Planning, Design, and Analysis of Tailings Dams

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Preface to Second Printing

Time is a stern judge, and this is no less true of books describing engineering than of the constructed works themselves. The reader of this second printing deserves an accounting of the verdict rendered by the near-decade that has passed since preparation of the original manuscript.

In a time dominated by information technology and the public perceptions it produces, there is less tolerance for the effects of mining and less weight given to the value of its metals. The general desire for a risk-free environment is nowhere more strongly expressed than through concern for environmental effects of both existing and abandoned tailings impoundments. The great majority of new ore deposits recently exploited worldwide have been for precious metals, with their attendant issues of cyanide control. In this regard, more emphasis is now being placed on pre-treatment and destruction of cyanide prior to discharge into tailings impoundments through a variety of new treatment processes. Many if not most milling operations for precious metals are now accompanied by heap leach extraction operations, and experience gained with leach pad liners is increasingly transferred to tailings pond liners, with emphasis on synthetic materials.

Inactive tailings deposits are receiving similar attention. Once situated in remote settings, many such long-abandoned deposits are now surrounded by urban encroachments. Windblown dispersal and potential ingestion of tailings solids containing heavy metals, arsenic, or selenium now demand considerable attention, albeit from such technical specialities as meteorological modelling and medical toxicology beyond the scope of this book. The problem of groundwater contamination also remains. Trends toward adoption of more specific and quantitative regulatory groundwater quality criteria at once clarify the problem and complicate its solution. Nevertheless, major advances have been made in treatment of acid drainage through biological methods including artificial wetlands. It is interesting now to observe that many times more technical effort is devoted to remedial groundwater and toxicological studies for these abandoned deposits than was ever allotted for their original design or operation. Similarly, for cases such as uranium, costs for permanent stabilization of abandoned tailings deposits are sometimes orders of magnitude greater than the value of all ore ever produced.

All this is not to say that understanding of more classical geotechnical matters has remained static. It is now recognized that consolidation of tailings slimes cannot be accurately addressed by Terzaghi theory and instead requires the application of finite-strain consolidation theory. This opens the door to more reliable prediction of pore pressures for stability evaluations involving slimes. Perhaps the most dramatic advances have occurred in evaluation of seismic stability, particularly in the understanding of tailings behavior following the onset of liquefaction. These advances have occurred in part through study of flow failure case histories, some of these being for tailings dams, and they promise to bring about better prediction and control of the destructive potential for tailings dam flow failures, seismic or otherwise.

If these trends toward increasing environmental and structural safety were merely due to shifts in public perception, they might be easily dismissed, but they are now enforced by what may be the most subtle yet significant change since this book first appeared; the increase in accountability through legal liability and litigation to which neither mine operator, design professional, nor regulator is immune. This makes it even more important that all aspects of mine waste disposal be firmly grounded in defensible and accepted standards, for environmental practices as well as engineering design. The fact that tailings dam failures yet occur - such as at Stava, Italy in 1985 with the loss of 268 lives - suggests that the fundamentals of tailings dam design and operation are still not universally observed. In spite of recent advances, fundamentals of tailings behavior, design practices, and environmental factors set forth in the book, although supplemented remain unchanged. To this extent the verdict of time seems a favorable one. I hope the reader will agree.

Indian Hills, Colorado — *Steven G. Vick*

February, 1990

Preface

The disposal of mine waste, chiefly tailings, has of late assumed an importance that transcends even the massive volumes of materials produced annually by mining operations. From an engineering standpoint, some tailings embankments class among the largest earth structures in the world. Aside from their significance in strictly engineering terms, tailings impoundments receive intense regulatory attention and public scrutiny. Because of the land areas they disturb and the varying toxicities of the mine wastes they retain, tailings impoundments are often the lightning rod for public opposition to mining projects.

Historically, tailings disposal began as the practice of dumping tailings into nearby streams and progressed to empirical design of impoundments by mine operators based on less than satisfactory principles of trial and error. Only within about the past 20 years have the principles of geotechnical engineering been applied to tailings embankments, ordinarily in the context of design practices for water-retention dams. Now, however, planning and design of tailings impoundments has become a multidisciplinary enterprise, one that requires a broader background in many diverse fields extending beyond the traditional application of geotechnical knowledge. This book is intended to provide a bridge between the various technical disciplines involved in tailings disposal and to illustrate the application of these fields to tailings disposal planning and design. In addition, the intent is to provide the reader with access to key sources of literature applying to tailings that heretofore have been scattered among various conferences, symposia, and journals in a wide range of technical fields.

The intended audience for the book falls into three main categories: geotechnical engineers, mining and metallurgical engineers, and regulatory personnel. Knowledge of soil mechanics fundamentals and basic earth dam design principles are assumed on the part of the reader and are well covered in other soil mechanics texts. For the geotechnical engineer, the message is that basic earth dam design criteria and concepts alone are not sufficient for tailings embankment design. To the extent that a dam cannot be designed without a thorough understanding of the materials it retains, geotechnical engineers will find themselves introduced to the metallurgical processes by which tailings are generated, the chemical nature of tailings, and the unique engineering behavior of tailings of various types. Also extending beyond rudimentary geotechnical concepts are the application of slope stability analyses, dynamic analyses, and seepage analyses to tailings embankments, accounting for the differences between tailings embankments and classical water-retention dams.

Because operational responsibility for tailings disposal usually falls to the mill superintendent, metallurgical engineers and to some extent mining engineers will be confronted with the problem of tailings disposal, and probably with little preparatory background in their formal training. Chapters of the book dealing with tailings disposal methods and impoundment layout may be of value in this regard. Mill operators will probably also find themselves subject to the incantations of geotechnical engineers from time to time, and perusal of chapters on analysis and design may be an aid to translation of the seemingly obscure geotechnical liturgy.

When assessing plans for tailings disposal, regulatory personnel will likely be faced with a host of issues outside their technical domain. In the absence of experienced technical assistance, the temptation is to rely strictly on well-known design techniques and design criteria for water-retention dams. However, a recurring theme throughout the book is that there are significant differences between tailings embankments and water-retention dams; any one of a number of disposal methods, embankment types, and design methods may be appropriate. Determining which is acceptable is a matter of ensuring that the disposal method and impoundment design are compatible with the specific site conditions as well as the physical and chemical nature of the particular type of tailings. Chapters treating the topics of tailings effluent chemistry, assessment of alternatives, seepage, liners, and reclamation will likely be of particular interest to regulatory personnel.

The organization of the book loosely follows the chronology of tailings impoundment planning and development, from conception to reclamation. Chapters 1 and 2 introduce the physical, chemical, and engineering nature of tailings, factors that provide the background essential to all subsequent disposal planning. Chapters 3, 4, and 5 present topics related to tailings disposal methods and impoundment siting, including factors related to surface-water hydrology, while Chapter 6 introduces concepts for systematic evaluation of alternative tailings disposal methods and sites. Progressing through the design process, Chapters 7, 8, and 9 detail methods for impoundment design and analysis that stem from classical geotechnical procedures. Chapters 10 and 11 address tailings impoundment seepage and methods to reduce it. Finally, Chapter 12 outlines the application of reclamation concepts to tailings impoundments.

Concepts of tailings disposal have changed at a rapid pace because of advancing technology and developing regulatory concerns. Many of the conclusions and opinions presented in this book will no doubt change under the influence of future advances, accounting for the attempt throughout to present a range of available options rather than to rigidly advocate one or another. To paraphrase Mark Twain, I can therefore ask only for indulgence at the hands of the reader, not justification.

Indian Hills, Colorado
May 1983

STEVEN G. VICK

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Separate mention is required of the contribution of William Highland of Dames & Moore. Mr. Highland contributed Chapter 10 on seepage analysis in its entirety, in addition to providing a sounding board for many of the author's concepts.

S.G.V.

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1

Nature and Production of Tailings

There is nothing so aggravating as silver milling. . . . When there was nothing else to do, one could always "screen tailings." That is to say, he could shovel up dried sand that had washed down to the ravine through the troughs and dash it against an upright wire screen to free it from pebbles and prepare it for working over. . . . Of all recreations in the world, screening tailings on a hot day, with a long-handled shovel, is the most undesirable.

Mark Twain, *Roughing It*

Solid wastes from mining operations are produced in staggering quantity and variety, probably the most well-recognized form being coarse pit-stripping waste rock. The term *tailings* is sometimes used in a generic sense to refer to any mine or mill waste in solid form, including rock-sized stripping waste, underground mine muck, and finely ground mill waste. However, because of the divergent characteristics of these various materials, a more narrowly defined nomenclature is necessary. For purposes of this book, tailings are defined as crushed rock particles that are either produced or deposited in slurry form. This definition encompasses the vast majority of finely ground mill or mineral processing wastes remaining after extraction of mineral values. It does not include pit-stripping waste, underground muck, or such process wastes as fly ash or retorted oil shale that are produced and handled in dry form.

In the mining and extractive metallurgy literature, primary emphasis is on extraction of mineral values from the parent ore. Inasmuch as tailings are simply a waste product, they have long been considered the stepchild of the extraction process and have seldom been treated as a separate entity with distinct and complex physical and chemical characteristics. This traditional but undeserved lack of emphasis on tailings as a unique and important material is changing rapidly because of growing public and technical awareness of mining impacts. Disposal of tailings is commonly identified as the single most important source of environmental impact for many mining projects. Controversy often surrounds siting, design, and seepage impacts of tailings impoundments, and development of some orebodies has been stymied altogether by lack of tailings disposal sites or methods considered suitable by regulatory agencies or the public.

Against this backdrop, the production and physical characteristics of tailings are explored in Chapter 1. The aim is to focus as much as possible on the nature of the tailings and liquid mill effluents produced, rather than on extraction of the mineral values. To the possible consternation of miners and metallurgists, tailings—not mineral values—are treated as the end product of the processing operation, and basic mineral processing techniques are interpreted in this light. The purpose of Chapter 1 is to provide an introduction to the diversity and complexity of tailings and liquid mill effluents. A very basic overview of the way in which tailings are produced is presented, followed by a discussion of transport and water-handling methods. Essential tailings phase relationships are then defined. The physical nature and index properties of various types of tailings are explored, followed by a discussion of the chemical nature of tailings effluents in relation to general health and toxicity considerations.

MINERAL PROCESSING

Fundamental to an understanding of the nature of tailings is a basic knowledge of how they are produced. Extraction of mineral values requires procedures as diverse as the ores processed, but some fundamental steps in the processes are common to many ores.

Central to most ore milling procedures are the initial steps of crushing and grinding. Then concentration of those particles that contain the highest mineral value is usually performed by one of several processes. Separation and removal of mineral values in the concentrate leaves the remaining barren particles as tailings. Optional processes following or supplanting concentration may include leaching or heating. The final stage in the process is recovering excess water from the tailings in preparation for pumping the tailings slurry to the disposal impoundment. These steps are illustrated in Figure 1.1.

Crushing

Crushing is usually performed in stages with the aim of reducing rock fragments from mine-run size to a size that can be accepted as feed to grinding equipment. Jaw and gyratory crushers, as shown in Figure 1.2, are common forms of *primary* crushing equipment and can often accept rock fragments as large as several feet in diameter.

Secondary crushing follows to reduce rock fragments from 10 in. to 12 in. size to about 20 mesh. Gyratory crushers are sometimes used for secondary as well as primary crushing, in addition to hammer mills and cone crushers, as shown in Figure 1.3.

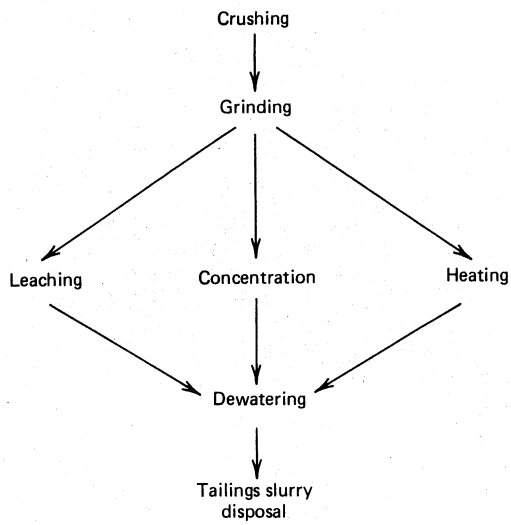


Figure 1.1 Procedures in tailings production.

Grinding

Grinding further reduces size of the fragments produced by crushing. Commonly used grinding equipment includes rod mills and ball mills, shown in Figure 1.4. These units operate by tumbling crushed ore together with heavy steel balls or rods in a rotating drum. Rod mills and ball mills typically produce minus No. 10 and minus No. 40 mesh materials, respectively, and are often operated in series, with product from the rod mills used as feed for ball mills. More modern *autogenous* or *semiautogenous* grinding mills can grind coarser feed material, sometimes directly from the primary crusher, and may replace rod mills.

Grinding represents the final stage in the physical reduction of ore from rock to tailings size. The gradation of the tailings produced will depend on both the degree of particle breakdown produced in grinding of hard rock fragments as well as the clay content in the original ore. Copper tailings, for example, often consist almost exclusively of silicate particles produced by grinding of the parent rock, whereas Florida phosphate tailings primarily reflect the extremely high clay content of the original ore rather than particle breakdown due to grinding.

The optimum degree of grinding is governed by mineral extraction efficiency. If leaching is the primary extraction method, then maximum extraction sometimes favors the high specific surface area of individual particles that accompanies fine grinding. Flotation processes, however, may be adversely affected by grinding to extremely fine sizes. Optimum particle size is determined by detailed metallurgical testing usually at pilot-plant scale.

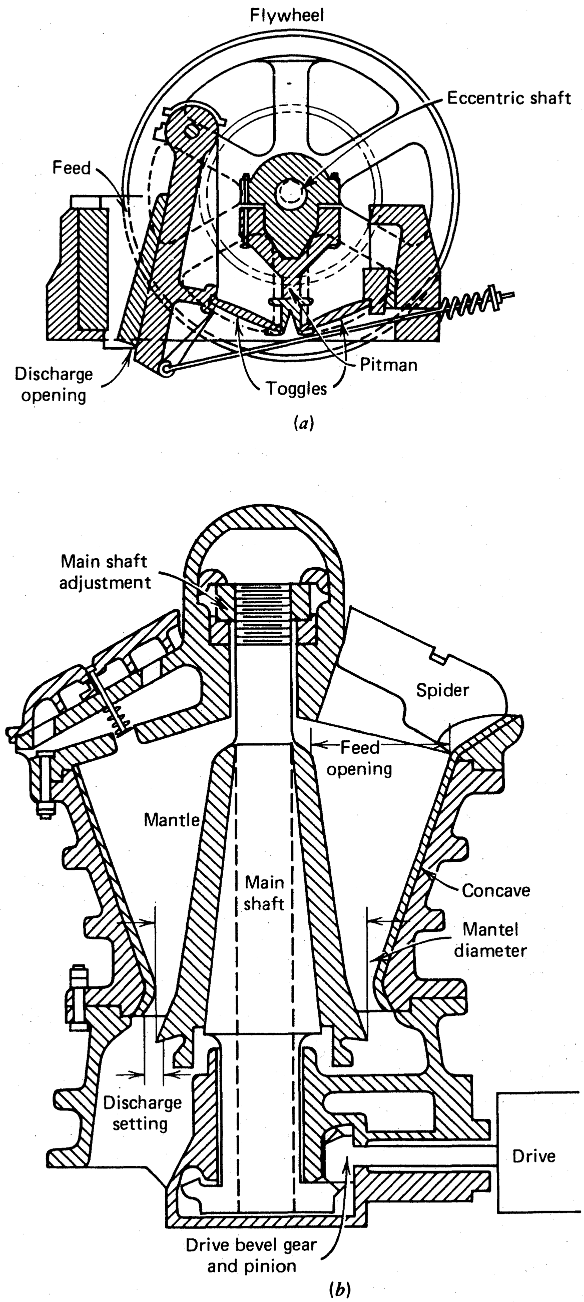
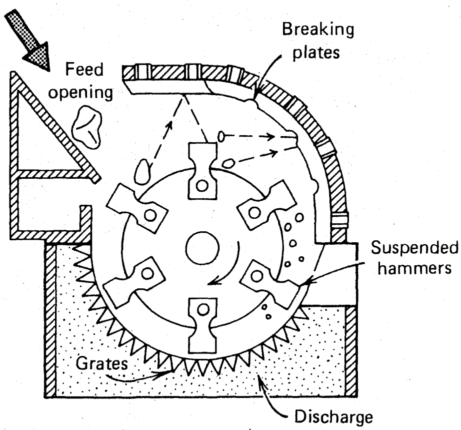
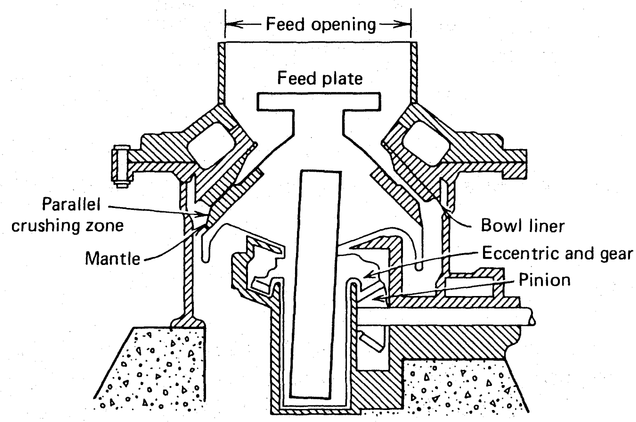


Figure 1.2 Primary crushers. (Reprinted from Cummins and Given, 1973.) (*a*) Jaw crusher. (*b*) Gyratory crusher.



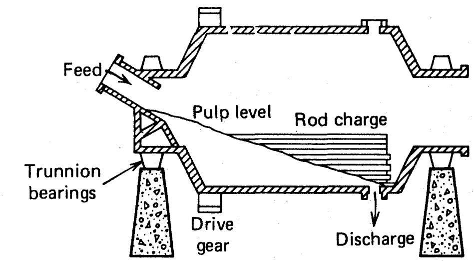
(a)



(b)

Figure 1.3 Secondary crushers. (Reprinted from Cummins and Given, 1973.) (*a*) Hammer mill. (*b*) Cone crusher.

As a result of the severe physical violence inflicted during crushing and grinding, the remaining tailings particles are usually sound and highly angular (Pettibone and Kealy, 1971). Even ground particles extending well into the silt size range show remarkable angularity (Hamel and Gunderson, 1973). The exceptions are ores consisting principally of shale, and those having a high clay content. Tailings derived from crushing and grinding of these ores will have particle shape and hardness reflecting the silt and clay particles in the parent material.



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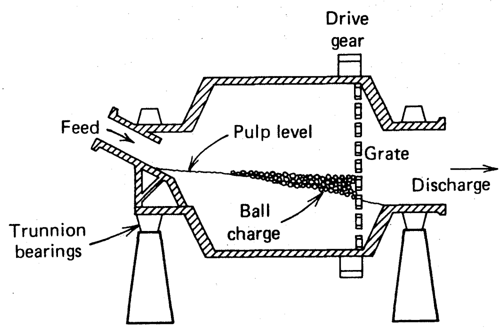


(*b*)

Figure 1.4 Grinding. (Reprinted from Cummins and Given, 1973.) (*a*) Rod mill. (*b*) Ball mill.

Concentration

The individual particles produced by grinding vary in mineral content. The purpose of concentration is to separate those particles with high values (*concentrate*) from those with lower values (tailings). Methods for concentration vary according to ore type, but three general classes are in use: gravity separation, magnetic separation, and froth flotation.

Gravity Separation

Gravity separation requires that the mineral and its host rock have considerably different specific gravities—for example, coal, iron, gold, or tin. Simple sluices and jigs or spiral classifiers can be used to collect the lighter (or heavier) particles, leaving the remaining particles as tailings. Gravity separation is usually performed with water.

Magnetic Separation

Magnetic separation is obviously most useful for extraction of iron from such magnetic ores as magnetite and magnetic taconite. Magnetic particles are collected from a belt or drum separator. Nonmagnetic particles remain as tailings.

Flotation

Froth flotation, by far the most widely used concentration method, is a highly complex physiochemical process. Individual mineral-bearing particles in water suspension are made water repellent and receptive to attachment to air bubbles. Particles with higher mineral content then rise to the surface of a froth. The froth is skimmed, and the remaining barren particles become tailings.

Concentration by froth flotation is the first step in the mineral processing sequence at which chemical reagents are introduced. The use of flotation procedures in a particular operation may therefore be an early indicator of possible problematic chemical constituents in the mill effluent and, ultimately, in the tailings impoundment.

The reagents used are very specific to the type of mineral or minerals being separated in the flotation process, and they vary considerably for different operations. Table 1.1 shows typical classes of reagents and their

Table 1.1 Froth Flotation Reagents

Class	Use	Compound
(1) Collectors	To selectively coat particles with a water-repellent surface attractive to air bubbles	Water-soluble polar hydrocarbons, such as fatty acids
(2) Modifiers		
(a) pH regulators	To change pH to promote flotation, either acid or basic	NaOH CaO Na_2CO_3 H_2SO_4 H_2SO_3
(b) Activators and depressants	To selectively modify flotation response of minerals present in combination	Metallic ions Lime Sodium silicate Starch Tannin Phosphates
(3) Frothers	To act as flotation medium	Pine oil Propylene glycol Aliphatic alcohols Cresylic acid
(4) Oils	To modify froth and act as collectors	Kerosene Fuel oils Coal-tar oils

use in flotation. Cyanide in low concentrations is also sometimes used as a flotation reagent in some cases for some ores—for example, lead and molybdenum.

Production of concentrate is the final stage of many milling operations. Concentrate is shipped to a smelter for refining or, in the case of coal, to the user. Other methods of processing, however, may be used to supplement or replace concentration. These processes, which include leaching and heating, may induce important changes in the nature of the liquid mill effluent.

Leaching

Leaching involves removal of mineral values from the ground particles by direct contact with a solvent, usually a strong acid or alkaline solution depending on the type of ore. H_2SO_4 is the most common acid-leach reagent, and acid leaching of uranium or copper oxide ores typically produces mill effluent discharged with the tailings having a pH in the range of 1–3. The chief alkaline leach reagents are hydroxides and carbonates of sodium or ammonium. Sodium cyanide with lime as a pH modifier is the common leaching reagent for extraction of gold and silver.

Leaching may change the physical characteristics of the tailings. Acid-leach processing at one Wyoming uranium mine, for example, results in conversion of montmorillonite clay minerals in the original ore to predominantly kaolinite in the tailings as a result of calcium-sodium replacement (Highland et al., 1981). Acid leaching of waste rock dumps has reportedly resulted in physical breakdown and alteration of rock fragments, but the extent to which physical breakdown due to leaching may occur in ground tailings particles is probably much less.

Heating

Heating either ground ore or the particle-slurry suspension is used in extraction of some minerals, principally oil from oil sands. Heating, or *calcining,* is also used in the production of phosphoric acid fertilizer from phosphate rock concentrate. Tailings produced are handled in slurry form.

Dewatering

Dewatering is the final milling process stage of significance in tailings production. Somewhat of a misnomer, the term *dewatering* in a mill process context refers not to complete drying of the tailings, but rather to the process of removing some of the water in the tailings-water slurry (*pulp*) following concentration. Recovered water and reagents are recirculated in the mill process for reuse where possible. Recycling of used process water is not feasible in some cases (for example, acid-leach uranium mills) because of the presence of contaminants that would reduce extraction efficiency. In such

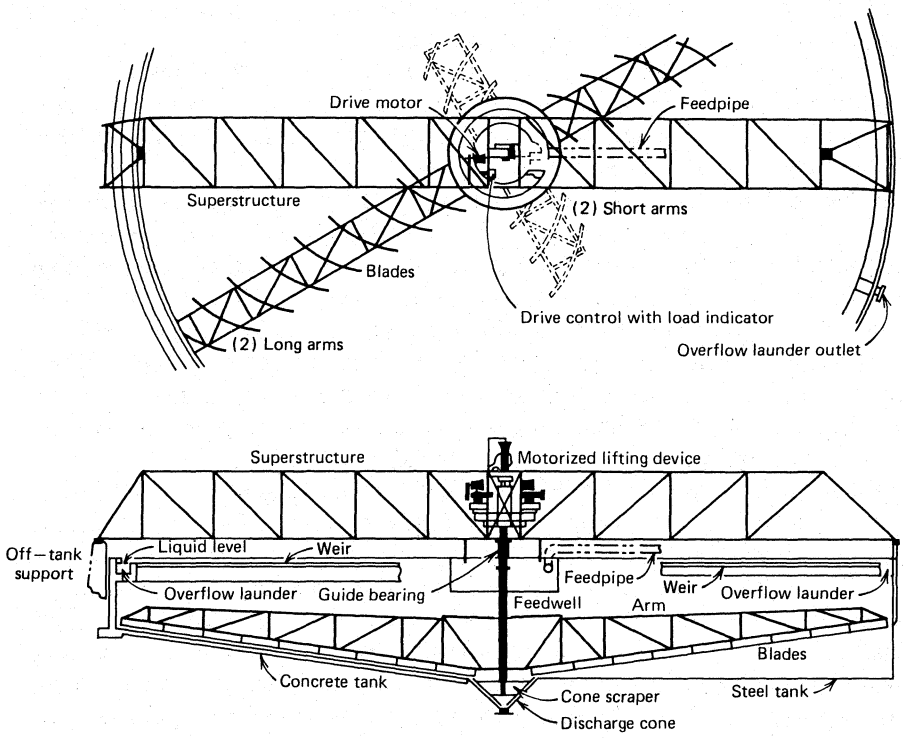


Figure 1.5 Thickener. (Reprinted from Cummins and Given, 1973.)

cases the inability to recycle process water has important implications on tailings disposal.

