1. INTRODUCTION

During the latter half of the 1970's the attention of the public, regulatory authorities and industry focused on the environmental impact of uranium and other mine wastes. This focus resulted in the intense evaluation of existing mine waste disposal methods and facilities and their impact on the environment. Additional regulatory control was implemented at both Federal and State levels. Research was undertaken by both State agencies and industry to develop alternative and better mine waste disposal methodology. While much of the initial focus was on uranium tailings, the results were generally found to be applicable to other ores and mine wastes.

The North American mining industry has changed dramatically in the last 4 years as a consequence of the depressed minerals market. This has resulted in the closure of many of the mines, requiring waste facility reclamation. Awareness of the difficulties of reclamation and of the long term environmental impacts has increased considerably. With the decline in the mining industry, both as an employer and in its economic contributions, there has been a commensurate decline in public and regulatory acceptance of detrimental environmental impacts. Consequently, the industry is forced with ever more stringent requirements, to limit the environmental impact of mine wastes, both during mining and following reclamation.

Also of considerable concern is the manner in which some of the regulations and requirements can be implemented to require the attainment of environmental protection standards which were not anticipated at the time of designing the facilities. The retroactive implementation of these standards, make reclamation much more difficult to achieve and considerably increases the costs. Industry must respond to existing regulations as well as anticipate future requirements, if it is to avoid remedial work and additional reclamation costs.
2.0 RECLAMATION AND LONG TERM CONCERNS

2.1 Types of Waste

Almost all wastes produced at a mining and milling operation can be divided into two classes.

Mine waste is that product which is mined but which is not processed before being placed on a waste dump.

Tailings is that product which, after mining and processing to remove the economic products, is discarded. Processing may range from simple mechanical sorting to crushing and grinding followed by physical or chemical processing.

Where simple physical processing is used, such as in placer or coal mining and processing, the tailings and mine wastes are often little different. However both products are removed from their original location, broken up and placed in piles in which the conditions of oxidation, seepage, leaching and erosion differ considerably from those at their original location. This considerably increases the potential for wind or water erosion of surface materials and their transport into the environment. The potential for oxidation and leaching is also increased with the result that dissolved contaminants may be carried away from the pile into the environment.

It is apparent that the concerns regarding potential environmental impacts are essentially similar for mine wastes and tailings. Indeed some mine wastes (from massive sulphide deposits for example) may have considerably greater pollution potential than some tailings (from gold placer mining for example).

Where the extraction process considerably alters the ore and where chemicals are added to liberate some of the contaminants in the ore, the potential for pollution migration is increased. If the tailings or mine wastes are hydraulically placed, then some of these contaminants may be in solution and the solutions are part of the mine waste product that must be prevented from entering the environment.

2.2 Visual Impacts and Landuse Constraints

Long term objectives for visual impact and landuse are the same for all mine wastes. It is desirable that following reclamation the waste facility blends into or is compatible with the surrounding terrain, and that the surface of the facility be capable of a land use equivalent to or better than the original surface. This usually requires the facility surface be stable, of gentle slope with positive drainage and with a topsoil cover layer.

2.3 Stability and Erosion

Dumps advanced by crest tipping are at a factor of safety of 1.0 on the front face. The stability of such dumps must be carefully evaluated and monitored during the operating phase of the mining. All other waste facility slopes are usually designed with substantial factors of safety and there is usually little risk of
failure. Section 3 of this paper discusses the design and monitoring of active dump slopes advanced by crest tipping.

The long term stability of waste facility slopes can decrease as a result of:

(i) increases in the groundwater table due to groundwater accumulation and due to changes in the permeability of the dump materials resulting from weathering and the inwashing of fines,

(ii) decreases in the dump material strength due to weathering.

Changes such as these were responsible for the tragic dump failure at Aberfan. Long term stability analyses must take into account the potential long term strength and phreatic surface changes.

