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Seepage Control for Tailings Dams

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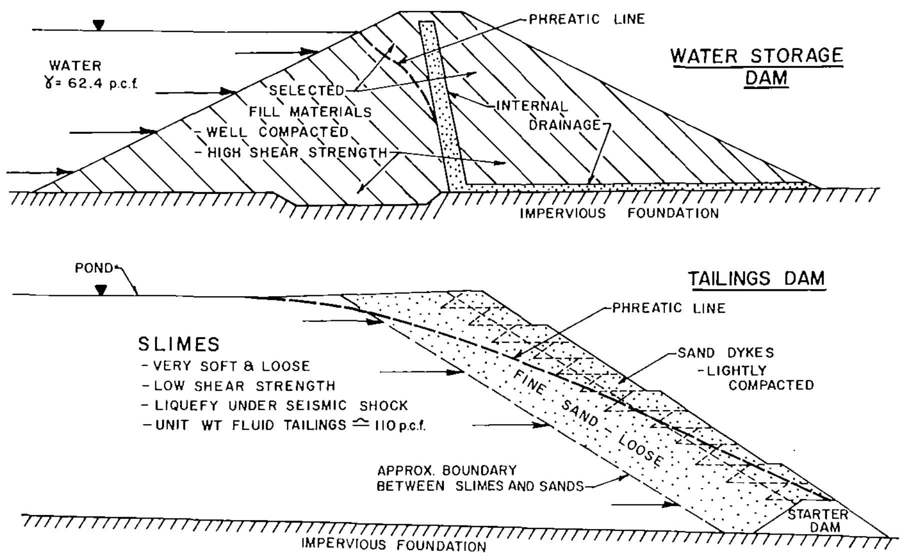
1. INTRODUCTION

Seepage control is a critically important aspect in the design, construction, and operation of tailings dams as it directly affects: the stability of the downstream slope; internal erosion due to piping; and pollution of ground and surface waters downstream of the dam. Methods for controlling seepage through embankments, abutments, and foundations have been extensively studied and developed over a period of many years in the field of conventional water storage dams. These procedures, which are considered standard practice in the conventional water storage dam field, are well documented in the published engineering literature. Moreover, these conventional seepage control procedures are directly applicable to the control of seepage flows in the embankments, abutments, and foundations of tailings dams. However, they must be suitably modified to account for the fact that a tailings slurry, rather than water is being stored behind the dam.

In the past, practically all tailings dams were constructed by some variation of the upstream method of construction. The original upstream method normally involved construction of a low earth "starter" dyke, 10 to 20 feet in height. This dyke was usually constructed from locally available borrow materials and was seldom subject to engineering design.

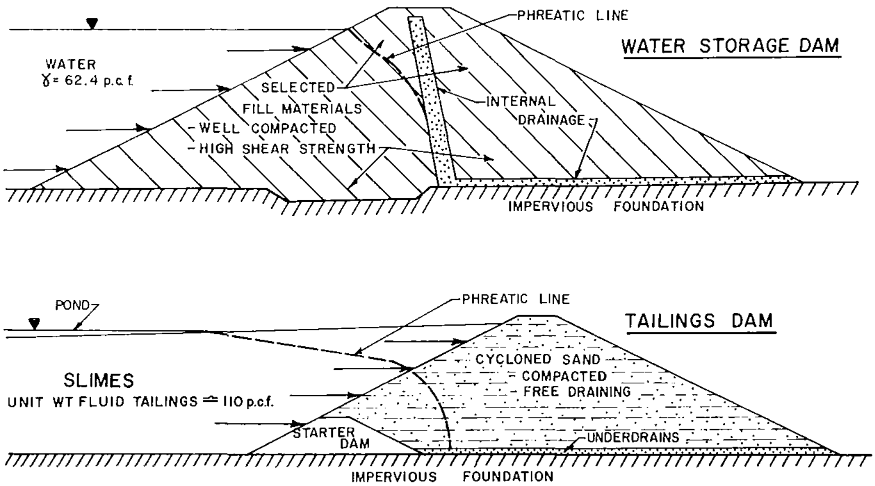
The tailings were discharged by spigotting off the top of the starter dyke. When the initial pond was nearly filled, the dyke was raised by borrowing material from the dried surface of the previously deposited tailings, and the cycle was repeated. As the height of such a dam increases, each successive dyke moves further upstream, and is underlain by the soft, previously deposited tailings. There is a limiting height to which such a dam can be raised before shear failure occurs, and the tailings flow out. In regions subject to seismic shocks, failure of this type of dam by liquefaction can occur at very low heights. In fact, the history of this method of dam construction is plagued with failures, some of them catastrophic. Figure 1 (9) presents a comparison between a conventional water storage dam and a tailings dam built using the upstream method of construction.



COMPARISON BETWEEN CONVENTIONAL WATER STORAGE DAM AND TAILINGS DAM BUILT USING AN UPSTREAM METHOD OF CONSTRUCTION

FIGURE 1

Current, good engineering practice is to use downstream methods of tailings dam construction for all major tailings dams. In areas of high seismic risk where failure of the tailings dam poses a threat to life and property, downstream methods of tailings dam construction should be used for all structures regardless of their height. The downstream method of tailings dam construction evolved from a blending of the engineering knowledge and experience available in the field of water storage dams, with the knowledge of the mining operators responsible for construction and operation of tailings dams. The downstream method of tailings dam construction involves constructing the dam in a downstream direction from the initial starter dam. Consequently, as the dam is raised, it can be constructed over a carefully prepared foundation base rather than over previously deposited slimes, as is the case for the upstream method. Figure 2 (9) presents a comparison between a conventional water storage dam and a tailings dam built using the downstream method of construction. As might be expected, the downstream method of tailings dam construction permits far better control of seepage flows and pressures than does the upstream method.



COMPARISON BETWEEN TYPICAL WATER STORAGE DAM AND TAILINGS DAM BUILT USING ONE OF THE DOWNSTREAM METHODS OF CONSTRUCTION

FIGURE 2

In some instances, dam safety and environmental pollution control regulations can have a major effect on the seepage control facilities required for either upstream or downstream tailings dams. In those instances where a closed circuit tailings pond is required (no discharge of effluent is permitted downstream of the dam), foundation cutoffs and foundation drainage wells may be necessary to prevent surface and embankment seepage from passing downstream of the dam.

The paper addresses the problem of seepage control for tailings dams. Conventional flow nets are presented to illustrate the effectiveness of various seepage control measures which are normally used in the design of water storage dams. The application of these measures to tailings dam design and construction is then discussed. Examples are presented illustrating some of the problems that can develop when uncontrolled seepage occurs. Also presented are several case histories illustrating seepage control measures incorporated into the design of several existing tailings dams.

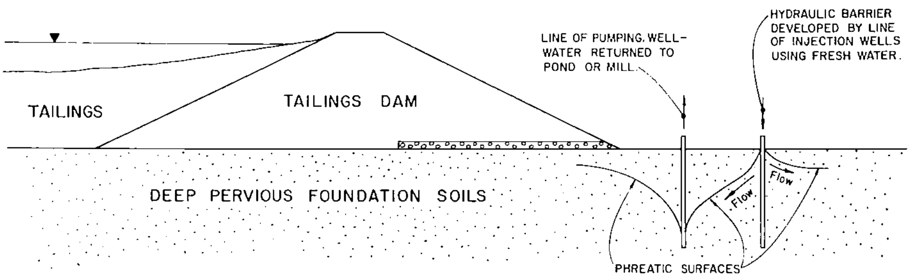
II. PUBLIC CONCERN AND GOVERNMENT REGULATIONS

Tailings dams are important structures that involve two aspects of public concern. One is the structural stability of the dam and the possible release, if failure occurred, of very large volumes of water and/or semi-fluid tailings. Such an event would not only cause extensive downstream pollution but would also pose a serious threat to life and property. The other aspect of public concern is the possibility of pollution under normal operation, in which polluted effluent might escape through or around the tailings dam and enter the streams or groundwater of the area.

The potential pollution hazard associated with storage of the tailings slurry varies with different mining operations, and ranges from very severe for the radioactive wastes associated with uranium mining, to none for mining processes which merely grind up an inert ore without the addition of toxic chemicals during processing. In between these two extremes are a wide range of conditions that present either short or long-term, potential, pollution problems.

In response to public demands, Governments throughout North America and in many other parts of the world have enacted legislation relating to the safety and pollution aspects of the design, construction, and operation of tailings dams. Accidents such as the Teton Dam failure have served to focus the attention of both the public and the legislators on the issue of dam safety, and a trend towards stricter regulations affecting all aspects of the design, construction, and operation of tailings dams should be anticipated.

Pollution control regulations can have a very important impact on tailings dam design, and, in particular, on seepage control measures that are required to satisfy the regulations. In those instances where the effluent seepage is considered harmful, extensive cutoff and seepage collection facilities may be required to prevent seepage from reaching and contaminating the surface and groundwaters of the area. This would be the case where the effluent was considered to contain harmful chemical agents or was radioactive. In recent years, public concern about the storage of radioactive waste from uranium mining operations has greatly increased. (24) In some cases, complete lining of the uranium tailings pond has been required with suitable underdrainage and monitoring systems installed. In instances where the tailings dam is constructed on pervious foundations of great thickness, and pollution control requirements do not allow the loss of water from the pond, a hydraulic barrier may be required. This method of seepage control which is rather costly and complicated involves the installation of two lines of wells downstream of the dam. The upstream line of wells are pumping wells which lower the groundwater table, and the downstream row of wells are injection wells which maintain the positive hydraulic barrier. The method is illustrated schematically on Figure 3.



HYDRAULIC BARRIER- USED TO PREVENT SEEPAGE DOWNSTREAM OF DAM

FIGURE 3

Obviously, regulations pertaining to dam safety and pollution are factors which must be carefully considered in designing and costing the tailings disposal facilities required for any new development. Satisfying these regulations unquestionably will add to the cost of tailings disposal and, in future, these extra costs must be considered a necessary part of the cost of production.

III. SEEPAGE THROUGH CONVENTIONAL DAMS

1. General

Tailings dams store water as well as tailings in their reservoirs. The volume of water stored and the location of the free water surface within the tailings pond varies from one mining operation to another. At one extreme, some tailings dams are designed to have water stored against their upstream face (8) and must perform the same function as a conventional water storage dam. (On a large number of projects the starter dam is designed in this manner with the remainder of the dam raised using sand tailings for dam construction and utilizing the slimes beach as the upstream impervious membrane). At the other extreme, some tailings dams have very wide slimes beaches against their upstream slope at all times and the free water surface in the tailings pond is located 1000 ft or more away from the dam. Most tailings dams fall somewhere between these two extremes such that they generally operate with a slimes beach several hundred feet wide against their upstream face. This slimes beach acts as the upstream impervious membrane for the tailings dam. In most instances the tailings dam itself is built of pervious, tailings sand. If, under unusual hydrological or operating conditions, pond levels should rise and flood the slimes beach we have the case of a sand dam with a free water surface against its upstream face. This, of course, is a most undesirable situation, the possible consequences of which are discussed in a following section.

In summary, it can be stated that tailings dams, like conventional water retention dams, store water. In some instances, and particularly for tailings starter dams, an appreciable depth of water may be stored against the upstream face of the tailings dam. In most instances a wide beach of slimes, acting as an upstream impervious membrane is located between the rising sand tailings dam and the free water in the pond. However, under unusual circumstances, this beach could be flooded bringing the free water surface up to the face of the sand dam. Obviously the seepage control measures that have been developed to protect conventional water storage dams have wide application to tailings dams. In the following sections of this paper these conventional seepage control measures will be reviewed and their application to various tailings dam designs considered.

2. Seepage Problems and Their Defences

The water stored behind a dam always seeks a means of escape. Control of this seepage presents a challenge to the designer because water will always find the path of least resistance for its escape route. This will take the seepage through pervious strata, joints, fissures, and cracks as they really exist, in and beneath the structure, rather than as assumed for purposes of the design analyses. For this reason, seepage control measures should always be conservatively designed and for important structures, instrumentation to measure piezometric pressures and seepage flows should be included as part of the design.

Seepage through dams may give rise to three basic problems that can create serious difficulties and in the extreme may lead to failure. These three problems are:

a) Piping - This occurs where exiting seepage flows pick up soil particles and move them out of the foundation or embankment. The continued removal of soil particles causes the unseen development of channels or pipes in the embankment or foundation. When these pipes connect back to the free water in the reservoir very large flows develop along the pipe and complete failure of the dam may occur. The Teton Dam failure (20) has been attributed to piping.

b) Slope Instability and Heaving - Seepage forces caused by the flow of water through the embankment or its foundations can cause instability of downstream slopes. If excess upward seepage forces develop in the foundation soils immediately downstream of the toe of the dam, heaving may occur.

c) Excess Water Losses - These occur when the embankment or its foundations are pervious. Apart from the obvious disadvantage of losing water, large seepage losses may or may not pose a problem for the dam. Generally, provided the seepage pressures associated with these large flows do not pose a stability or uplift problem and provided adequate protection against piping is given by properly designed filters, fairly large seepage flows can be accepted. However, it is normally considered good engineering practice to minimize seepage flows by the introduction of relatively impervious elements in the upstream section of the dam and its foundation.