The most common means of dewatering is by *thickeners,* illustrated in Figure 1.5. Thickeners consist of a tank with rotating arms that convey the settled tailings solids to the center of the tank, where they are collected and pumped to the disposal area. Hydrocyclones are sometimes also used for dewatering.

Other types of dewatering devices less commonly used include drum, disc, and belt filters. These devices function by vacuum suction of water from the tailings slurry through a cloth or screen.

TAILINGS HANDLING AND MILL WATER RETURN

Tailings Transport and Discharge

Tailings collected from the thickener are almost universally transported in slurry form to the tailings impoundment. This practice results more from convenience than design, since tailings in the mill are already mixed with water and further dewatering and drying that would be necessary for dry handling is usually economically prohibitive.

Transport of tailings slurry is sometimes by gravity flow through open launders but more commonly through pipes, either with or without pumping as dictated by relative elevations of the mill and tailings impoundment as well as by pipe length and headlosses.

Piping and pumping systems for tailings slurry are difficult to design, complicated to operate, and on the whole constitute an art beyond the scope of explanation of this book. However, an understanding of basic tailings transport considerations is valuable to the extent that impoundment siting is governed to a large degree by feasible distances for tailings transport.

The tailings slurry is usually abrasive and of high viscosity. The common measure of slurry density is *pulp density* (defined as the weight of solids per unit weight of slurry), which ranges from about 15% to 55% and is most commonly in the range of 40–50% produced by typical thickeners.

McElvain and Cave (1973), Faddick (1980), and Wasp et al. (1977) outline design procedures for tailings distribution systems. Design of tailings pipelines is complicated by a number of conflicting constraints. Unlike water, tailings slurry is subject to a minimum flow velocity below which the tailings will settle from suspension and plug the pipe with disastrous results. Minimum settling velocity is unique to each situation and depends on pulp density of the slurry as well as size, gradation, and specific gravity of the solid particles. Excessively high velocities, on the other hand, cause rapid pipe wear as well as high headloss that must be overcome by larger pumps and higher pumping costs. Pipe segments on steep downslopes must usually be provided with energy-dissipating drop boxes to prevent both excessive pipe pressures and high velocities.

As a practical matter, most tailings pipelines operate within a velocity range of about 5–10 ft/sec, depending on the coarseness and size distribution of the tailings, the pulp density of the slurry, and other factors. Pipe wear, which is often sufficiently severe to destroy ordinary steel pipe within a few years, can be combated by using rubber-lined steel pipe. Within recent years, high-density polyethylene (HDPE) pipe has found wide application for tailings transport over low to moderate pipe pressure ranges. In design of tailings pipelines, provisions must always be made for pipe drainage in the event of mill shutdown so that tailings do not settle from the motionless slurry, plugging the line.

Deposition of an above-water tailings beach around the perimeter of a tailings dam is usually desirable and sometimes mandatory for structural purposes. Beach deposition may be accomplished by either single-point discharge or spigotting, as shown in Figure 1.6. Single-point discharge requires that the open end of the tailings discharge pipe be relocated periodically to form a series of adjacent and overlapping deltas. Spigotting accomplishes the same purpose by discharging tailings slurry through closely spaced spigots in the pipeline (usually about 50–150 ft apart), without the need for

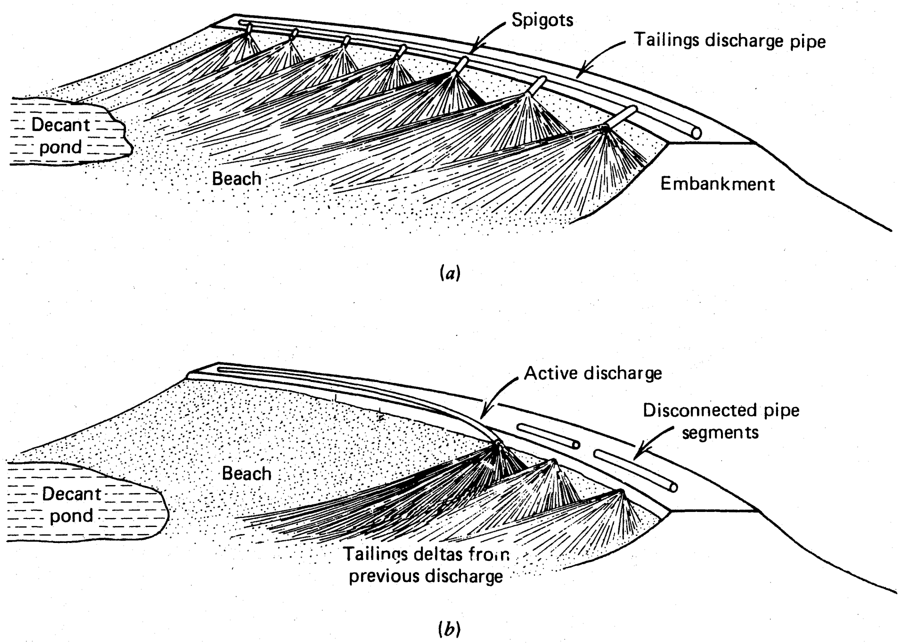


Figure 1.6 Peripheral discharge methods. (*a*) Spigotting. (*b*) Single-point discharge.

frequent relocation of the pipeline or disconnection of pipe segments. Spigots are usually individually valved for control and distribution of discharge.

Decanting of Ponded Water

Following discharge of tailings into the impoundment area, much of the coarser tailings particles tend to settle from suspension relatively near the point of discharge. Remaining coarse particles, finer particles, and colloidal particles are carried further to the ponded water, or *decant pond,* where they eventually sediment in standing water. Where possible, water is decanted from the pool and returned to the mill for reuse. Pumps or siphons mounted on floating barges are often used for this purpose. Also used are decant towers, vertical concrete risers with intake ports that extend from the bottom of the impoundment upward through the tailings deposit. Barge and tower methods for water return are illustrated in Figure 1.7.

The use of decant towers requires a concrete conduit extending beneath the tailings deposit and through the dam to exit beyond its toe. Because of the possibility of conduit rupture due to high imposed stresses or subgrade settlement, and the resulting potential for internal erosion and eventual collapse of the dam, decant towers are not the preferred method for mill water return. Other objections to decant conduits and towers center around the

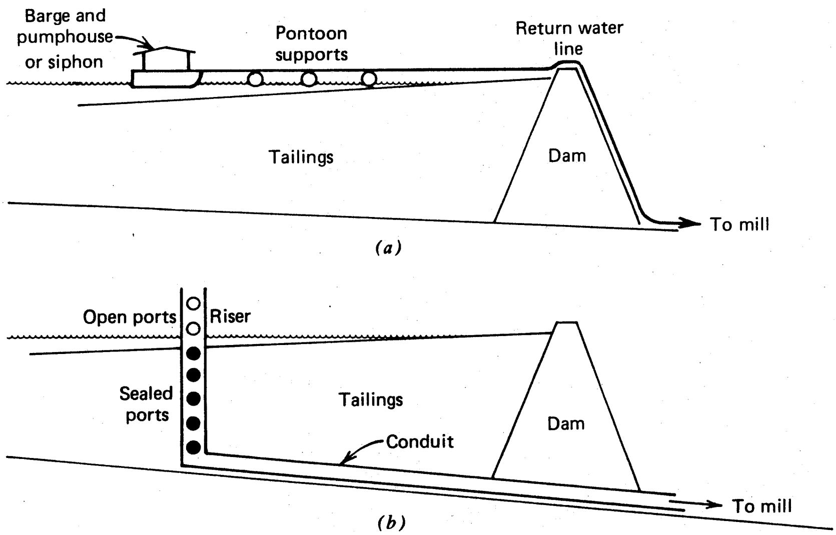


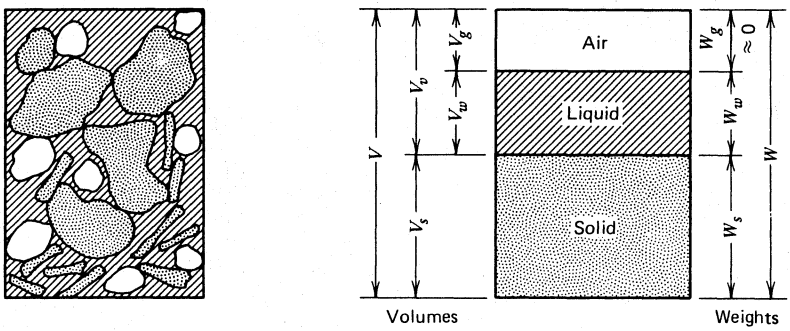
Figure 1.7 Decant methods. (*a*) Floating barge. (*b*) Decant tower.

susceptibility of concrete to acids and sulfates that may be present in the mill effluent, and around the fact that omission of properly constructed seepage collars has caused piping-related failure of embankments in the past.

Decant barges, on the other hand, do not suffer from these disadvantages. In addition, barges offer the flexibility for relocation to various areas of the decant pond. This flexibility offers important advantages in control of the level and location of the pond within the impoundment. Where maintenance of a minimum beach width is required for embankment stability purposes, this factor can be significant. Also, some tailings impoundments are divided into adjacent segments to allow for sequential discharge and operation. In such cases, a single decant barge can be shifted between segments, unlike decant towers, which would require a separate system for each segment. Soderberg and Busch (1977) provide a comprehensive description and comparison of various decant methods.

TAILINGS PHASE RELATIONSHIPS

An essential element in physical characterization of tailings is to relate the solid, liquid, and air phases of the mass of material, whether in slurry or settled form. Figure 1.8a illustrates schematically the three phases as they would typically exist in a mass of settled or slurry tailings. Figure 1.8b shows an idealized representation of the three phases completely separated from one another, with weight and volume components defined for the three phases. The solid and liquid phases are always present; air may or may not exist in the tailings mass, depending on its degree of saturation.



(a) (b)

Figure 1.8 Tailings phase relationships. (Reprinted from Lambe and Whitman, 1969.) (*a*) In-place assemblage of tailings particles with air-filled voids and water-filled voids. (*b*) Separation of air, liquid, and solid phases by weight and volume.

Fundamental parameters that describe tailings phase relationships include pulp density for slurry, and void ratio, water content, porosity, specific gravity, and degree of saturation for settled tailings. These parameters are defined below with reference to Figure 1.8b:

void ratio $$e = \frac{V_v}{V_s}$$

degree of saturation $$S = \frac{V_w}{V_v}$$

porosity $$n = \frac{V_v}{V}$$

specific gravity (solids) $$G = \frac{\gamma_s}{\gamma_w}$$

where γ_s = unit weight of solids $= \dfrac{W_s}{V_s}$

and γ_w = unit weight of water $= \dfrac{W_w}{V_w}$

water content $$w = \frac{W_w}{W_s}$$

pulp density $$P = \frac{W_s}{W}$$

The equation $Gw = Se$ follows from the above definitions. From this equation, expressions can be derived to relate the various phase relationships. The following are often useful in determining tailings density, weight, and volume:

pulp density (% solids) P $= \dfrac{W_s}{W} = \dfrac{1}{1 + w}$

degree of saturation S $= \dfrac{V_w}{V_v} = \dfrac{\gamma_d\, w\, G}{G\gamma_w - \gamma_d}$

void ratio e $= \dfrac{V_v}{V_s} = \dfrac{G\gamma_w}{\gamma_d} - 1$

$$= \frac{(1 + w)G\gamma_w}{\gamma_t} - 1$$

porosity n $= \dfrac{e}{1 + e}$

total unit weight (total density) $\gamma_t = \dfrac{W}{V} = \dfrac{G + Se}{1 + e}\gamma_w$

$$= \frac{1 + w}{1 + e} G\gamma_w$$

dry unit weight (dry density) $\gamma_d = \dfrac{W_s}{V} = \dfrac{G}{1 + e}\gamma_w$

$$= \frac{G\gamma_w}{1 + (w\, G/S)}$$

$$= \frac{G\gamma_w}{1 + [(1 - P)/P]G/S}$$

$$= \frac{\gamma_t}{1 + w}$$

In addition to characterization based on phase relationships, tailings are often described on the basis of gradation using the terms *sands* and *slimes*. Sands represent the plus 200 sieve fraction of the *whole tailings* (tailings of the gradation produced by the mill), and slimes correspond to the minus 200 component. Nearly all whole tailings contain both components, but to widely varying degrees. The tendency of coarser and finer particles to settle within different areas of a tailings impoundment, for example, gives rise to the term *slimes zone* to describe that portion of the tailings deposit consisting predominantly of minus No. 200 sieve size material.

For a given deposit of sand tailings, *relative density, D_r*, provides a useful description of in-place density in relation to the loosest and densest states that the tailings may attain. For sand tailings with a given in-place void ratio or dry density,

$$D_r = \frac{\gamma_{d_{\max}}}{\gamma_d} \times \frac{\gamma_d - \gamma_{d_{\min}}}{\gamma_{d_{\max}} - \gamma_{d_{\min}}} \times 100\%$$

$$= \frac{e_{\max} - e}{e_{\max} - e_{\min}} \times 100\%$$

where $\gamma_{d_{max}}$ = dry density of tailings in densest condition
$\gamma_{d_{min}}$ = dry density of tailings in loosest condition
e_{max} = void ratio of tailings in loosest condition
e_{min} = void ratio of tailings in densest condition

In conventional soil mechanics, the use of relative density is restricted to clean sands with less than about 10% fines. Minimum density (or maximum void ratio) is determined by pouring dry material into a mold, and maximum density (or minimum void ratio) is determined using a vibrating table. In order to extend the use of relative density to sand tailings, which seldom comply with such rigid restrictions on percent fines, various investigators have determined maximum and minimum densities on predominantly sand tailings having up to 40% passing the No. 200 sieve (Pettibone and Kealy, 1971). A high-energy compaction test is used for determination of maximum density in such cases. For this reason caution is required in comparing tailings relative density to values determined according to conventional soil mechanics practice. In addition, it is important to note that relative density computed on the basis of dry density will reflect differences in specific gravity, which may in some cases vary widely from point to point within a single tailings deposit. Computation of relative density on the basis of void ratio (which requires separate determination of specific gravity) can compensate for errors that would otherwise result from use of dry densities.

TYPES OF TAILINGS

The nature of tailings varies according to the ore being milled and the particular processing operation. Pertinent index properties by which tailings are characterized on a general basis include gradation, specific gravity, and plasticity. The following sections provide a general discussion of the nature and production of tailings of various types.

Coal

Processing of coal may be necessary depending on the characteristics of the seam, mining methods, and quality requirements of the final user. Where practiced, processing is usually for the purpose of removing fine coal and rock (usually shale) particles that would produce excessive ash or interfere with burning, or to remove pyrites to reduce sulfur content.

Processing of steam coal relies to a major extent on gravity separation (Cummins and Given, 1973), the product of which, commonly designated *coarse refuse*, is produced and handled in essentially dry form. Increasingly, however, wet cleaning and froth flotation are practiced, producing a tailings-type waste slurry referred to as *fine refuse* or *sludge* (Backer et al., 1977).

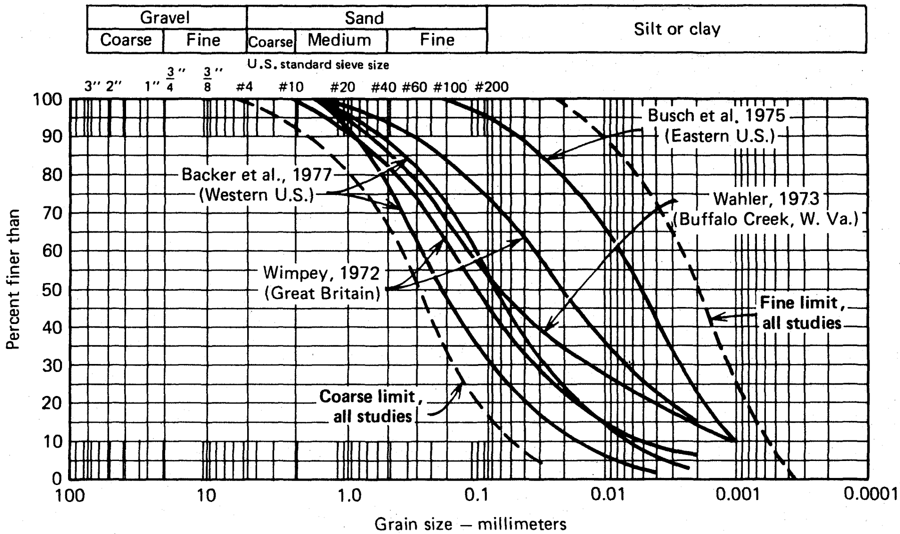


Figure 1.9 Average gradations of fine coal refuse.

Average gradations of fine coal refuse from several studies are shown in Figure 1.9. The average curves for each study summarize what is generally a much wider collection of gradation curves. However, it is apparent from Figure 1.9 that fine coal refuse is indeed a generally fine-grained material consisting principally of coal particles and silt–clay particles from the associated shale. This composition has two effects on the characteristics of the fine refuse: low specific gravity due to coal content, and some plasticity resulting from the presence of clay minerals. Table 1.2 summarizes average specific gravity and plasticity data from several sources.

There is much evidence to indicate that gradation, specific gravity, and plasticity of fine coal refuse vary regionally (Backer et al., 1977). Thus, general characteristics must be interpreted carefully with reference to a particular mine.

Table 1.2 Specific Gravity and Plasticity of Fine Coal Refuse

Location	Specific Gravity	Liquid Limit (%)	Plasticity Index (%)	Source
Eastern U.S.	1.5–1.8	35–50	0–13	Busch et al., 1975
Western U.S.	1.4–1.8	—	—	Backer et al., 1977
Buffalo Creek, W.V.	1.4–1.6	20–40	2–12	Wahler, 1973
Great Britain	1.7–2.4	30–60	3–30	Wimpey, 1972

Tar Sands

Production of crude oil from tar or oil sands requires large-scale mining and processing operations, such as those in the Athabasca Tar Sand region in northern Alberta. Ore is a poorly indurated sandstone that requires essentially no crushing or grinding. Petroleum values are initially recovered by aerating the ore slurry in hot water and steam, and secondary recovery is by froth flotation (Nyren et al., 1979). Sodium hydroxide is a process additive, resulting in effluent pH in the range of about 8–9.

Gradation data for oil sands tailings produced by existing operations is shown in Figure 1.10. While the tailings are relatively clean and coarse overall, a significant sludge (slimes) fraction is present that consists of a mixture of silt, clay, and oil residue. Poor sedimentation and consolidation characteristics of the sludge have complicated tar sands tailings disposal. Scott and Dusseault (1980) discuss sedimentation, consolidation, and permeability characteristics of oil sands sludge. The sludge may settle to void ratios as high as 10, occupying disproportionately large volumes in the disposal impoundment. Extremely high rates of oil sands tailings production (up to 250,000 T/day) have required unusually high rates of tailings embankment height increases, with adverse effects on embankment stability (Mittal and Hardy, 1977).

The oil sands tailings gradations shown are unique to the Athabasca deposits. Gradations and clay contents of other oil sands tailings may be governed by the particle size of the particular formation mined or by crushing and grinding.

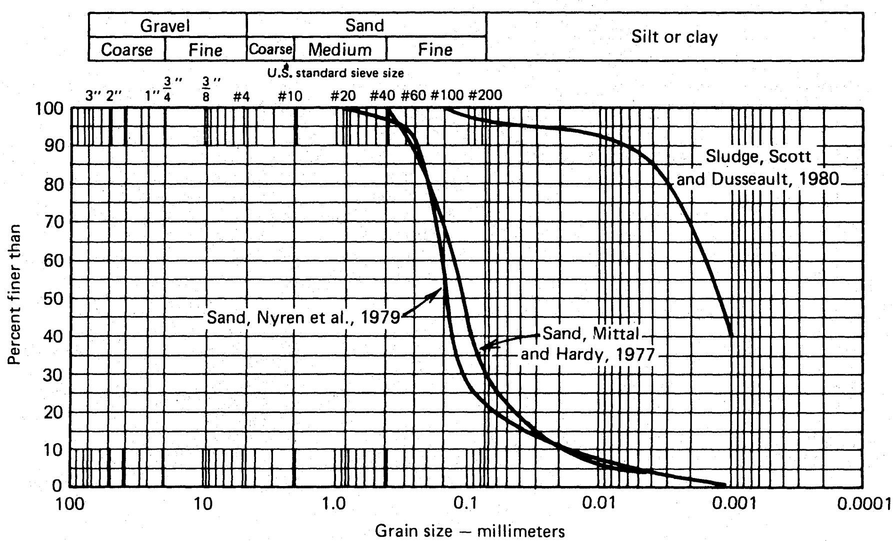


Figure 1.10 Gradations of tar sands tailings.

Lead–Zinc

Lead and zinc commonly occur in association and are often extracted together, sometimes also in combination with silver. Concentration is by froth flotation methods, often at a slightly alkaline pH.

Lead and zinc ores are often associated with either quartzitic or dolomitic host rock, producing hard, angular tailings particles typical of "hard-rock" mining operations in general. At many lead–zinc mines in the United States, tailings are separated in the mill into sands and slimes fractions by cycloning. Sand is customarily used for underground backfill in western U.S. mines and for tailings dam construction in eastern mines.

Figure 1.11 shows typical "average" gradation curves for lead–zinc tailings, for both whole tailings representative of the mill discharge and for slimes tailings typical of those remaining after removal of sands from the whole tailings feed. The gradation differences between the two materials are substantial, and the characteristics of tailings discharged to lead–zinc tailings impoundments will depend on whether sand separation is practiced.

Lead–zinc tailings are generally of low plasticity and clay content, even for the slimes fraction (Lagergren and Griffith, 1973; Mabes et al., 1977). Specific gravity ranges reported in the literature are shown in Table 1.3. High specific gravities apparent in some cases have been associated with the presence of pyrite in the tailings. With a specific gravity of approximately 5.0, even small quantities of pyrite or related minerals can have a major influence on the specific gravity of the tailings. Pyrite may be encountered in

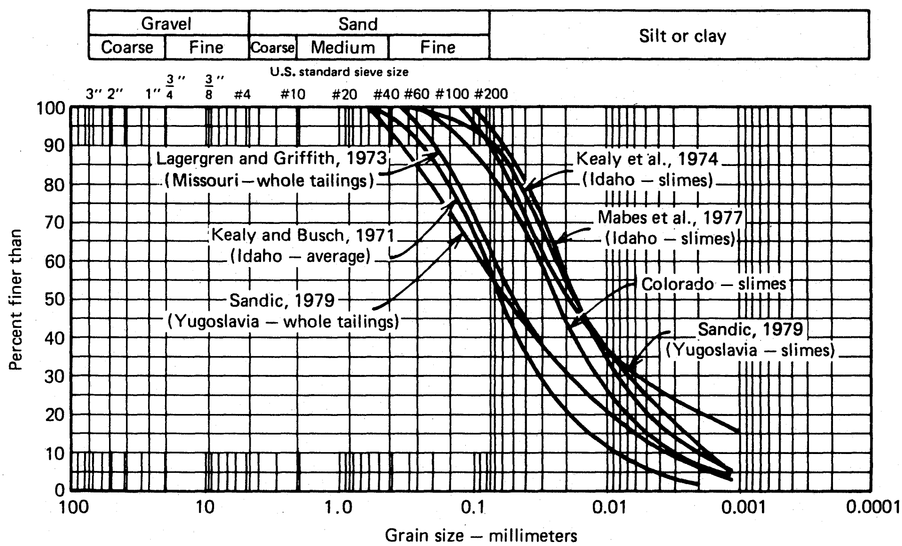


Figure 1.11 Gradations of lead–zinc tailings.

