SUBSURFACE DRAINAGE OF TAILINGS IMPOUNDMENTS: SOME DESIGN, CONSTRUCTION AND MANAGEMENT CONSIDERATIONS

by
Dirk van Zyl1 and A. MacG. Robertson2

ABSTRACT

Subsurface drainage of tailings impoundments to improve stability, reduce hydraulic head on liners, improve water return to mill and stabilize tailings is getting more attention lately. This paper deals with a number of the design, construction and tailings management considerations to establish effective subsurface drainage systems for tailings impoundments. The design considerations include capacity calculations of subsurface drainage systems, spacing of outlets, spacing of parallel drains, use of geotextiles and alternative drainage systems specifically for tailings impoundments. Poor control during construction of drains and during first filling with tailings usually leads to the loss of dewatering capacity, and even complete clogging of drains. Some practical considerations are discussed, such as the influence of bottom surface slope of tailings impoundment, depositional procedures to minimize drain clogging and installation of subsequent drains at higher levels in the tailings impoundment.

INTRODUCTION

Subsurface drainage for the improvement of tailings impoundment stability has been employed for some time now. Due to stringent regulatory requirements on the uranium industry to limit seepage releases through liners and to cover the tailings impoundment upon decommissioning, considerable attention is being paid lately to the use of subsurface drainage. Although a number of papers on subsurface drainage of uranium tailings impoundments have been published lately, e.g., Robertson, et.al. (1978), Staub (1978) and Charlie and Martin (1980), the technology is not new. The report by the National Building Research Institute (1959) on gold tailings impoundments in South Africa recommended under drains, with drainage material designed according to filter criteria, and perforated outlet pipes. Numerous other publications have been published since 1959 dealing with the various aspects of subsurface drainage of tailings impoundments, e.g., Coates and Yu (1977); Soderberg and Busch (1977); Klohn (1979) and Gray and Somogyi (1978).

Experience gained in the design, construction and management of tailings impoundments during the last 20 years has led to a better appreciation of the difficulties involved with the design and construction of subsurface drainage. The most important single factor

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in the design of subsurface drainage for tailings impoundments is an appreciation of the depositional techniques to be used for constructing the impoundment and its influence on the successful operation of the selected drainage system. This controls the position of the drain, the material selected for its construction and the control during the first coverage of the drain with tailings. Although there are some similarities between the drainage systems used in water retaining structures and those used in tailings impoundments, some very important differences exist. It is not always possible to select and control construction of the material overlying subsurface drains in tailings impoundments. This selection and control can at best be done by cycloning the tailings material in the direct vicinity of the drains. However, in most instances, natural sedimentation must be relied on for construction. Information on changes in the tailings material characteristics with time, e.g., a finer grind, is usually not available at the time of the subsurface drainage design. Monitoring of the pore pressure in tailings impoundments with piezometers throughout the life of the project is an important part of successful long-term subsurface drainage.

This paper will discuss some of the design, construction and management considerations for establishing successful long-term subsurface drainage for tailings impoundments. The special considerations for uranium tailings impoundments are highlighted.

**WHY SUBSURFACE DRAINAGE?**

Subsurface drainage of tailings impoundments is important for the following reasons (Robertson, et al., 1978; Staub, 1978; Charlie and Martin, 1980).

(i) The stability of a tailings impoundment is improved by lowering the phreatic surface through subsurface drainage. A larger "shell" of unsaturated material is formed which improves the static and seismic stability of the impoundment.

(ii) The water return to the plant can be improved by subsurface drainage. A large part of the tailings impoundment is kept unsaturated.

(iii) The reduced water content of an underdrained impoundment will reduce movement of tailings material if a failure of the embankment should occur. A combination of a well-drained structure and the storage of as little water as possible on the impoundment surface during operations will prevent a failure from being as catastrophic as the Bafokeng failure (Jennings, 1979).

(iv) Subsurface drainage systems placed above the liner of lined impoundments, such as uranium tailings impoundments, reduces the hydraulic head on the liner, thereby reducing the quantity of seepage through the liner.
(v) By reducing the moisture content of the tailings during operation, long-term seepage from the impoundment is reduced. The overall quantity of contaminated seepage released from the impoundment is also reduced. Groundwater impact is therefore limited.