There are three basic defences used for the control of seepage. These are:

a) Filters - to prevent piping and heaving. Basically, filters are designed to permit the free discharge of the seepage water but to prevent the movement of soil particles. (The basic rules for design of filters are presented in the following section).

b) Seepage Reduction - to reduce water pressures and seepage forces in the critical exit areas downstream of the dam. The methods used include: impervious cutoffs, grout curtains, and upstream impervious blankets.

c) Drainage - to reduce water pressures in the embankment and foundation soils. The methods used include: internal vertical interceptor drains, horizontal blanket drains, strip drains, toe drains, and relief wells.

Normally, the above three methods of seepage control are used in combination. For example, the seepage reduction methods used must almost be perfect, if they are to greatly reduce downstream seepage flows and pressures. As this is seldom possible to achieve, the seepage reduction methods are usually combined with downstream drainage to ensure the desired end-result. Similarly, all drains must be designed to meet the filter requirements as their function is to get the water out of the surrounding soil without loss of soil particles.

Filter Design - The filter design criteria specified by different designers and/or agencies (5, 17, 18, 19, 23) show some variations but basically follow the criteria originally set out by Terzaghi and confirmed by Bertram (1) about 40 years ago. The author suggests the following criteria for filter design:

Rule 1: $\dfrac{\text{The 15\% size of the filter}}{\text{The 85\% size of the protected soil}}$ should be less than 5.

Rule 2: $\dfrac{\text{The 50\% size of the filter}}{\text{The 50\% size of the protected soil}}$ should be less than 25.

Rule 3: The filter material should be smoothly graded and its grain size curve should approximately be parallel to that of the protected soil, in the finer range of sizes.
Gap-graded materials are not acceptable.

Rule 4: $\dfrac{\text{The 15\% size of the filter}}{\text{The 15\% size of the protected soil}}$ should be greater than 5.

Rule 5: The filter should not contain more than 5 per cent of particles, by weight, finer than the No. 200 sieve, and the fines should be cohesionless.

Rule 6: The maximum size of filter aggregate should not exceed 3 inches (protects against segregation).

Rule 7: For bases of plastic clay soils with low permeability, concrete sand may be used for the filter (ASTM C33)

Rule 8: For bases of non-plastic silt, rock flour, or varved silt, asphalt sand may be used for the filter (ASTM D1073)

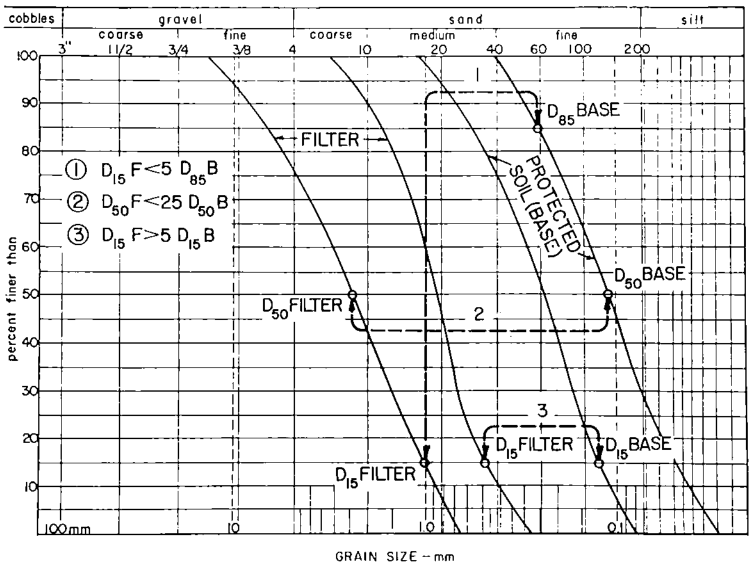
Rule 9: For base material that ranges from more than 10% larger than a No. 4 sieve to more than 10% passing a No. 200 sieve, the filter design should be based on the material passing the No. 4 sieve.

Rule 10: To avoid movement of filter into drain pipe perforations or joints:

$$\frac{\text{D85 filter}}{\text{Slot width}} \quad 1.4 \text{ and} \qquad \frac{\text{D85 filter}}{\text{hole diameter}} \quad 1.2$$

Rules 1, 2 and 3 are to ensure that the filter will not allow migration of particles of the protected soil. Rules 4 and 5 are to ensure that the filter has sufficient permeability. Rule 6 is intended to minimize the problem of particle segregation during placement. Rules 7 and 8 cover the case of placing filter against fine-grained, core materials. The sands are fine enough to prevent the migration of fine particles, are coarse enough to be free draining, and are sufficiently cohesionless to act as "crack stoppers" for the core. Rule 9 ensures that the filter design for broadly graded base material adequately protects the finer portion of the base. Rule 10 applies to the case of filter material placed around slotted or perforated drain pipe.

Figure 4 (18) illustrates the application of Rules 1,2,3 and 4. Rules 1 and 2 use the finest D85 and D50 gradation for the base against the coarsest D15 and D50 for the filter. Rule 4 uses the coarsest D15 for the base against the finest D15 for the filter.



GRADATION REQUIREMENTS FOR FILTERS

(AFTER REFERENCE 23-TENTATIVE DESIGN GUIDE FOR MINE WASTE EMBANKMENTS IN CANADA,

FIGURE 4

Where graded filters are required, each successive filter must satisfy the filter criteria. Most drains are zoned and consist of an outer zone to prevent the movement of fines from the embankment or foundation soils and an inner zone of higher permeability to carry away the seepage flows. The coarser, inner zone must also satisfy the filter criteria relative to the outer zone to ensure that soil particles from the outer filter do not move into the coarse inner zone.

Relief Well Design - Relief wells are used as a means of relieving uplift pressures in pervious foundation soils that pass beneath the dam. They normally consist of an 18" to

24" diameter outer hole and a 6" to 10" diameter inner well screen. The annular space between the drill hole and the well screen is filled with suitable filter material. Relief wells are usually located at the downstream toe of the dam where they are accessible for both observation and maintenance. Quite often relief wells are used in conjunction with one of the seepage reduction measures (grouting, upstream impervious blanket, etc.) to control seepage gradients at the downstream toe of the dam. Relief wells flow by gravity and hence the amount of pressure relief that they can provide is controlled by the elevation of their discharge pipe. In some instances, the relief wells are allowed to discharge at, or slightly above ground surface, whereas in others they discharge into a buried collector pipe which leads either to a pumped sump or discharges at a lower topographic elevation.

Relief wells are commonly spaced at 50 to 100 ft centres. Flows should be measured and piezometers should be installed between relief wells to measure their effectiveness. In the event the original installation is inadequate, additional relief wells can be installed between the original wells. Relief wells must be able to maintain their initial capacity for long periods of time or be restored or replaced. A comprehensive 5 year investigation by the U.S. Army Corps of Engineers in 1972, (22) of relief wells on the Mississippi River showed that the specific yield of 24 test wells decreased 33% over a 15 year period. Incrustation on well screens and in gravel filters was believed to be the major cause (iron bacteria growth on screens and in filters from precipitation of iron oxides and hydroxides and calcium carbonates in gravel filters). This is an item which must be considered when a relief well system is included as part of the seepage control measures.

Relief wells must be designed to discharge water, without loss of solids. To ensure permanent performance without movement of soil particles, the well screens and their surrounding granular fill must satisfy the filter criteria outlined in a previous section of the paper. Detailed procedures for the design of relief well systems are presented in the list of references (5,13,21,23) appended to this paper.

3. Flow of Water Through Soils

<u>The Flow Net</u> - The flow of water through soils is directly proportional to the hydraulic gradient (Darcy's Law). The general differential equation for the steady flow of water

through isotropic soils can be expressed mathematically by La Place differential equations. The graphical solution of the La Place equations is called the flow net. (2,5).

Flow nets may be sketched by hand, developed from model studies using dyes, or by using electrical analogs. More recently, digital computer solutions using either finite-element or finite-difference methods have been developed (7,12,16). The flow net is a grid formed by the intersection of two sets of orthogonal lines. One set of lines, the flow lines, represent the direction of flow of the water, the other set of lines, the equipotential lines, are contours of pressure head. The top flow line defines the phreatic surface of the seepage flow. Piezometric pressures at any point are predicted from the equipotential lines. Seepage flows are predicted from the flow net using the following relationship:

$$q = k \cdot h \cdot \frac{n_f}{n_d} \quad \text{per unit of length}$$

Where:

q = rate of seepage flow

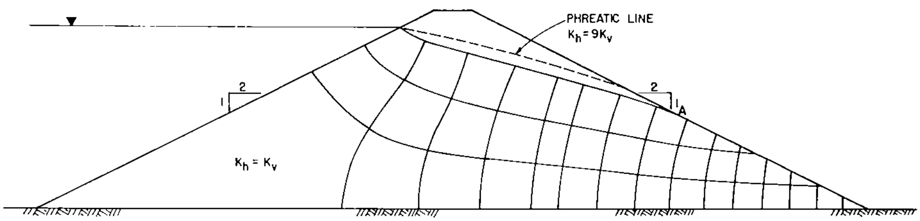
k = coefficient of permeability of the soil

h = the hydraulic head acting across the structure

n_f = number of flow paths in the flow net

n_d = number of equipotential drops in the flow net

Figure 5 presents a flow net for a homogeneous sand dam, having no seepage control features and resting on an impervious foundation. To simplify the presentation we have assumed that the vertical and horizontal permeabilities for the sand dam are equal. In practice this is seldom the case as the horizontal permeability is normally several times greater than the vertical permeability. This has a major effect on the shape of the flow net and the seepage control measures required. This aspect of the problem will be addressed in a following section.



FLOW NET FOR HOMOGENEOUS DAM - NO UNDERDRAINS

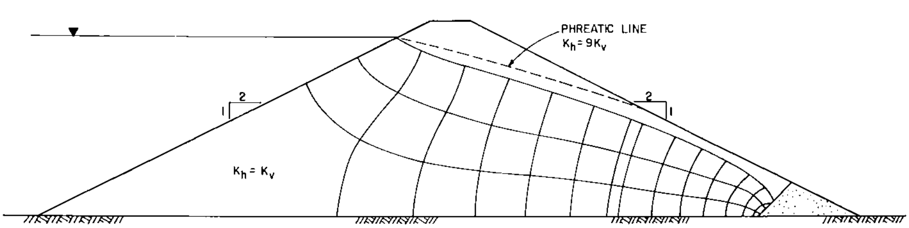
FIGURE 5

On Figure 5 the phreatic line (top flow line) intersects the downstream slope of the dam at Point A. Below Point A the slope is saturated and water is flowing from the slope. These conditions create the following two problems:

1) High piezometric pressures in the downstream slope, which may cause slope instability.

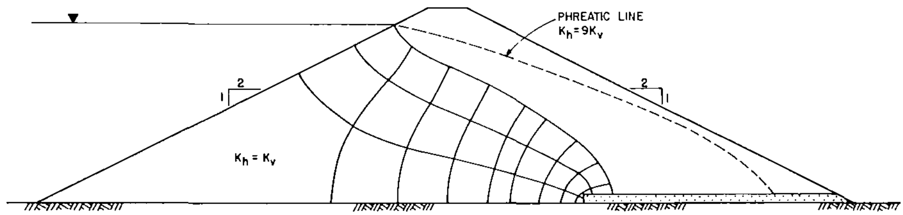
2) High exit gradients for the seepage flows, which may cause piping due to internal erosion.

The potential instability and piping problems of Figure 5 can be cured by the introduction of suitable drainage measures. Figure 6 presents the flow net for the same sand dam ($k_h = k_v$) with a filtered toe drain added. The effects of the toe drain are readily apparent. The seepage flow no longer intersects the outer face of the dam and instead now intersects the filtered toe drain entirely within the dam section. However, the benefits derived from the toe drain may be somewhat misleading because, although the toe drain effectively treats the piping and surface sloughing problems it may not solve the slope instability problem, as the phreatic line remains high in the embankment and a large part of the embankment remains saturated.



FLOW NET FOR HOMOGENEOUS DAM - WITH TOE DRAIN

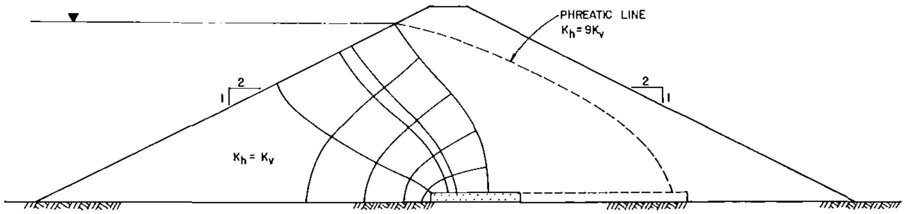
FIGURE 6



FLOW NET FOR HOMOGENEOUS DAM - WITH BLANKET DRAIN

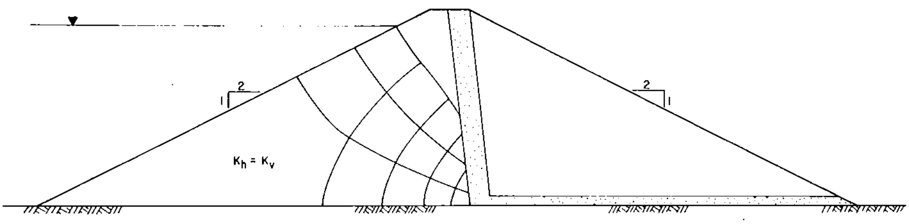
FIGURE 7

Figure 7 presents the flow net for the same sand dam with a downstream blanket drain added. The blanket drain is much more effective than the toe drain in lowering the phreatic line and draining the downstream portion of the sand dam. Figure 8 presents a variation of the blanket drain whereby a strip drain, running parallel to the axis of the dam, is installed at about the centreline of the dam. Finger drains connect the strip drain to a toe drain at the downstream toe of the dam. For the assumed conditions ($k_h = k_v$) the strip drain is extremely effective and the downstream half of the dam is dry. Another type of extremely effective drain that is often used in homogeneous dam sections is the inclined or chimney drain. Figure 9 presents the flow net for this case. Finger drains connect the inclined drain to the toe of the dam.
Drains should be very conservatively designed to ensure that their capacities are adequate to handle the maximum probable seepage flows and still maintain the line of seepage within the drain zone. For estimating the seepage flows that will be intercepted by the drain, the highest probable permeability should be used for the surrounding soil and the computations carried out based on flow nets.