Table 1.3 Specific Gravity of Lead–Zinc Tailings

Location	Specific Gravity	Source
—	2.8–3.4	Soderberg and Busch, 1977
Idaho	2.9	Kealy and Busch, 1971
Idaho	2.9–3.0	Mabes et al., 1977
Colorado	3.3–3.6	Unpublished data

diverse types of orebodies and is not unique to any particular type of tailings.

Gold–Silver

Like lead and zinc, gold and silver are frequently associated in orebodies and are often extracted in combination. Concentration by froth flotation is nearly universal for low-grade disseminated deposits. Gold–silver concentrate may be the end product of the milling operation. Alternatively, concentrate may be further processed in some mills by sodium cyanide leaching, with obvious effects on the character of the liquid tailings effluent. Cyanide leaching, if performed, requires slightly alkaline conditions, usually achieved by addition of lime.

Gradation curves for gold–silver tailings, shown in Figure 1.12, span a

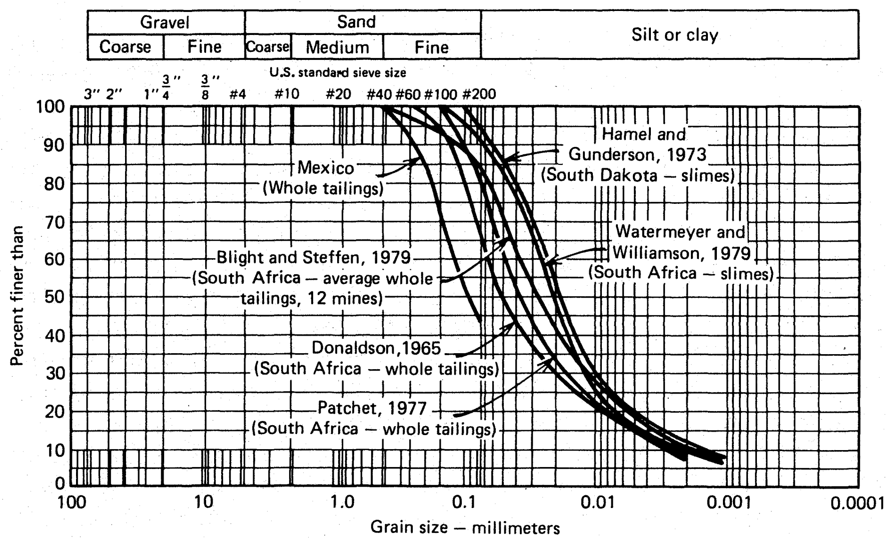


Figure 1.12 Gradations of gold–silver tailings.

wide range, a reflection of the fact that gold–silver extraction efficiency by in-mill (or *vat*) leaching is directly related to the specific surface area of the particles and therefore to the fineness of the grind. The extraction efficiency necessary for economic operation is, in turn, related to the ore grade being processed and is balanced by higher costs and flotation inefficiencies for grinding to finer sizes. Rich ores may therefore result in relatively coarse tailings, with the more finely disseminated but more common ores producing finer tailings.

The gradation of tailings produced by the mill are designated "whole tailings" in Figure 1.12 (although whole tailings are referred to as "slimes" in South African terminology). Curves labeled "slimes" in Figure 1.12 represent tailings discharged to the impoundment after separation of sands for use in either dam construction or underground mine backfilling.

Gold–silver ores generally contain little clay, and the tailings produced are usually low to nonplastic (Hamel and Gunderson, 1973). Specific gravity of 2.6–2.7 is reported by Soderberg and Busch (1977) and 3.1 by Hamel and Gunderson (1973). Higher specific gravities are probably due to the presence of pyrite in gold tailings reported by several investigators (Hamel and Gunderson, 1973; Blight and Steffen, 1979).

Unlike disseminated ores, placer gold deposits are mined by hydraulic or dredge methods and rely on gravity separation without grinding or concentration. Finer particles, sand, and silt are released into the stream while the gravel and cobble-sized "tailings" are stacked in piles that dominate the landscape in placer gold mining areas of California, Colorado, Alaska, and the Yukon.

Copper

The majority of copper ore produced is mined by large-scale open-pit methods from hard-rock ores, which are concentrated by froth flotation in the mill.

Gradation curves for whole tailings from the United States and Canada, shown in Figure 1.13, fall within a surprisingly narrow range, with one exception where the shale-related ore produces high clay content. In general, the whole tailings are relatively coarse, with about 45% passing the No. 200 sieve on the average, although variations from mine to mine can be expected. Finer grinds may be produced by mills that extract other minerals in association with copper—for example, silver or zinc. Also shown in Figure 1.13 are gradations for copper slimes at mines where sands are removed for use in tailings dam construction. Whole tailings are usually nonplastic. Slimes tailings may exhibit low plasticity, as summarized in Table 1.4.

Specific gravities quoted in the literature range from 2.6 to 2.8 (Volpe, 1979) and 2.7 to 3.0 (Mittal and Morgenstern, 1976). As is the case for many of the hard-rock minerals, higher specific gravities, where encountered, usu-

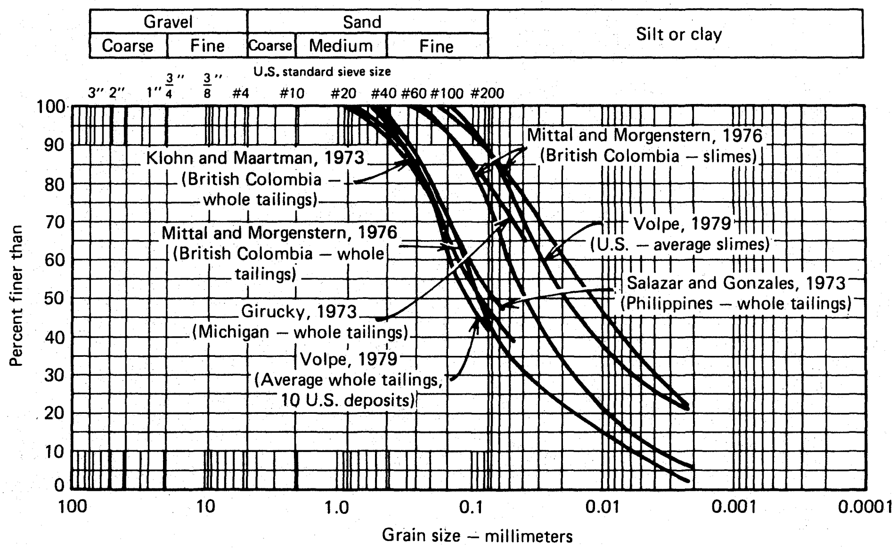


Figure 1.13 Gradations of copper tailings.

ally result from the presence of pyrite in the ore that is rejected during flotation and discharged with the tailings.

Molybdenum

Although molybdenum has historically been extracted in North America chiefly as a by-product of copper operations, development of new molybdenum mines promises to make molybdenum tailings of increasing importance. Concentration in the mill is by froth flotation, often at neutral to slightly alkaline pH.

Gradation curves for molybdenum tailings are shown in Figure 1.14. The materials are relatively coarse and similar to whole copper tailings, with about 40% passing the No. 200 sieve. The whole tailings are nonplastic, with the slimes fraction showing a typical liquid limit and plasticity index of about

Table 1.4 Plasticity of Copper Slimes Tailings

Location	Liquid Limit (%)	Plasticity Index (%)	Source
Western U.S.	40 (avg)	13 (avg)	Volpe, 1979
British Columbia	0–30	0–11	Mittal and Morgenstern, 1976

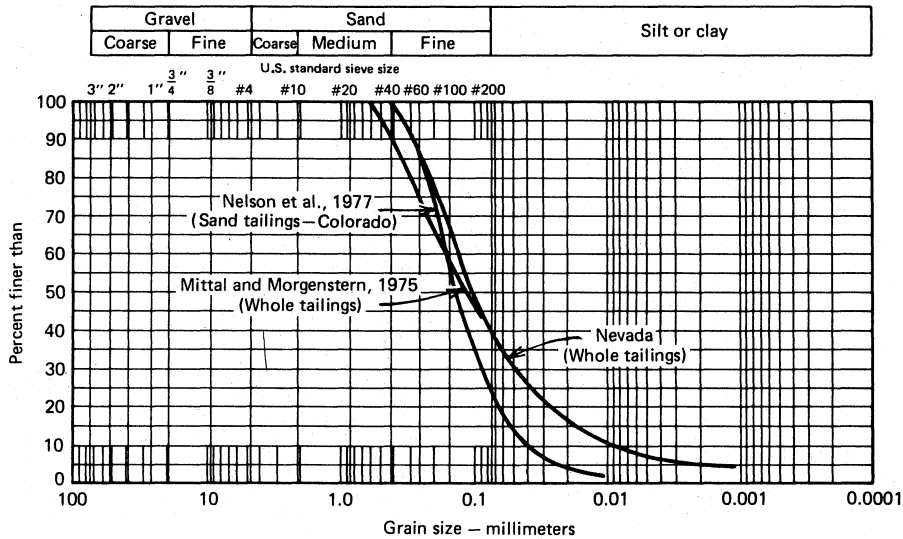


Figure 1.14 Gradations of molybdenum tailings.

30% and 4%, respectively. Quoted specific gravities in the literature are in the range of 2.6–2.8 (Nelson et al., 1977; Soderberg, 1977).

Nickel

Historically, the major North American nickel sources have been the sulfide deposits of the Subury, Ontario, district, consisting of nickeliferous pyrrhotite. Concentration is by flotation, sometimes in conjunction with magnetic separation (Taggart, 1945). Reported gradation of Ontario nickel tailings is shown in Figure 1.15. Tailings high in pyrrhotite may have unusual chemical and physical properties produced by oxidation of ferric sulfides, as discussed in subsequent portions of this chapter.

Newly developed sources of nickel may include ocean-floor manganese nodules and laterite ores. Laterites are highly weathered residual soils usually found in tropical environments, and tailings derived from laterite ores may contain high concentrations of clay and/or mica that result in characteristics very different from and more difficult than those of the classical sulfide ores.

Taconite

The main ore in the United States for production of iron is low-grade taconite, containing primarily hematite with lesser amounts of magnetite, limonite, and related minerals. Concentration is usually by gravity separation, often followed by magnetic separation. These concentration processes result in relatively coarse tailings almost exclusively in the sand size range. At

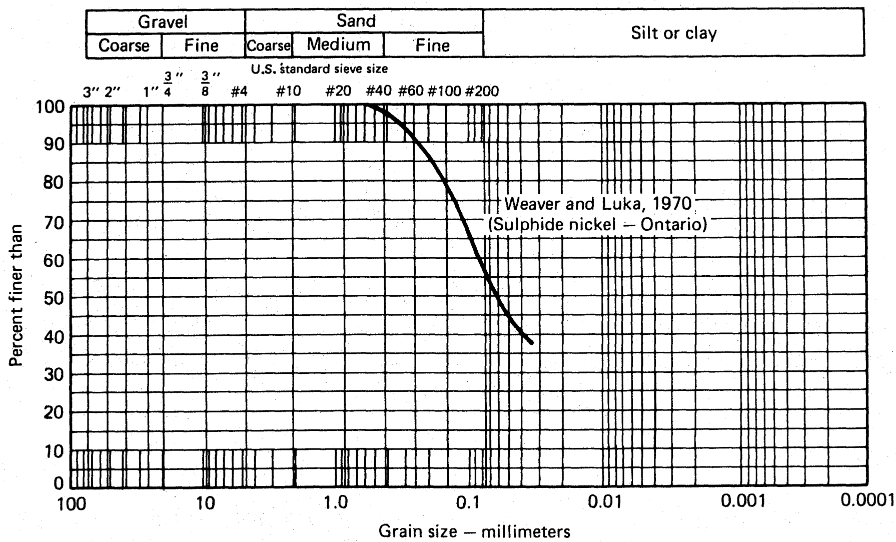


Figure 1.15 Gradation of nickel tailings.

some operations, further concentration by flotation methods is performed to liberate additional low-grade or nonmagnetic mineralization. Flotation requires crushing to very fine sizes. Thus, a given operation may produce either or both types of tailings. Where both fine and coarse tailings are produced, the fine tailings usually predominate in terms of volume.

Gradations of both types of tailings materials are shown in Figure 1.16.

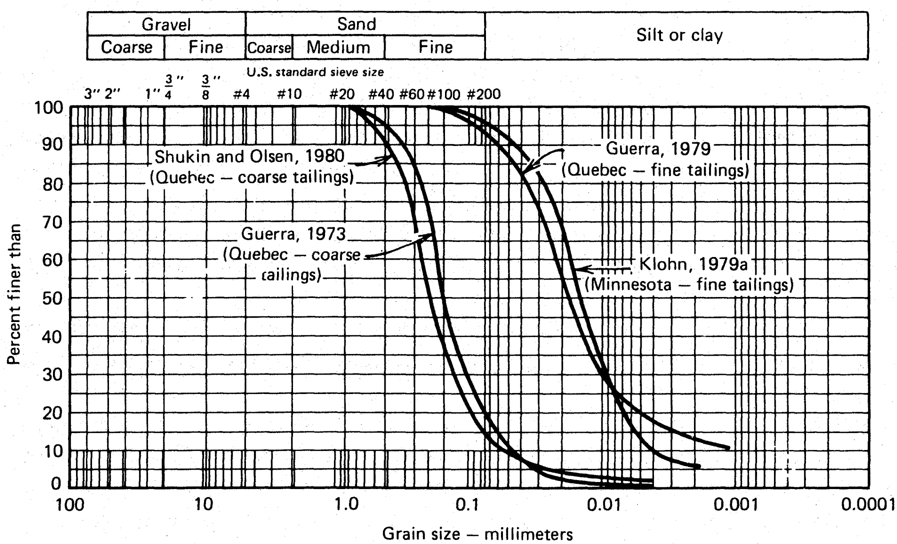


Figure 1.16 Gradations of taconite tailings.

Table 1.5 Specific Gravity of Taconite Tailings

Location	Tailings Type	Specific Gravity	Source
Quebec	Fine	3.0–3.4	Guerra, 1979
Quebec	Coarse	3.0	Guerra, 1973
Minnesota	Fine	3.1	Klohn, 1979a

The difference in gradation is immediately apparent. However, Guerra (1973) reports both types of materials to be nonplastic. Specific gravities reported in the literature are shown in Table 1.5. High specific gravities result from unrecovered iron minerals.

Phosphate

Together the various classes of phosphate tailings exhibit among the most complex and bizarre behavior of any type of tailings material. To arrive at the end product of phosphoric acid, used in fertilizer, phosphate rock is beneficiated to produce two different types of tailings, which include the notorious phosphatic clays. The concentrate is then converted to phosphoric acid, producing gypsum tailings. The two procedures and the tailings produced are discussed separately in this section.

Beneficiation Tailings

The nature of the host rock for phosphate ores varies widely, including shale in the western United States, apatite in South America, and what is essentially a clayey sand in the southeastern United States. Following mining, and crushing if necessary, the ore is washed to remove fines. The fines are discharged at neutral pH as tailings known as *slimes* or, in the case of Florida materials, *phosphatic clays*. Effluent associated with this process is essentially neutral, with a pH of 7–8 (Bromwell and Raden, 1979). Phosphate values are retained in the coarser particles. These values are usually liberated by a subsequent flotation process, producing the phosphate concentrate. The remaining flotation tailings are usually known as *sands* in industry terminology.

Gradation of typical sand tailings is shown in Figure 1.17. Sands are usually produced in relatively small quantity and seldom present a disposal problem.

The phosphate slimes, on the other hand, are among the finest tailings produced by any type of processing operation. Slimes derived from western U.S. shale ores are generally finer than the No. 100 sieve and predominantly in the silt and clay size range. The Florida phosphate slimes, however, are not only extremely fine grained, as shown in Figure 1.15, but are also com-

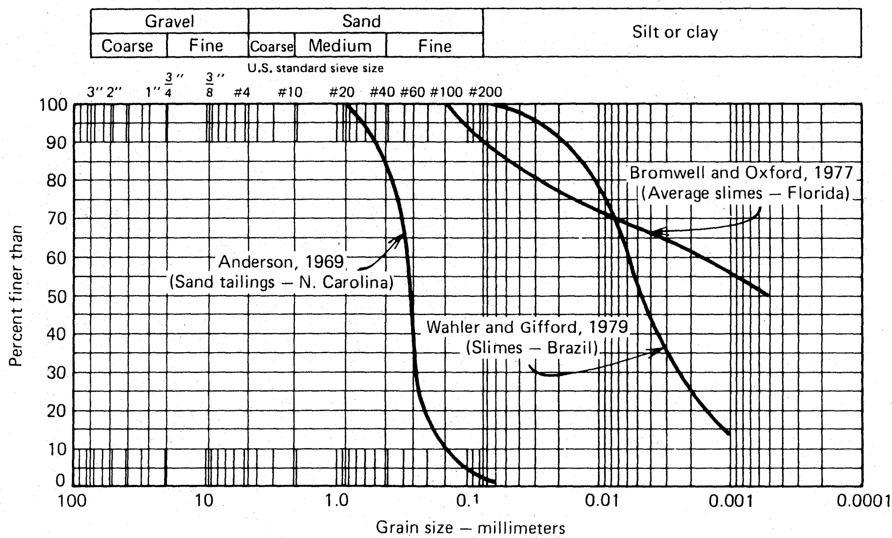


Figure 1.17 Gradations of phosphate tailings.

posed of highly active clay minerals, including montmorillonite, attapulgite, kaolinite, and illite (Bromwell and Raden, 1979). As a result of the exceedingly slow consolidation characteristics of these clay minerals, the slimes have extremely high void ratios, in the range of 5–10, even after several years in an impoundment. The resulting deposit occupies enormous land areas, ties up great quantities of water in the voids, and stymies reclamation efforts for many years. Specific gravity of Florida phosphate slimes ranges from 2.5 to 2.8, and liquid limit and plasticity index range from 120% to 200% and 90% to 150%, respectively (Bromwell and Oxford, 1977). Development of means to effectively dewater and reduce the volume of phosphate slimes has been the subject of considerable research (Martin et al., 1977; Smelley et al., 1980; Scheiner et al., 1982).

Gypsum Tailings

Phosphate concentrate is converted to phosphoric acid (P_2O_5) for use in fertilizer manufacture by processes that include heating (calcining) to remove organics and treatment with sulfuric acid. The result is gypsum tailings, consisting principally of calcium sulfate with hydrated water. The effluent discharged with the gypsum tailings typically has a pH of 1–3 and is high in fluoride (Williams, 1973). Gypsum tailings may also contain trace levels of radioactive Ra-226 derived from the parent ore (May and Sweeney, 1982).

Gypsum tailings fall almost exclusively within the silt size range (Figure 1.18) and are nonplastic. Specific gravities range from 2.3 to 2.4 for Idaho deposits (Vick, 1977) and 2.2 to 2.3 for Mississippi materials. Care must be

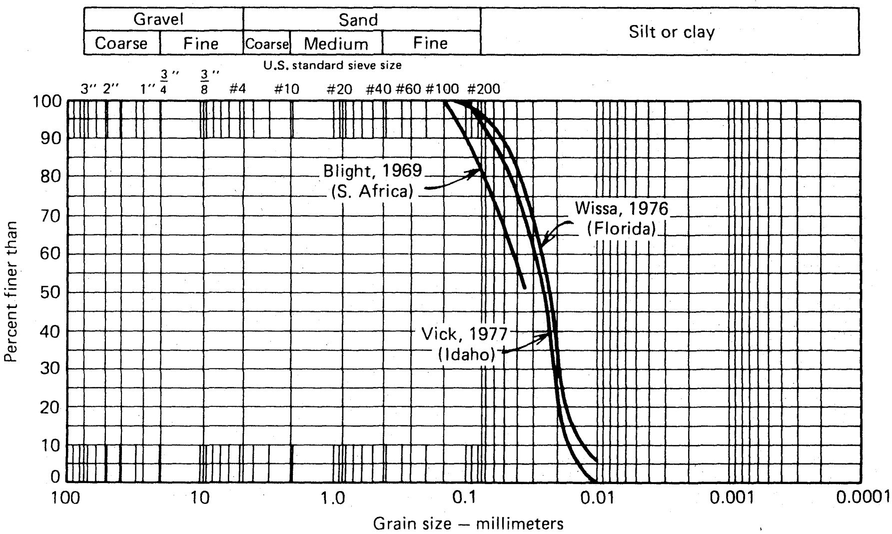


Figure 1.18 Gradations of gypsum tailings.

exercised in determining gypsum tailings water contents to avoid driving off hydrated water bonded to the crystals.

Gypsum is unique among tailings materials in two respects: it exhibits in-place cementation, and it experiences major, long-term creep deformation under constant stress. These factors influence the design of gypsum tailings embankments in that slopes are often steeper than usual because of cementation. Also, major creep-related cracking can lead to formation of solution cavities and sinkholes in embankments (Wissa, 1976; Vick, 1977).

Bauxite

Processing of bauxite ores to yield aluminum oxide produces a type of tailings known as *red muds*. The commonly used Bayer extraction process essentially involves leaching aluminum oxides by washing the ground ore in hot caustic soda solution. The mineral composition of the remaining tailings includes hematite and complex silicates; quartz and clay minerals, however, are notably absent (Somogyi and Gray, 1977). The effluent is highly alkaline, with pH ranging from about 12 to 13 (Somogyi and Gray, 1977; Parekh and Goldberger, 1976).

Gradation of bauxite tailings from various sources is summarized in Figure 1.19. While some materials contain up to 30% sand, most of the tailings are in the silt size range. The materials exhibit low plasticity, with a liquid limit of about 46% and a plasticity index of about 7–9%. Specific gravity is highly variable depending on iron content, with quoted values in the range of 2.6–3.1 (Parekh and Goldberger, 1976) and 2.8–3.3 (Somogyi and Gray, 1977).

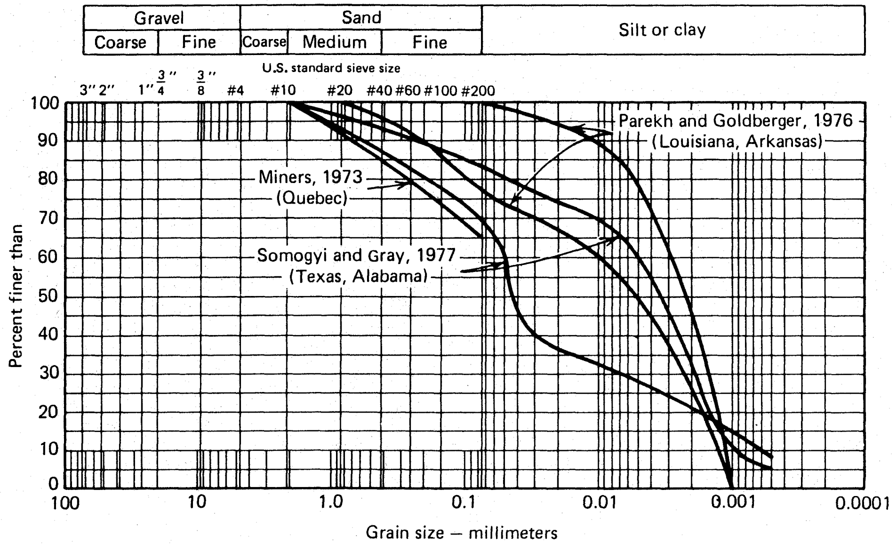


Figure 1.19 Gradations of bauxite tailings.

In spite of their low plasticity and low clay content, bauxite tailings share some of the properties usually associated with clayey tailings, such as low sedimentation–consolidation rates. Methods such as sand layers at the impoundment base have been studied as a means to accelerate consolidation and reduce the impoundment volume occupied by these and similar types of tailings (Rickel et al., 1982). Also, like gypsum tailings, bauxite tailings show a degree of time-dependent creep under constant stress (Somogyi and Gray, 1977).