(vi) The stabilization of the tailings during operations through dewatering will reduce the settlement of the impoundment surface after reclamation. Small differential settlements of the reclamation cap will have less influence on its integrity than large differential settlements.

DESIGN ASPECTS

For the purposes of this discussion, two different sets of seepage boundary conditions will be considered:

(i) Flow from a pool to a blanket drain.

(ii) Flow into parallel drains constructed on a low permeability liner.

These two conditions are shown schematically in Figures 1 and 2.

The successful long-term operation of subsurface drains is dependent on an adequate capacity to remove all the water entering the drains. Furthermore, the drainage materials used in constructing the drains should be free-draining to secure complete drawdown of the phreatic surface, especially in the case of parallel drains. The magnitude of \( t \) (see Figure 2) is determined by the drawdown realized at the drains.

The following quantities must be calculated to design the subsurface drainage system:

- \( q \) - the discharge rate per unit length of drain.
- \( L_{dr} \) - the effective drain length (see Figure 1) which will determine the width of the drainage blanket.

Blanket Drains

The calculation of seepage conditions through tailings impoundments was described by van Zyl and Harr (1977). Special attention was given to closed form solutions.

The selection of seepage boundary conditions for the estimation of \( q \) and the position of the phreatic surface is usually a very difficult aspect of the design. The spatial distribution of the coefficient of permeability in the tailings and foundation can only be assumed in the design stages. Even the very best estimates of the coefficients of permeability from field and/or laboratory measurements are usually only within one order of magnitude. It is usually assumed that the coefficient of permeability of tailings in an impoundment
decreases with increasing distance from the discharge point. This is especially prevalent where spigot discharge is used. This effect is discussed by Kealy & Busch (1971) and Abadjiev (1976).* 

Closed form solutions for the calculation of seepage conditions, though limited in application, are much easier and less expensive to use than numerical solutions such as finite element or finite difference techniques. Furthermore, if the seepage boundary conditions have to be assumed, a strong argument can be made for the use of closed form solutions, at least in the initial stages of design. Closed form solutions are available for the analysis of homogeneous, isotropic, steady state seepage through a variety of sections on impermeable foundations (e.g., see van Zyl & Harr, 1977). The approach by Abadjiev (1976) (see expressions in Appendix) consider a specific set of non-homogeneous, isotropic steady state conditions.

By pleading steady state seepage through a homogeneous, isotropic section on an impermeable foundation, a number of 'real' conditions are not satisfied, e.g., spatial variation of the coefficient of permeability in the tailings. Also, intermittent discharge of tailings on the beach add some flow to the steady flow from the pool. Furthermore, the foundation material should be at least one order of magnitude less permeable than the tailings for the assumption of an 'impermeable foundation' to be valid. However, although the estimates of the discharge rate, obtained from steady state seepage analysis of homogeneous, isotropic sections are usually conservative, they are acceptable for drain design. All such estimates are made by multiplying the coefficient of permeability of the material by some geometric factor. If the coefficient of permeability of the tailings can at best be estimated within one order of magnitude through measurements and the magnitude of the coefficient of permeability of the material used in drains is less certain, then it is clear that the use of expensive, time-consuming numerical analyses are not warranted for drain design.

It can be shown by considering various closed-form solutions, e.g., Dupuit (see Harr, 1962), Kozeny and vertical upstream face (van Zyl & Harr, 1977), and the method proposed by Uginchus (1960), that all these methods of analysis give about the same flow rate for L/h ≈ 3 (see Figure 1 for definition of L and h). Tailings impoundments usually have L/h ≈ 3 so that the simplest closed form method can be used for calculating flow rate. By rewriting the Dupuit solution for the boundary conditions in Figure 1, the following expression is obtained:

\[ q = \frac{k h^2}{2 \Gamma} \]  

where \( k \) = coefficient of permeability

* An error was detected in the solution presented by Abadjiev (1976), the correct solution is given in the Appendix to this paper.
FIGURE 1
FREE SURFACE FLOW TO BLANKET DRAIN

FIGURE 2
FLOW INTO PARALLEL DRAINS ON LINER
It was shown by van Zyl and Harr (1977) that the effective length of the drain, \( L_{\text{dr}} \) (Figure 1) can be taken as about 0.1 h for \( L/h \geq 2 \).