FLOW NET FOR HOMOGENEOUS DAM - WITH STRIP DRAIN

FIGURE 8



FLOW NET FOR HOMOGENEOUS DAM - WITH INCLINED DRAIN

FIGURE 9

Conversely, the lowest probable permeability values should be used for the drain, when determining the drain cross-section required. There are several factors which might add to the originally estimated seepage flows and therefore the required drain capacity may be unknown at the time the drains are designed. Such factors might include highly pervious zones in the foundations not found during design, development of cracks in the impervious core, windows in the grout curtain, drainage from consolidating slimes, etc. For these reasons, the author considers that drains should be designed initially to handle seepage flows several times greater than the maximum anticipated values.

Two types of drains are commonly used. One type utilizes a perforated pipe surrounded by filter material for the water bearing element, whereas the other type utilizes a coarse, drain rock zone surrounded by filter material. Pipe drains located beneath dams can become potential sources of internal erosion, should they collapse at some future date. Collapse can be caused by such events as corrosion of the pipe, excessive vertical load on the pipe, pulling apart at joints due to large strains and/or settlements, etc. Repair of a collapsed drainpipe which is discharging soil and water can be difficult without destroying thd drainage system. For these reasons if pipe drains are used they must be very conservatively designed and must have sufficiently generous surrounding filter zones that should the pipe collapse or pull apart, the surrounding filters will fill the void and prevent uncontrolled internal erosion from developing. The major advantage of using pipe drains as opposed to graded filters is, of course, the much greater hydraulic capacity that can be achieved. However, because of the inherent risks involved with the use of pipe drains, the writer considers that graded filters are preferable except perhaps in those instances where large hydraulic capacity is required, and suitable granular materials or quarried rock are not available.

Flow nets are used to estimate the volume of water that any particular drainage system must carry. As previously pointed out, these estimates should be generously high and the drain should be capable of handling flows several times those values indicated by the computations. Cedergren (5) outlines procedures for computing the size of a graded filter drain required for any given volume of seepage. Where pipe drains are used the required pipe drain sizes can be selected using standard hydraulic tables, once the gradients and pipe type have been chosen. All drain designs should be based on the premise that the hydraulic gradient will remain within the drain material.

Anistropic Soils - In the previous examples the soils were assumed to be isotropic, that is, possessing the same permeability in all directions. In actual fact, this is never the case for either naturally deposited soils or artificially deposited soils such as might be placed in an embankment. For well-stratified soil deposits, the horizontal permeability may be more than 10 times the vertical permeability. Even for compacted embankments where great care has been taken to minimize horizontal stratification, the horizontal permeability is likely to be in the order of 4 to 9 times the vertical permeability.

For embankments constructed of tailings sand placed in horizontal lifts, values of the ratio of k_h/k_v ranging between 4 and 10 have been found to reasonably represent field conditions. For embankments constructed by on-dam cycloning methods, a much more homogeneous sand deposit is obtained. Observations would suggest that for such deposits the k_h/k_v ratio is much less than that for hydraulically placed sands and may even approach unity. For spigotted tailings beaches, a ratio of $k_h/k_v = 9$ appears to provide a reasonable fit with observed piezometric pressures.

The flow net solutions presented for the isotropic case ($k_h = k_v$) can be applied to the anistropic case by means of a geometric transformation (2, 5). This involves either reducing all dimensions in the direction of k_{max} by the factor $\sqrt{k_{min}/k_{max}}$ or increasing all dimensions in the direction of k_{min} by the factor $\sqrt{k_{max}/k_{min}}$. The transformed section thus produced can then be used to draw up the conventional flow net. Once this is done, the flow net obtained is transposed back onto the true section. The resulting flow net will likely consist of rectangles rather than squares and the flow lines and equipotential lines are unlikely to intersect at right angles. Seepage quantities can be computed using the relationship:

$$q = \bar{k} \cdot h \cdot \frac{n_f}{n_d}$$

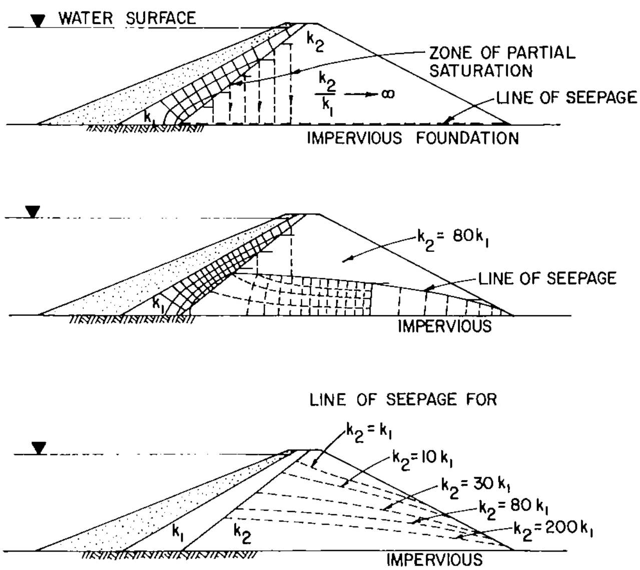
where $\bar{k} = \sqrt{k_{min} \times k_{max}}$ = effective coefficient of permeability

The higher the ratio of horizontal permeability to vertical permeability the higher the phreatic line and the larger the zone of saturation in the dam. The effects of increasing the ratio of $\frac{k_h}{k_v}$ from 1 to 9 is illustrated by the heavy dashed line on Figures 5 to 8 inclusive. It should be noted that the small toe drain shown on Figure 6 was inadequate for the case of $k_h/k_v = 9$ and consequently the phreatic line emerges on the downstream face of the dam.

The inclined drain (chimney drain) is one method of drainage that can be used to effectively intercept seepage, where the ratio of horizontal to vertical permeability is high and foundation drainage cannot effectively prevent saturation of a large part of the downstream section of the dam. Figure 9 illustrates such a drain.

<u>Seepage Reduction Features</u> - In those instances where the foundation and/or embankment soils are highly pervious, large seepage losses may develop unless some method for reducing seepage is incorporated into the design. For embankments, upstream impervious zones or central impervious cores are normally used for this purpose. Figure 10 (4) illustrates the effect of the relative permeabilities of the upstream impervious zone and downstream shell on the position of the phreatic line in the downstream shell, assuming no drains are installed at the foundation contact. The addition of a strip or blanket drain would draw the phreatic line down as shown on Figure 11 and would greatly improve the stability of the downstream shell.

For pervious foundations some type of foundation cutoff is normally used. If the pervious zone is not excessively thick, a cutoff trench, excavated without bracing and subsequently backfilled with compacted impervious soil, provides

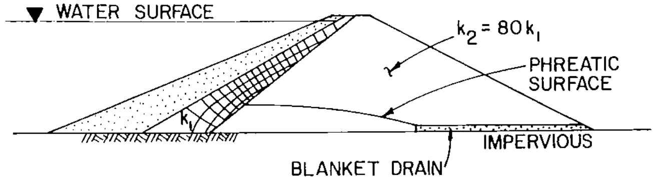


EFFECT OF RELATIVE PERMEABILITY OF UPSTREAM SLOPING CORE AND DOWNSTREAM ZONE ON POSITION OF LINE OF SEEPAGE IN DOWNSTREAM ZONE

(AFTER CEDERGREN – REFERENCE 4)

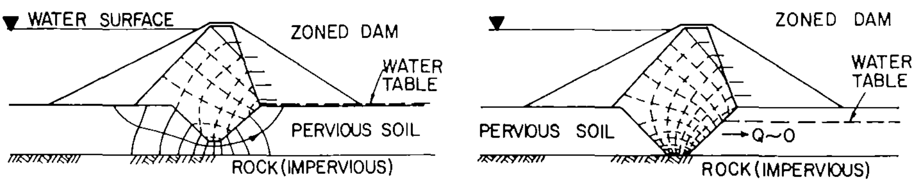
FIGURE 10

a positive water barrier. Where the pervious zone is very thick some other method of cutoff must be used. Such methods include: slurry trench cutoffs, cement and bentonite cutoffs, and concrete cutoffs. To be effective, foundation cutoffs must thoroughly penetrate the pervious strata. Partial penetration of the pervious strata by the foundation cutoff may not significantly reduce the seepage. Figure 12 (5) shows the relative effectiveness of partial cutoffs. This figure also illustrates the high seepage gradient that develops along the base of the cutoff and on its downstream face in both the foundation and embankment zones. To prevent the possibility of piping developing at these locations, suitable filters must be provided.

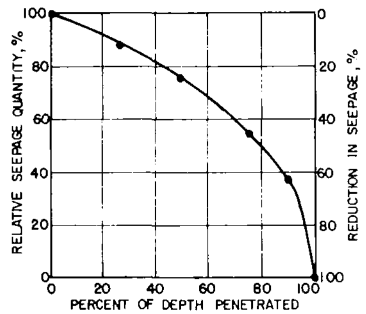


USE OF BLANKET DRAIN TO CONTROL PHREATIC LINE OF FIGURE 10

FIGURE 11



(a) PARTIAL CUTOFF (b) COMPLETE CUTOFF



(c) RELATIONSHIP BETWEEN DEPTH OF CUTOFF AND SEEPAGE FLOWS

(AFTER CEDERGREN- REFERENCE 5)

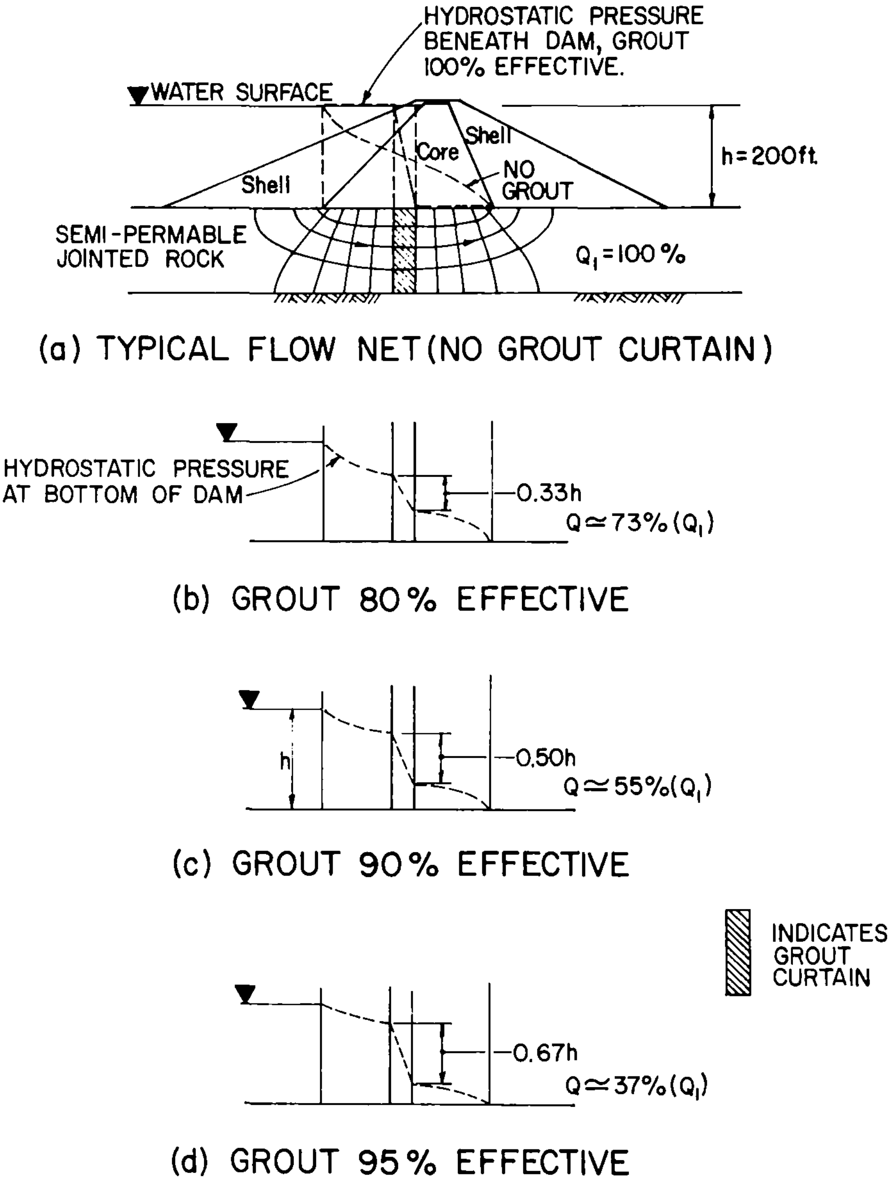
FIGURE 12

Where the pervious foundation is bedrock, grout curtains are often used to seal the pervious rock. Grouting is not a foolproof method of providing a cutoff in the bedrock because it is extremely difficult to ensure that all cracks and fissures have been intersected and filled with grout. Past experience has shown that even very thorough grouting of bedrock foundations has not always produced the desired results (3). Where the upper few feet of bedrock is fissured and cracked, surface treatment combined with shallow blanket grouting is often used to seal the pervious rock. The surface treatment usually consists of slush grouting or shotcreting all the surface cracks. Blanket grouting is usually accomplished by drilling shallow (20 ft) grout holes on a 10 ft x 10 ft grid and grouting them up under low pressure. The purpose of the surface treatment of the bedrock is to adequately seal all cracks and fissures that exist beneath the core and transition filters to protect against the possible loss of soil into such fissures. Another important detail is the provision of suitable filters and drains between the earthfill dam and the bedrock downstream of the grout curtain.