Uranium

Uranium tailings have received considerable recent attention because of their radioactive properties and the consequent concern for long-term disposal. Perhaps as a result of overriding environmental considerations, the physical nature of the tailings themselves has received relatively little attention.

Uranium ores are processed using leaching techniques with either acid or alkaline reagents, depending on carbonate content of the ore. Acid leaching predominates in the United States, Canada, and Australia, and produces liquid effluent with pH in the range of 1–2. The pH of alkaline-leach effluent is usually about 10. Acid-leach process water cannot be recirculated through the mill. Consequently, large quantities of water are discharged into the tailings impoundment, which must either be evaporated or treated prior to release, usually by lime neutralization and/or barium chloride precipitation.

Uranium tailings gradation curves from the literature are shown in Figure 1.20. While the tailings are generally coarse, clay content varies widely

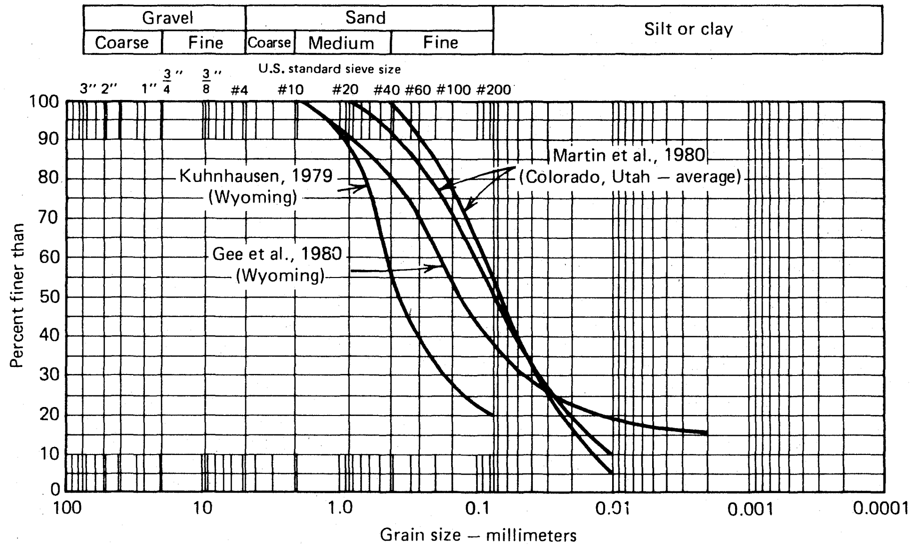


Figure 1.20 Gradations of uranium tailings.

according to the nature of the ore. In some cases, acid leaching has been found to alter the mineralogical character of the clay fraction, from montmorillonite in the ore to principally kaolinite in the tailings slimes. Also, it should be pointed out that the data in Figure 1.20 are representative of acid-leach tailings; alkaline-leach processes usually require grinding to finer sizes, predominantly minus 200 mesh (Clark, 1974).

Whole uranium tailings are usually nonplastic because of the considerable sand content. Plasticity of the slimes fraction, however, varies from low to high depending on the clay content and mineralogy of the parent ore. Johnson (1980) reports specific gravity of 2.6–2.7 for uranium tailings.

Both uranium tailings and liquid effluent contain low levels of radioactive radium-226 with a half-life of 1,620 years, in addition to nonradioactive constituents. Ra-226 has been shown to be concentrated mainly in the slimes fraction of the tailings, with lesser amounts in the sand (Borrowman and Brooks, 1975). Radioactive materials may enter the environment and the food chain via two primary pathways: water and air. Radium contained in the liquid effluent may enter groundwater via seepage, where it may be subject to eventual ingestion. In addition, radium contained in the solid tailings particles decays to gaseous radon-222. Radon released into the air quickly decays into small, short-lived daughter particles, including polonium-218, lead-214, bismuth-214, and polonium-214, which may be ingested through the lungs (Clark, 1974).

These properties have resulted in strict regulatory restrictions on disposal methods for uranium tailings. Impoundment liners are commonly required for seepage reduction. After abandonment of the tailings impoundment, covering of the tailings with a substantial thickness of inert soil is often prescribed to reduce radon emanation. As with other forms of low-level radiation, however, the degree of health and environmental hazard posed by uranium tailings remains controversial and poorly understood.

Trona

Trona ore is mined for conversion to soda ash, widely used in manufacturing and many industrial products. Like coal, trona occurs in seams between shale layers, and the mined ore necessarily incorporates some shale impurities. Crushed ore is beneficiated by hot water solutioning. The process leaves behind the insoluble tailings (principally shale particles) known as *insols*, together with large quantities of waste water, which are usually evaporated in the tailings impoundment. The effluent is alkaline, with a pH of 9–11.

Typical trona tailings gradation is shown in Figure 1.21. The tailings contain a coarse sand fraction consisting of angular but soft shale particles. The finer materials consist of silt and clay, with plasticity varying from low to high. Specific gravity is about 2.5 to 2.7.

Potash

Potash (KCl) is mined from sedimentary deposits principally for use in fertilizer supplements. Potash tailings comprise two fractions: a coarse, sandy fraction containing a large proportion of salt, and a clay fraction. Effluent

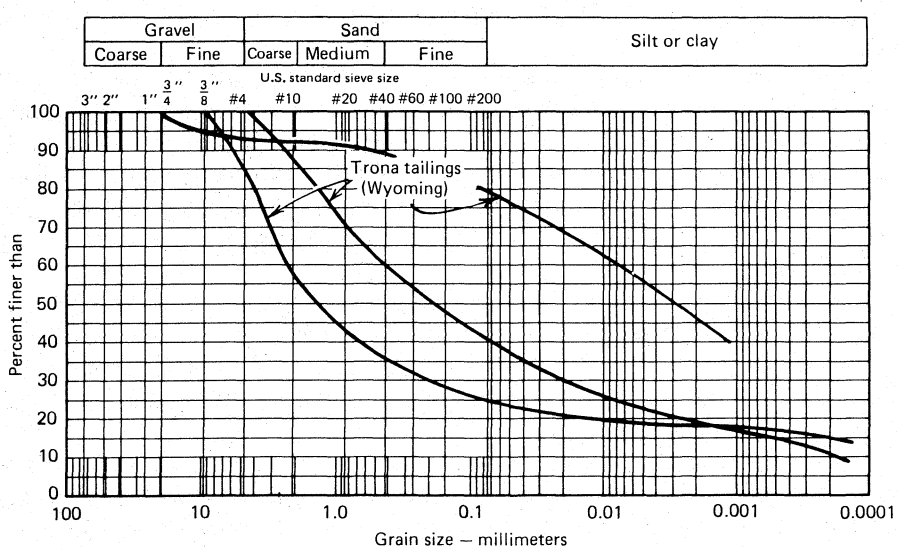


Figure 1.21 Gradations of trona tailings.

consists of salt brine. Problems encountered in potash tailings disposal stem from the susceptibility of the coarse fraction to solutioning and from slow sedimentation characteristics of the clays (Coates and Yu, 1977).

Classification of Tailings Types

The types of tailings discussed in this section cover such a wide variety of physical characteristics that generalization is difficult. Not only do the types of tailings vary, but tailings within any one ore type may differ substantially according to mill process and the nature of the orebody. Within wide limits, however, some generalizations may provide a useful summary.

Table 1.6 divides the various types of tailings into four general categories according to both gradation and plasticity. The first category, soft-rock tailings, are those derived principally from shale ores, including fine coal refuse and trona insols. While these tailings ordinarily contain some sand-sized materials, the clayey nature of the slimes significantly influences the physical character and behavior of the material as a whole.

Sands usually predominate for the second category, hard-rock tailings, which includes the lead–zinc, copper, gold–silver, molybdenum, and nickel types. Tailings are primarily finely crushed silicate particles. Slimes, while they may be present in substantial proportions, are derived from the crushed host rock rather than clay and do not usually exert an overwhelming influence on the behavior of the tailings as a whole. Other hard-rock tailings not specifically addressed in this chapter may include those from cobalt, tin, platinum–palladium, tungsten, chromium, titanium, and mercury mining and milling. Basic information on the mineralogy of the ore, grinding operations, and concentration procedures will usually permit reasonably valid correlations with physical characteristics of the hard-rock tailings reported herein.

Fine tailings, the third category, are those having little or no sand and include phosphatic clays, bauxite red muds, fine taconite tailings, and slimes from tar sands tailings. The characteristics of slimes predominate for these materials to the extent of rendering them largely incompetent from a structural standpoint. These materials may require long periods of time for sedimentation and consolidation, and may require large impoundment areas and volumes.

Coarse tailings are those whose characteristics are determined on the whole by the sizable coarse sand fraction or, in the case of gypsum tailings, by nonplastic silt that behaves more or less like a sand. This group includes the coarse fraction of tar sands, uranium, gypsum, coarse taconite, and phosphate sand tailings.

Because tailings in any one category share the same broad physical characteristics, disposal problems are usually somewhat similar. Thus, when dealing with tailings from ores where there is little information available on disposal practices, comparison with tailings in the same general category

Table 1.6 Summary of Physical Tailings Characteristics

Category	General Character
Soft-rock tailings	
Fine coal refuse Trona insols Potash	Contain both sand and slime fractions, but slimes may dominate overall properties because of presence of clay
Hard-rock tailings	
Lead–zinc Copper Gold–silver Molybdenum Nickel (sulfide)	May contain both sand and slime fractions, but slimes are usually of low plasticity to nonplastic. Sands usually control overall properties for engineering purposes.
Fine tailings	
Phosphatic clays Bauxite red muds Fine taconite tailings Slimes from tar sands tailings	Sand fraction generally small or absent. Behavior of material, particularly sedimentation–consolidation characteristics, dominated by silt- or clay-sized particles and may pose disposal volume problems
Coarse tailings	
Tar sands tailings Uranium tailings Gypsum tailings Coarse taconite tailings Phosphate sand	Contain either principally sands or nonplastic silt-sized particles exhibiting sandlike behavior and generally favorable engineering characteristics.

may provide useful general guidelines. In addition, changes in grinding at a particular mill may, for example, produce considerably finer material, which may change the category in which the tailings reside and introduce new and different disposal problems. It is important to recognize, however, that the above classifications reflect only the broad physical characteristics and engineering behavior of various tailings types; chemical characteristics and environmental considerations may be more important than physical behavior in determining disposal practices in some cases.

NATURE OF LIQUID TAILINGS EFFLUENTS

The physical nature of tailings cannot be considered separately without also accounting for the chemical characteristics of the associated liquid mill effluent. As briefly discussed for uranium tailings, design of the tailings impoundment may be dictated not by the nature of the solids, but rather by that of the effluent. Inasmuch as the overall degree of conservatism used in tailings impoundment design depends on the hazard posed by the materials it contains, a general understanding of the chemical constituents of mill effluent is required.

Classes of Effluents

As discussed in previous portions of this chapter, milling processes involving chemical alteration of the ore include flotation and leaching. During flotation a variety of organic chemicals may be added, including fatty acids, oils, and polymers. These organic chemicals are seldom of major concern in mill effluent because they have generally low concentrations and comparatively low toxicity. However, pH adjustment during flotation may have significant effects on inorganic mill effluent constituents and this effect is accentuated if acid or alkaline leaching is practiced. The chemical–mineralogical constituents present in the ore are the most important factor in determining the chemical nature of the mill effluent, and pH adjustment during milling may liberate a number of these constituents from the host rock. As a result, pH is often a useful indicator of the general types of constituents in mill effluent, and several effluent categories can be defined on this basis.

1. *Neutral.* This condition is produced by simple washing or gravity separation operations where pH is not substantially altered. Chemical constituents in the effluent will be primarily limited to those in the host rock that are soluble at neutral pH. Levels of sulfates, chlorides, sodium, and calcium may be somewhat elevated for effluents of this class.
2. *Alkaline.* Raising the pH of the effluent may also result in elevated concentrations of sulfates, chlorides, sodium, and calcium constituents, among others. While some metallic contaminants may be present, extensive mobilization of cationic heavy metals in very high concentrations does not usually occur.
3. *Acid.* Reducing pH raises equilibrium levels of many metallic contaminants, and acid-leach effluents may show high levels of such cationic constituents as iron, manganese, cadmium, selenium, copper, lead, zinc, and mercury, if present in the host rock. Acid effluents also exhibit elevated concentrations of such anions as sulfates and/or chlorides. Low pH effluents class among the most troublesome of liquid mill wastes.

Table 1.7 shows constituent levels for several acid and alkaline mill effluents. These levels are not intended to be typical or representative in a general way, since there are substantial variations from ore to ore and from

Table 1.7 Comparison of Acid and Alkaline Mill Effluents (in mg/l)

	Acid			Alkaline			
Constituent	Uranium Acid Leach[a]	Gypsum Phosphoric Acid Plant[b]	Copper Lead Zinc[c]	Lead–Zinc Flotation[d]	Lead Flotation[d]	Trona	Tin[c]
pH	2	2.6	6.4	10.1	8.1	10.9	11.0
Aluminum	2,000					0.04	
Arsenic	0.2					0.004	0.05
Calcium	500						
Cadmium	0.2	0.4		<0.02	<0.02	<.001	
Copper	50	0.3	0.2	1.9	1.9	0.002	0.18
Fluoride	5	503		2.4		17	
Iron	1,000					0.35	
Lead	7	0.2	0.0	0.1	<0.1	0.006	0.05
Manganese	500		0.6	0.5	1.1	0.02	0.84
Mercury	0.07					0.005	
Molybdenum	100						
Selenium	20					<.001	
Sodium	200						
Zinc	80	12.2	1.0	0.2	0.4		3.4
Cyanide					0.5		
Sulfate	30,000					1,530	
Chloride	300			4		4,380	
Nitrate	500						

[a] U.S. Nuclear Regulatory Commission, 1979.
[b] Williams, 1973.
[c] Down and Stocks, 1977.
[d] Williams, 1975.

mill to mill. However, they do illustrate general differences in chemical constituent levels, particularly heavy metals, in the two categories of effluents.

Another special class of effluents consists of radioactive or other toxic materials, in particular uranium and gold–silver effluents. Both acid- and alkaline-leach uranium extraction processes liberate radioactive radium-226 and thorium-230, but acid-leach processes do so to a greater extent. It is possible to reduce radium-226 to low levels by lime neutralization and/or coprecipitation with barium chloride (Clark, 1974; Averill et al., 1980). Treatment is mandatory if effluent is to be released from the impoundment.

In the case of gold–silver mill effluents, cyanide, a necessary reagent if leaching is practiced, is the troublesome constituent. Cyanide is also sometimes used in concentration processes—for example, lead and tungsten flotation (Williams, 1975; Brawner, 1979). Cyanide is relatively unstable and readily degrades to the less toxic cyanate form in the presence of oxygen (Ripley et al., 1978). Natural degeneration of cyanide comes about by a number of mechanisms, including acidification and volatilization by CO_2 absorption from the air, photodecomposition, oxidation, and biological decomposition. These processes will eventually result in reduction of cyanide concentrations in the impoundment effluent, but considerable time may be required depending on concentration levels. Also, the presence of metal-complexed cyanides may limit the minimum cyanide concentration attainable by natural degradation. Treatment methods for cyanide removal are available. Acidification, for example, can be used to accelerate natural decomposition. More common methods are alkaline chlorination or hydrogen peroxide oxidation, procedures performed in the mill prior to effluent discharge.

Another class of toxic effluents are those containing arsenic. Where present in association with the ore, arsenic may be liberated in the effluent during milling. Coates and Yu (1977) note that for gold-bearing arsenopyrite ores the arsenic must first be removed by roasting to allow for efficient leaching and then be disposed in a suitable area, preferably not the tailings impoundment.

Toxicity

The levels of various constituents in mill effluents can be judged meaningfully only in relation to levels that are harmful to humans, plants, and animals. Various guidelines have been proposed in the United States for drinking water, livestock water, and irrigation water to which a given mill effluent assay can be compared. This is not to say that mill effluent will be used for any of these specific purposes. However, the guidelines are useful as a baseline for determining relative levels of contaminant concentrations in impoundment seepage derived from mill effluents.

Table 1.8 shows guidelines from several sources. Presented first in the table are primary drinking water standards, followed by secondary standards. Recommended standards for livestock and irrigation use are presented separately in Table 1.8.

Careful inspection of Table 1.8 shows that the nature of hazard varies greatly depending on the particular constituent. For example, an elevated constituent concentration of 1.0 mg/l may cause either instant death or laundry staining, depending on whether the constituent happens to be arsenic or manganese. A basic knowledge of health and toxicity effects is necessary for a proper understanding of the level of hazard posed by a particular mill effluent. Table 1.8 does indicate that the most highly toxic contaminants include, among others, cadmium, chromium, lead, mercury, and selenium. These are among the metallic elements most likely to be mobilized in low-pH effluents if present in association with the ore. Such constituents as chlorides and sulfides common to both acidic and alkaline effluents, however, do not ordinarily pose a major toxicity hazard, although overall groundwater quality and potential use may be adversely affected by seepage containing these constituents.

Pyrite Oxidation

A final comment is necessary concerning a special chemical problem afflicting some tailings: oxidation of sulfides, particularly pyrite and pyrrhotite. Pyrite (FeS_2), where present in an ore, is usually rejected from the concentrate during flotation. Its ultimate fate is to be deposited with the tailings material as ground particles that are often visible to the naked eye or whose presence is indicated by high specific gravity. Although the occurrence of pyrite is specific to a particular orebody, pyrite can be associated with a variety of tailings, including lead–zinc, nickel, gold, copper, coal, and uranium.

Pyrite oxidizes in the presence of free oxygen, producing acidic conditions. The basic mechanism of this reaction is the combination of the metal sulfide with water and oxygen to yield a metal hydroxide and sulfuric acid. Kleinmann et al. (1981) show that, in addition to chemical oxidation, a bacterium (*Thiobacillus ferrooxidans*) is of prime importance and bacterial oxidation may in fact be the dominant process in later stages of acid formation.

In operating tailings impoundments, the tailings are usually saturated and the voids are filled with water, limiting chemical oxidation where pyrites are present. Following termination of active discharge into the impoundment, however, water levels within the tailings drop, introducing air into the void spaces and promoting conditions conducive to pyrite oxidation. This may produce acidic conditions, even if the tailings effluent was initially alkaline or neutral. Low pH liberates a host of cationic metallic contaminants in solution, together with sulfates and other dissolved anions. For example, Blair et al. (1980) and Cherry et al. (1980) describe an abandoned pyrite-

Table 1.8 Water Quality Standards and Toxicity of Inorganic Chemical Mill Effluent Constituents (mg/l)

Constituent	1962 Drinking Water Standards[a]	Interim Primary Drinking Water Regulations[b]	Water Quality Criteria, 1972[c]	Effects on Humans, Animals, and Plants
Primary				
Arsenic (As)	0.05	0.05	0.1	Highly poisonous and possibly carcinogenic in humans. Arsenic poisoning can range from chronic to severe and may be cumulative and lethal.
Barium (Ba)	1.0	1.0	1.0	Barium is a general muscle stimulant, especially of the heart muscle. It can cause nerve block leading to nervous disorders and increase blood pressure by vascular constriction. Ingested barium salts can cause irreversible damage to some tissues.
Cadmium (Cd)	0.01	0.01	0.01	Cadmium is concentrated in tissue, and humans can be poisoned by contaminated foods, especially fish. Cd may be linked to renal arterial hypertension and can cause violent nausea. Cd accumulates in liver and kidney tissue. It depresses growth of some crops and is accumulated in plant tissue.
Chromium (Cr)	0.05	0.05	0.05	Cr^{+6} is toxic to humans and can induce skin sensitizations. Human tolerance of Cr^{+3} has not been determined.

Cyanide (CN)	0.2	0.2	0.2	Cyanide poisoning is well-known, but lethal effects usually occur when biological detoxification mechanisms are overwhelmed. Otherwise the body is capable of detoxifying low CN levels.
Fluoride (F)	—	approx. 2.0	—	Has beneficial effects on teeth at low levels. Livestock may be susceptible to chronic fluoride poisoning at high levels.
Lead (Pb)	0.05	0.05	0.05	A cumulative body poison in humans and livestock. Humans may suffer acute or chronic toxicity. Young children are especially susceptible.
Mercury (Hg)	—	0.002	0.002	Hg is biologically magnified, accumulating in the brain, liver, and kidneys of animals. Hg poisoning may be acute or chronic.
Nitrate (NO_3^- as N)	10.0	10.0	10.0	Nitrate concentration in excess of 10 mg/l can cause methemoglobinemia in infants, which can be fatal.
Selenium (Se)	0.01	0.01	0.01	Toxic effects in humans comparable to As, and possibly carcinogenic. Se is easily accumulated by vegetation to levels toxic to grazing animals.
Silver (Ag)	0.05	0.05	0.001	Main effects are cosmetic, causing skin discoloration. Toxic to algae at low levels.
Uranium (U)	5.0	—	—	Level based on chemical toxicity and possible kidney damage, rather than radioactive properties.
Radium (226 + 228)	3.0 pCi/l	5.0 pCi/l gross alpha		The effects of radiation on humans are considered harmful and to be avoided.

Table 1.8 (*Continued*)

Constituent	1962 Drinking Water Standards[a]	Interim Primary Drinking Water Regulations[b]	Water Quality Criteria, 1972[c]	Effects on Humans, Animals, and Plants
Other standards for domestic water supply				
Chlorides (Cl)	250	—	250	May affect water taste at high levels. Permissible concentrations for livestock up to 3,000 mg/l.
Copper (Cu)	1.0	—	1.0	Small amounts considered nontoxic and necessary for human metabolism. Large doses may induce vomiting or liver damage. Toxic to fish and aquatic plants at low levels.
Iron (Fe)	0.3	—	0.3	Essentially nontoxic but causes taste problems in water.
Manganese (Mn)	0.05	—	0.05	Affects water taste and may stain laundry. Toxic to animals at high concentrations.
Sulfate (SO_4^-)	250	—	250	Can affect water taste and have laxative effects on occasional users. 600 mg/l can be tolerated by humans, 1,000 mg/l for livestock.
Total dissolved solids (TDS)	500	—	—	High TDS objectionable because of water taste, laxative effects, and effects on irrigated crops above this level.
Zinc	5.0	—	5.0	Zinc is necessary and beneficial, but may affect water taste at high levels. Toxic to some plants and fish.
pH	—	—	—	Generally recommended between 6.5 and 8.5 because of mobilization of trace metals outside this range. Fish mortality can be expected at pH less than 5.0.

Standards for livestock use				
Aluminum (Al)	—	—	5.0	No evidence of either major benefit or high toxicity to animals.
Cobalt (Co)	—	—	1.0	Toxic only at very high levels, with a wide margin of safety from recommended level.
Vanadium (V)	—	—	0.1	Toxic to chicks at high levels.
Standards for irrigation				
Aluminum (Al)	—	—	5.0	Toxic to plants.
Boron (B)	—	—	0.75	Essential to plant growth but damaging to sensitive crops at high levels.
Cobalt (Co)	—	—	0.05	Toxic to plants, including beans, tomatoes, and corn.
Molybdenum (Mo)	—	—	0.01	Forage crops may become toxic to grazing animals. More recent data suggest levels up to 0.15 mg/l may be acceptable.
Nickel (Ni)	—	—	—	May depress plant growth or be toxic in excess of 0.5–1.0 mg/l.

[a] U.S. Public Health Service, 1962.
[b] U.S. EPA, 1975.
[c] U.S. NAS/NAE, 1972.

bearing uranium tailings deposit that was initially discharged at a mill effluent pH of 8 following neutralization treatment. About 15 yr later, pyrite oxidation within unsaturated portions of the tailings had produced pH as low as 3, accompanied by extensive mobilization of metallic and radiologic contaminants in solution.

Chemical problems notwithstanding, pyrrhotite-rich tailings have been used to advantage in some applications because of unusual self-cementing properties produced by pyrrhotite in high concentrations. Patton (1952) describes the use of pyrrhotite-rich tailings as a cementing agent for underground backfill for mines in Quebec, and Thomas (1971) reports the results of related studies for underground mines in Australia. Oxidation rates may be high enough to generate considerable heat.