The width of the drain is furthermore controlled by the coefficient of permeability of the finest layer in the drain. Consider Figure 3. The value of \( L_{\text{dr}} \) must be such that the flow rate \( q_1 \) through the tailings is equal to the available flow rate \( q_2 \) into the drain. The magnitude of \( q_2 \) can be calculated by the extension of Darcy's Law

\[
q_2 = k_2 \cdot i_2 \cdot A_2
\]  

where \( k_2 = \text{coefficient of permeability of the finest layer in the drain}; \)

\( i_2 = \frac{1}{A_2} \)

\( A_2 = L_{\text{dr}} \times \text{unit length of drain} \)

\[ \therefore \quad q_1 = k_2 \cdot L_{\text{dr}} \]  

It is recommended that the width of drain used be at least 50% more than the value of \( L_{\text{dr}} \) obtained from eq. 3 to account for possible blinding of the drain, etc. It is further recommended that the minimum drain width be 2 m.

The drain must transport all the water flowing into it. The spacing of outlets is therefore dependent on the capacity of the drain, i.e., the capacity of drainpipes, if used, otherwise the capacity of the coarsest drainage layer.

The capacity of drain pipes can be estimated by using Manning's equation:

\[
q = \frac{1.486}{n} \cdot R^{2/3} \cdot S^{1/2} \cdot A
\]

where \( n = \text{Manning's Roughness Coefficient} \)

\( R = \text{Hydraulic radius of pipe} \)

\( R = \frac{A}{P} \)

\( A = \text{Area of pipe in square feet} \)

\( P = \text{Wetted perimeter of pipe in feet} \)

\( S = \text{Hydraulic slope of pipe} \)

Typical values of \( n \) for various drain pipe materials are given in Table 1.

The capacity of the coarsest drainage layer can be estimated by using equation 2 in the form
FIGURE 3
TYPICAL SECTION OF DRAIN
\[ q_3 = k_3 i_3 A_3 \]  
where \( k_3 \) = coefficient of permeability of coarse drainage material  
\( i_3 \) = slope of the drain (or slope of ground on which the drain is installed)  
\( A_3 \) = cross-sectional area of coarse drainage material

**TABLE 1**

Typical Values of Mannings n  
(after Schwab et.al., 1966 and Advanced Drainage Systems, Inc., 1978)

<table>
<thead>
<tr>
<th>Material</th>
<th>Min.</th>
<th>Design</th>
<th>Max.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asbestos cement</td>
<td>0.009</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cast iron, coated</td>
<td>0.011</td>
<td>0.013</td>
<td>0.014</td>
</tr>
<tr>
<td>Cast iron, uncoated</td>
<td>0.012</td>
<td></td>
<td>0.015</td>
</tr>
<tr>
<td>Clay or concrete drain tile (4-12 in.)</td>
<td>0.011</td>
<td>0.017</td>
<td>0.017</td>
</tr>
<tr>
<td>Concrete</td>
<td>0.010</td>
<td>0.014</td>
<td>0.017</td>
</tr>
<tr>
<td>Corrugated plastic</td>
<td>0.013</td>
<td>0.016</td>
<td>0.017</td>
</tr>
<tr>
<td>Metal, corrugated</td>
<td>0.021</td>
<td>0.025</td>
<td>0.0255</td>
</tr>
<tr>
<td>Steel, riveted and spiral</td>
<td>0.013</td>
<td></td>
<td>0.017</td>
</tr>
<tr>
<td>Vitrified sewer pipe</td>
<td>0.010</td>
<td>0.014</td>
<td>0.017</td>
</tr>
<tr>
<td>Wood stave</td>
<td>0.010</td>
<td>0.013</td>
<td></td>
</tr>
<tr>
<td>Wrought iron, black</td>
<td>0.012</td>
<td></td>
<td>0.015</td>
</tr>
<tr>
<td>Wrought iron, galvanized</td>
<td>0.013</td>
<td>0.016</td>
<td>0.017</td>
</tr>
</tbody>
</table>