Erosion can occur as a result of both wind (Schwendiman et al 1980) and water (Winters 1983). Design measures to limit erosion have been reviewed by Li et al (1983), Beedlow and Parker (1985), Beedlow (1984) and Nelson et al (1983). This is a major mechanism for long term dispersion of mine waste contaminants and has become an important consideration in the design of long term reclamation for waste dumps and tailings impoundments alike.

2.4 Leaching and Seepage

Leaching occurs in both waste dumps and tailings impoundments. Wastes with acid generating potential can produce highly contaminated seepage flows, which in the long term may be the most significant deleterious impact of the mine on the environment. Such leaching is proportional to the rate at which water can enter the waste deposit. Control measures include measures which limit infiltration. In areas with dryer climates, very low infiltration rates can be achieved with good surface drainage. In wetter areas, lower permeability surface covers help to reduce infiltration.

On a number of mining properties it has proved possible to prevent acid generation in the waste dump by including in the dump waste which is rich in calcium carbonate or other neutralizing minerals.

With modern facilities contaminated seepage is probably the single largest mechanism for the migration of contaminants to the environment. Methods for its control are discussed further in Section 4.

3 STABILITY OF WASTE DUMPS

3.1 Failure Mechanisms

The various failure modes that occur in mine waste embankments have been summarized by Caldwell and Moss (1984) who review the methods of analysis. These failure modes are illustrated in Figure 1.
SURFACE OR EDGE SLIDE  

SHALLOW FLOW SLIDES  

BASE FAILURE (SPREADING)  

BLOCK TRANSLATION  

CIRCULAR ARC FAILURE  

FOUNDATION CIRCULAR FAILURE  

FIG. 1 MINE WASTE EMBANKMENTS  
POSSIBLE FAILURE MODES  
(AFTER CALDWELL AND MOSS, 1981)
Surface or edge slides may occur as material moves down the slope. This mode of failure is most likely to occur in crest tipped embankments and is best evaluated by the equations describing the stability of an infinite slope. If sufficient water enters the slope and flows parallel to the face, a shallow flow slide may occur.

Dumps placed on flat ground of competent soil are least likely to fail. However, if the flat ground is covered by a thin layer of weak material, base failure may occur. If the ground is inclined, base failure is more likely to occur. This mode of failure has been experienced in both end-dumped and layer placed embankments.

Block translation can occur where a dump is formed on inclined ground and the soil cover is relatively thin and weak. Unusually high water tables in the embankment, earthquakes or the decay of organic material beneath the dump may start such a failure.

Circular arc failure through the dump material is most common where the dump material contains a significant percentage of fine grain soil. Similarly, a circular arc failure surface may develop through a deep foundation soil deposit of fine grained soils.

### 3.2 Deformations as an Indication of Failure

Deformations occur in a slope as a result of stresses and shear displacements in the mass of material forming the slope. Some of these deformations, such as consolidation, are not indicative of failure while others, such as shear displacement along the failure surface, are. To predict failure it is necessary to distinguish deformations which indicate failure from those which do not. This requires an understanding of the failure mode and the deformations that accompany it.

Analytical techniques which are currently available for the analysis of dump deformations are either cumbersome and expensive to use (McCarter 1981) or are not sufficiently accurate to enable pre-construction estimates to be made of the consolidation and failure deformations. Instead, failure criteria are usually based on experienced gained as the dump is constructed. The rate of deformation, and a change of the rate of deformation are generally good indicators of the behavior of a slope. They may be used to establish criteria indicative of failure.

The slices method of limiting equilibrium analysis may be used to obtain an estimate of the stresses which occur along a failure surface and to determine which portion of the failure surface is in failure. Knowing the stresses and the zones that are in failure, the nature of the deformations that will result can be inferred. This inference may be used to design a monitoring system and to interpret the results of deformation monitoring. The $y = 0$ method of analysis is used to illustrate the approach.