Figure 13 (5) shows the relative effect of different degrees of grout curtain effectiveness, on piezometric pressures under a dam. The figure also indicates how different degrees of grout curtain effectiveness influence the seepage flows. For example, even if the grout curtain is 90% effective, the seepage through the grout curtain is 55% of the quantity that would occur with no grout curtain. As the effectiveness of a grout curtain must always be somewhat suspect, a second line of defence should always be provided. Normally, this second line of defence is drainage. In the example shown in Figure 13, generous strip or blanket drains would be provided along the foundation contact immediately downstream of the core to collect and remove seepage flows that pass through the grout curtain.

In the previous example which is taken from Cedergren (5) the grouted zone was assumed to be 30 ft wide. A 90% effective grouted zone is assumed to have a permeability of one-tenth that of the ungrouted rock; if grouting is 95% effective, the permeability of the grouted zone is one-twentieth that of the ungrouted rock.

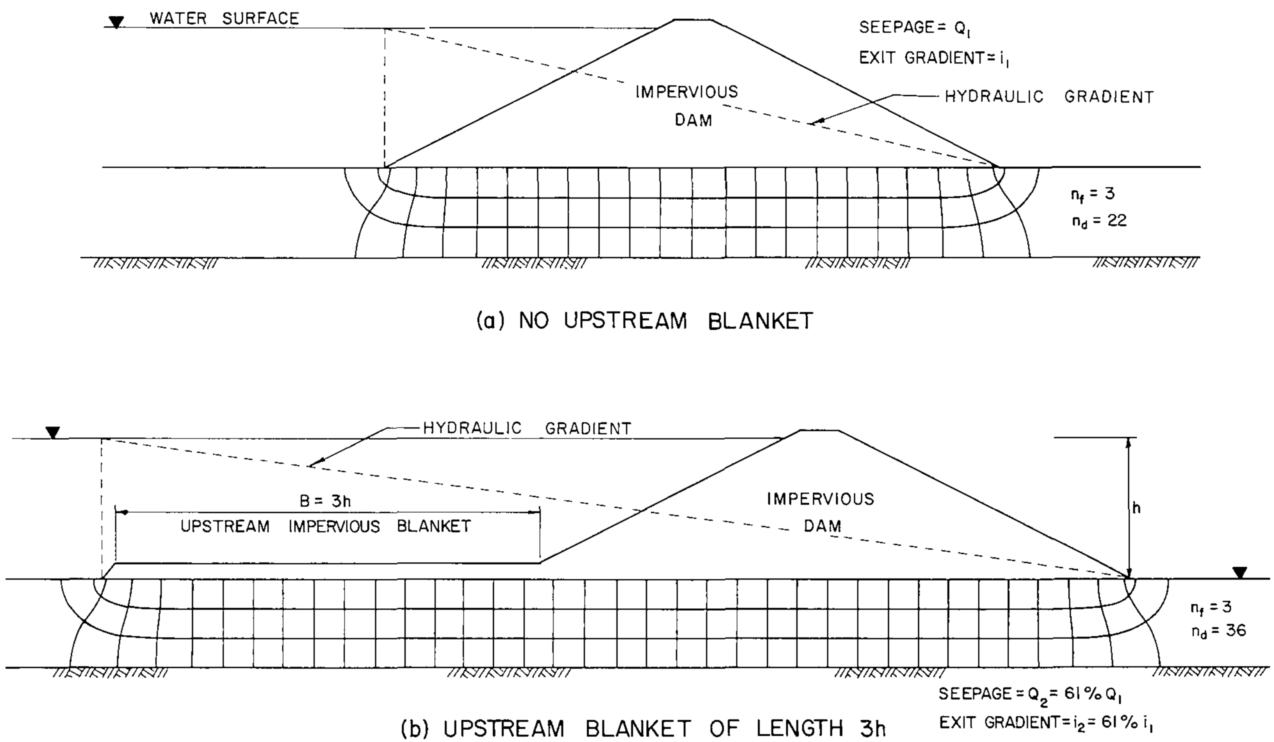
Upstream impervious blankets are often used, as an alternative to installing cutoffs, to reduce seepage through pervious foundations. Upstream impervious blankets serve to increase the length of the seepage path through the foundation, thereby reducing both the seepage flows and the hydrostatic pressures within the foundations beneath the dam. The greater the distance that the impervious blanket is extended upstream, the lower both the seepage flows and the hydrostatic pressures under the dam. Figure 14 presents flow nets for a homogeneous impervious dam, resting on a pervious foundation, with and without an upstream impervious blanket. The beneficial effects of the upstream impervious blanket in reducing seepage flows and uplift pressures is indicated on the figure. The critical area where seepage and heaving may occur is at the downstream toe of the dam. Drainage facilities such as a downstream blanket drain and/or downstream toe drain normally would be installed in this area to collect the seepage and protect against heaving and/or piping.



EFFECTIVENESS OF GROUT CURTAIN ON
HYDROSTATIC PRESSURE AND SEEPAGE

(AFTER CEDERGREN—REFERENCE 5)

FIGURE 13



FLOW NET ILLUSTRATING THE EFFECT OF AN UPSTREAM IMPERVIOUS BLANKET ON HYDROSTATIC PRESSURES AND SEEPAGE FLOWS

FIGURE 14

IV. SEEPAGE THROUGH TAILINGS DAMS

1. General

The conventional tailings dam is constructed from the coarser fraction of the tailings and utilizes a broad spigotted beach of fine tailings as its water barrier. As previously pointed out, tailings dams fall into two broad categories, based on their method of construction. These are:

a) Upstream Method of Construction - the dam is raised by moving upstream over top of previously deposited tailings. Tailings excavated off the spigotted beach are often used for this purpose although cycloned sand or borrow materials may also be used. (Reference Figure 1).

b) Downstream Method of Construction - the dam is raised by moving downstream over a prepared foundation, usually including underdrains. Cycloned sand is often used as the construction material although borrow materials may also be used. (Reference Figure 2).

Both the upstream and downstream types of tailings dam normally rely on a spigotted beach of fine tailings to provide a water barrier. Although such beaches are relatively impervious, some seepage still occurs. The amount of seepage depends on the physical properties of the fine tailings, the height of the dam, and the width of the spigotted beach. For a given tailings dam, the wider the spigotted beach, the more effective the impervious barrier it provides, as the length of seepage path is increased and the phreatic line through the dam is lowered. Conversely, the narrower the spigotted beach, the greater the seepage losses through the dam and the higher the phreatic line.

Obviously, seepage control measures are required to control the seepage through the dam and prevent the phreatic line from emerging on the downstream face of the dam. For all downstream methods of dam construction this problem is easily handled using conventional drainage methods. Moreover, in those cases where the initial starter dam must be made impervious, to store mill start-up water, and a foundation cutoff is required, the cutoff for the starter dam also becomes the cutoff for the ultimate tailings dam.

For upstream methods of tailings dam construction, seepage control is a little more difficult. Ideally, the starter dam should be pervious so that it performs as a toe drain for the ultimate dam and keeps the phreatic line inside the dam. Upstream drains might be combined with the pervious starter dam to further improve underdrainage. Where the starter dam is required to be impervious for the storage of mill start-up water, other methods must be used to control seepage through the ultimate tailings dam. One procedure that can be used is to design the starter dam in such a manner that its upstream section provides the impervious membrane and the downstream section acts as a drain to provide a safe exit for the phreatic line. Another, more complex method of drainage would be to provide an underdrainage system upstream of the starter dam, collect the seepage in pipe drains, and carry outlet pipes through the dam to discharge the seepage water. The outlet pipes would have to be valved so that the mill start-up water would not flow out through the system prior to the deposition of fine tailings. Two potential disadvantages of such a system, are the risks involved in running pipes through the tailings dam over the entire life of the operation (i.e. corrosion, collapse of pipes, settlement damage, piping along pipelines, etc.) and the difficulties involved in ensuring that the upstream drainage system remained effective over the entire life of the operation and did not become clogged with fine tailings.

In the writer's opinion, the downstream methods of tailings dam construction are preferable to the upstream methods because the downstream methods lend themselves to sound engineering analyses and design. Consequently, tailings dams designed and constructed by these methods can be built to whatever standards are necessary to satisfy any particular site conditions and/or regulatory requirements. In recent times, use of downstream methods of tailings dam construction has become increasingly popular as tailings dam heights have increased and better engineered designs are required to ensure safe structures. Presently, great emphasis is being placed on the environmental aspects of tailings storage, with the requirement that seepage flows be carefully controlled and, in many instances, not allowed to leave the pond area and enter the surrounding surface or groundwaters. Under these conditions, the downstream methods of construction allow greater flexibility in selecting the most suitable seepage control procedures for any given set of conditions. The remainder of this paper deals with seepage control measures as applied to downstream methods of dam construction.

2. Seepage Control Measures

Quantity of Seepage - The first step in the design of seepage control measures for a given tailings dam is to estimate the quantities of seepage water that the system will be required to handle. Generally, there are five sources of seepage water during the construction of most tailings dams. These sources are:

i) The free water in the tailings pond.

ii) The construction water associated with dam building. (This may be cyclone underflow or the water used for transporting the sand for a hydraulic fill).

iii) The water from the spigotting operation used to form the impervious beach.

iv) The consolidation water squeezed out of the slimes as they consolidate in the pond.

v) Precipitation falling on the tailings dam.

The quantity of seepage to be expected from the free water in the tailings pond can be estimated using approximate flow nets based on the anticipated width of beach and the permeabilities of the materials involved. Estimating the quantity of seepage due to spigotting is much more difficult, however, a reasonable approximation can be made by assuming that the spigotting completely saturates the beach so that in effect the free water in the pond extends to near the top of the spigotted beach. A flow net drawn for this condition should provide an estimate of the combined seepage, due to both the free water in the pond and the effects of spigotting.

Estimating the quantity of construction water that the drains must handle varies from a simple exercise for the case of on-dam cycloning, where all the construction water seeps into the downstream sand dam, to a complex exercise for the case where large volumes of hydraulic fill, transport water flow across the dam and then exit either upstream into the tailings pond or downstream behind the seepage recovery dam. For the on-dam cycloning, all the water contained in the cyclone underflow is assumed to reach the underlying drain. This value can be estimated with reasonable accuracy from the density of the underflow and the recorded tonnage of sand placed each day. For the case of hydraulic fill placement, where large volumes of water flow across the sand dam, the seepage loss into the dam may be estimated by assuming downward seepage under a hydraulic gradient of 1 and using the expression:

$$q = \bar{k}\ i\ A$$

where q = rate of vertical seepage

A = total area over which the water is flowing

i = hydraulic gradient (1 for this case)

$\bar{k}$ = effective coefficient of permeability = $\sqrt{k_h \times k_v}$

The fourth potential source of seepage, consolidation water squeezed out of the slimes as they consolidate, is more difficult to quantify. In effect, this action adds an additional increment of pore pressure to the normal hydrostatic pore pressure that would exist in the slimes if they were completely consolidated under their own weight.

Mittal and Morgenstern (14) examine this problem and suggest a method for estimating the resulting combined seepage flows. However, when the writer attempted to apply this procedure to a particular tailings dam, the result obtained appeared to be appreciably higher than the actually measured values. Apart from the uncertainties in assessing the effective permeability of the slimes, another possible explanation for this difference is that the excess pore pressures due to consolidation exist only in the fine slimes, which are located out in the pond hundreds of feet from the dam. Consequently, most of the excess head is used up in flowing through the low-permeability, spigotted beach before reaching the free-draining, sand dam, drainage face.

In arid climates, seepage due to precipitation falling on the dam is generally a very minor item. However, in high rainfall climates precipitation can be a major contribution and may cause the sand dam to be almost fully saturated during long periods of heavy rainfall.