The coefficient of permeability of natural drainage material is difficult to estimate. A useful estimate of the coefficient of permeability of clean sand can be obtained from Hazen’s formula:

\[ k = D_{10}^2 \]  
where \( D_{10} \) = diameter of particles with 10% passing by mass in mm  
and \( k \) in cm/sec

However, if turbulent flow takes place, as in coarse drainage material, other estimates of the coefficient of permeability must be used. Typical values of the coefficient of permeability of various coarse drainage materials are presented in Table 2.

**Parallel Drains**

The design concepts of parallel drains are well-developed in the agricultural field and numerous detailed publications on the subject are available (Luthin, 1957; Luthin, 1966; Schwab, et.al. (1966); U.S. Dept. of the Interior, 1978). Although solutions are available in the publications above for various seepage boundary conditions,
TABLE 2
Coefficient of Permeability of Typical Drainage Materials
(after U.S. Dept. of the Navy, 1971)

<table>
<thead>
<tr>
<th>Drainage Material:</th>
<th>k’ (cm/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D10 Size:</td>
<td></td>
</tr>
<tr>
<td>1 mm</td>
<td>0.5 to 1.7</td>
</tr>
<tr>
<td>2 mm</td>
<td>4 to 10</td>
</tr>
<tr>
<td>Size Range:</td>
<td></td>
</tr>
<tr>
<td>25 mm to 37 mm</td>
<td>37</td>
</tr>
<tr>
<td>10 mm to 13 mm</td>
<td>28</td>
</tr>
<tr>
<td>2 mm to 5 mm</td>
<td>2.7</td>
</tr>
<tr>
<td>0.22 mm to 0.42 mm</td>
<td>0.07</td>
</tr>
</tbody>
</table>

this paper will only consider solutions for homogeneous, isotropic soils subjected to steady state flow. An impermeable layer is assumed to be directly beneath the drains. The flow rate per unit length of drain, q, is given by the coefficient of permeability multiplied by some geometric quantity.

Water is assumed to infiltrate from a pool on the tailings impoundment under partially saturated conditions down to a phreatic surface, as shown in Figure 2.

The flow rate, q, into each drain can be estimated from (see Figure 2 for definition of t and s)

\[ q = \frac{4kt^2}{s} \]  \hspace{1cm} (7)

where \( k \) = coefficient of permeability of material

The simplified relationship between t and s can be expressed as follows (if it is assumed that complete drawdown takes place at the subsurface drain - McWhorter and Sunada, 1977).

\[ t^2 = \frac{ws^2}{4k} \]  \hspace{1cm} (8)

Because of the layers of slimes in the impoundment, it can be taken in general that:

\[ w < k \]  \hspace{1cm} (9)
The magnitude of $k$ can be taken as the horizontal coefficient of permeability which is about equal to the permeability of the coarsest layer. Furthermore, the infiltration rate, $W$, is controlled by the layer with the lowest coefficient of permeability. Depending on the magnitude of the lowest coefficient of permeability (which can be estimated by using Hazen's formula, see Mabes, et. al, 1977 for range of applicability of this formula) it can be assumed that

$$0.001 \leq W \leq 0.1k$$  

(10)

Substituting the lower value ($W = 0.001k$) into eq. 8, it is obtained that:

$$t = 0.016 \text{ s}$$  

(11)

which implies that if $s = 300$ ft.; $t = 4.8$ ft.

The rest of drain design, i.e., width of drain and capacity of drain can be estimated in the same way as before (see equations 2, 4, and 5).

Filters and Filter Criteria

Since the publication of the first filter criteria, numerous studies have been done to prove their validity or to improve upon them, e.g., Bertram (1940), Karpoff (1955), Thanikachalam (1975). Although numerous variations exist for filter criteria, the criteria presented in Table 3 are recommended.