### 3.3 $y = 0$ Analysis

The $y = 0$ method of analysis [Robertson (1977), Caldwell and Moss (1981)] is explained with reference to the slope and potential failure surface shown in Figure 2. If a crack were to form along
FIG. 2  P FORCE DISTRIBUTION BY THE Y = 0 ANALYSIS METHOD
DE there would be no forces effective across this boundary and the factor of safety against sliding of the failing segment can be calculated as follows:

Take moments about the center of the slip circle:

\[ F = \frac{\xi r.FR_i}{\xi r.DF_i} = \frac{\xi RF_i}{\xi DF_i} \]  

(1)

where RF is the maximum resistance to sliding that can be developed along the sliding surface and DF is the force tending to produce sliding. It is convenient to define a force P such that:

\[ \xi RF_i = \xi DR_i + P \]  

(2)

Substituting Eq (2) in Eq (1)

\[ F = \frac{\xi RF_i}{\xi DF_i} (\xi DF_i + P) + P = 1 + \frac{P}{\xi DF_i} \]  

(3)

If P is positive, F > 1.0 and the failing segment will not slide while if P is negative, F < 1.0 and the slope has failed. If a force P is applied to the failing segment then from Eq (2) \( \xi RF_i = \xi DF_i \) and it follows that the forces along the failure surface will be such that F=1.0. This P may be visualized as that external force which must be applied to bring the failure surface to the limit of failure.

Equations for the solution of DF and RF can be developed using all the common slices methods of analysis (Bishop 1955, Morgenstern & Price 1965). Some solutions are presented in Robertson (1977) and Caldwell and Moss (1981).

The value of P on the interslice boundaries has been plotted on the lower graph as has F, the factor of safety of the failing segment. The failing segment is that portion of the slope above the failure surface to the left of the slice boundary under consideration. Between A and F along the failure surface, P is increasing; this indicates that the first two slices on the left have a reserve of strength on the failure surface and require a positive P force to be applied to make them fail. Between F and E the slope of the P force curve is negative; this indicates that each slice has a strength deficit along its base. At point E the P force is zero, hence F = 1.0; this implies that if a tension crack forms between D and E the slope would just fail.

The inference is that the zone between F and E is in failure and is supported by the reserve strength between A and F. Substantial shear movements are expected between F and E, and tension cracks will form in the vicinity of D. A bulge may form in the slope above A due to the pressure from the failed zone. The surface between B and D is expected to have a vector of movement down and out parallel to E G due to shear movement.
3.4 Deformations Preceding Failure

To illustrate the approach, the slope and failure surface shown in Figure 3 (a) is investigated. The slope material has both a cohesion and a friction strength.
For a slope height, $H = 30\text{m}$, the variations of the $P$ force and factor of safety are shown in Figure 3(b). Since the slope of the $P$ force curve is positive along its entire length, all parts of the potential failure surface have a reserve of strength. No shear displacements are anticipated on the failure surface. If this is true of all potential failure surfaces through the slope, then no substantial failure deformation is anticipated at the surface of the slope.

For a slope height, $H = 45\text{m}$, the variations of $P$ and $F$ are shown in Figure 3(c). The negative slope of the $P$ curve between A and B on the potential failure surface indicates a strength deficit at the base of each slice. These slices must "slide" down the failure surface and push against the toe slices located between E and C, which have a reserve of strength.

Thus, as the slope is excavated from $30\text{m}$ depth to $45\text{m}$ depth, movements may be anticipated between A and B; a tension crack may develop in the vicinity of B. The largest movements should occur just to the left of B. Since the reserve of strength in the toe region is sufficient to support the slope, it will not fail, although some toe bulging may be observed.

If excavation proceeds to a depth of $60\text{m}$, the resulting $P$ and $F$ curves are shown in 3(d). The strength deficit under the slices between C and D is greater than the reserve that exists between E and C. $F$ is less than 1.0 and the slope will fail before a depth of $60\text{m}$ is reached. The largest movements can be expected to the left of the crack position, 'D'. Little shear movement is expected in the toe region until complete slope failure actually takes place.

