwall can be achieved but only through careful design and construction and the use of a large volume of construction material. The liner and its protective shell should be at least 12 m thick, the freeboard should be no greater than 4 m, a filter may be required between the liner and highwall, soil cement may be required on the inside of the liner, and a partially dewatered tailings derived sand beach should completely encircle the inside of the clay liner.

Such a structure would be, in essence, an engineered embankment like an earthfill dam but without a "downstream" shell and a steeper "upstream" shell. The steep upstream shell is made possible only because the freeboard height would be carefully controlled. By contrast, a traditional embankment dam would be built in stages so that the freeboard would vary from 3 to 12 m, thus requiring a gentler slope.

Because the clay liner would completely encircle the tailings, it could be considered as a below grade form of the more traditional "ring-dike" method of tailings containment. The proposed below grade "ring-dike" would be an engineered embankment with a steep sloped inner shell. The rock behind the highwall makes the outer shell unnecessary.

In summary, the proposed construction along the highwall of an open pit is nothing more than an engineered embankment with two uniquely appropriate design features: 1) a steep sloped inner shell made possible by careful control of freeboard, and 2) the absence of an outer shell, its purpose being fulfilled by in-place rock behind the highwall.

In general, tailings disposal design is complicated by the effect of groundwater flow parameters, the large time span involved, the size of the waste pits being considered and the chemical characteristics of the waste. The effectiveness of various design changes, such as clay liner thickness, can only be determined if these factors are considered. In the Morton Ranch case, with large pit areas and low groundwater velocities, large increases in clay liner thicknesses are necessary to reduce the amount of leakage by any substantial amount. The alternative solution of reducing the permeability of the liner by addition of soil cement (assuming it will be acid resistant) allows for thinner liners. Such liners reduce potential leakage much more efficiently than uncemented liners of the same thickness or unlined pits with wastes placed above the water table.

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