The filter criteria in Table 3 are based on the assumption that high flow gradients exist in the proximity of the drain (in the order of 10 to 50). Agricultural drains are usually subjected to much lower gradients, in the order of 1 or less (Winger and Ryan, 1971). For the latter condition, 'gravel envelopes' are presently advocated (U.S. Dept. of Interior, 1978). Because of the large $L/h$ ratios present in tailings impoundments it is expected that low flow gradients are also present. Gravel envelopes may, therefore, also be considered for tailings impoundments.

The following are the most important considerations for gravel envelopes (Winger and Ryan, 1971):

(i) Drainage material must be reasonably well graded and free of substances which could change the permeability with time. To be well graded, a material must have

$$\frac{D_{60}}{D_{10}} \gg 4 \text{ for gravel}$$

$$\frac{D_{60}}{D_{10}} \gg 6 \text{ for sand}$$  

(12)*

*Coefficient of uniformity.
TABLE 3

Recommended Filter Criteria

\[
\frac{D_{15} \text{ filter}}{D_{85} \text{ base}} \leq 5 \\
5 \leq \frac{D_{15} \text{ filter}}{D_{15} \text{ base}} \leq 20 \\
\frac{D_{50} \text{ filter}}{D_{50} \text{ base}} \leq 25 \\
\frac{D_{85} \text{ filter}}{\text{Slot width}} \leq 1.4 \text{ to } 2.0 \\
\frac{D_{85} \text{ filter}}{\text{Hole diameter}} > 1.2
\]
(ii) All material should pass the 1½" sieve and not more than 5% should pass the No. 50 (U.S.) sieve. The maximum size for a lower envelope is 3/8" while the smallest size for the upper envelope is the No. 30 (U.S) sieve. Figure 4 shows some gravel envelopes for various base materials. These envelopes were developed on the basis of theoretical analyses and laboratory and field observations.

The authors are not aware of the use of 'grave' envelopes' or anything similar for the subsurface drainage of tailings impoundments. However, it is an area which should be investigated, especially with respect to successful long-term drain performance.

The replacement of granular filter materials by woven or non-woven synthetic materials (geotextiles**) is becoming more widespread. Geotextiles are mostly used for economic reasons i.e. when certain granular materials are not readily available. The characterization of geotextiles as drainage materials is still being done by various researchers and the 'filter criteria' which are presently advocated can only be considered as tentative. The criterion used for designing filters adjacent to perforated pipes can also be used to design a filter adjacent to a woven textile (or a woven textile adjacent to natural soil), i.e.

\[
\frac{D_{85}}{D_{2}} \geq 2 \quad (14)
\]

Marks (1375) produced an extensive study of the use of geotextiles for subsurface drainage. Some of the findings of this study were as follows:

Geotextile filter systems undergo a stabilization; wherein, after a small initial loss of fines, the remaining sand and silt particles interfere with one another and prevent further migration of the remaining fines. The interaction of the soil particles immediately behind the fabric create a filter media within the soil mass. Thus, the effective area of the drain is increased to provide a more natural transition from soil mass to filter. The natural transition greatly reduces the seepage velocities near the boundary between filter and protected soil.

*Coefficient of curvature.
**The terms plastic filter, filter fabric and cloth filter are also used in the literature.
<table>
<thead>
<tr>
<th>Gravel</th>
<th>Sand</th>
<th>Fines</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Coarse to medium</td>
<td>Fine</td>
</tr>
<tr>
<td>U.S. standard sieve sizes</td>
<td></td>
<td></td>
</tr>
<tr>
<td>[Diagram showing gravel envelopes for base materials]</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**FIGURE 4** - EXAMPLES OF GRAVEL ENVELOPES FOR VARIOUS BASE MATERIALS (AFTER WINGER & RYAN, 1971)

- a) COARSE SILT LOAM
- b) VERY FINE SAND
- c) MEDIUM SAND
- d) COARSE SAND
Geotextile filter systems investigated were found to perform as well as conventional aggregate systems under all protected soils considered.

The experimental laboratory tests showed that one major advantage of geotextile filters is the elimination of the serious problem of segregation during placement of conventional aggregate filters.

Filtration tests produced permeability decreases in aggregate filter systems of about one order of magnitude through internal gradation mechanisms of the aggregate filter system since soil loss did occur during filtration.