This example serves to illustrate how and where surface deformations may be anticipated in a slope which is cut in material possessing both a cohesion and friction strength.

Mine waste dumps are usually constructed by advancing a dump of cohesionless material over a foundation material. "Over the crest" dumping techniques are normally used. The most common form of failure of high waste dumps is base spreading on a relatively shallow foundation soils layer overlying bedrock. This failure mode has been investigated in the field and the laboratory by Blight (1981) and is illustrated in Figure 4.
(A) Active and passive wedges in first time failure on flat foundation

(B) Active and passive wedges developed in failure on sloping foundation

FIG. 4 Base failure modes as demonstrated by laboratory models (After Blight, 1981)
This failure mode can be analyzed by the \( y = 0 \) slices method. An example is shown in Figure 5. The analysis investigates what

**FIG. 5** CHANGE IN P FORCE AND FACTOR OF SAFETY DISTRIBUTIONS WITH DECREASE IN FOUNDATION STRENGTH OR INCREASE IN BASE ANGLE
happens if the strength of the foundation layer is reduced or if the slope of the foundation layer is increased. Figures 5 (a) to (c) show the variation of $P$ and $F$ along the failure surface $CBA$ for three different cases. In each case there is a deficit of strength along that portion of the failure surface which occurs in dump material, i.e. along $AB$. The deficit is the same in all three cases and amounts to $p$. For the first two cases 5 (b) and (c) there is a sufficient reserve of strength in the toe region, $CB$, to hold-in the mass of the dump. In 5 (d) this reserve is insufficient and the toe will kick out. The dump will fail by base spreading. In practice different values of $p$ must be investigated to determine the location of the potential failure surface.

The critical failure surface usually intersects the crest point. Breakback behind the crest is often a secondary effect which occurs after the first failure. Large movements of the crest and slope face precede bulging and kicking out of the toe. These movements are often masked by further dumping.

3.5 Considerations in Dump Monitoring

1. The $y = 0$ method of analysis may be used to illustrate the development of progressive failure (Robertson 1977) and this shows that failure starts in the zone below the crest. Failure (of the type illustrated) progresses towards the toe which fails just before general slope failure. Deformations at the top of the slope therefore occur during the entire process of failure surface development while those at the toe are most pronounced just before failure occurs. This implies a longer period of warning from monitoring at the crest than the toe but with more "noise" and a less precise indication of the ultimate failure. The warning signal at the toe is more distinct but may occur too late to be of value.

2. The mobilization of failure conditions on a portion of the potential failure surface does not necessarily imply dump failure. Thus large deformations can occur at the crest with perfectly satisfactory dump performance. The same cannot be said for the toe area.

3. Deformations at the dump crest due to settlement and consolidation can, to some degree, be separated from shear deformations as a result of the greater amount of horizontal movement associated with the latter.

4. Consolidation settlement usually decreases with time. Where vertical deformations continue at the same rate, or accelerate, after dumping is stopped progressive failure is indicated.

5. Stability analyses often indicate a failure surface which intersects the slope at the crest line. Dump failures of this type are frequently observed. Monitoring stations must be located at the crest or sometimes on the front slope. Monitoring stations located back from the crest are of little value if the failure surface is likely to intersect the crest.

6. The amount of movement that is likely to occur before failure determines the sensitivity of the monitoring equipment required. Movement varies with the type of dump material, the dump height and the location at which monitoring will be done. Stiff cut rock slopes may fail after a few centimeters of movement and sensitivity to tenths of a centimeter, or less, is required.
Experience with waste dumps, where total movement may be measured in meters, suggests that settlement sensitivities of the order of 2.5 cm are usually adequate at the crest. McCarter (1981) reports sensitivities he has found suitable for other forms of measurement.