Of the above five outlined sources of seepage water, construction water from cyclone underflow or hydraulic fill placement operations is usually several times greater than all other sources combined. In designing the drains, the highest probable seepage flows that can enter the drains should be used and the drains should be assigned their lowest probable permeabilities and gradients. This is essential, as once constructed and buried, the drains must continue to perform satisfactorily throughout the life of the structure and, in many cases, for many years after the mining operation is completed. The author strongly recommends that all drains should be sized to handle flows several times the largest probable flows computed using the above outlined methods.

Closed Circuit Tailings Ponds - Closed circuit tailings ponds are required when pollution regulations forbid the discharge of tailings effluent from the tailings pond. This means that all water entering the tailings pond, which includes surface runoff as well as the tailings transportation water, must be stored in the pond and recycled through the mill. In those instances where the tailings effluent is considered toxic, seepage losses through the dam, its abutments and foundations, and the tailings basin itself, are required to be reduced to minimal values. These requirements can place severe limitations on the tailings dam design, construction, and operation and invariably add appreciably to the costs of the structure.

As previously discussed, conventional tailings dam design maintains a wide, spigotted beach in front of the dam. The wider this beach, the further the free water in the pond is kept from the sand dam. This tends to lower the phreatic line (at least in areas where the beach is not saturated by spigotting), and reduce seepage flows through the dam. However, for closed circuit operations where spring runoff and/or large rainstorms must be stored in the pond, large fluctuations in pond levels may occur. Such fluctuations may flood the beach and place the free water surface against the sand dam. As spigotted beaches commonly have gradients in the order of 1 percent, a 5 ft increase in pond levels would flood a 500 foot wide beach. Correspondingly, a 10 ft rise in pond levels would place 5 ft of free water against the sand dam. This would cause large seepage flows through the dam with the inherent risk of flow concentration and resulting piping due to internal erosion of the fine tailings sand. Such action can be extremely dangerous as it could lead to failure of the dam.

Obviously, the water balance in a closed circuit tailings pond must be carefully monitored and sufficient freeboard must be maintained to safely store floods from spring runoff or rainstorms. Where a spigotted beach is used as the upstream impervious membrane, it must be wide enough to absorb the expected maximum flood surcharge without having the free water surface rise up against the sand dam. If this cannot be achieved, then some type of impervious zone must be placed on the upstream face of the sand dam to prevent the development of excess seepage flows. This problem is usually most severe in the first few years of operation when the storage capacity of the pond is low and large fluctuations in pond levels are required to store flood flows. This situation may require the use of an impervious zone on the upstream face of the dam during the first few years of operation, reverting back to a spigotted beach as the pond capacity becomes large enough to absorb the floods with acceptable fluctuations in pond level.

The other aspect of closed circuit tailings pond design that requires special attention is the requirement that seepage losses out of the system must be minimized. This usually requires the inclusion of some seepage reduction features and a seepage recovery dam. In addition, a well-designed underdrainage system is required, which, of course, should be provided for all tailings dams.

Seepage Reduction Features - Minimizing seepage losses from the main tailings dam usually requires the inclusion in the tailings dam design of one or more of the seepage reduction features covered previously for conventional water storage dams. Where seepage losses must be minimized, some type of positive cutoff is required for the dam foundations and abutments. The simplest and most positive procedure is to excavate a cutoff trench through the pervious foundation and abutment soils and backfill the trench with compacted impervious soils. If a high water table and/or a great thickness of pervious soils make this method impracticable, then a slurry trench or some similar deep cutoff installation might be required. The cutoff is connected to the impervious zone of the starter dam and the ongoing construction of the tailings dam continues as normal.

Where the foundations and/or abutments are pervious bedrock, grouting may be required to seal off the rock beneath the impervious zone of the starter dam. The grouting program usually involves one or more of three basic types of treatment. These are:

Surface Treatment - used to seal up fissures and joints in the exposed rock surface. Slush grouting or shotcreting is normally used for this purpose.

Blanket Grouting - used to seal shallow fissures and joints that extend below surface of the rock. Grouting, using low pressures and holes drilled to shallow depth (20') and spaced on a grid at about 10' centres, is normally used for this purpose.

Curtain Grouting - used to seal pervious rock to great depths. One or more lines of deep drill holes are grouted under high pressure until no more grout is accepted by the rock. This is a complex and expensive foundation treatment and is not normally used with tailings dams.

Surface treatment of fissured and jointed bedrock foundations is very important where large hydraulic gradients are likely to develop across the impervious zone. Open cracks and fissures in the bedrock may permit the flow of water under high heads, which in turn can erode the core of the dam and cause piping to develop. One of the secondary protections against such piping is the provision of suitable filters and adequate drains downstream of the impervious core.

Downstream Seepage Recovery Dam - A seepage recovery dam is required immediately downstream of the main tailings dam to collect and store, for ultimate pumping back into the tailings pond, all seepages that emerge downstream of the tailings dam. The seepage recovery dam is designed as a conventional water storage dam and must have impervious foundations and abutments to ensure that none of the intercepted water escapes. Where the natural foundations and abutments are pervious, positive cutoffs must be provided. These may include such devices as: excavated cutoff trenches, slurry cutoff trenches, grout curtains in broken rock, etc. In the case where pervious foundations exist to great depth below the dam, such that it is not practicable to install a positive impervious cutoff, a hydraulic barrier, such as illustrated on Figure 3, might be installed downstream of the seepage recovery dam. However, this is not a desirable solution and every effort should be made in selecting a closed circuit tailings dam site, to find a location where a positive cutoff can be constructed under the seepage recovery dam.

The major source of seepage flows into the seepage recovery dam is normally the construction water used to build the main, sand tailings dam. Lesser contributors are precipitation, and seepages through the dam and its foundations and abutments. Where relief wells are required to relieve hydrostatic pressures under the main tailings dam, the flows from these wells are also collected behind the seepage recovery dam.

The seepage recovery dam is the last line of defence against seepage losses from the tailings pond facility. Groundwater quality monitoring is normally carried out immediately downstream of the seepage recovery dam to assess the effectiveness of the seepage control features. Such water quality monitoring should be established well in advance of the start of operations of the tailings dam to establish the natural base levels for the groundwater. As natural groundwaters often contain what might be considered unallowably high concentrations of metals, fibres, or other undesirable elements, it is essential that the content of the natural groundwaters be clearly established before tailings storage starts. In the event that ongoing water quality observations showed that contamination was occurring downstream of the reclaim dam, further seepage control measures, such as pumped drain wells and/or injection wells would have to be considered.

Storage of tailings from uranium mining operations pose special problems (24) that are beyond the scope of this paper The major item of concern is the movement of radionuclides in seepage water, escaping from the tailings pond. The regulatory agencies tend to favour licensed repositories such as used for powerplant and nuclear weapons waste. The mining companies tend to favour unlined tailings ponds. In between these two extremes are such methods as:

1) Disposal in mine workings

2) Disposal in lined and capped reservoirs

3) Double lined tailings ponds

4) Single lined ponds with monitoring systems and some form of backup.

As is the case for all tailings disposal problems, disposal of radium tailings is a site specific problem, the solution for which is likely to vary from site to site. However, there is little doubt that one of the most critical factors to be considered at all sites is the control of seepage flows containing radionuclides.

3. Seepage Problems

Water related problems probably cause the greatest concern, during the life of a tailings dam. The most serious problem of this type is the possibility of overtopping. This, of course, cannot be allowed to happen as it likely would result in failure of the tailings dam with subsequent loss of tailings and water. Consequently, if such a threat develops, it must be corrected by raising the dam and/or reducing the inflow of tailings and runoff into the pond. In the extreme case, where neither of these measures can be carried out in time to stop overtopping, an emergency spillway is required. Although emergency spillage must be considered a last resort, particularly where discharge from the tailings pond is considered to be a pollutant, it is a far better alternative than overtopping the dam and causing a large and sudden uncontrolled discharge of tailings and effluent.

Next to the threat of overtopping, seepage problems generally pose the most serious concern during the operating life of a tailings pond. As previously discussed, seepage through a tailings dam may give rise to three basic types of problem. These are:

a) Piping, which occurs when seepage flows remove fine soil particles.

b) Slope instability and heaving, which occur when excess hydrostatic forces, due to seepage flows, develop.

c) Large Water Losses, which may contribute to a) and b) above, in addition to causing downstream pollution.

Piping is considered to be one of the most serious problems that can develop in a tailings dam, as, in the extreme, it could result in total failure of the dam. As previously discussed, properly designed drains and filters are the protections against piping. Drainage controls the phreatic line and prevents it from emerging on the downstream slope of the dam (Reference Figs 5 and 7). Drainage also reduces seepage pressures and exit hydraulic gradients. The lower the exit hydraulic gradient the less the likelihood of piping occurring. Filters permit the escape of seepage water, but prevent the movement of soil particles. All drains should be designed to act as filters.

One design feature, common to many of the older tailings dams, was the use of decant towers with discharge lines running through the base of the dam to a downstream pumphouse. As previously discussed for drainage pipes, dam designers avoid, whenever possible, passing conduits through the dams. The reason is that they represent a risk, as a potential source of seepage and piping problems. This risk is even greater for tailings dams which are constructed of easily erodible tailings. Seepage collars around the pipelines do not guarantee their safe performance and may, in fact, give the designer a false sense of security. Poor backfilling procedures with or without seepage collars can lead to piping failures. Other factors which may have an adverse effect on pipelines passing through dams are: corrosion and ultimate collapse of the pipeline, collapse of the pipeline under high fill loads, and the pulling apart of pipe connections owing to large settlements and/or strains in the foundation soils.

Decant towers may also fail, although the risk of this occurring for a well-designed tower is probably less than that associated with the possible failure of a decant pipeline passing through the dam. Factors which may affect decant towers include: large negative skin friction forces caused by the settling tailings, shear movements in the tailings, and ice forces during spring breakup. For these reasons, some procedure other than the use of decant towers, with discharge lines running through the dam, should be employed for reclaiming water from the pond. A floating or movable pumphouse, located near the shore of the tailings pond, is a suitable method now in common use.

Figure 15 is a photograph illustrating piping that developed at the contact between a sand tailings dam and its abutment. The piping developed when a large spring runoff caused the tailings pond to rise, drown the beach, and come in contact with the upstream face of the sand dam. The rise in tailings pond levels had been predicted a year earlier and the dam had been provided with an upstream impervious zone of soil. Unfortunately, in the area where the piping occurred, careless spigotting had eroded away a small portion of the upstream impervious protection. Seepage occurred at this location, which happened to be near the contact between the sand dam and the abutment. The seepage flows concentrated along this contact, which was an impervious boundary, and the observed piping began to develop. Fortunately, the operators were aware of the seriousness of the problem and implemented repairs as soon as the seepage flows were noted. Repairs consisted of dumping impervious fill over the upstream face of the dam to block off the seepage entrance and filling of the downstream erosion area with sand and gravel filter material. The repairs were successful and no further piping occurred.

FIGURE 15 - PIPING AT DOWNSTREAM FACE OF SAND TAILINGS DYKE.

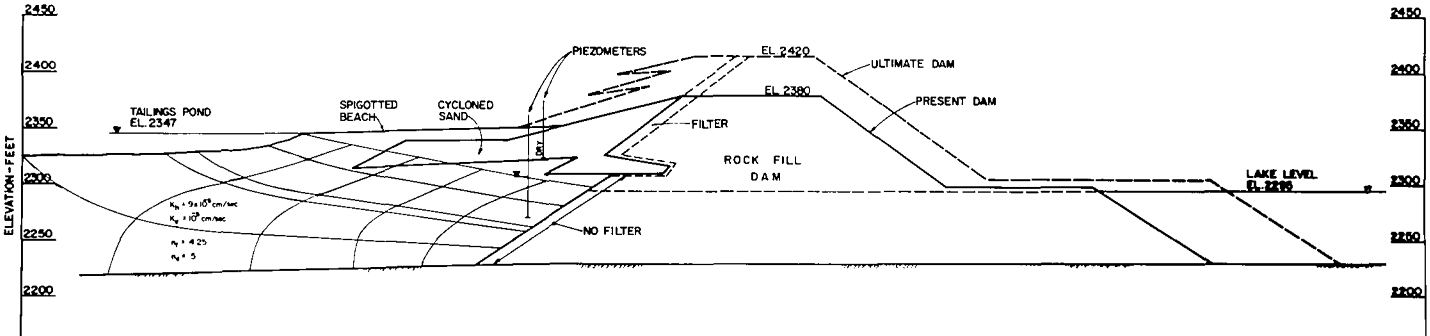
Figure 16 presents a section through a tailings dam constructed from mine waste rock. The dam was built in two stages. Stage 1 involved constructing a rockfill to close off a small bay on a large lake. The rockfill, which was deposited through a maximum of 80 ft of water, extended a few feet above lake level. Tailings were then discharged into the bay from the top of the rockfill and water was reclaimed from the pond by pumping. The operation proceeded smoothly until the top surface of the tailings emerged from the pond and a tailings beach developed. Concurrently, the water levels in the pond rose slightly above those in the adjacent lake. At this time, a sudden piping failure occurred and a significant volume of tailings and effluent flowed through the rockfill dam and discharged into the lake, much to the chagrin of the regulatory agency which was concerned about possible damage to fish life.