In a recent study @emery (1980) found that long-term water flow through a soil and geotextile system was controlled by changes in the soil (compaction due to flow and bacterial growth) regardless of the type or the opening size of filter used. This was more prevalent when fine soils were located adjacent to the geotextile.

The important conclusion from these two studies is that the percentage of fines in soil layers adjacent to geotextiles should be limited to prevent clogging of the fabric and/or the formation of a filter cake adjacent to it. It is, therefore, recommended that geotextiles not be used directly against tailings, but that a granular filter material be used between the tailings material and the geotextile. It is recommended that a granular drainage layer adjacent to a geotextile layer should not have more than 2% passing the No. 200 sieve. Perforated drain pipes should not be wrapped in geotextile before installation. This is not good practice as clogging of the geotextile may occur at the perforations due to migration of fines or build-up of bacterial growth or chemical deposits.

Drain Lay-out

The drains should be positioned to fulfill the required function, be it reduction in pore pressure for stability requirements or increase in water return, etc. As a first approximation for design, it is recommended that a blanket drain be positioned underneath the final crest position, see e.g. Figure 1. This position will usually secure adequate long-term lowering of the phreatic surface for stability purposes. In order to secure sufficient drainage during the initial stages of construction, an extra blanket drain may have to be installed against the starter embankment.

Typical details for drainage blankets and outlets are indicated in Figure 5. Trench cut-offs and outlets, as shown, are usually installed where liners are not used. When a liner is used, outlets are located above the liner. In this case, special provisions may have to be considered for outlets to secure maximum outlet efficiency. This can be done by providing steps along the length of the drain against which outlets are positioned, see Figure 6.

It is recommended that geotextiles be covered with at least 4
(a) BLANKET DRAIN DETAIL

(b) OUTLET DRAIN

FIGURE 5 - TYPICAL DRAIN DETAIL
FIGURE 6 - TYPICAL DRAIN OUTLET WHEN DRAIN TRENCHES NOT USED
inches of granular filter material (special filter sand or coarse tailings) as indicated in Figure 5. This will prevent the direct washing of tailings across the geotextile, whereby the possibility of blinding the drain is increased.

A vertical drain system can be installed to improve drainage (Gray and Sonogyi, 1978). The vertical drains can consist of specially formed cones or walls of coarse tailings deposited by a number of cyclones installed at regular spacing within the tailings impoundment. Considerable difficulty will be experienced with access to the cyclones to service them and to elevate them.

An alternative to the vertical drains formed by cycloning is the use of sandbags to form vertical 'drainpipes', these 'pipes' can be filled with coarse drainage material and connected to a blanket drain with regular outlets; or a wall of sandbags can be used as vertical drains. The sandbags will act as an extra filter layer. However, some difficulties may be experienced with clogging of the burlap walls of the sandbags, as would be the case with geotextiles directly in contact with tailings.

The chimney drain scheme shown in Figure 7 is recommended. A drain is installed on the ground surface with regularly spaced outlets. Vertical chimneys of about 18 to 24 inch boiler pipe filled with coarse drainage material (typically pea gravel to 0.5 inch) is installed at about a 300 ft. spacing, (or as required by design). Another drainage blanket is installed when the impoundment is filled with about 12 to 15 ft. of tailings at the chimneys. This blanket is keyed into the chimneys which act as outlets. The top blanket is finally responsible for drainage of the impoundment. The position of the top blanket depends on the beach formation on the impoundment. It should be installed only when a beach is formed such that the pool position does not cause the settlement of large amounts of fines on the blanket drain, thereby blinding it.

Klohn (1979) presents some alternative drainage schemes for various construction methods and embankment sections.

When embankments with clay cores are used, blanket drains (or vertical drains) should be provided on the upstream side of the clay core to reduce the hydraulic head on the core.

The disposal of uranium tailings in mined out pits is becoming more prevalent. The hydraulic head on the side walls can be reduced by installing a blanket drain on the pit perimeter with a sump and pump system to remove water from the drains. The use of the chimney drain system described above, can also be considered, however, its installation must be related to the depositional pattern used.