3.6 Methods of Deformation Monitoring

Current monitoring techniques have been summarized by McCarter (1981). These include:

(i) On-site inspections
(ii) Surveying
   a. leveling
   b. electronic distance measurement
   c. location by intersection
(iii) Photogrammetry
(iv) Extensometers
   a. above surface
   b. buried
(v) Inclinometers
   a. surface or buried tiltmeters
   b. borehole inclinometers
(vi) Acoustic emission
(vii) Laser beacon
(viii) Settlement cells

The suitability of the different techniques is dependent on the material, height and construction method used for the dump. A number of monitoring system applications are illustrated in Figure 6.
FIG. 6 EXAMPLES OF SOME MONITORING SYSTEM APPLICATIONS
(TAKEN FROM McCARTER, 1981)
4.1 Subsurface Contaminant Migration

Subsurface releases of contaminants occur when soluble contaminants migrate downwards and laterally from the impoundment by advective flow dispersion and diffusion. Advective flow occurs when dissolved solids travel at the same velocity as the water which is transporting it. Diffusion occurs as a result of movements of contaminant molecules independent of any advective movement of groundwater. Dispersion occurs in part by molecular diffusion and in part by variation in velocity within the porous medium. Thus, a concentrated release at a point will tend to disperse along the flow path.

Chemical effects along the flow path may temporarily prevent molecules of contaminants from migrating at the advective velocity, resulting in a retardation of the contaminant flow. Thus, both physical and chemical controls may be used to contain contaminants.

4.2 Physical Containment

The quantity of flow, Q, carrying contaminants along a flow path can be determined from Darcy's Law:

\[ Q = K_i A \]

where:

- \( K \) is the hydraulic conductivity of the porous medium in which flow is occurring,
- \( i \) is the hydraulic gradient along the flow path, and
- \( A \) is the area of the flow path.

To reduce the quantity of contaminated flow from an impoundment, any one, or a combination, of \( K, i, \) or \( A \) can be reduced, by installing appropriate design elements. It is often possible to achieve sufficient reduction in a single quantity to reduce \( Q \) to acceptably low values. An example is the installation of a synthetic liner with a sufficiently low \( K \) value. Often it is preferable, for either long-term security or economic considerations, to select a combination of design elements, which together provide the required reduction, possibly with some redundancy. For waste dumps and many tailings impoundments it is preferable to install the seepage control measures in a manner which prevents water entry into the deposit and therefore the initial mobilization of the contaminants. In the long term all water entering a deposit must escape either as seepage or discharge from the deposit. Prevention of water entry may be more effective than trying to limit water exit. Some of the alternative design concepts are mentioned below.
4.2.1 Design Concepts Reduction $k$

(i) Liners (or covers)

Liners can be one of a number of types:

(a) Natural geologic liners occur when the hydraulic conductivity of the foundation soils or rocks are sufficiently low that $Q$ is reduced to an acceptable value. Generally mass permeability values of less that $1 \times 10^{-6}$ cm/sec are required. Fractures in, and heterogeneity of, the foundation soils are the main concerns.

(b) Clay or low permeability soil liners may be used to form a core in the embankment, to line the impoundment basin or cover the dump or deposit. Admixtures of bentonite (Kozicki 1985) or other low hydraulic conductivity, natural materials additives may also be considered. Some of the design, construction and field testing criteria are reviewed by vág Zyl (1983). Hydraulic conductivity values of less than $1 \times 10^{-7}$ cm/sec are required. Such liner materials must be checked to ensure that they are not degraded by the waste materials. A number of papers describe suitable evaluation techniques for acid and alkaline wastes (Relva and Martin, 1982) and for organic chemicals (Daniel, 1985). Clay layers used as cover sealers are sensitive to the deformations of the underlying tailings or wastes. Excessive deformations may result in cracking.

(c) Flexible membrane liners may be divided into two common types:

- flexible polymetric membranes;
- spray applied asphaltic liners.