The cause of the failure was obvious, no filter had been provided on the upstream face of the rockfill dam. Consequently, as soon as water levels in the pond exceeded those in the lake, seepage flows developed towards the lake. These flows carried the fine tailings into the voids in the rockfill and the resulting piping failure occurred. A significant feature of this failure is that it occurred under a differential head of less than 1 ft.

Repair of the failure has involved pushing the free water surface in the pond as far back as possible from the rockfill dam. This was accomplished by placing a wide zone of cycloned sand over the existing spigotted beach. The intent of pushing back the pond and widening the beach was to lower the phreatic line and reduce the exit hydraulic gradients at the face of the "unfiltered" rockfill. It was also thought that the cycloned sand zone would help the situation by acting as a drain.

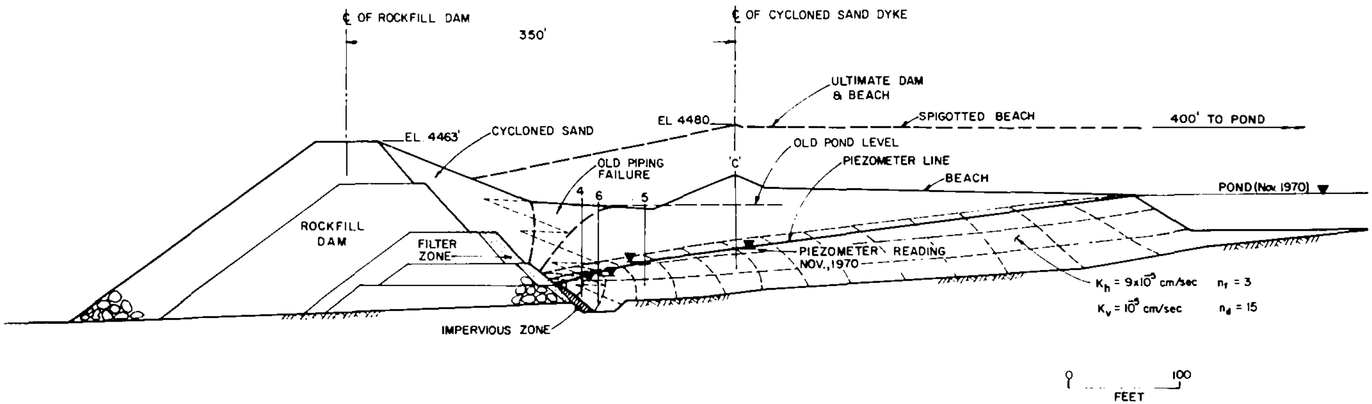
These remedial measures have been successful and the pond has now been raised to its ultimate height without further incident. Piezometric readings, taken when the dam and pond had reached the elevations shown on Figure 16, indicate that the phreatic line is below most of the cycloned sand zone. An approximate flow net is also shown on Figure 16. This flow net, which was drawn using the permeability parameters indicated on the figure, agrees reasonably well with the observed piezometric pressures, and confirms that pushing the pond away from the face of the dam has greatly reduced the exit hydraulic gradients at the face of the rockfill.

Figure 17 presents a section through a tailings dam constructed of mine waste rock, which suffered a piping failure in its early years. Also shown on the figure are the remedial measures undertaken by the owner and their effects on the piezometric line. An approximate flow net is also presented on this figure. Reasonable agreement is obtained between the observed piezometric pressures and those predicted by the flow net.



SECTION THROUGH ROCKFILL TAILINGS DAM SHOWING FLOW NET

FIGURE 16



SECTION THROUGH REPAIRED TAILINGS DAM SHOWING MEASURED PIEZOMETRIC LINE AND ESTIMATED FLOW NET

FIGURE 17

The original design called for the placement of protective filters on the upstream face of the rockfill dam. Cycloned sand was to be placed over the filters and a spigotted beach was to be developed beyond the cycloned sand zone. The intent was to operate the pond in such a manner that a generous beach would be maintained at all times between the free water surface and the face of the dam.

Despite these good intentions the tailings dam suffered a sudden piping failure which caused the loss of large volumes of water and fine tailings. Approximately 10,000,000 gallons of water were discharged and a peak flow of 48,000 gallons per minute was reached. The sudden discharge caused considerable property damage downstream. A subsequent investigation into the failure indicated that:

1) The filter zones for the second and third lifts of rockfill were end-dumped from the top of the lift rather than placed in thin layers. No filters were placed on the fourth and fifth lifts of rockfill.

2) At the failure area the tailings beach was poorly developed with the free water in the pond close to the upstream face of the dam.

3) Sinkholes and fluctuations in seepage flows through the dam had occurred in this area at least twice during the summer prior to failure, indicating seepage was carrying the fine sand into the rockfill dam. The operators had filled these sinkholes with tailings and noted that seepage flows decreased and that no tailings were passing through the dam. This was interpreted to indicate that no problem existed.

The owners' and their consultants' solution to the problem was to reduce the possibility of further piping occurring by pushing the free water surface in the pond well back from the dam. This was done by building a second dyke across the tailings pond and then building a spigotted beach off the upstream face of the dyke as shown in Figure 17.

The remedial measures have worked well and no further piping problems have developed. The piezometric levels shown in Figure 17 were measured approximately one year after the failure had occurred. Since then the tailings dam has been raised to its full height without further difficulty and is currently being prepared for abandonment.

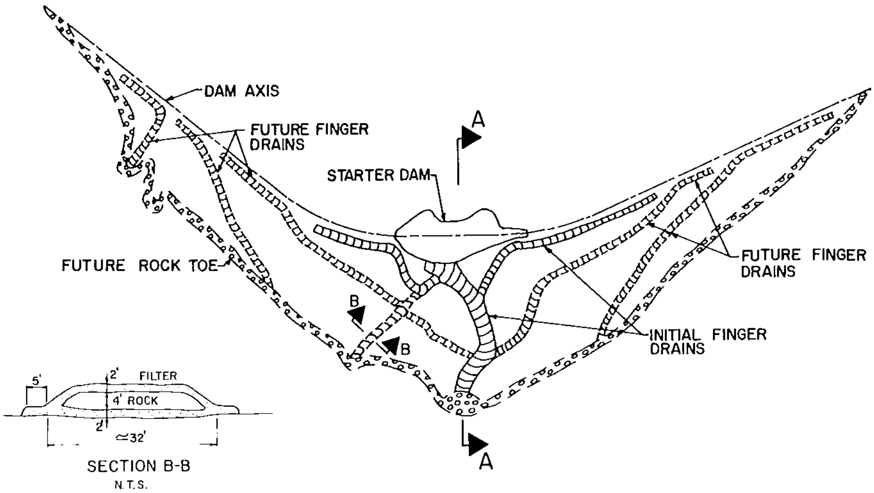
4. Case Histories Illustrating Modern Tailings Dam Designs

The seepage control features required for any given tailings dam must be based on a study of the particular requirements of the project. Obviously, these requirements will vary from project to project and will depend on such items as: foundation conditions at the site, materials to be used for construction of the dam, requirements for the starter dam (i.e. does it have to store start-up water for the mill), method of dam construction, and requirements to be satisfied for the pertinent regulatory agencies. To illustrate some of the seepage control methods currently in use several tailings dams have been selected for brief discussion.

Gibraltar Mines Ltd - Gibraltar Mines (10) is situated in central British Columbia, about 40 miles north of the town of Williams Lake. The mine is a low-grade copper, open-pit operation and has a capacity in the order of 40,000 tons per day. The present tailings facility will provide for storage of about 220,000,000 tons of tailings.

The Gibraltar tailings pond is located in a separate valley about 4 miles north of the mine and mill area. A plan view of the ultimate tailings dam, showing the layout of the extensive finger drain system, is presented on Figure 18. These finger drains, which constitute the underdrainage system, consist of a central, highly-pervious, quarried rock core, suitably protected by a surrounding filter zone. The drains are conservatively designed to handle flows several times larger than the maximum anticipated seepage. A typical section through a finger drain is also shown on Figure 18.

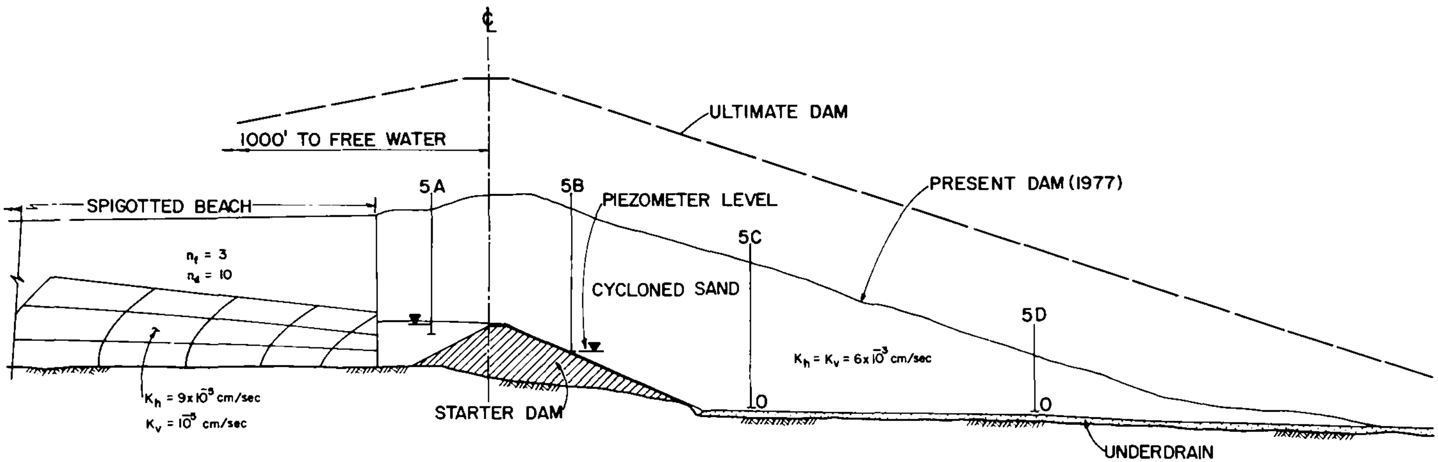
When completed, the dam will have a maximum height above stream bed of approximately 400 ft, and a crest length of 8,000 ft. Initially a small, 100-ft high, impervious, starter dam was constructed on the centreline of the ultimate sand tailings dam. The dam is being constructed from cycloned sand, using the centreline method of construction, which is a variation of the downstream construction method. The tailings are cycloned on the dam, using portable cyclones which are supported on skid-mounted, steel towers. The sand underflow from the cyclones is deposited directly onto the dam and assumes a natural slope that generally ranges between 3.5 and 4 to one. The transportation water in the "ropy" sand underflow, which equals approximately 500 gallons per minute, seeps downward into the underlying drainage system and is collected downstream behind a seepage recovery dam. The overflow from the cyclones is spiggoted to form a beach upstream of the sand dam. This beach, which is in the order of 1000 ft in width, provides the upstream impervious zone for the dam. The Gibraltar tailings dam is not operated as a fully, closed circuit, tailings facility, as some effluent discharge is permitted downstream of the seepage recovery dam.



PLAN VIEW-GIBRALTAR TAILINGS DAM FIGURE 18

A section through the Gibraltar tailings dam, at its point of maximum height, is presented on Figure 19. Also presented on this figure are the piezometric readings that exist at this section. From an examination of these readings it can be seen that piezometric levels are extremely low and in effect are located in the underdrains, downstream of the starter dam. The cycloned sand has a relatively high permeability and is sufficiently free-draining that the construction water from the cyclone underflow does not appear to appreciably affect the piezometric levels beyond the small area being filled. The wide spigotted beach at this site is very beneficial in reducing seepage from the tailings pond.

Also shown on Figure 19 is an approximate flow net, drawn up using the indicated permeability parameters. Using the approximate flow net, the seepage loss through the dam, for the present section is computed to be approximately 80 gallons per minute. As this dam has been shut down by strikes for almost a year it presents a unique opportunity to compare the computed seepage with the actual



SECTION A-A

GIBRALTAR DAM-MAXIMUM SECTION AT STATION 36+00 SHOWING MEASURED PIEZOMETRIC LINE AND ESTIMATED FLOW NET

FIGURE 19

measured seepage, with no contributions from construction water to be concerned about. In this case the average measured seepage is a value which compares reasonably well with the computed value.

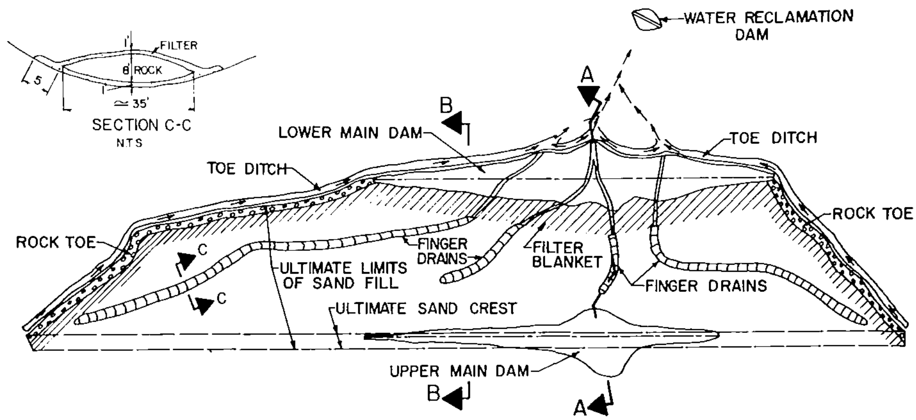
One of the requirements of the Gibraltar tailings dam design is that it be able to withstand moderate earthquake shocks without risk of failing by liquefaction. The usual protective measures against liquefaction are either to compact the sand to a high density or to prevent it from becoming saturated. At this site, prevention of saturation by means of good drainage was the protection selected. Recent analyses, based on dynamic testing of samples of the cycloned tailings sand (11) confirm that the well-drained, sand dam will safely withstand a maximum earthquake having a Richter Magnitude of 7.0, and located 25 miles from the site.