A related problem, acid production by oxidation of thiosalts, occurs during disposal of effluent from milling of massive sulfide ores and is common for some metal mines in eastern Canada. Thiosalts are sulfate compounds in solution in the mill effluent, including thiosulfate ($S_2O_3^{-2}$), trithionate ($S_3O_6^{-2}$), and tetrathionate ($S_4O_6^{-2}$), liberated during grinding and upgrading, particularly if liquid SO_2 is added in the milling process (Rolia and Barbeau, 1979). Thiosalt oxidation may be a particularly insidious problem, since resulting acid production may not occur until effluent reaches receiving streams from the impoundment via either direct discharge or seepage. The culprit in thiosalt oxidation is *Thiobacillus thiooxidans*, a bacterial cousin of the organism that contributes to pyrite oxidation. Biological treatment methods are available to remove thiosalts from the mill effluent stream (Guo and Jank, 1980).

SUMMARY

No rational plan or design for tailings disposal can be carried through without an appreciation of the physical nature of the tailings and the chemical characteristics of the liquid mill effluent. The combined physical and chemical properties of the material dictate on the most fundamental level the type of disposal facility required and the degree of conservatism in its design consistent with the hazard posed by the materials.

Understanding of tailings begins with a knowledge of the processes by which they are produced, and even a limited familiarity with these processes can give important clues to the nature of the tailings. While tailings characteristics vary over wide ranges in nearly every respect, the type of ore being processed usually allows for some reasonably valid generalizations about the general physical nature of the material.

In much the same way, a basic understanding of processing techniques can give useful indications of the types of chemical constituents to be expected in the liquid effluent. Here, however, it is necessary to account for not only the types of contaminants but also their expected concentrations.

The potential hazard posed by specific contaminants at specific concentrations can be judged only in the context of individual toxicity levels. The assessment of chemical hazard and toxicity often involves not only scientific evidence but also social judgments and political processes, and it cannot be expected that these controversies will be resolved in the context of any particular tailings disposal plan. Ideally, however, the informed designer will be in a position to address the degree of possible hazards with neither undue complacency nor unjustified panic.

In tailings disposal planning and design, the nature and production of the particular type of material under consideration is the cornerstone of all subsequent efforts. The problems that will subsequently have to be addressed during detailed design follow from and are defined by the physical and chemical characteristics of the tailings and effluent. Engineering designs are often akin to puzzles, and while considerably more effort will be required to fit all the pieces into place, topics addressed in Chapter 1 will provide important clues to the size and shape of the puzzle itself.

2

Engineering Behavior of Tailings

If you get careless or go romanticizing scientific information, giving it a flourish here and there, Nature will soon make a complete fool out of you. It does often enough anyway even when you don't give it opportunities.

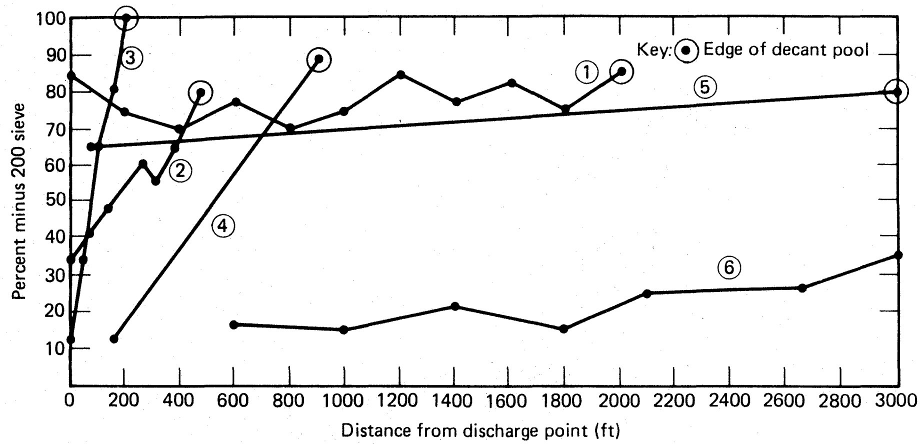
Robert Pirsig, *Zen and the Art of Motorcycle Maintenance*

DEPOSITIONAL CHARACTERISTICS

Central to an understanding of tailings behavior is the nature of the depositional processes that tailings undergo. As discussed in Chapter 1, tailings are deposited hydraulically, usually by some form of peripheral discharge method, either spigotting or rotating single-point discharge. This results in an above-water tailings beach and a slimes zone associated with the ponded decant water. For most types of tailings, the beach slopes downward to the decant pond on an average grade of 0.5–2.0% within the first several hundred feet, with beaches on the steeper end of the range usually resulting from higher pulp density and/or coarser gradation of the whole tailings discharge. At more distant points on exposed beaches, the beach slope may flatten to as little as 0.1%. At these more distant locations, depositional processes may come to resemble natural stream channel sedimentation, with continually shifting braided flow channels and backwater regions.

This depositional process produces a highly heterogeneous beach deposit. In the vertical direction, tailings beach deposits are frequently layered, with percent fines typically varying as much as 10–20% over several inches in thickness. If discharge points or spigots are widely spaced, variations in fines content of 50% or more can occur over short vertical distances. Such extreme layering produced by thin slimes layers within otherwise sandy beaches may result from periodic encroachment of ponded water onto the beach where thin layers of fines settle from suspension.

Horizontal variability is usually also significant, with coarser particles settling from the slurry as it moves over the beach, and finer suspended or colloidal particles settling only when they reach the still water of the decant pond to form the slimes zone. Particle size sorting that occurs along the



Curve	Tailings type	G	%–200, Mill discharge	Pulp density of mill discharge	Source
1	gold	–	–	–	Blight and Steffen, 1979
2	copper	2.7	45	45	Volpe, 1979
3	lead-zinc slimes	3.4	75	< 40	Unpublished
4	–	2.7	38	30	Soderberg and Busch, 1977
5	–	2.7	60	50	Soderberg and Busch, 1977
6	copper	3.0	–	–	Unpublished

Figure 2.1 Grain-size segregation along tailings beaches.

beach has been studied in laboratory models (Kealy and Busch, 1971; Jerabek and Hartman, 1965). As they settle from the slurry, particles are transported along the beach surface by saltation and rolling. Hydraulic separation results in a general tendency of finer particles on the beach to be carried and deposited further from the point of discharge.

Measurements of lateral grain-size variability on actual tailings beaches, however, present a somewhat more complicated picture than model studies indicate. Figure 2.1 summarizes measurements of fines content as a function of distance for several tailings beach deposits. The degree of grain-size segregation ranges from high to almost nonexistent. The degree of sorting obviously depends on the gradation characteristics of the whole tailings discharge; slurry with a wide range of particle sizes is more likely than slurries containing poorly graded materials to exhibit beach grain-size segregation. However, inasmuch as most mill grinding processes produce more or less similar ranges of particle size variation, gradation alone is not sufficient to explain the observed differences. Soderberg and Busch (1977) conclude that pulp density of the discharged slurry controls the degree of beach particle-size segregation, with lower pulp densities tending to promote a greater degree of segregation. They suggest that differences in pulp density of as little as 10–20% can produce major changes in lateral particle-size distribution.

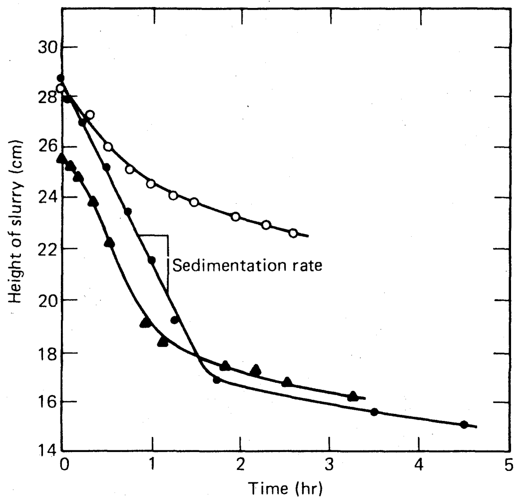


Figure 2.2 Typical sedimentation test results on copper slimes samples. (Reprinted from Mittal and Morgenstern, 1976, by permission of the National Research Council of Canada.)

Deposition of slimes is by entirely different processes than those for tailings beaches. Sedimentation of slimes from suspension in ponded water does not involve sorting by saltation or particle rolling, but rather it is a relatively straightforward process of vertical settling. The rate of slimes sedimentation can have important effects on the size of decant pond necessary for water clarification and on the quantity of water available for mill recycle. Sedimentation rates may be determined in the laboratory by pouring a homogeneous slurry at the desired pulp density in a glass cylinder. The advance of the interface between the water and settled solids is recorded with time. Typical sedimentation test results are shown in Figure 2.2, with the linear portion of the curve yielding the sedimentation rate. Following completion of sedimentation, void ratio of the settled slimes can be

Table 2.1 Slimes Sedimentation Rates

Slimes Type	Specific Gravity	Plasticity Index (%)	Sedimentation Rate (ft/hr)	Source
Copper	2.7	10	0.31	Mittal and Morgenstern, 1976
	2.7	9	0.14	Mittal and Morgenstern, 1976
Phosphatic clay	2.8	125	0.17	Keshian et al., 1977 (field tests)
Copper–zinc	2.9	0	0.38	Unpublished
	4.0	0	0.54	Unpublished

measured in the cylinder, giving an indication of the initial void ratio that the slimes will assume under field conditions. Typical sedimentation rates for various slimes, as shown in Table 2.1, are remarkably similar considering the ranges of plasticity and specific gravity for the various slimes compared.

In the absence of laboratory sedimentation tests, an empirical rule for decant pond size is that it should allow 5 days of retention time and provide about 10–25 acres of surface pond area per 1,000 T of tailings discharged per day (Coates and Yu, 1977).

DENSITY

In-Place Density

Estimation of in-place density may be particularly important during early stages of tailings impoundment planning. The settled density of the tailings usually controls the impoundment volume required for a given mill tailings production rate.

In-place density can be expressed in terms of either dry density (γ_d) or void ratio (e). Table 2.2 shows typical values measured in actual impoundments for various types of tailings. Within the range of void ratios or dry densities for a particular type of tailings shown in Table 2.2, lower void ratios or higher dry densities usually correspond to greater depths within a deposit. Conversely, highest void ratios or lowest dry densities are usually associated with near-surface materials shortly after deposition.

In-place dry density depends primarily on three factors: specific gravity, type of tailings (sands or slimes), and clay content. Because of variations in all three factors, dry density spans a wide range, from as low as 14 pcf for clayey phosphate slimes to in excess of 110 pcf for high specific gravity lead–zinc and taconite tailings.

General trends are better shown by void ratio, which removes the masking effect of specific gravity variations. Grain size and clay content control in-place void ratio. For most hard-rock tailings and even soft-rock tailings derived from shales (for example, fine coal waste and trona), in-place void ratio for sands generally ranges from about 0.6 to 0.9. Slimes from these tailings types of generally low to moderate plasticity show higher in-place void ratios, ranging from about 0.7 to 1.3. Exceptions include slimes of highly plastic clay or unusual composition, notably phosphatic clays, bauxite, and oil sands tailings slimes. In-place void ratios for slimes of this class are very high, ranging from about 5 to 10. The large impoundment volumes occupied by these materials often result in significant disposal problems.

In-place density exhibits considerable scatter within any single tailings deposit, but it generally increases with depth because of the compressibility of the hydraulically deposited tailings. Figure 2.3 shows average dry density as a function of depth for several types of tailings. Slimes tailings generally

Table 2.2 Typical In-Place Densities and Void Ratios

Tailings Type	Specific Gravity	e	γ_d (pcf)	Source
Fine coal refuse				
Eastern U.S.	1.5–1.8	0.8–1.1	45–55	Busch et al., 1975
Western U.S.	1.4–1.6	0.6–1.0	45–70	Backer et al., 1977
Great Britain	1.6–2.1	0.5–1.0	55–85	Wimpey, 1972
Oil sands				
Sands	—	0.9	87	Mittal and Hardy, 1977
Slimes	—	6–10	—	Mittal and Hardy, 1977
Lead–zinc				
Slimes	2.9–3.0	0.6–1.0	93–113	Mabes et al., 1977
	2.6–2.9	0.8–1.1	80–103	Kealy et al., 1974
Gold–silver				
Slimes	—	1.1–1.2	—	Blight and Steffen, 1979
Molybdenum				
Sands	2.7–2.8	0.7–0.9	92–99	Nelson et al., 1977
Copper				
Sands	2.6–2.8	0.6–0.8	93–110	Volpe, 1979
Slimes	2.6–2.8	0.9–1.4	70–90	Volpe, 1979
Taconite				
Sands	3.0	0.7	110	Guerra, 1973
Slimes	3.1	1.1	92	Klohn, 1979a
	3.1–3.3	0.9–1.2	97–105	Guerra, 1979
Phosphate				
Slimes	2.5–2.8	11	14	Bromwell and Raden, 1979
Gypsum	2.4	0.7–1.5	60–90	Vick, 1977
Bauxite				
Slimes	2.8–3.3	8	20	Samogyi and Gray, 1979
Trona				
Sands	2.4–2.5	0.7	92	Unpublished
Slimes	2.4–2.5	1.2	68	Unpublished

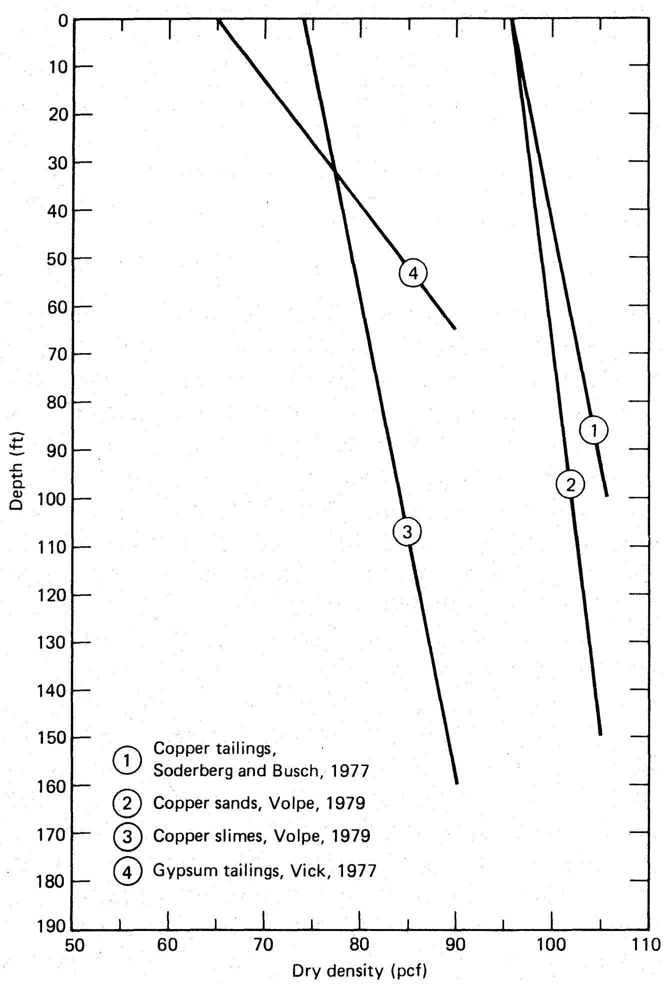


Figure 2.3 Increase in average in-place density with depth.

show an average increase of about 10 pcf per 100 ft of depth, with slightly smaller rates of increase for the less compressible sands of about 5–10 pcf per 100 ft. High rates of density increase for gypsum are not representative of tailings in general since they are produced by long-term creep deformation of individual grains in addition to compression of the soil structure.

Relative Density

Relative density (D_r) of hydraulically deposited beach sands has important influences on dynamic strength behavior. As explained in Chapter 1, relative

Table 2.3 Minimum and Maximum Densities of Sand Tailings

$\gamma_{d_{min}}$ (pcf)	$\gamma_{d_{max}}$ (pcf)	e_{max}	e_{min}	Reference
75–96	99–112	0.72–1.23	0.51–0.68	Mittal and Morgenstern, 1975
85–99	105–129	0.99–1.32	0.51–0.67	Pettibone and Kealy, 1971

density is a measure of the in-place density with respect to the densest and loosest states that the material can attain in laboratory tests. Mittal and Morgenstern (1975) and Pettibone and Kealy (1971) report the results of minimum and maximum density determinations on a number of different hard-rock tailings sands as summarized in Table 2.3. Relative density is sensitive to minimum and maximum densities used, and the data in Table 2.3 exhibit considerable variation. For the data reported by various investigators, there appears to be little or no correlation between minimum or maximum density and sand tailings gradation parameters. Therefore, it is necessary to perform minimum and maximum density determinations specific to the individual tailings sand under consideration.

In-place relative density of sand tailings deposited hydraulically is summarized from several sources in Table 2.4. In addition to hydraulically spigotted tailings, data in Table 2.4 include cycloned tailings sands. All the materials, however, were deposited without mechanical compaction. Although relative density can vary considerably from point to point within a deposit, it appears that many beach sand tailings deposits can attain average relative densities in the range of 30–50% from spigotting or similar procedures. Relative densities in this range can be achieved only by relatively clean sands, either those from which slimes have been removed by cycloning or those that undergo a high degree of particle-size segregation during normal slurry spigotting.

Table 2.4 Average In-Place Relative Density of Sand Tailings

Type	D_r (%)	Reference
Tar sands	30–50	Mittal and Hardy, 1977
Molybdenum sands	31–55	Nelson et al., 1977
Cycloned copper sands	33–54	Klohn and Maartman, 1973
Cycloned copper sands	45–68	Mittal and Morgenstern, 1977
Cycloned copper sands	10–55	Brawner, 1979
Cycloned lead-zinc sands	30	Sandic, 1979
Lead–zinc sands	17–43	Unpublished
Copper sands	37–60	Unpublished

ENGINEERING PROPERTIES

Permeability

More than any other engineering property of tailings, permeability is difficult to generalize. Average permeability spans five or more orders of magnitude, from 10^{-2} cm/sec for clean, coarse sand tailings to as low as 10^{-7} cm/sec for well-consolidated slimes. Permeability varies as a function of grain size and plasticity, depositional mode, and depth within the deposit. General ranges of permeability are shown in Table 2.5.

Average tailings permeability decreases with increasing fines content (percent minus 200 sieve), but percent fines is not the most useful indicator of permeability. Mittal and Morgenstern (1975) demonstrate that average permeability for sand tailings is best predicted by the well-known Hazen's formula:

$$k = d_{10}^2$$

where k = average permeability
d_{10} = grain size in millimeters for which 10% of the particles pass by weight

Mabes et al. (1977) show that Hazen's formula can be extended in application to nonplastic slimes tailings, and Bates and Wayment (1967) describe the application of Hazen's and similar formulas to cycloned sands.

Estimates of average permeability on the basis of grain size, however, cannot account for several important factors that control the permeability of the tailings deposit as a whole. These factors are discussed below.

Effects of Anisotropy

Because of their layered nature, tailings deposits exhibit considerable variation in permeability between the horizontal and vertical directions. Reported

Table 2.5 Typical Tailings Permeability Ranges

Type	Average Permeability, cm/sec
Clean, coarse, or cycloned sands with less than 15% fines	10^{-2}–10^{-3}
Peripheral-discharged beach sands with up to 30% fines	10^{-3}–5×10^{-4}
Nonplastic or low-plasticity slimes	10^{-5}–5×10^{-7}
High-plasticity slimes	10^{-4}–10^{-8}

measurements of anisotropy in the literature are few, but the ratio of horizontal to vertical permeability, k_h/k_v, is generally in the range of 2–10 for reasonably uniform beach sand deposits and for underwater-deposited slimes zones. Transition beach zones between areas of relatively clean sands and slimes are likely to have higher anisotropy ratios due to interlayering of finer and coarser particles. For tailings deposits where discharge procedures are not well controlled, resulting in extensive sand–slime interlayering, k_h/k_v can be as great as 100 or more.

Effects of Distance from Discharge

The degree to which tailings permeability can be expected to vary as a function of distance from the point of discharge has been a topic of considerable debate. The classic conceptual model proposed on the basis of model studies by Kealy and Busch (1971) is shown in Figure 2.4 and consists of a zone of high-permeability sands near the point of discharge, a zone of intermediate permeability, and a low-permeability slimes zone. The relative width of each zone depends on the relative proportion of sands and slimes size fractions in the mill tailings discharge and on the location of the ponded water relative to the point of discharge. Data shown in Figure 2.5 indicate that some tailings deposits do indeed correspond to this conceptual model, with a systematic reduction in both horizontal and vertical permeability with greater distance from the point of discharge.

On the basis of studies of copper tailings deposits, however, Volpe (1979) suggests that the overall variation in average permeability with distance is not very significant, only about a factor of 10, for tailings discharged at pulp densities of 45–50%. Data presented by Soderberg and Busch (1977) show even less systematic variation for some deposits, which exhibit almost random variations in permeability with distance.

The degree of particle-size segregation during beach deposition and discharge procedures appears to control the extent of systematic permeability variation with distance. Deposits exhibiting the largest variation are likely to

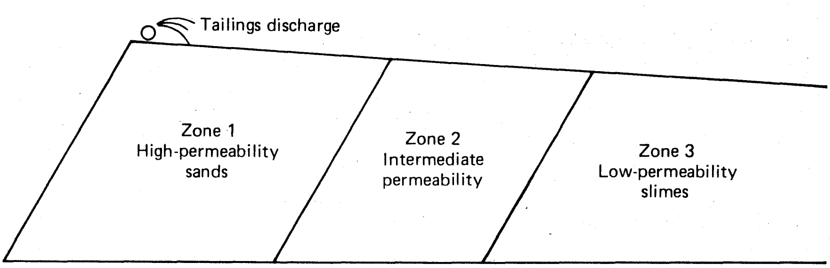


Figure 2.4 Conceptual model of permeability variation within a tailings deposit. (From Kealy and Busch, 1971.)

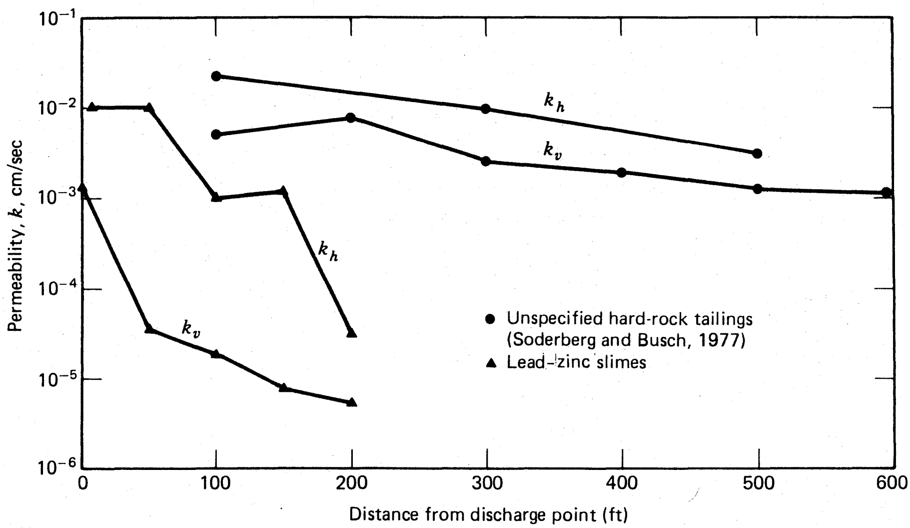


Figure 2.5 Variation in permeability and anisotropy with distance from discharge for well-segregated tailings beaches.

be those where a reasonably wide range of particle sizes is present in the mill discharge, where the discharge is at a low pulp density, and where discharge points or spigots are sufficiently closely spaced to minimize deposition of slimes layers on the beach. Unless mechanical separation by cycloning is practiced during discharge, permeability variation with distance is difficult to estimate for a given type of tailings without sampling the particular deposit.

Effects of Void Ratio

The influence of void ratio (or dry density) on tailings permeability has been extensively studied in the laboratory. Figure 2.6 shows relationships between void ratio and average permeability for a variety of tailings. Although absolute permeabilities vary greatly, the change with decreasing void ratio is reasonably consistent for most tailings sands and low-plasticity slimes. Over the range of void ratios encountered with depth in most tailings deposits, sands may show a permeability decrease of about a factor of 5. The permeability of slimes, on the other hand, may decrease by roughly a factor of 10 because of their higher compressibility. As a result of the greater permeability decrease exhibited by slimes layers, which generally control vertical permeability, the anisotropy ratio k_h/k_v may tend to increase with depth in a deposit of interlayered sands and slimes.