Both of these membrane types can be used to line impoundments of complex geometric shape. A review of liner types available in North America, their characteristics and installation requirements, has been undertaken by Firlotte and Gould (1984) as part of CANMET's National Uranium Tailings Program. They conclude that, of the membranes reviewed, high density polyethylene (HDPE), Hypalon (CSPE), and chlorinated polyethylene (CPE) all exhibit sufficiently good characteristics to warrant further study, as do catalytic airblown asphalt or asphaltic-elastomeric compounds. While the membrane materials have extremely low permeability (probably less than $1 \times 10^{-11}$ cm/sec) the operative permeability, resulting from liner defects, which are typically used are $1 \times 10^{-8}$ and $1 \times 10^{-10}$ cm/sec respectively for asphaltic and polymeric liners respectively.

It is questionable how well these assumed operating values reflect actual field installations, as testing procedures for large expanses of liners are imprecise. A number of liner failures have been reported, often the result of manufacturing or installation errors. Such liners are very thin and the long-term degradation remains a prime concern, particularly in the chemical and radiation environments of some tailings solutions (refer Mitchell D.H. 1984), the harsh climatic conditions of some northern properties and when subject to deformations associated with foundation settlements. They are usually not considered suitable as a cover material as they can be disrupted by burrowing animals. Experience
indicates that membrane liners can be a practical and effective solution in the short-term. Long-term suitability remains to be demonstrated.

Membrane liners can be effectively combined with other liners such as clay liners or natural geologic liners. The second liner limits releases from membrane liners imperfections and remains as a durable long-term liner. This combination may be particularly effective where the contaminant concentration and/or the seepage driving forces reduce with time.

(ii) Reducing the Hydraulic Conductivity of the Tailings or Waste

Tailings often are of relatively low hydraulic conductivity themselves. Their hydraulic conductivity depends on the grading of the tailings and the tailings density. The slimes zones may have very low hydraulic conductivity \(1 \times 10^{-8}\) cm/sec while sand beach zones may have relatively high hydraulic conductivity \(1 \times 10^{-4}\) cm/sec. If segregation had been prevented, a typical value might be \(1 \times 10^{-6}\) cm/sec, with a range of \(1 \times 10^{-7}\) to \(1 \times 10^{-4}\) depending on the tailings density. When the tailings are discharged in a slurry they have a low density. Under load (of incumbent tailings or cover materials), and when drained, they consolidate to a denser state. Thus the hydraulic conductivity can reduce with time to levels which are typical of natural liner materials. The low hydraulic conductivity of the slimes zones and of the densified tailings can be used to advantage for controlling seepage losses from the tailings impoundment in the long-term. It requires that appropriate tailings placement techniques be used in a manner which achieves a partially saturated deposit as discussed in the next section. Air bubbles in the tailings pores impede seepage flow.

Reduced dump surface permeabilities can be achieved by selectively placing more clayey materials as a final surface layer and by compacting this upper surface layer.

4.2.2 Design Concepts Reducing i

The hydraulic gradient, i, is determined by the head loss, h, and the flow path length, L.

\[ i = \frac{h}{L} \]  

(5)

Reductions in i may be brought about by both a decrease in h or an increase in L. For a synthetic liner, L is usually very small, resulting in an extremely high value of i over the liner thickness. For natural geologic liners L is considerably greater and for this reason lower values of k are acceptable for natural geologic liners than for synthetic membranes.

(i) Underdrains

The value of h applied to a liner can be drastically reduced by installing an underdrain over the liner. If water reaching this drain is drained away, then the hydraulic head on, and consequently the seepage through, the liner may be small. Some design, construction and management considerations for the installation of underdrains are given by van Zyl and Robertson (1980). The
installation of underdrains also has the effect of increasing the consolidation in the tailings pile and therefore decreasing its hydraulic conductivity and of reducing the total volume of water (and dissolved contaminants) available for seepage. Hydraulic heads effective on cover layers are typically much less than on liners. Thus preventing inflows is often easier to achieve than preventing seepage outflows.