Brenda Mines Ltd - Brenda Mines (10) is situated on a mountain plateau west of Okanagan Lake in South Central British Columbia approximately 40 miles from Kelowna, B.C. The mine produces copper and molybdenum concentrates from a low-grade, open-pit operation with a capacity of approximately 28,000 tons per day. For a planned mine life of 20 years approximately 200 million tons of tailings must be safely stored.

The mine is situated at the head waters of a stream flowing eastward into Okanagan Lake. Because the Okanagan Valley is one of the major tourist and recreational areas of Southern British Columbia, it was made a basic requirement for development of the mine that the tailings facilities be completely closed circuit.

The valley in which the tailings dam and tailings pond are situated has a steep gradient and is relatively narrow, requiring a high dam to provide the necessary storage volume. The dam was designed originally to rise 400 feet above the stream bed. Recent modifications in design have increased that height to approximately 500 feet above the stream bed.

A plan view of the ultimate tailings dam is presented on Figure 20. Also shown on this figure is the layout of the finger drain, underdrainage system and a typical section through one of the drains. As was the case at the Gibraltar tailings dam these drains, which consist of a quarried rock core, surrounded by a filter zone, are very conservatively designed and can handle several times the maximum estimated seepage flows.



PLAN VIEW- BRENDA TAILINGS DAM FIGURE 20

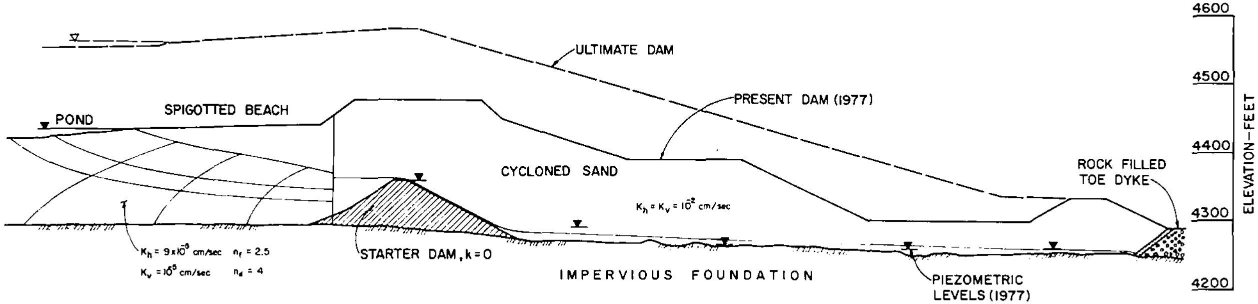
The dam will have an ultimate crest length of approximately 7,000 feet, a maximum base width of approximately 1,800 feet, and a maximum height above the downstream toe of about 500 feet. This dam is also being raised by the centreline method of construction (a type of downstream construction) which produces a vertical upstream face of interfingered cycloned sand and slimes. The final downstream sand slope will be approximately 3.5 horizontal to 1 vertical. Total sand requirements will be approximately 32,500,000 cubic yards.

An impervious starter dam, having a maximum height of 125 feet was constructed on the centreline of the ultimate sand tailings dam. The starter dam, which was constructed of rockfill with an upstream impervious zone and foundation cutoff trench, was used to store mill start-up water. At the downstream toe of the ultimate tailings dam a 175 ft high rockfill toe dam was constructed. This large toe dam provides confinement to the lower portion of the sand tailings dam and serves to retain the embankment so that it could be sited next to the edge of a steeply dropping section of the valley. The tailings dam is being constructed from cycloned sand. The tailings are cycloned in a building located high on the left abutment and the sand underflow transported to the dam by sand line. The cycloned sand and water are deposited on the dam in large cells, following procedures similar to those used for placing hydraulic land fills. Large volumes of construction water are associated with the sand placement, this water seeps vertically downwards and into the underdrainage system. The volume of water involved is in the order of 2,000 gallons per minute. This water, which is collected by the underdrains is discharged behind the seepage recovery dam from where it is pumped back to the mill for re-use. The overflow from the cycloning operation is spigotted off the upstream face of the dam to provide a wide beach between the free water in the pond and upstream face of the sand dam. This beach provides the impervious upstream zonefor the dam.

A typical section through the Brenda Dam, outside of the maximum central gully section, is presented on Figure 21. Also shown on this figure are the piezometric levels that exist at this section. An examination of this figure indicates that the underdrainage system is working effectively, with the phreatic surface being at the base of the sand fill and controlled by the finger drains. The large volumes of water used for placing the cycloned sand have some effect on piezometric levels, causing them to rise temporarily in the immediate vicinity of the filling operation.

Also shown on Figure 21 is an approximate flow net, drawn up using the indicated permeability parameters. Using this flow net, the estimated seepage loss through the dam is computed to be approximately 85 gallons per minute. This value is considered to be of the right order of magnitude and is very small compared to the volume of construction water that the drainage system must handle. Even if one assumes that the spigotting operation saturates the entire beach, the computed volume of seepage only increases by approximately 20%.

Figure 22 presents a section through Brenda Dam at its highest point, where it crosses the central gully section of the valley. It will be noted that water levels at this section are higher than those shown for the adjacent section. There are thought to be three reasons for this situation. First, during the early days of dam construction, it is believed that the filters surrounding the large finger drain became partically fouled by silty construction water, which concentrated in this low area. Second, as this was the lowest spot on the sand dam the finer sands and silt sizes were concentrated in this area. Third, this is an area that tends to collect foundation seepage water from the adjacent higher areas, in addition to those seepages attributable to the small lake of construction water which continuously ponded on the sand surface as the dam was raised. The combination of only partially effective underdrainage, an accumulation of finer silty sands, and an excess supply of construction

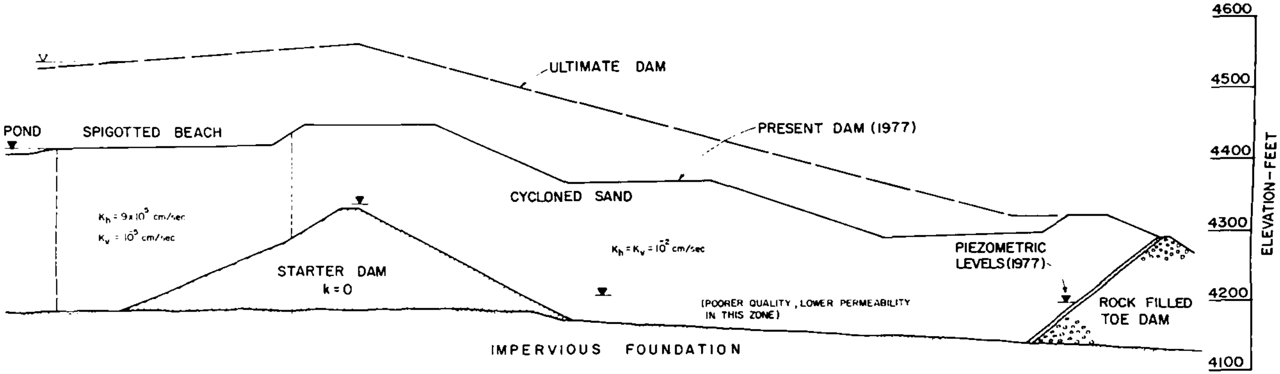


SECTION B-B

BRENDA DAM — SECTION AT STATION 38+00

SHOWING MEASURED PIEZOMETRIC LINE AND ESTIMATED FLOW NET

FIGURE 21



BRENDA DAM — MAXIMUM SECTION AT STATION 50+00

SHOWING MEASURED PIEZOMETRIC LINE

FIGURE 22

water is believed to be the cause of the observed higher water levels in this area. Once construction of the dam is completed, it is anticipated that the observed water levels will drop down close to the level of the drains.

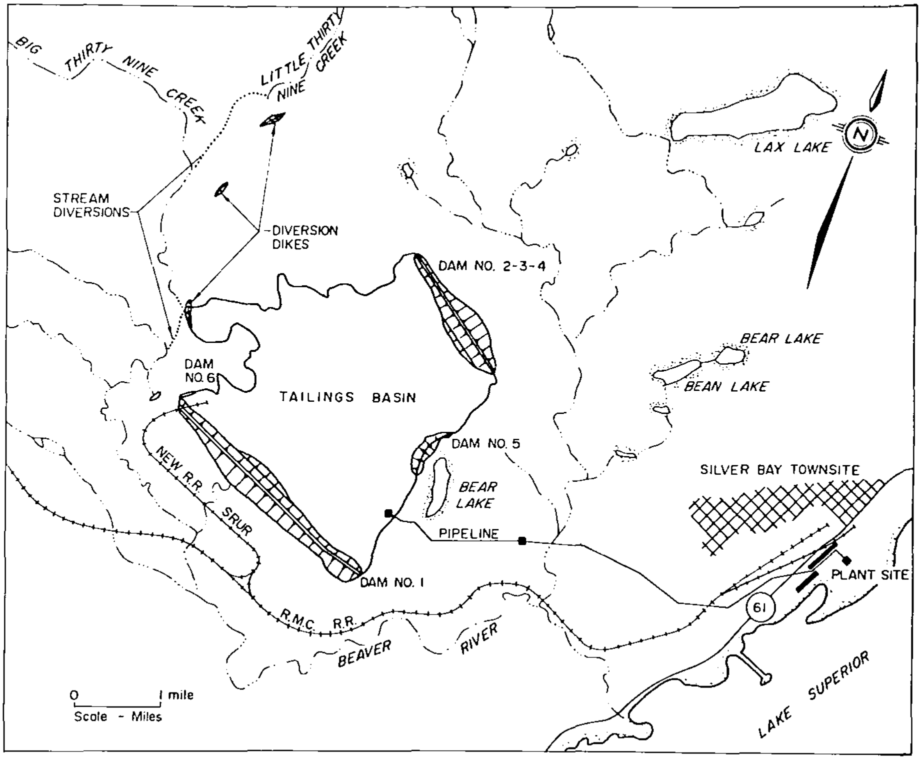
Brenda like Gibraltar, is designed to withstand moderate earthquake shocks. Also, like Gibraltar, the protection against liquefaction under modest earthquake shock has been to keep the downstream, sand shell well drained. Recent analyses confirm that the design will safely withstand the maximum probable earthquake anticipated for this area. (Richter Magnitude 6.5 at 15 miles from the dam).

At both the Gibraltar and Brenda tailings dams, the most critical period is during construction, when large volumes of construction water must be handled by the underdrainage systems. Once construction of the dams is completed and planned reclamation works push the free water in the ponds far back from the dams, the volumes of water seeping through the dams will be small and the downstream shells will be dry.

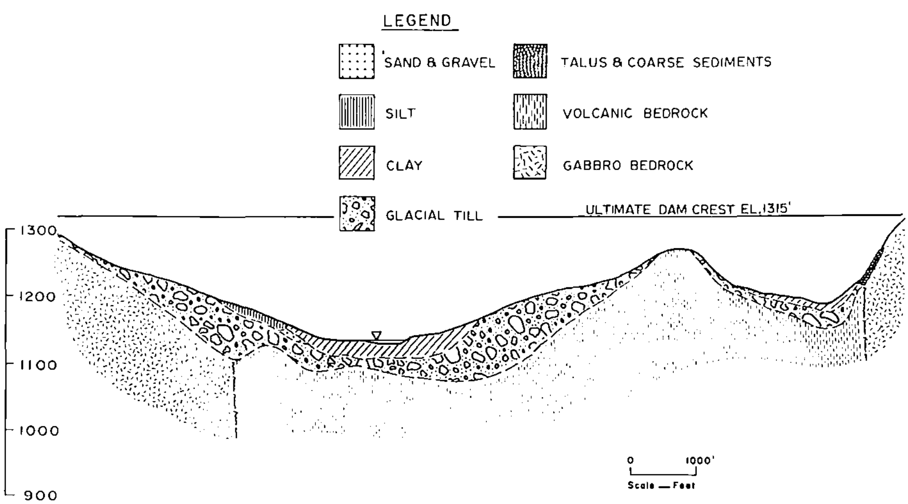
Reserve Mining Company - Reserve Mining Company (8) operates a large taconite, open-pit operation located at Babbitt, Minnesota on the Mesabi Iron Range. The crushed taconite is shipped by rail to Silver Bay, Minnesota where it is concentrated and pelletized into iron ore pellets. The tailings, along with 415,000 gallons per minute of water, are discharged from launders to Lake Superior. At full production 88,500 long tons per day of crude taconite are processed at the Silver Bay plant. Of this, about 29,500 tons become iron ore pellets and 59,000 tons become tailings.