Phosphate slimes and oil sands sludge constitute exceptions to generalizations about void ratio effects. In spite of high plasticity and clay content,

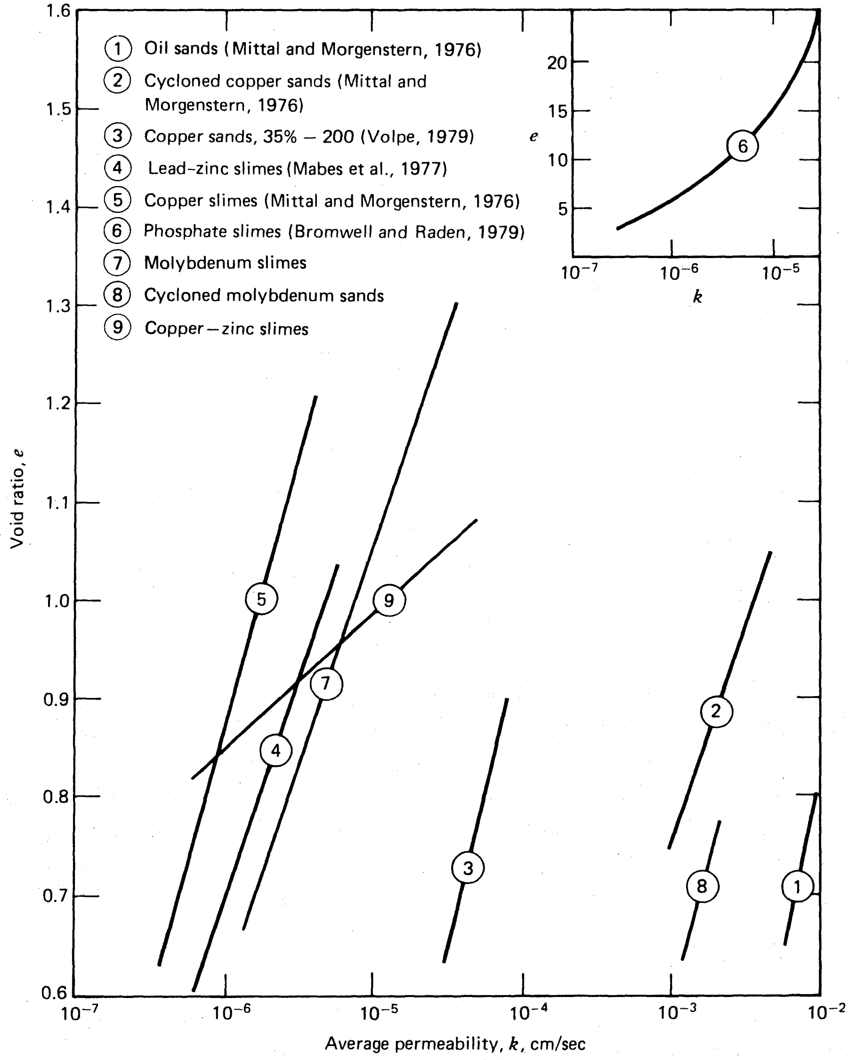


Figure 2.6 Variation in average permeability as a function of void ratio.

permeability of these materials can be as great as 10^{-4} cm/sec at the extremely high void ratios that follow sedimentation. Permeability may approach 10^{-8} cm/sec or lower at void ratios produced by high degrees of consolidation.

Compressibility

Because of their loose depositional state, high angularity, and grading characteristics, both sands and slimes tailings are more compressible than most natural soils of similar type. Compressibility is determined in the one-dimensional compression (consolidation) test, most commonly used to evaluate compressibility of clays in conventional soil mechanics. Interpretation of tests on tailings, however, is complicated by the fact that tailings do not always share with natural clays the well-defined break between "recompression" and "virgin compression" portions of the loading curve. Some slimes tailings may exhibit preconsolidation effects similar to those shown by clays according to classical soil mechanics theory. Most sand tailings, however, show broad curvature of the void ratio–log stress relationship even after preconsolidation. Meaningful interpretation of compressibility coefficients therefore requires specification of the stress range over which they apply. Typical values for the compression index, C_c, determined in one-dimensional compression tests are shown in Table 2.6, together with approximate stress ranges over which the values were determined and corresponding initial void ratios.

The effect of stress history on compressibility of slimes is similar to that on natural clays (Lambe and Whitman, 1969). Overconsolidation produces a flatter initial recompression portion of a void ratio–log stress compression curve. Overconsolidation may be produced by desiccation or capillary suction on exposed, above-water slimes deposits. For most slimes deposits, however, the effect of preconsolidation is limited in both magnitude and vertical extent. Over typical depth ranges, compression in the normally consolidated state is usually of principal interest.

As illustrated by the data in Table 2.6, the differentiation between sands and slimes is the most fundamental factor influencing the compression index. For sand tailings, C_c usually ranges from about 0.05 to 0.10. Most slimes of low plasticity show C_c values ranging from 0.20 to 0.30, about three to four times higher than values for sands. For both sands and slimes, another factor of importance is the density or void ratio that the materials initially assume upon deposition. The looser or softer the initial state, the higher the compression under loading.

Also of interest in Table 2.6 are several materials with unique compressibility characteristics. Phosphatic clay slimes are unusually compressible, because of high initial void ratios and the presence of highly active clay minerals. Also, both bauxite slimes and gypsum tailings exhibit unusual time-dependent deformation characteristics, due probably to interparticle bonding and/or creep of the individual grains. For these materials, compressibility is a function of load duration and is difficult to evaluate by conventional methods.

Consolidation

The time rate of consolidation for materials conforming to Terzaghi theory can be divided into primary and secondary phases (Lambe and Whitman, 1969). Primary consolidation governs the rate of pore pressure dissipation

Table 2.6 Typical Values of Compression Index, C_c

Material	Initial Void Ratio e_0	Compression Index C_c	Stress Range (psf)	Source
Taconite, fine tailings	1.37	0.19	500–20,000	Guerra, 1979
Copper slimes	1.3–1.5	0.20–0.27	20–20,000	Mittal and Morgenstern, 1976
	—	0.28	—	Volpe, 1979
Copper sands (cycloned)	1.10 (D_r = 0)	0.05 0.11	200–2,000 2,000–20,000	Mittal and Morgenstern, 1975
	—	0.09	—	Volpe, 1979
Tar sands	1.0 (D_r = 0)	0.06	200–20,000	Mittal and Morgenstern, 1975
Molybdenum, beach sands	0.72–0.84	0.05–0.13	500–20,000	Nelson et al., 1977
Gold slimes	1.7	0.35	3,000–100,000	Blight and Steffen, 1979
Lead–zinc slimes	0.7–1.2	0.10–0.25	1,000–12,000	Kealy et al., 1974
Fine coal refuse	0.6–1.0	0.06–0.27	—	Wimpey, 1972
Phosphate slimes	>20	3.0	100–1,600	Bromwell and Raden, 1979
Bauxite slimes	1.6–1.8	0.26–0.38[a]	1,000–20,000	Samogyi and Gray, 1977
Gypsum tailings	1.3	0.07[a] 0.28	500–5,000 5,000–20,000	Vick, 1977

[a] Compressibility dependent on load duration.

under constant load, which can have important implications for certain classes of stability and seepage problems.

Primary consolidation for sand tailings occurs so rapidly that it is difficult to measure in the laboratory. The few available data suggest that the coefficient of consolidation c_v varies from about 5×10^{-1} to 10^2 cm^2/sec for beach sand deposits. For slimes tailings c_v is generally about 10^{-2}–10^{-4} cm^2/sec, in the same range as typically exhibited by natural clays. Reported data from the literature for both sands and slimes tailings are summarized in Table 2.7.

Quite unlike natural clays, however, slimes tailings show little consistency in the variation of c_v with void ratio. Data for copper slimes reported

Table 2.7 Typical Values of Coefficient of Consolidation, c_v

Material Type	c_v cm²/sec	Source
Copper beach sands	3.7×10^{-1}	Volpe, 1979
Copper slimes	1.5×10^{-1}	Volpe, 1979
Copper slimes	10^{-3}–10^{-1}	Mittal and Morgenstern, 1976
Molybdenum beach sands	10^2	Nelson et al., 1977
Gold slimes	6.3×10^{-2}	Blight and Steffen, 1979
Lead–zinc slimes	10^{-2}–10^{-4}	Kealy et al., 1974
Fine coal refuse	3×10^{-3}–10^{-2}	Wimpey, 1972
Bauxite slimes	10^{-3}–5×10^{-2}	Somogyi and Gray, 1977
Phosphate slimes	2×10^{-4}	Bromwell and Raden, 1979

by Mittal and Morgenstern (1976) are shown in Figure 2.7. For these materials, c_v increases with decreasing void ratio, the opposite of behavior generally shown by natural clays at void ratios corresponding to stresses in the range of the preconsolidation pressure. On the other hand, data reported by Blight and Steffen (1979) indicate slightly decreasing c_v for decreasing void ratio. The picture is further complicated by phosphate slimes, which show constant c_v over a wide range of void ratios (Bromwell and Raden, 1979) and by copper–zinc and molybdenum slimes, which show no consistent trend (Kealy et al., 1974).

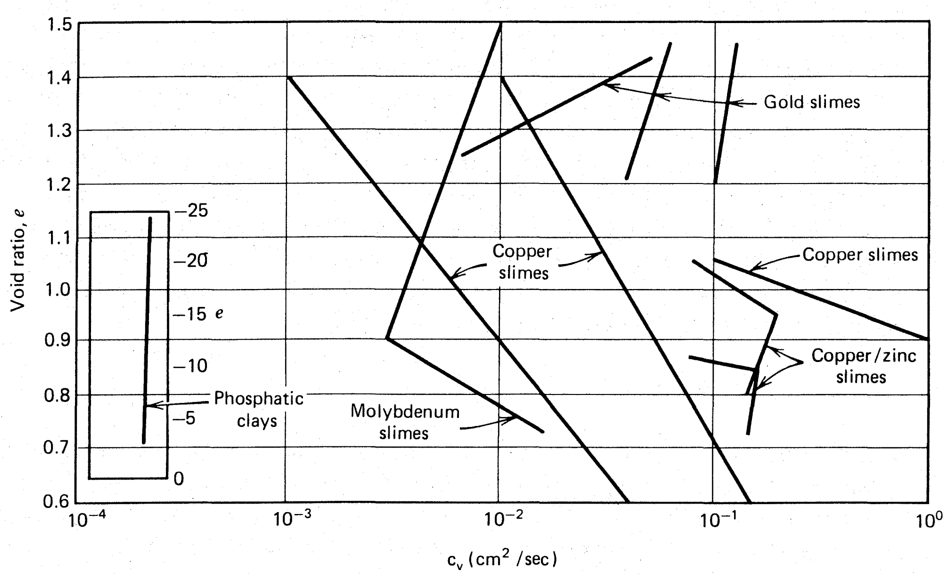


Figure 2.7 Variation in c_v with void ratio for various slimes tailings. (Copper slimes: Mittal and Morgenstern, 1976; gold slimes: Blight and Steffen, 1979; phosphatic clays: Keshian et al., 1977; copper–zinc and molybdenum slimes: unpublished.)

The coefficient of consolidation can be expressed in terms of permeability and stress–strain characteristics in one-dimensional compression by the following relationship:

$$c_v = \frac{k}{\gamma_w m_v}$$

where k = permeability
γ_w = unit weight of water
$m_v = \frac{\partial \epsilon}{\partial \sigma}$ (ϵ = strain, σ = stress)
= coefficient of volume change

Thus, the change in c_v with void ratio is related to the corresponding change in permeability and the rate of change in strain as a function of stress. Unlike natural soils, where permeability change usually has the dominant influence, the behavior of tailings slimes is more complex. Permeability trends may dominate at some void ratios and stress–strain characteristics at others. This complexity precludes valid generalizations concerning the effects of void ratio on c_v and suggests that any such variation must be determined by laboratory tests on each unique material.

For most types of tailings, secondary compression produces continuing deformation under constant load even after pore pressure dissipation associated with primary consolidation is essentially complete. Secondary compression of sand and nonplastic slimes tailings may be attributable to continuing particle rearrangement and grain-to-grain slippage under the influence of load. Lee et al. (1967) suggest that continuing fracture propagation at grain-to-grain contacts promoted by the presence of water may be responsible for secondary compression of some highly angular natural soils, and these observations likely also apply to angular tailings particles. Secondary compression of tailings, however, is usually small and relatively insignificant from a practical standpoint compared to primary consolidation. The exception is for such materials as gypsum tailings, where creep effects dominate consolidation behavior to the extent that classical Terzaghi consolidation theory no longer applies (Vick, 1977).

Drained Shear Strength

Their generally loose depositional state notwithstanding, tailings have high drained (effective-stress) shear strength owing primarily to their high degree of particle angularity. It is not uncommon for tailings to show an effective friction angle ($\bar{\phi}$) 3–5° higher than that of similar natural soils at the same density and stress level. With rare exceptions, tailings are cohesionless and show a zero effective cohesion intercept $\bar{c}$ in properly performed and interpreted laboratory tests.

In the laboratory $\bar{\phi}$ is commonly measured using either consolidated-drained (CD) triaxial or direct shear tests, or consolidated-undrained (CU) triaxial tests with pore pressure measurements. Direct shear tests on tailings must be run at a slow rate of strain and under fully saturated testing conditions in order to avoid buildup of either positive or negative pore pressures which can yield erroneous results, usually manifested as an apparent cohesion intercept. Backpressure saturation with pore pressure measurement in triaxial tests can eliminate these difficulties.

Effects of Density

The effect of initial density (or void ratio) on the effective-stress strength of tailings is surprisingly small. Over the range of densities commonly encountered in tailings deposits, $\bar{\phi}$ seldom varies more than about 3–5° for sand tailings. Similarly, overconsolidation, if present, usually has relatively little effect on $\bar{\phi}$ for slimes tailings.

Effect of Stress Level

The most important factor influencing $\bar{\phi}$ for tailings is the stress range over which it is measured. Even at relatively low levels of applied stress, stresses at the point-to-point contacts of the angular grains are very high, producing particle crushing. The result is often curvature of the strength envelope, especially at low applied stress. Figure 2.8 shows a particularly pronounced envelope curvature for a loose tailings sand, with resulting tangents ranging from 41° to 29° for stresses from 0 to 3,000 psf. Data at higher stress ranges

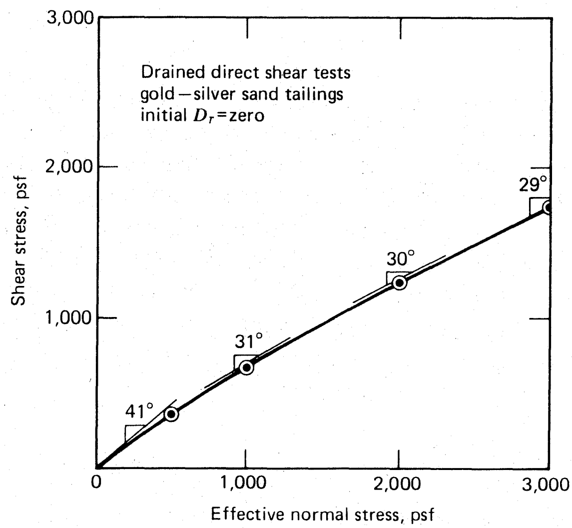


Figure 2.8 Strength envelope curvature at low stress levels.

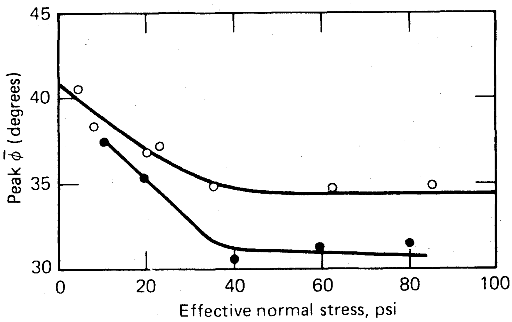


Figure 2.9 Variation in $\bar{\phi}$ with stress level. (From Mittal and Morgenstern, 1975, by permission of the National Research Council of Canada.)

and for denser sand tailings are shown in Figure 2.9. For these materials, the combined effects of particle crushing and dilatency are most pronounced at stresses up to about 40 psi; $\bar{\phi}$ at higher stresses remains relatively constant.

The stress dependence of $\bar{\phi}$ shown by tailings is well documented in the literature for other types of materials. Lee and Seed (1967) report the effects of particle crushing on loose, angular natural sands, and similar effects are noted by Marsal (1973) for angular rockfill fragments. Effective cohesion intercepts for tailings sometimes reported in the literature usually result either from testing errors previously discussed or from erroneous linear extrapolation of curved failure envelopes to low stress levels.

Typical Values

Typical values of $\bar{\phi}$ for various materials are shown in Table 2.8, based on laboratory tests of both undisturbed and remolded samples. In most cases, the tests were performed on samples either at an initial density representative of that of the tailings deposit or at the minimum density for the particular material. For most materials, $\bar{\phi}$ falls generally within the range of 30–37°, averaged over a wide stress range. There is little variation between $\bar{\phi}$ for sands and slimes tailings, except for fine coal refuse, which may contain layers of more highly plastic clay resulting in lower maximum values. In general, only tailings showing in-place cementation, notably gypsum, have a significant effective cohesion intercept.

Undrained Shear Strength

Undrained shear strength implicitly accounts for the pore pressures generated by rapidly-applied shear stresses, and is important in evaluating

Table 2.8 Typical Values of Drained Friction Angle $\bar{\phi}$

Material	$\bar{\phi}$ (degrees)	Effective-Stress Range (psf)	Source
Copper			
Sands	34	0–17,000	Mittal and Morgenstern, 1975
	33–37	0–14,000	Volpe, 1975
Slimes	33–37	0–14,000	Volpe, 1975
Molybdenum beach sands	32–38	—	Nelson et al., 1977
Taconite			
Sands	34.5–36.5	—	Guerra, 1979
Slimes	33.5–35	—	Guerra, 1979
	27–32	—	Klohn, 1979a
Lead–zinc–silver			
Sands	33.5–35	—	McKee et al., 1979
Slimes	30–36	—	McKee et al., 1979
Gold slimes	28–40.5	0–20,000	Blight and Steffen, 1979
Fine coal refuse	22–39	0–6,000	Wimpey, 1972
	22–35	0–25,000	Wimpey, 1972
Bauxite slimes	42	0–4,000	Somogyi and Gray, 1977
Gypsum tailings	32 ($\bar{c}$ = 500 psf)	0–10,000	Vick, 1977

the flowlike behavior exhibited by many tailings deposit failures. In the laboratory, undrained shear strength is most commonly determined by CU triaxial tests, which produce the total-stress strength parameters ϕ_T (total-stress friction angle) and c_T (total-stress cohesion). Unlike the often illusory effective-stress cohesion, total cohesion, which may be exhibited by some tailings in CU tests, is a "real" phenomenon. Its measurement, however, requires sufficient backpressure during testing to prevent pore water cavitation (Seed and Lee, 1967).

Typical Values

Typical values of ϕ_T and c_T for various types of tailings are shown in Table 2.9, as determined from CU triaxial tests performed mainly on undisturbed

Table 2.9 Typical Total-Stress Strength Parameters

Material	Initial Void Ratio e_0	Total Friction Angle ϕ_T (deg.)	Total Cohesion c_T (psf)	Source
Fine coal refuse	0.5–0.8	16–24	600–1,500	Wahler, 1973
Molybdenum sands	0.8	14	800	Unpublished
Copper tailings, all types	—	13–18	0–2,000	Volpe, 1979
Copper beach sands	0.7	19–20	700–900	Wahler, 1974
Copper slimes	0.6	14	1,300	Wahler, 1974
	0.9–1.3	14–24	0–400	Wahler, 1974
	1.1	14	0	Unpublished
Lead–zinc slimes	0.8–1.0	21	0	Unpublished
Gold slimes	—	28	0	Blight and Steffen, 1979
Bauxite slimes	—	22	100	Somogyi and Gray, 1977

and normally consolidated samples. The strength values correspond to initial void ratios generally typical of those encountered within most tailings deposits. ϕ_T is in the range of 14–24°, roughly 15° less than $\bar{\phi}$ for similar materials. Total cohesion c_T, where present, ranges generally up to about 1,500 psf. There is little apparent variation between sands and slimes materials. Within the range of initial void ratios at which the tests were performed, there appears to be little systematic effect of void ratio on ϕ_T for the rather limited available information. Void ratio, however, does appear to influence c_T. Initial void ratios for denser materials in the range of 0.5–0.8 are associated with higher total cohesion, whereas total cohesion for looser materials at initial void ratios in excess of about 1.0 (principally slimes) tends to be lower or absent altogether.

Other Approaches

Alternative approaches to determining undrained shear strength as a function of effective consolidation stress, or depth, include undrained direct shear tests and field vane tests.

Some natural clays exhibit "normalized" undrained strength behavior according to the SHANSEP concept proposed by Ladd and Foott (1974). Consolidated-undrained shear strength is best measured in direct shear tests, which, unlike triaxial tests, do not induce artificially high undrained shear strength due to isotropic consolidation. A rapid rate of shearing alone is not sufficient to produce true undrained conditions during direct shear testing; normal stress must be varied throughout shearing to produce constant-volume conditions.

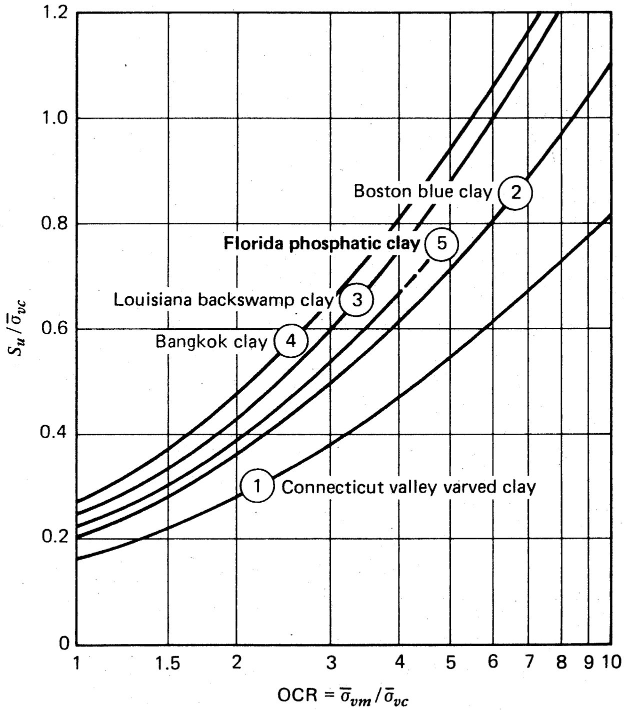
According to the SHANSEP concept, the normalized strength ratio $S_u/\bar{\sigma}_c$ is independent of the magnitude of effective consolidation stress $\bar{\sigma}_c$ and varies only as a function of the overconsolidation ratio (OCR). Data presented in Figure 2.10a show that phosphate slimes exhibit such normalized strength behavior.

Figure 2.10b shows the results of similar testing on slurry-sedimented samples of nonplastic lead–zinc slimes. $S_u/\bar{\sigma}_c$ is not independent of consolidation stress, and the divergence between the two stress-level curves increases with increasing OCR. In general, this tailings material does not conform to normalized behavior. However, Figure 2.10b also shows that, for normally consolidated material (OCR = 1.0), the divergence is small, and for practical purposes $S_u/\bar{\sigma}_c = 0.20$. Interestingly, this value compares reasonably well with data in Figure 2.10a for normally consolidated phosphate slimes. While the SHANSEP concept may not apply in a strict sense to tailings, it may be useful for conservative lower-bound estimates of the undrained shear strength of some types of normally consolidated slimes.

Undrained shear strength has also been determined directly in the field by the vane shear device (Wahler, 1973; Wimpey, 1972), with rapid rates of shearing used in an attempt to maintain undrained conditions during testing. Others, however, have advocated the use of field vane shear tests for determining effective-stress strength parameters of tailings (Blight, 1970; Blight and Steffen, 1979) by using slow rates of shearing to produce full drainage.

For nonplastic materials, such as tailings slimes, even rapid shearing does not insure the constant-volume conditions required for determination of true undrained shear strength. Moreover, the presence of thin sandy seams that would promote drainage during testing, even with rapid shear rates, is not only possible but probable in a given slimes deposit because of the layered nature of the material. The use of the vane shear test to determine drained strength parameters is afflicted by similar problems. Slow rates of shearing sufficient to prevent pore pressure buildup for materials of "average" characteristics do not account for the possible presence of very fine-grained or plastic layers within the tested zone with much slower pore pressure dissipation characteristics. The actual drainage characteristics during shear are difficult to determine with certainty in tailings irrespective of vane rotation rates.