CONSTRUCTION AND MANAGEMENT CONSIDERATIONS

The most important consideration during construction of the underdrainage system is the protection of the integrity of the system; e.g. prevent pipe breakage, protect the geotextiles, prevent segrega-
tion of granular drainage material.

Drainage pipes can be broken during construction by coarse drainage material being dumped onto it. Winger and Ryan (1971) suggested that 1\(\frac{1}{2}\) inch stone is the largest drainage material which can be used without causing damage to clay pipes.

Construction traffic can also cause damage to drainpipes. The collars of open collar drain pipes can be wrapped in geotextile to prevent complete loss of drainage ability should it be damaged. This should only be done in the areas where construction traffic can cause damage and when sufficient granular material is used between the drain-pipe and the tailings so that clogging of the geotextile on the collars is prevented.

Some geotextiles are sensitive to ultraviolet light exposure and should be covered as soon as possible. Rodents can damage geotextiles by eating holes in it. Careful observation of outlet pipes and drainage blankets during first filling will help in the detection of such damage: the drainage water will have sediment in it and sinkholes may appear on top of the drainage blanket. Careful control during construction should ensure that geotextile layers overlap sufficiently in drainage blankets, see Figure 5a.

Segregation of granular drainage material can be limited by limiting the range of particle sizes in the material and/or by adding some water to the drainage material before placing it.

It is recommended that the drainage material in geotextile lined trenches (see Figure 5) be compacted lightly to ensure good contact between the fabric and the foundation material. This is especially important on the trench bottoms.

The most critical time for the control of subsurface drainage systems in tailings impoundments is during the first filling with tailings. The movement of fines across the drain can easily blind the drain and special depositional techniques may have to be employed to prevent it. An engineer should be on site during first filling to make any changes necessary to prevent blinding of drains. The subsurface drainage system is an expensive element of the tailings impoundment and should not be damaged through lack of control during the life of the impoundment.

High flow rates across a drainage blanket can destroy the top drainage layer. Discharge of tailings slurry can initially be done parallel to the drainage blanket instead of perpendicular to it to reduce the flow velocity over the top of the drain.

It may be necessary in some instances to remove the initial layer of tailings from the drain by hand if blinding is suspected.

The installation of piezometers to measure pore pressure at a number of locations on a tailings impoundment is very important if the operation of subsurface drains is to be monitored sufficiently. The
Piezometers should be installed when the impoundment is constructed, or shortly thereafter, to reduce installation costs. A monitoring program must be established as part of the overall tailings management program. Pore pressure results must be analyzed regularly so that irregularities in drain performance can be detected early. As part of the monitoring program, drain discharge must be measured regularly, this can be done through the installation of measuring weirs. A good appreciation of drain performance can be obtained by analyzing piezometer and drain discharge data regularly.

**Drain Maintenance**

Drain clogging can occur through a number of mechanisms, e.g. pipe breakage, movement of silt and clay-size material into the filter material and/or drain pipe perforations or openings, growth of vegetation through drainage material, chemical deposition in the filter material and drain-pipes such as iron, manganese or calcium carbonate and bacterial growth. Some of these problems can be remedied through mechanical means, such as a Roto-rooter or high pressure water jets to remove vegetation and chemical build-up. Calcium carbonate can usually be removed by pumping an acidic solution into the drain-pipes and filter.

The formation of chemical and bacterial deposits in subsurface drains is usually a function of the chemical environment at the drain. The role of bacteria and the physical and chemical factors contributing to, e.g. ochre formation are reasonably understood, but there are few long-term satisfactory techniques for predicting or controlling the clogging of drains by ochre. Ochre is described as a sticky, gelatinous, yellow to reddish mass of ferric hydroxide plus organic material that can clog subsurface drains. As the ochre ages and loses water, it becomes dark reddish-brown, granular, and hard. Clogging of subsurface agricultural drains by ochre deposits is widespread throughout the United States and Europe. (Lidster and Ford, 1979).

Although there are a large number of different bacteria capable of precipitating iron, some filamentous forms and a large number of rod bacteria are identified with the problem. Iron bacteria are present in the ground and surface waters throughout the temperate and tropical zones (Lidster and Ford, 1979).