Controversy over Reserve's practice of discharging tailings into Lake Superior had its beginnings in 1969. This controversy extended through numerous court actions. On June 1, 1977, Reserve began construction of its on-land tailings disposal facility at Mile Post 7. A general plan of the tailings disposal area is presented on Figure 23.

A very serious aspect of Reserve's tailing disposal problem is the existence of asbestos-like fibres in the tailings. A fibre is defined as a particle with a three to one aspect ratio. It is alleged by the regulatory agencies that these asbestos-like fibres are similar or identical to amosite asbestos. Asbestos fibres were claimed to be a health hazard because of their alleged link with cancer when inhaled. It was further alleged that these asbestos-like fibres were also a health hazard when ingested with water. As a consequence



GENERAL PLAN OF TAILINGS DISPOSAL FACILITY
RESERVE MINING FIGURE 23



GEOLOGICAL SECTION THROUGH DAMSITE NO. 1
-RESERVE MINING FIGURE 24

of these allegations and the actions of the courts, Reserve Mining is legally required to provide a watertight tailings pond, operated as a closed circuit system, so that no asbestos-like fibres escape into the surface or groundwaters of the area. Reserve is also required to provide stringent dust control measures to prevent loss of fibres into the atmosphere.

From the outset, all dams were to be constructed by the downstream method, using the coarse tailings (minus 3/4" material) as a major construction material. Originally, Reserve proposed placing the excess coarse tailings, not required for dam building, west of the tailings pond. Also, the upstream faces of the tailings dams were to be sealed with a beach of fine tailings, a procedure conventionally followed for most tailings dam designs. However, the State objected to both the stockpiling of waste coarse tailings above the pond and the use of an exposed tailings beach to provide the upstream impervious facing for the dam. Their concern was the dust, allegedly containing potential cancer-causing, asbestos-like fibres, that might emanate from both these sources. Reserve's proposal to vegetate the waste piles and keep all tailings beaches wet by spraying when inactive was not acceptable to the MPCA. Consequently, in order to minimize any dust problem, Reserve agreed to:

- eliminate the tailings beaches by depositing all fine tailings underwater.

- eliminate the coarse tailings stockpile by also placing all coarse tailings underwater in the tailings pond.

- maintain essentially zero visible dust emisions from the dams and basin by minimizing the size of working areas and using dust suppressants on all inactive areas.

The stringent regulations imposed by the regulatory agencies concerning possible water losses from the tailings basin, coupled with the requirement that all tailings be disposed of underwater, made seepage control a major concern for this project. Fortunately, the tailings basin is relatively watertight as can be seen from Figure 24, which presents a section through the basin. The tailings dams are designed as water retention dams with a compacted upstream impervious zone and a compacted coarse tailings (minus 3/4" material) downstream zone. The starter dam is also an impervious water storage dam, which stores up to 30 ft of start-up water, under whose surface the first fine tailings will be stored. A typical section through the largest dam is presented on Figure 25.

Some of the seepage reduction features which have by necessity been built into the dam designs are:

1) Compacted, impervious, glacial till membrane on the upstream face of the dam.

2) Foundation cutoff trench carried into impervious glacial till or clay.

3) Slush grouting or shotcreting of rock surface under the impervious zone of the dam and the filters.

4) Blanket grouting on the rock abutments

5) At least 1 row of deep grout holes on the rock abutments.

The basic drainage features include:

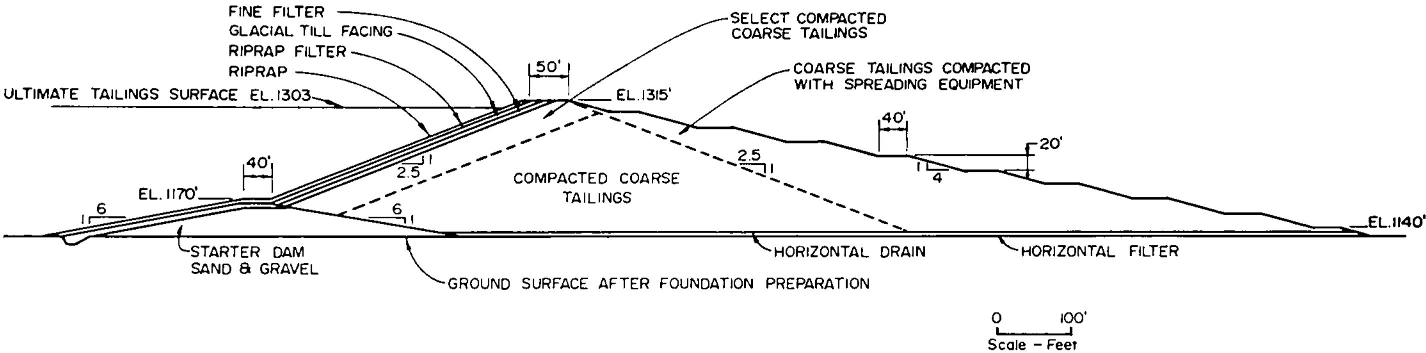
1) Filter and drain zones downstream of upstream impervious zone.

2) Foundation blanket drain downstream of starter dam.

3) Relief wells across the downstream toe of the dam.

In addition, seepage recovery dams are provided downstream of each structure. As the anticipated seepages will be inconsequential, the main function of these dams is to collect surface runoff water which has flowed over the tailings dam. The state considers such water to be polluted as the dams are constructed of tailings.

Extensive stream diversions have been necessary to control the volumes of water that must be stored in the closed circuit tailings pond (Reference Figure 23). All diversions have had to be designed to handle the maximum probable storm. Similarly, the tailings pond has also been designed



TYPICAL SECTION THROUGH DAM NO. I - RESERVE MINING

FIGURE 25

to handle a maximum probable storm and still maintain a safe freeboard. Water reclaim is by floating, pump barge, having a maximum capacity of 10,000 gallons per minute.

Piezometers for measuring the seepage pressures that develop in the foundation soils downstream of the dams have been installed. Also installed are wells for monitoring water quality.

GCOS Tar Island Tailings Dyke - Our last example, which is taken from previously published reports (6,15,16), illustrates a case where internal drains have been used to control the phreatic line. The dam is being constructed of sand, using hydraulic placement procedures. The hydraulically placed sand is compacted to a high density, using vibratory compaction equipment. At this particular project the tailings pond contains a great depth of water, which is in contact with the upstream face of the sand dam. Under these conditions a fairly high phreatic line would be expected to develop through the dam under steady seepage conditions. A further factor, which adds to the seepage from the pond, is the transportation water used to place the sand on the dam. Locally, where hydraulic fill placement is underway, the phreatic line is raised by the added seepage due to this construction water.

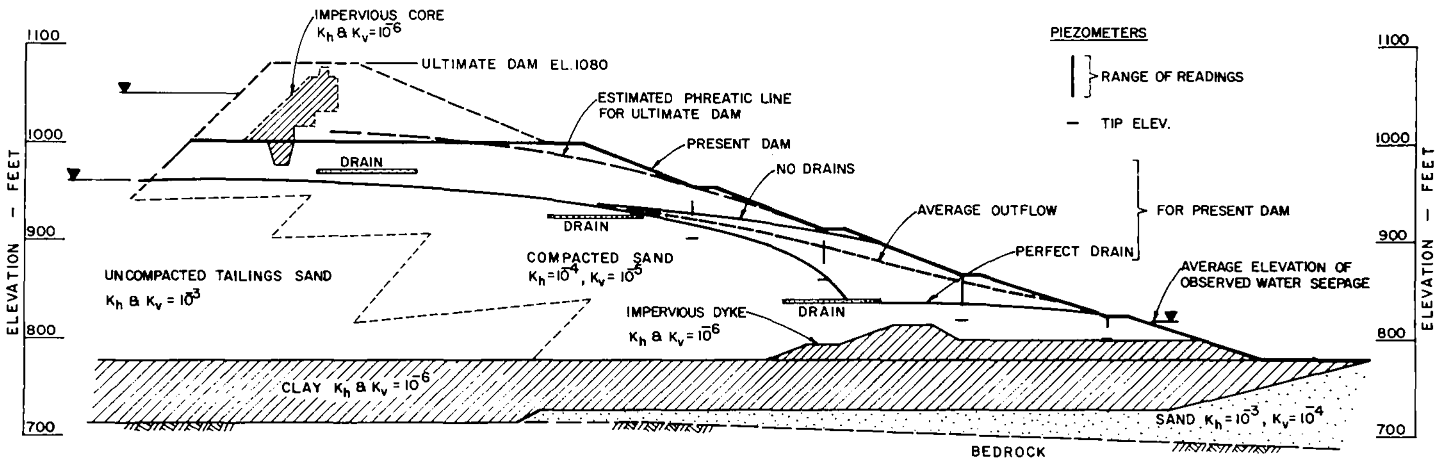
Seepage analyses (16) using finite element procedures, were made for a large number of conditions. According to these analyses the existing internal drains require a greater capacity to increase their ability to draw down the phreatic line. The three phreatic lines shown on Figure 26 indicate where the analyses place the phreatic line for the three assumed conditions of:

a) no internal drains

b) perfect lower internal drain

c) existing drain flows

The phreatic line obtained using the existing drain flow conditions provides a fairly good fit with the observed piezometric readings. The higher piezometric pressures that are measured in the vicinity of the second internal drain were considered to reflect the hydraulic fill operations in this area. Using the same parameters as were used for this "best-fit" condition an attempt was then made to predict where the ultimate phreatic line might occur. Extrapolated



TYPICAL SECTION THROUGH
G.C.O.S. TAR ISLAND TAILINGS DAM
(AFTER MORGENSTERN AND KAISER–REFERENCE 20)

FIGURE 26

drain flows, based on past observations, were used in carrying out the computations. The predicted location of the final phreatic line is also shown on Figure 26. Although such predictions should be considered as approximate estimates only, because of the large number of variables which can affect such an extrapolation, they are nonetheless considered to be valid indicators of the trend which will develop as the dam is raised.

In this example, the dense sand embankment is considered stable, even though it has a relatively high phreatic line. The only point of concern is the probable development of local sloughing, slumping, and minor soil erosion at points where concentrated seepage might develop in the downstream slope of the dam during construction. Filters will be placed at such locations to control seepage and prevent piping from developing.

CONCLUSIONS

1. Seepage control is a critically important aspect in the design, construction, and operation of tailings dams. Uncontrolled seepage can lead to such problems as piping, slope instability and heaving, and excess water losses.

2. Seepage control methods developed for conventional water storage dams are directly applicable to controlling seepage flows through tailings dams. However, they must be suitably modified to satisfy the specific requirements of any given tailings dam design.

3. Pollution control regulations can have a major effect on the seepage control facilities required for any given tailings dam. In those instances where no discharge of effluent is allowed into the downstream ground or surface waters, extensive seepage reduction features may be required. These measures may materially add to the costs of the tailings storage facilities.

4. Downstream methods of tailings dam construction allow the greatest flexibility for selecting the most suitable seepage control measures required to satisfy any given set of site conditions and/or regulatory agency requirements.

5. Observational data from several operating tailings dams indicate that flow nets may be used to estimate seepage flows and to determine the location of the phreatic surface

through the beach and tailings dam with sufficient accuracy for most design purposes. The greatest unknown is the effective permeability of the tailings beach and the writer recommends that the maximum probable values for both the permeability and the permeability ratio (k_{max}/k_{min}) be used for all computations. This procedure will produce conservatively high values for both the seepage quantity and the location of the phreatic line.

6. Drains should be designed using the highest probable seepage flows that can enter the drains and the lowest probable permeability for the drains themselves. All drains should be sized to handle flows several times the largest value computed on the above basis. This philosophy is considered essential to handle such unknowns as: highly pervious foundation zones not found during design, development of cracks in an impervious zone, "windows" in a grout curtain, drainage from consolidation slimes, high pond levels which flood the slime beaches, loss of drain's capacity with time, due to plugging with fines or precipitation of salts, etc. For tailings dams constructed using on-dam-cycloning or hydraulic fill procedures, the largest volumes of seepage that must be handled by the drains comes from the transportation water.

7. Drainage is particularly important for tailings dams located in areas of high seismic risk. Loose, saturated tailings are subject to liquefaction under earthquake shocks. The basic protective measures against liquefaction are compaction and drainage. For tailings dams located in areas of low to medium seismic risk, drainage of the downstream shell will in most instances provide adequate protection against liquefaction. For tailings dams located in areas of very high seismic risk, both drainage and compaction are considered necessary.

8. Seepage control measures unquestionably add to the total costs of the tailings storage facilities. However, they greatly increase the overall safety of the tailings dams and when properly designed and constructed satisfy both the safety and pollution control requirements of the regulatory agencies. The additional costs associated with constructing adequate seepage control measures should be considered a necessary part of the cost of building and operating a mine in today's society.

ACKNOWLEDGEMENTS

The data presented in this paper has been drawn from many sources, a major source being our clients, to whom we extend our thanks for use of the data developed on their projects. The writer also wishes to acknowledge the assistance given him to several members of his staff who helped gather the data presented in this paper. In this regard, special acknowledgement is extended to Robert C.Y. Lo who carried out the analyses required for drawing up the several flow nets, and to Eric D. Pharey who prepared all the figures.

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