Field vane shear data from copper tailings in Figure 2.11 illustrate this problem. Along with the vane shear data are shown both effective-stress and undrained shear strengths predicted from laboratory testing. The vane shear data are bracketed by the undrained and effective-stress strength envelopes, making it difficult to specify just what drainage conditions the vane shear tests actually represent. While vane shear tests may yield useful index data, in the same category as standard penetration blowcounts, they should be



(*a*)

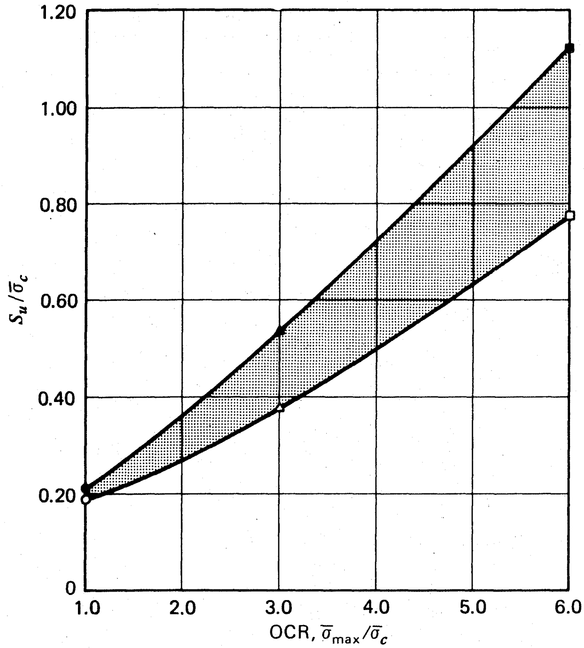
Figure 2.10(*a*) $S_u/\bar{\sigma}_c$ from undrained direct shear tests. Phosphate slimes compared to natural clays. (Reprinted from Bromwell and Raden, 1979.)

used with great caution in developing shear strengths, either drained or undrained, for use in analysis.

Stress–Strain Behavior in Triaxial Shear

The stress–strain characteristics of tailings in triaxial shear are generally similar to those of loose to medium dense natural soils of similar gradation. At densities typically resulting from hydraulic tailings sedimentation, significant dilatency seldom occurs. Stress–strain data spanning a wide range of tailings types and confining pressures are often marked by relatively high strain at failure and little or no reduction in post-failure strength at large strains.

Figure 2.12 shows deviator stress, effective-stress ratio, and pore pressure versus strain curves generally typical of those exhibited by most tailings in CU triaxial loading at initial void ratios representative of those that occur within the deposit. Deviator stress (or shear stress) versus strain curves typically rise continually throughout the test to strains at or in excess of 5 to



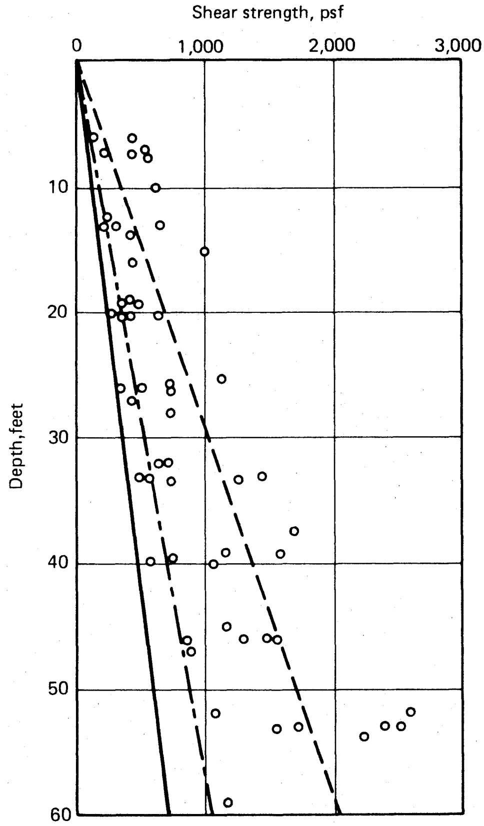
Symbol	Maximum applied effective stress $\bar{\sigma}$ max (psf)	Applied effective stress at start of shear $\bar{\sigma}_c$ (psf)	Overconsolidation ratio, OCR = $\bar{\sigma}_{max}/\bar{\sigma}_c$
○	1000	1000	1.0
●	5000	5000	1.0
△	3000	1000	3.0
▲	6000	2000	3.0
□	3000	500	6.0
■	6000	1000	6.0

(*b*)

Figure 2.10 (*b*) $S_u/\bar{\sigma}_c$ from undrained direct shear tests. Nonplastic lead–zinc slimes sedimented samples.

10%, often without showing a well-defined peak or subsequent post-failure reduction.

Post-peak reductions in shear stress, where encountered, most commonly occur for samples at lower initial void ratios corresponding, for example, to materials that have received mechanical compaction. In such cases, post-peak reduction in strength is most pronounced at lower confining stresses (less than 1,000–2,000 psf). Peak shear strength is commonly developed at roughly 2–4% axial strain in such tests, and post-peak shear strengths at large strains may be as little as 50% of peak values. This reduction becomes less pronounced as confining stress increases, and at higher confining



— — Effective-stress strength predicted from $\bar{\phi}$ = 35°

- —— - Undrained shear strength predicted from ϕ'_T = 20°

——— Undrained shear strength predicted from S_u/σ_c = 0.20

o Vane shear data (Wahler, 1973)

Figure 2.11 Comparison of laboratory and vane shear strength data.

stresses, the axial strain corresponding to peak shear stress usually increases to 5% or more.

Typical pore pressure versus strain response is also shown in Figure 2.12. At typical initial void ratios, pore pressure generally follows a trend similar to that of shear strength, but with a tendency to reach a peak value, then remain constant or decrease slightly at large strains. For tailings at high initial void ratios corresponding to initial sedimentation, pore pressure may not reach a maximum value until strain in excess of 10% or more has occurred.

Effective-stress ratio versus strain usually shows a peak that is most

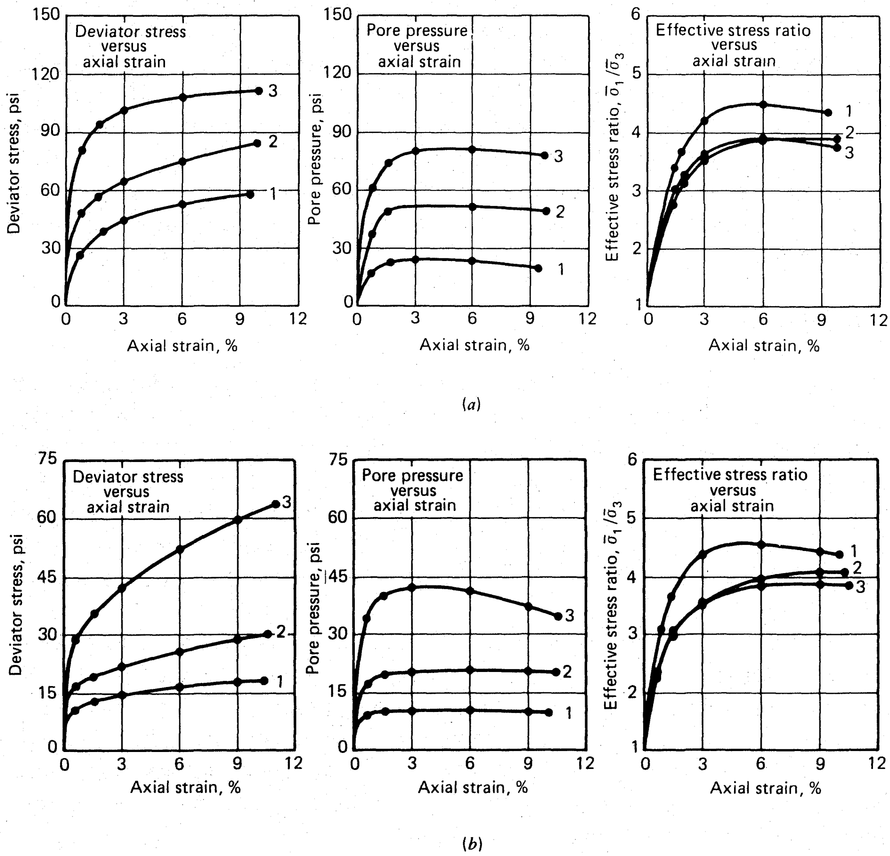


Figure 2.12 Typical stress–strain characteristics. (*a*) Fine coal refuse: 1 $\bar{\sigma}_c = 40$ psi; 2 $\bar{\sigma}_c = 80$ psi; 3 $\bar{\sigma}_c = 120$ psi, $e_o = 0.3–0.8$. (Reprinted from Wahler, 1973.) (*b*) Copper slimes: 1 $\bar{\sigma}_c = 15$ psi; 2 $\bar{\sigma}_c = 30$ psi; 3 $\bar{\sigma}_c = 60$ psi, $e_o = 0.9–1.2$. (Reprinted from Wahler, 1974.)

pronounced for tests at lower confining stresses, as shown in Figure 2.12. The peak effective-stress ratio occurs at higher strain and is more poorly defined as confining pressures increase. This behavior is in good agreement with that reported by Lee and Seed (1967) for angular natural sands at low relative densities.

Data on tailings presented by Highter and Tobin (1980), however, indicate a very pronounced peak and subsequent post-peak reduction in shear stress, as measured by Bishop's brittleness index, even at high confining pressures. These data also suggest that the normalized difference between peak and post-peak shear strength may increase with increasing confining stress. This behavior does not appear to correspond well with most available data on typical tailings materials, and the reason for the discrepancy is not immediately apparent. However, one possible explanation may lie in testing methods. The high degree of post-peak strength reduction noted by Highter and Tobin was produced in stress-controlled tests, as opposed to the strain-controlled tests used nearly exclusively for testing of tailings outside a research context. As noted by Bishop (1973), strain-controlled tests may not accurately reflect post-peak strength reductions that occur under undrained conditions, suggesting that stress-controlled tests may be necessary to evaluate post-peak strength reduction in cases where this is a critical factor in design—for example, where large strains may occur rapidly within the tailings deposit.

Cyclic Strength

Among the factors that influence cyclic strength of a given soil deposit are relative density, grain characteristics, method of deposition, aging effects, and previous seismic history (Seed, 1976). In contrast to most natural soil deposits, tailings are highly angular, have been deposited by very specific hydraulic mechanisms, and have been recently deposited and unlikely to have experienced previous seismic shaking. Consequently, cyclic strength of natural soils may not correlate directly to that of tailings at a similar relative density.

As with natural cohesionless soils, cyclic strength of tailings is governed by relative density, but relative densities of tailings are lower than those for many natural soil deposits. As a result, tailings may undergo large cyclic strains after a relatively few cycles of stress reversal. Attainment of approximately 10% cyclic strain, commonly used as a strain-related failure criterion, often more or less coincides with initial liquefaction (pore pressure equal to confining pressure) for many tailings samples.

Figure 2.13 summarizes data on tailings cyclic shear strength at densities typical of those produced by hydraulic deposition. The data in Figure 2.13 are plotted in the commonly accepted format of the ratio of cyclic shear stress to confining stress ($\Delta\sigma/2\bar{\sigma}_c$) versus number of cycles for either initial liquefaction or 10% cyclic strain, whichever occurs first during testing. The number of cycles required to produce failure conditions is highly sensitive to applied cyclic shear stress ratio for all materials shown. Relative density, even over the limited range exhibited by hydraulically deposited tailings, also has a major influence on cyclic shear strength.

It is of interest to note from Figure 2.13 that slimes tailings overall have generally higher cyclic shear strength than do sands or undifferentiated tailings. This appears to be in conceptual agreement with observations reported by Seed (1983), Finn (1982), and others of the greater liquefaction resistance of silty compared to clean materials—both natural soils and tailings. Corresponding to the cyclic shear strength of tailings is development of pore pressure during cyclic shear. Finn et al. (1978) describe methods for predicting cyclic pore pressure buildup in sand tailings, while Moriwaki et al. (1982) present pore pressure data for slimes tailings.

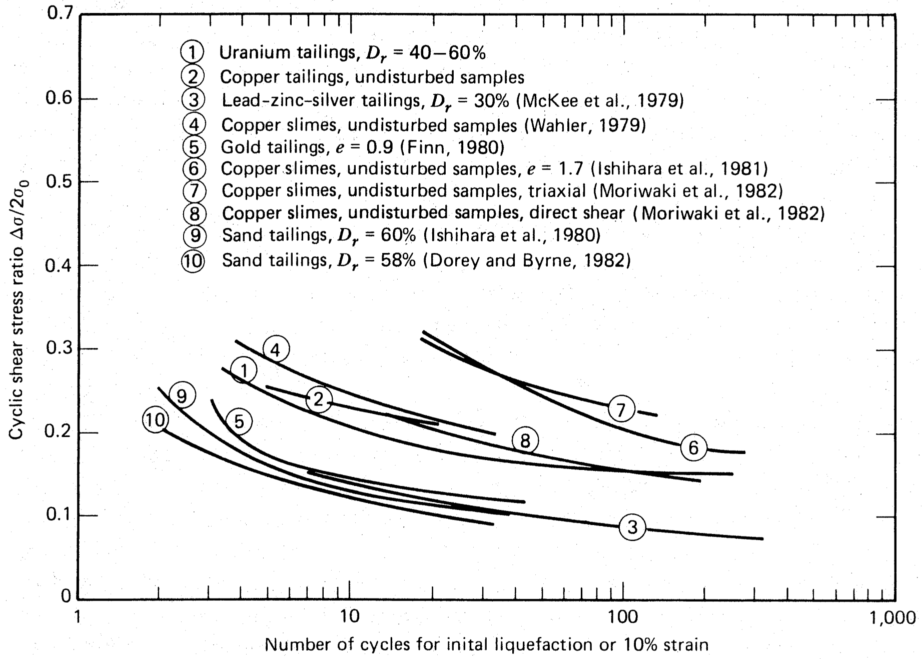


Figure 2.13 Cyclic shear strength (from isotropically consolidated cyclic triaxial tests unless otherwise noted).

In a study of hydraulic fill placed for earth dams, Meehan (1976) compared data from 120 cyclic triaxial tests. While likely lacking the particle shape of tailings, many of the materials tested were probably similar to tailings deposits with regard to gradation, relative density, and mode of deposition. These studies of hydraulic fill, as well as similar studies for nonplastic silt (Donovan and Singh, 1976), indicate that, at low effective confining stress, cyclic shear resistance is relatively constant. Thus, the ratio $\Delta\sigma/2\bar{\sigma}_c$, which implies a direct proportionality between cyclic shear resistance and effective confining stress, may be misleading if applied over all depth ranges, particularly for relatively shallow tailings deposits. Data on copper sand and slimes tailings presented by Volpe (1975) suggest that cyclic shear strength is directly proportional to effective confining pressure only at effective stresses higher than about 2,000 psf. Thus, it is necessary to account explicitly for the effect of confining pressure and depth when determining cyclic shear strength of tailings deposits.

SUMMARY

The engineering behavior of tailings is determined not only by the characteristics of the material itself, but also by the nature of the deposit. Tailings deposition by spigotting usually results in two distinct classes of material: sands deposited by hydraulic sorting mechanisms, and finer grained slimes deposited by sedimentation. Sands and slimes may be encountered either in reasonably distinct zones within a deposit, or they may be highly interlayered and poorly defined, with the degree of differentiation dependent on such factors as gradation of the whole tailings, pulp density of the discharged slurry, and spigotting procedures.

Engineering properties are sometimes significantly different for sands and slimes. Sand tailings generally share the engineering behavior of loose to medium-dense deposits of natural sand. Slimes, on the other hand, are a complex material that may exhibit properties similar to natural sands in some cases, natural clays in others, or a combination of both in still others.

Permeability is probably the most variable property exhibited by tailings and may span a range of five orders of magnitude depending on the type of tailings and depositional factors specific to a particular impoundment. Compressibility is not so widely varying, but tailings are generally more compressible than are corresponding types of natural soils because of the looser state they usually assume upon deposition. Consolidation characteristics are a function of both permeability and compressibility, and as a result are highly complex. Consolidation characteristics are important, however, in evaluating the time rate of pore pressure dissipation within a tailings deposit.

The drained, or effective-stress, shear strength of tailings, both sands and slimes, is often higher than that for similar natural soils because of the high degree of particle angularity exhibited by most tailings. This same characteristic, however, often results in an effective-stress friction angle that is sensitive to applied stress level. Undrained shear strength, on the other hand, may be particularly important in evaluating susceptibility of tailings deposits to flow-type sliding. Here the initial density of the material is a significant factor. In addition, the difference between peak and post-peak strength, a function of the material stress–strain characteristics and testing procedures, may be important. Cyclic strength of tailings used in evaluating seismic behavior of the deposit is also sensitive to initial density, and this factor may make tailings more susceptible than similar natural soil deposits to seismic liquefaction.

In summary, the engineering behavior of tailings must of necessity be interpreted in the context of classical soil mechanics theories for behavior of natural soils. However, proper application of these theories requires accounting for the unique characteristics of tailings and recognizing that there are cases where the classical soil behavior theories may not apply in their conventional sense.

3

Tailings Disposal Methods

The tailing heap constitutes the chief ornament of the concentrator or mill and often enough appears to be merely a slope of dead-white silt, although it is occasionally stained in a gaudy fashion by chemical action on the metal residues.

Otis Young, *Western Mining*

Central to tailings disposal planning are not only the nature and engineering behavior of tailings, but also an understanding of the various methods available for tailings disposal. Dams, embankments, and related types of surface impoundments are by far the most common disposal methods and remain of primary importance in tailings disposal planning. Changing economic and regulatory conditions, however, increasingly require consideration of different or more innovative methods, such as underground backfill, in-pit backfill, and offshore disposal. The purpose of Chapter 3 is to present an overview of the full range of options available for tailings disposal in the context of planning. More detailed siting and design considerations for surface impoundments are treated in subsequent chapters.

SURFACE IMPOUNDMENTS

Surface disposal of tailings uses dams and embankments of various types to form impoundments that retain both the tailings and mill effluent. In the not-so-distant past, tailings were routinely discharged from the mill into the nearest surface water course. As this practice came to be viewed with disfavor, it required only a modest advance in technology to dam the water course, forming an impoundment in which the tailings could settle from suspension to form a deposit that was stable, at least temporarily. The prevalence of surface disposal stems partly from this historical background and also from the fact that a reasonably large surface impoundment allows for clarification of discharged mill effluent and its subsequent return to the mill for reuse.

Surface impoundments are formed by two general classes of impounding structures: water-retention type dams and raised embankments, as discussed in subsequent sections.

Water-Retention Type Dams

Water-retention type dams for tailings disposal are constructed to their full height prior to the beginning of discharge into the impoundment. As implied by their name, water-retention type dams used for tailings disposal differ little from conventional water storage structures in appearance, design, or construction. Fill usually consists of native soil borrow of various types. Typical internal zoning, such as that illustrated in Figure 3.1, includes an impervious core, drainage zones, appropriate filters, and upstream riprap. Design of filters, internal seepage control, and slopes is according to conventional earth dam technology. Upstream slopes of tailings storage dams, however, do not experience rapid drawdown and as a result can often be steeper than those of their conventional water storage counterparts.

Water-retention type dams are best suited to tailings impoundments with high water storage requirements. Examples include impoundments with large storm runoff inputs or situations where mill process constraints prevent recirculation of discharged mill effluent and require large water storage volumes or evaporation areas.

Raised Embankments

Surface impoundment structures most commonly consist of raised embankments, which differ from conventional water-retention type structures in that construction of the embankment is staged over the life of the impoundment. Raised embankments begin initially with a *starter dike* constructed usually of natural soil borrow and sized to impound often the initial two to three years' mill tailings output plus appropriate allowances for storage of flood inflows. Subsequent raises of the embankment are scheduled to keep pace with the rising elevation of the tailings and floodwater storage allowance in the impoundment. Embankment raises may be constructed using a wide range of materials, including natural borrow soils, pit mine waste, underground development muck, hydraulically deposited tailings, or cycloned sand tailings.

The advantages of raised embankments are significant. First, because construction expenditures are distributed over the life of the impoundment, initial project development costs are reduced to those necessary for construction of the starter dike. Spacing expenditures over a longer time results in a lower discounted total cost and produces cash-flow benefits that are often important in financial considerations related to mine startup.

Also, because the total volume of fill required for the ultimate embankment need not be available initially, there can be much more flexibility in the selection of materials for embankment construction. Mine waste rock or sand tailings can provide ideal construction materials if dam raising can be paced to their production rates during mining or milling. In some cases where suitable natural soils are not available, the flexibility in use of mine

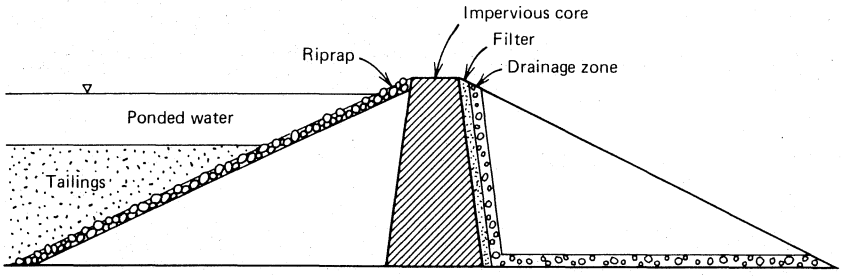


Figure 3.1 Water-retention type dam for tailings storage.

waste materials afforded by raised embankments becomes essential. Even where natural soils are readily available, waste rock, for example, must be disposed of in any event. If haul distances to the tailings embankment are not excessive, these materials provide essentially "free" dam fill, except for the additional cost associated with compaction.

Unlike water-retention type structures built initially to completion, construction of a raised embankment becomes the responsibility of the mine operator, whether performed using mine staff and equipment or by an independent contractor. Considerably more planning effort and attention to scheduling is required to raise the dam many times during the life of the operation.

Raised embankments may assume many configurations, each with unique characteristics, requirements, advantages, and pitfalls. Raised embankments, regardless of the type of material used in their construction, fall generally into three classes: upstream, downstream, and centerline. These designations refer to the direction in which the embankment crest moves in relation to its initial starter dike position as the embankment increases in height.

Upstream Method

The upstream raising method is shown in Figure 3.2. Initially a starter dike is constructed, and tailings are discharged peripherally from its crest to form a beach, as in Figure 3.2a. The beach then becomes the foundation for a second perimeter dike, as shown in Figure 3.2b. This process continues as the embankment increases in height.

Central to the application of the upstream method is that the tailings form a reasonably competent beach for support of the perimeter dikes. As a general rule, no less than 40–60% sand in the discharged whole tailings is necessary. This usually precludes use of the upstream method for tailings in the *soft-rock* or *fine* categories defined in Chapter 1, or when the sand fraction is removed from the whole tailings for use as underground mine backfill. The major advantages of the upstream method are cost and simplic-

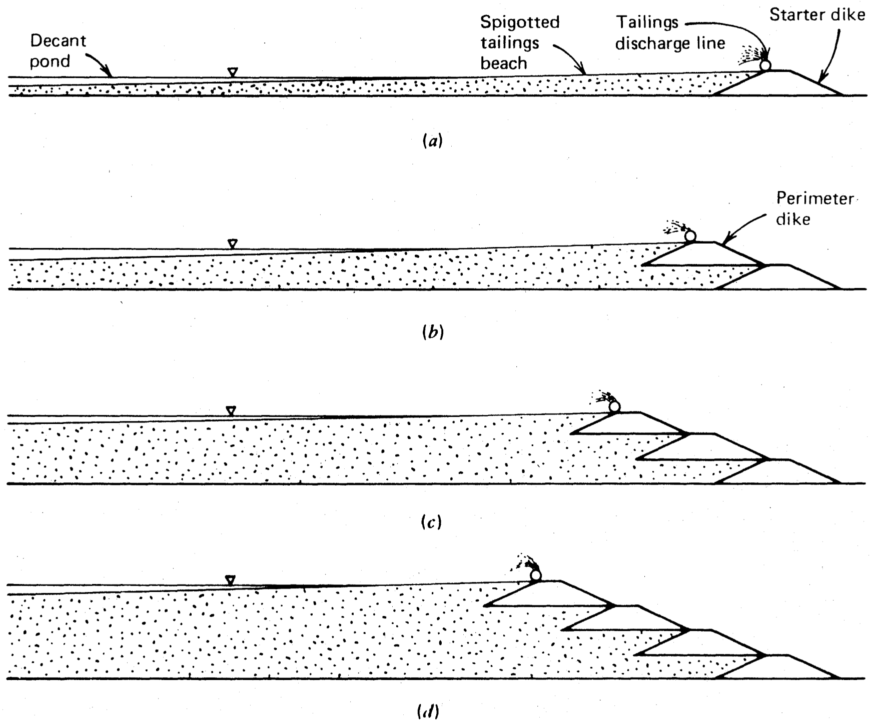


Figure 3.2 Sequential raising, upstream embankment.

ity. Only minimal volumes of mechanically placed fill are necessary for construction of the perimeter dikes, and large embankment heights can be attained at very low cost. Construction of the perimeter dikes is a simple and ongoing operation that can be routinely performed with minimal equipment and personnel. Beach sand tailings often provide a convenient source of fill for perimeter dikes, with excavation from the beach and placement by either dragline or bulldozer.