(ii) Partially saturated layered tailings

A partially saturated, low segregation, high density tailings deposit can be obtained by a combination of layered tailings placement and underdrainage. Air drying of the layers of tailings induces suction pressures in the tailings resulting in partial saturation, increased tailings densities and reduced hydraulic conductivity. If thickened tailings discharges are used to place uniform layers of tailings, then the degree of tailings segregation can be minimized. The semi-dry or sub-aerial technique of tailings placement has been employed in other countries for many years (Robertson et al, 1978). The technique is well suited to dry hot climates such as the southwestern United States. Because of the extreme climatic conditions at some northern mines and limited evaporation or high rainfall in other areas, the potential benefits of this technique may be considerably reduced. Concerns exist regarding the effects of ice lensing in the deposits. This technique has been used at Key lake and its application is described by Knight and Haile (1983). Operating experience is as yet limited. It is anticipated that a considerably reduced piezometric head, i, will be imposed on the impoundment liner at the time of abandonment.

(iii) Dry tailings

By filtering the tailings in the mill, most of the tailings solution (and the contained dissolved contaminants) can be removed, prior to placing the tailings in the impoundment. If the entry of additional surface water can be prevented, then the free water that is available for seepage is small. This effectively reduces h on any "liner" and the rate of contaminated seepage to the maximum rate at which the residual water can drain from the tailings. In practice, some infiltration will occur at the tailings surface; the rate will depend on a variety of factors relating to the cover, surface drainage and weather. Placement of "dry" tailings also results in a non-segregated tailings of relatively high density and consequently relatively low permeability.

The filterability of tailings varies greatly, dependent largely on the percentage and nature of the clay fraction. Pre-conditioning of the tailings (by floculants and other agents) can considerably improve their amenability to filtration. Different filtration equipment, including belt filters, belt presses, filter presses, centrifuges and cyclones must be evaluated to establish the most suitable. Filter cakes of differing moisture contents will be obtained, and the cost of filtration can vary enormously for the different equipment types and tailings. The handleability, trafficability and stability of such "dry" tailings will vary according to the moisture content. These characteristics should be evaluated before selecting the filtering equipment, transportation
and placement equipment or designing the impoundment management system. A review of the methods and production of dry tailings is provided by Robertson et al., 1982. Successful "dry" tailings systems have been operating for a number of years, at the Miniere Dong-Trieu Mine in France and at the Barton Mine in New York State. Dry tailings disposal is being tried and evaluated in the new in-pit tailings disposal system at Rabbit Lake Mine for the Collins Bay B-Zone orebody (Gulf Minerals Canada Ltd. 1981).

(iv) Hydraulic balancing

Hydraulic balancing is achieved by maintaining an essentially zero head difference between the piezometric head in the tailings and in the surrounding receiving waters. Since there is no head difference, $h$ and consequently $i$ is zero. No advective flow occurs and the mechanism for contaminant migration is diffusion. This principle is substantially achieved by placing tailings below water level in a natural or artificial lake. To prevent pressure effects due to groundwater flux the tailings should be surrounded by an envelope of high hydraulic conductivity. This principle was initially recognized and proposed as a containment measure by Gulf Minerals Canada Ltd. (1981) for the Rabbit Lake mine, Collins Bay B-Zone orebody tailings. Since flooding will only occur on impoundment decommissioning the results have yet to be demonstrated. Evaluations of contaminant migration from the uranium tailings at the Beaverlodge Mine (Eldorado Resources, 1983) which were placed largely in Fookes Lake, provides considerable confidence in the effectiveness of this concept.

4.2.3 Design Concepts Reducing $A$

For a given tonnage of waste, only three factors influence the seepage area: the geometry of the waste impoundment; the volume of water stored in the waste impoundment; and, the density of the waste.