Siltation and ochre clogging often coincide, which results in the cementation of the mineral grains and a concrete-like structure develops with age. Removal of such deposits have been attempted by Roto-rooters and high pressure jet cleaners with limited success (Lidster and Ford, 1979).

Lidster and Ford (1979) report that in Florida, iron concentrations of 0.1 to 0.3 mg/l are sufficient for the development of a clear to reddish slime mass of ochre in pipes. Iron concentrations of 0.4 to 1.5 mg/l may contribute to serious clogging of drainpipes provided conditions are favorable for the development of iron precipitating organisms. Ochre can occur in water with a pH range of 2.5 to 8.5 because there are many different types of bacteria that can influence
ochre development. Ochre deposits will, therefore, not develop in uranium tailings impoundments where the pH is lower than 2.5 (acid leach process) or higher than 8.5 (alkaline leach process). However, if long-term pH changes do take place, such clogging may be observed. It seems that ochre deposits can develop in most other tailings impoundments if the correct chemical atmosphere is present.

Treatment of ochre clogged drains can be done with 2 percent SO2 gas after the 'loose' ochre is removed by high pressure jet. (Grass and McKenzie, 1972). Hazards of potential air and water pollution and in handling the SO2 are major drawbacks of this method. Another method which is presently investigated to remove ochre formation is the use of dry pelletized sulfamic acid (HSO3NH2). The sulfamic acid solution should be held in the drain for a minimum of 24 hours by plugging the drain. The effluent from a treated drain can be neutralized with sodium carbonate prior to discharging it to the environment (Lidster and Ford, 1979).

Although no reports of bacterial clogging of subsurface drains of tailings impoundments have been published, it is possible that such clogging has occurred without it being interpreted as such. Although the rehabilitation of such clogged drains is expensive and difficult, it may be necessary and more economic procedure than other solutions, such as pore pressure reduction through pumping or the construction of rock buttresses to improve stability.

CONCLUSIONS

Although the design of subsurface drains is a relatively simple matter, technically, expert control during construction of the drains and first filling with tailings are necessary to secure successful long-term operation of subsurface drains. Regular monitoring of pore pressures in the impoundment and flow rates from the drains will lead to a good appreciation of drain performance.

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APPENDIX

EXPRESSIONS FOR SEEPAGE ANALYSIS ASSUMING EXPONENTIAL CHANGE IN COEFFICIENT OF PERMEABILITY (AFTER ABADJIEV, 1976)

(i) Dupuit principle and boundary conditions

\[ q = ky \frac{dy}{dx} \quad A.1 \]

at \( x = 0 \) \( y = \frac{q}{k_0} \) \quad A.2

at \( x = L \) \( y = H \) \quad A.3

see Figure A.

(ii) Expressions obtained by Abadjiev (1976)

\[ q = \frac{1}{k_0} \frac{aH^2}{2} e^{AL} - 1 \]

\[ y^2(x) = \frac{2}{a} q \frac{a}{k_0} (e^{ax} + \frac{a}{q} \cdot \frac{k_0}{2} - 1) \]

The latter expression is dimensionally incorrect.

(iii) Derivation of correct expressions

From A.1 above

\[ q e^{ax} dx = y dy \]

\[ \frac{q e^{ax}}{k_0} - \frac{y^2}{2} = C \]

From substitution of boundary condition.

\[ y^2 = \frac{2q}{k_0} a \left( e^{ax} + \frac{a}{q} \cdot \frac{k_0}{2} - 1 \right) \quad A.4 \]

\[ q = \frac{aH^2}{2(e^{aL} - 1)} + \frac{aq^2}{2k_0(e^{aL} - 1)} \quad A.5 \]

Solving the quadratic

\[ q = -\left( e^{aL} - 1 \right) \pm \sqrt{\left( e^{aL} - 1 \right)^2 + 2aH^2} \quad A.6 \]

Expressions A.4 and A.6 are the correct solutions and are also dimensionally correct.
$k(x) = k_0 e^{-ax}$

**FIGURE A**