(i) Geometry of the waste impoundment. Generally speaking, the greater the average depth of the waste, the lower the area covered by it. Both the seepage area and the area of environmental impact is decreased.

(ii) Volume of water stored. Tailings impoundments are often used to store excess process water. Minimizing the additional stored water, minimizes the volume of waste product in the tailings dam and consequently both the seepage area $A$ and hydraulic head, $h$.

(iii) Increased density reduces the total volume of waste, hence $A$, as well as decreasing the hydraulic conductivity of the waste and the total volume of solutions available for seepage. Waste placement and management techniques which result in increased tailings density have multiple beneficial effects.
4.3 Chemical Containment

Concepts for the prevention of pollution from mine waste by chemical means may be divided into two groups:

(i) Chemical pre-conditioning of the waste;
(ii) Hydrogeochemical attenuation along the seepage flow path.

4.3.1 Chemical pre-conditioning

Chemical pre-conditioning of tailings may be broadly grouped into three:

(i) neutralization of acid leachate;
(ii) fixation of solids;
(iii) specific constituent removal.

A review has been made of these tailings treatment techniques for uranium mill wastes by Sherwood and Serne (1983). Acidic solutions tend to be high in dissolved heavy metals and other pollutants. Over much of the U.S.A. deep residual soils with a high neutralizing capability exist which can be very effective in limiting the migration of contaminants. The northern Canadian environment tends to have limited buffering capacity to absorb acid seepage, with the result that acid seepage tends to persist and contaminants may be carried a substantial distance.

Neutralization, while not common in the U.S.A., is a requirement in Canada. Where the tailings have acid generation potential, the addition of excess lime may be required for long-term pH control. Neutralizing agents may consist of limestone, lime, soda ash, caustic soda or a combination of these. The advantages and disadvantages of these have been summarized by Sherwood and Serne (1983).

Fixation processes are those treatment processes that produce a physically stable, leach-resistant material from the tailings, sludges and slurries. Four stabilization processes have been tested on Canadian uranium tailings (Ritcey et al. 1982). These include lime, cement and asphalt based agents sold under a number of trade names. It was concluded that numerous disadvantages exist and, if used at all, they would be limited to the most toxic wastes.

A large variety of processes exist for the selective removal of specific constituents. An example of effective selective detoxification of tailings is the now fairly widely practiced techniques of cyanide destruction in cyanide gold tailings. These processes are highly ore and constituent specific in their technical and cost effectiveness. It is occasionally cost effective to modify the extraction process to avoid introducing non-desirable reagents or to remove one or more specific contaminants. General detoxification (both radiologically and chemically) of the tailings is not yet feasible. Continued research will add to our capability and options in this area.
4.3.2 Hydrogeochemical Attenuation

Hydrogeochemical attenuation of the dissolved solids in the seepage water occurs along the flow path. These processes occur both in the waste impoundment (Dave et al., 1981, and Feenstra et al., 1981) and in the flow path from the (Taylor and Antomaria, 1978). The radioactive constituents are highly reactive and tend to be immobilized within a relatively short distance. The anions such as chlorides and sulphates tend to be least reactive and tend to be carried quicker and further in the pollution plume. These persistent contaminants are often useful precursors which may be used to evaluate and predict the growth of any contaminant plume.

Hydrogeochemical attenuation may often be relied upon to prevent the movement of specific contaminants beyond finite boundaries. The techniques for predictive modelling are reasonably well developed as described by Bush and Markos (1984).

The deep residual soils found in many parts of the U.S.A. usually have a high retardation capacity. In northern Canada the surficial soils and rocks, are relatively little weathered and have a generally low chemical buffering capacity. They often provide relatively low retardation to contaminant migration. It may be possible to enhance this retardation by installing a layer or barrier which is geochemically more effective. Such an application is described by Opitz and Sherwood (1984).
REFERENCES