Use of the upstream raising method, however, is limited to very specific conditions and incorporates a number of inherent disadvantages. Factors that constrain the application of the upstream method include phreatic surface control, water storage capacity, and seismic liquefaction susceptibility.

The location of the phreatic surface is a critical element in determining embankment stability. For upstream embankments constructed by tailings spigotting, there are few structural measures for control of the phreatic surface within the embankment. Figure 3.3 shows that the most important factors influencing the phreatic surface location are the permeability of the foundation relative to the tailings, the degree of grain-size segregation and lateral permeability variation within the deposit, and the location of the

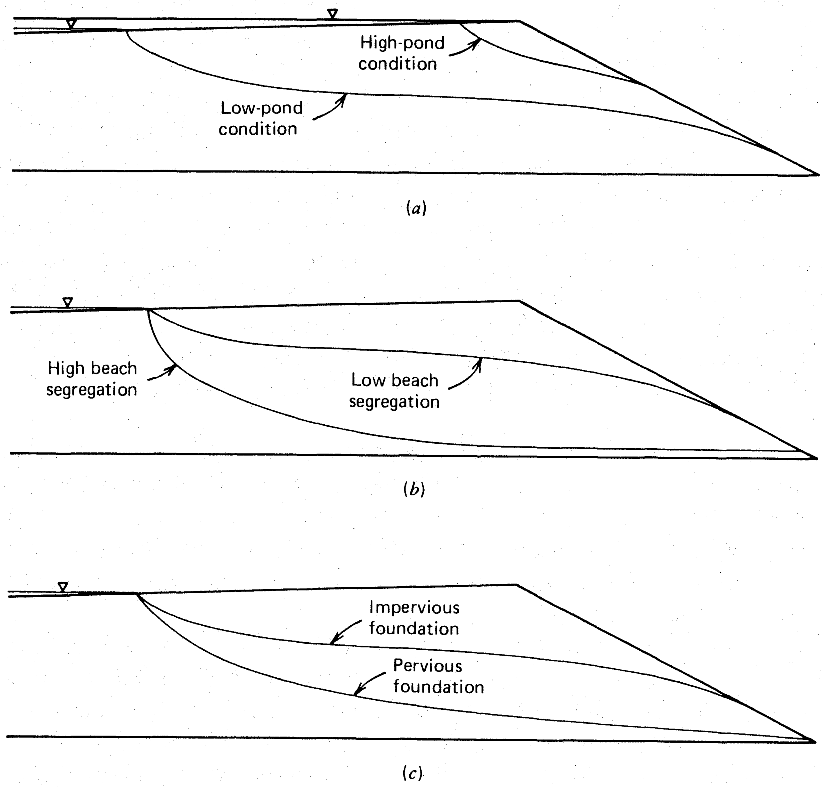


Figure 3.3 Factors influencing phreatic surface location for upstream embankments. (*a*) Effect of pond water location. (*b*) Effect of beach grain-size segregation and lateral permeability variation. (c) Effect of foundation permeability.

ponded water relative to the embankment crest. Although cycloning can be used to promote segregation of sands and slimes within the deposit and such measures as underdrains can be used to have the effect of increasing foundation permeability, pond water location is the only factor influencing the phreatic surface that can be controlled during operation.

As shown in Figure 3.3a, pond water encroachment on the tailings beach produces very high phreatic conditions near the embankment face, which endanger stability. In extreme cases, overtopping and consequent embankment breaching result. Many if not most failures of upstream embankments can be attributed to inadequate separation distance between the decant pond and the embankment crest. Ponded water can be pushed back from the embankment crest during operation by proper tailings spigotting and decant procedures. Increase in decanting rates lowers the pond elevation and increases the pond–crest separation distance. Similarly, increased tailings spigotting at critical areas of encroachment can be used to push the water back by building a higher and wider beach. However, inasmuch as peripheral spigotting is often curtailed during the winter in very cold climates, the usefulness of spigotting for pond water control may be limited.

While these water-control measures can be effective during normal operation, control of ponded water and its effects on the phreatic surface is difficult under the influence of appreciable flood or normal runoff inflows. For example, assuming a typical 1% tailings beach slope, each 1-ft rise in the elevation of ponded water will produce 100 ft of pond water encroachment on the beach. For this reason, upstream embankments are poorly suited to conditions where water accumulation is anticipated due to flooding, long-term accumulation of seasonal runoff, or high rates of mill water accumulation. In general, upstream embankments cannot be used for water retention. Near-total diversion of runoff and flood inflow is essential for this raising method.

The susceptibility of upstream embankments to liquefaction under severe seismic ground motion is well documented (Dobry and Alvarez, 1967). The low relative density and generally high saturation within the tailings deposit can result in liquefaction-induced flow of the tailings, with disastrous consequences. Upstream raising methods are clearly inappropriate in areas of high seismic potential.

Finally, the rate at which upstream embankments can be safely raised is limited. Raising rates are determined by the rate of mill tailings production and topographic configuration of the impoundment site. Rapid rates of height increase can produce excess pore pressures within the deposit, particularly in slimes zones because of the lower coefficient of consolidation of this material. Mittal and Morgenstern (1976) suggest that excess pore pressures in slimes will result from raising rates in excess of 15–30 ft/yr. Failures attributable to excess pore pressures in relatively clean sand tailings have been experienced at raising rates of 150 ft/yr (Mittal and Hardy, 1977).

Upstream embankments, while providing the simplest and least costly raising method, are subject to a number of very critical constraints. Proper use of the method can be justified only when these constraints are thoroughly investigated and satisfied.

Downstream Construction

The downstream raising method is illustrated in Figure 3.4. Initially tailings are discharged behind a starter dike. Subsequent raises are constructed by placing embankment fill on the downstream slope of the previous raise. This method is amenable to the incorporation of structural measures within the embankment (for example, impervious cores and internal drains) for positive control of the phreatic surface. Use of these measures can allow for storage of significant water volumes directly against the inner face of the embankment. In some cases, however, peripheral spigotting of a wide and well-controlled tailings beach can result in good phreatic surface control without

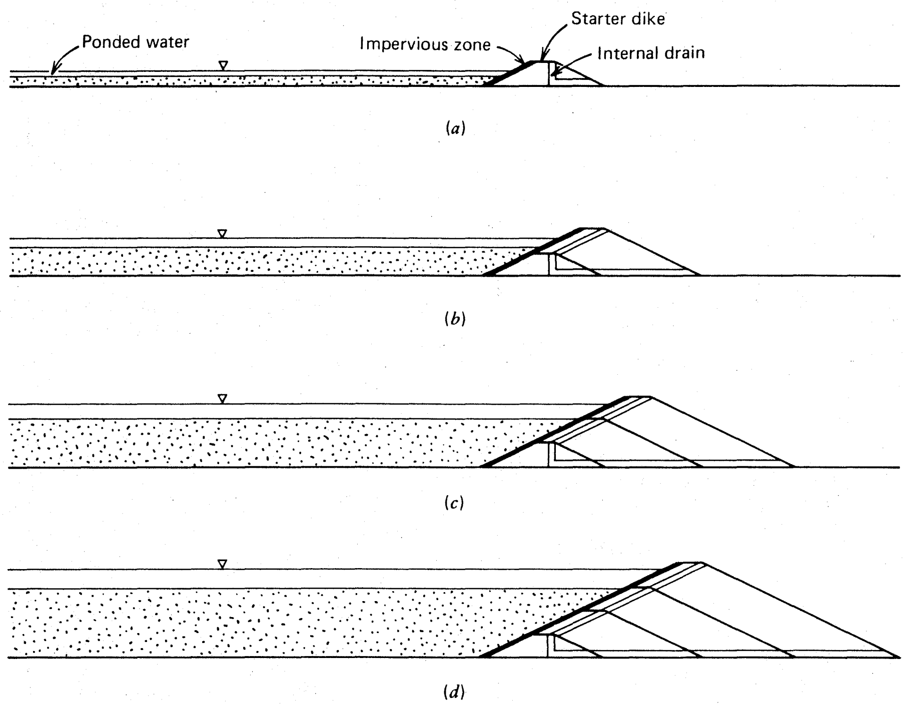


Figure 3.4 Sequential raising, downstream embankment.

the need for internal impervious zones and drains, provided that ponding of water against the inner face of the embankment does not occur and provided also that the embankment itself is sufficiently pervious.

In general, downstream raising methods are well suited to conditions where significant storage of water along with the tailings is necessary. Because the phreatic surface can be maintained at low levels within the embankment and because the entire body of the fill can be compacted, downstream raising methods are liquefaction resistant and can be used in areas of high seismicity. Unlike upstream embankments, raising rates are essentially unrestricted because the downstream raises are structurally independent of the spigotted tailings deposit. Downstream raising methods are essentially equivalent in structural soundness and behavior to water-retention type dams.

Downstream raising methods, however, require careful advance planning. Because the toe of the dam progresses outward as its height increases, sufficient space must be left during layout of the starter dike to prevent encroachment of the dam toe on property lines, roads, utilities, diversion ditches, or topographic constraints. The ultimate height of the embankment is often determined by such restrictions at the toe.

The major disadvantage of the downstream raising method is the com-

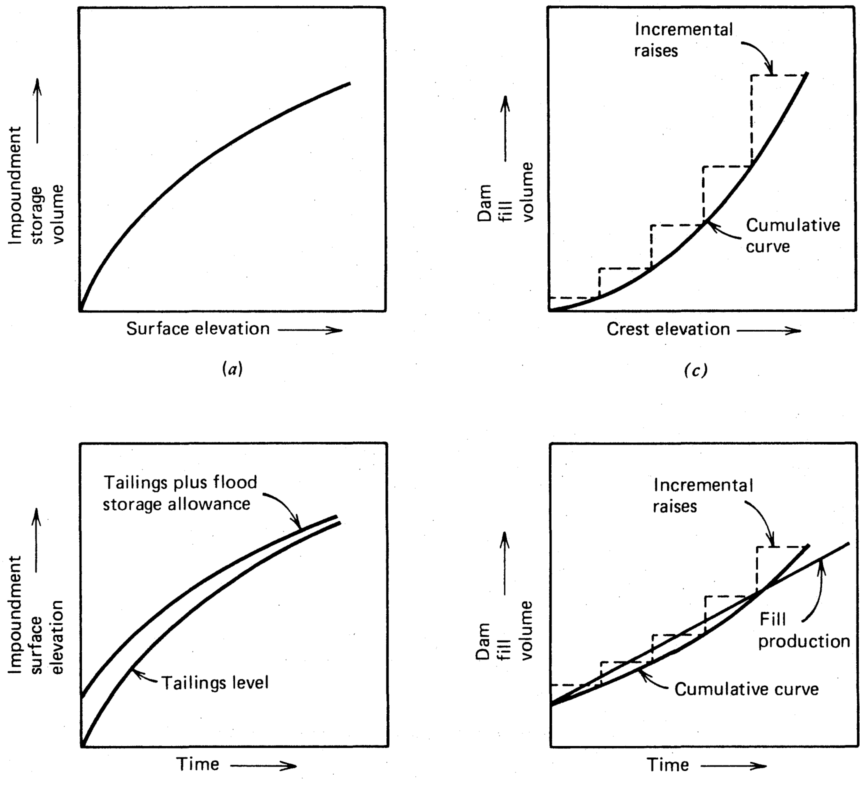


Figure 3.5 Typical elevation and time relationships for impoundment and embankment fill volumes. (*a*) Impoundment volume–elevation curve. (*b*) Impoundment elevation versus time. (c) Dam fill volume versus elevation. (*d*) Dam fill volume versus time.

paratively large volume of embankment fill required and the corresponding high cost. The availability of fill for various raises of the dam may also impose constraints on construction. In particular, if mine waste or sand tailings are used for embankment construction, these materials will be produced at a more or less constant rate. The volume of fill required for each successive downstream raise, however, often increases exponentially as the embankment increases in height. Consequently, advance planning is required to ensure that fill material production rates will be sufficient at all times during the life of the embankment.

This problem is illustrated in Figure 3.5. Figure 3.5a shows the elevation–volume curve for the impoundment, which is strictly a function of topography. For a given rate of mill tailings discharge, the impoundment elevation versus time curve in Figure 3.5b can be derived. The elevation of the tailings surface starts at zero and increases with time, but typically at a decreasing rate. In addition, an impoundment depth increment sufficient for storage of storm runoff inflow must also be accounted for, as shown by the higher curve in Figure 3.5b. Storm runoff volume is usually constant over time, and the resulting impoundment depth allowance at time zero represents the starter dike flood storage capacity. Although the volume remains constant, the depth required to retain this volume usually decreases with time because of the typical increase in impoundment area at higher elevations.

Figure 3.5c shows the volume of dam fill required as a function of dam crest elevation, a relationship determined by the cross-section of the dam and topography along the dam alignment. For the downstream method, each raise of constant height requires increasingly greater fill volumes to construct. Figure 3.5d shows the dam fill volume required as a function of time, derived from Figures 3.5b and 3.5c. The volume of fill required to keep the dam crest above the elevation of the tailings (plus flood storage allowance) increases exponentially with time. Superimposed on Figure 3.5d is a constant rate of fill production, such as mine waste, assuming that the starter dike is constructed of natural soil borrow. Although curves must be established for each individual case, in the example shown fill production is adequate initially, but then becomes insufficient at higher dam elevations after longer periods of time. This problem can be resolved by constructing a higher initial starter dike of native soil borrow, shifting the fill production curve in Figure 3.5d upward.

Centerline Method

The centerline raising method is a compromise between the upstream and downstream methods in many respects. As a result, it shares to a degree the respective advantages of the two methods, while mitigating their disadvantages. The centerline method is depicted in Figure 3.6. Initially, a starter dike is constructed, and tailings are peripherally spigotted from the dike crest to form a beach. Subsequent raises are constructed by placing fill onto the beach and onto the downstream slope of the previous raise. The centerlines of the raises are coincident as the embankment progresses upward, giving rise to the method's name.

Because internal drainage zones can be provided within the embankment, control of the phreatic surface is not so sensitive to the location of ponded water as it is for upstream construction. Peripheral spigotting is necessary to form a reasonably competent and above-water beach for support of the nominal fill that must be placed upon it during raising operations. However, the width of the firm beach area need not be large, and often even predominantly slimes tailings contain a small sand fraction that is sufficient to deposit an adequate though narrow beach near the point of discharge.

Unlike downstream embankments, the centerline method cannot be used for permanent storage of large depths of water. The water can be allowed to rise temporarily during floods, however, without adversely affecting stabil-

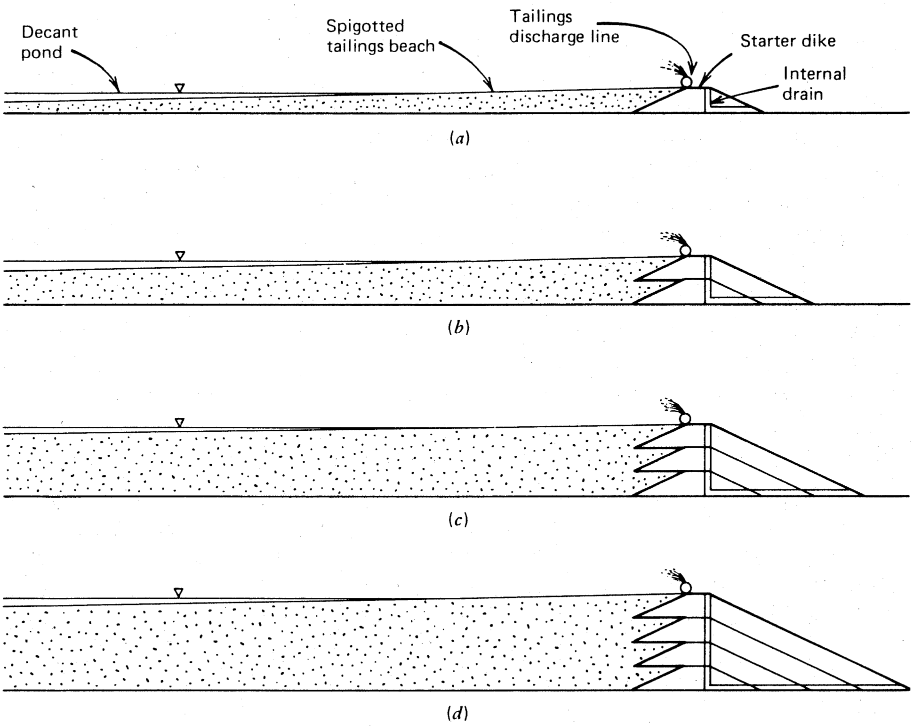


Figure 3.6 Sequential raising, centerline embankment.

ity of the structure provided that proper internal impervious and/or drainage zones are incorporated in the design.

The overall raising rate is not generally restricted by considerations related to pore pressure dissipation. However, the height of fill placed upon the beach in the upstream portion of the embankment is sometimes restricted by the undrained shear strength of the beach materials. Because the main body of the embankment fill can be compacted and saturation levels controlled by internal drainage, the centerline method has generally good seismic resistance. In the event of liquefaction of the beach tailings, limited failure of portions of the upstream fill placed upon the beach can occur. Nevertheless, as long as the central and downstream portions of the embankment remain intact and if water is not ponded directly against the embankment, the overall integrity and stability of the embankment as a whole is generally considered to be unaffected.

The volume of fill required for a given embankment height is intermediate between that for upstream and downstream methods, resulting also in intermediate costs. The compatibility between fill requirements and fill production rates (for embankments using mine waste or cycloned sand tailings) may be a problem, as discussed for the downstream method, although not to the same degree.

Comparison of Surface Impoundment Options

The selection of an appropriate surface impoundment option for a particular tailings disposal problem requires that the compatibility of the method to specific site conditions, mill tailings and effluent production, and mine production be carefully addressed. Suitability of various surface impoundment options to different conditions is summarized in Table 3.1.

Of particular interest in many cases are comparisons of different embankment types on the basis of cost. To the extent that embankment cost is proportional to total fill volume, comparison of the various embankment types in Figure 3.7 is instructive. For equivalent embankment heights, and for the particular configurations shown, downstream or water-retention type embankments require roughly three times more fill than an upstream embankment on the basis of comparative cross-sectional area. A centerline embankment would require about twice as much fill as an upstream embankment of similar height. The divergence between fill volumes for the various embankment types becomes greater with increasing height.

Embankment fill costs are a significant item in many cases, especially for high embankments and large tailings production rates. However, the contribution of embankment fill costs to the total cost of tailings disposal varies widely. In some cases, costs for impoundment area topsoil stripping, impoundment lining, or reclamation may far outweigh embankment fill costs, making comparisons between embankment types on the sole basis of fill cost potentially misleading.

Selection of embankment types has often been by unlikely combinations of historic precedent, empirical observation, and regulatory requirements rather then by strictly rational evaluation of each alternative method in a particular setting. Historically early-day (and sometimes latter-day) miners "designed" tailings embankments by trial and error. Embankment types that tended to fail in disproportionate number were gradually eliminated from the repertoire. In what was very much a process of natural selection, embankment types that proved to be poorly adapted to their environment were obliterated by their own failures. Thus, evidence that might have provided valuable lessons for contemporary designers was usually swept downstream along with the failed debris.

Curiously, however, this empirical process has resulted in historic precedents in different mining districts that sometimes bear a general relationship to more rational design principles. For example, upstream embankments are widely used in copper mining regions in the southwestern United States and in South African gold mining districts. The arid climate in these areas puts water at a premium, resulting in decant ponds that are kept low by mill recycle and few problems with water accumulation. Low seismicity in these areas has also contributed to the generally successful use of upstream methods. Upstream methods are also widely used in the lead–zinc–silver districts in northern Idaho but for entirely different reasons, again due his-

Table 3.1 Comparison of Surface Impoundment Embankment Types

Embankment Type	Mill Tailings Requirements	Discharge Requirements	Water-Storage Suitability	Seismic Resistance	Raising Rate Restrictions	Embankment Fill Requirements	Relative Embankment Cost
(1) Water-retention	Suitable for any type of tailings	Any discharge procedure suitable	Good	Good	Entire embankment constructed initially	Natural soil borrow	High
(2) Upstream	At least 40–60% sand in whole tailings. Low pulp density desirable to promote grain-size segregation	Peripheral discharge and well-controlled beach necessary	Not suitable for significant water storage	Poor in high seismic areas	Less than 15–30 ft/yr most desirable. Greater than 50 ft/yr can be hazardous	Natural soil, sand tailings, or mine waste	Low
(3) Downstream	Suitable for any type of tailings	Varies according to design details	Good	Good	None	Sand tailings or mine waste if production rates are sufficient, or natural soil	High
(4) Centerline	Sands or low-plasticity slimes	Peripheral discharge of at least nominal beach necessary	Not recommended for permanent storage. Temporary flood storage acceptable with proper design details	Acceptable	Height restrictions for individual raises may apply	Sand tailings or mine waste if production rates are sufficient, or natural soil	Moderate

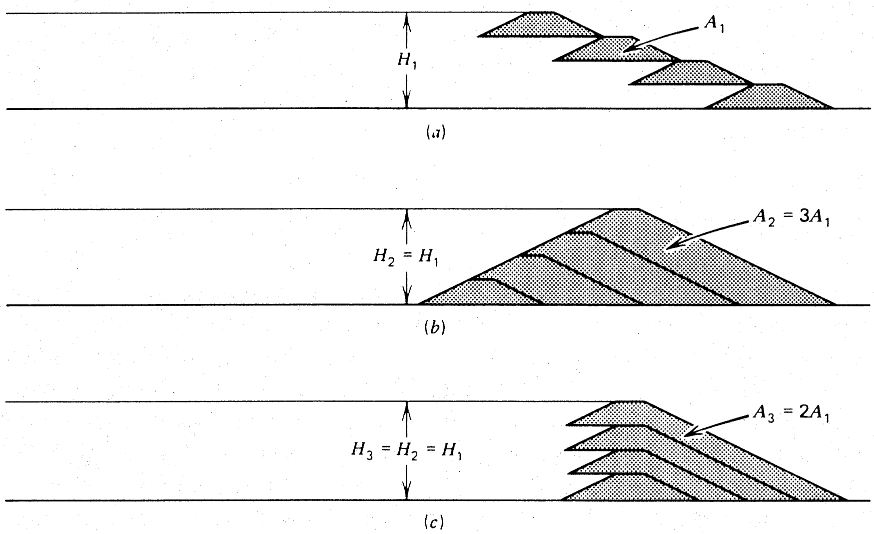


Figure 3.7 Comparison of fill volumes for various embankment types. (*a*) Upstream. (*b*) Downstream or water-retention type. (c) Centerline.

torically more to accident than design. Here, steep topography dictates impoundment sites that are often in valley bottoms. The prevalence of permeable alluvium at these sites produces inherently low phreatic surfaces in the upstream embankments that play a major role in their generally adequate performance.

On the other hand, precedent in the Missouri lead and British Columbia copper districts often favors downstream or centerline embankments using cycloned sand tailings. In Missouri, for example, high rainfall and steep topography produce large water inflows and put natural soil borrow of suitable moisture content at a premium, making downstream construction using cycloned sands attractive. Also, seismic risk in both areas weighs against upstream construction.

More recently, the regulatory arena has sometimes produced precedents that, although more sophisticated, are sometimes no less empirical. For example, water-retention type dams are sometimes favored by regulatory agencies because of their familiarity with this type of embankment in conventional water dam design and less extensive experience with other embankment types equally well suited for tailings disposal.

In spite of its sometimes dubious background, the role of precedent in establishing preferred embankment types within various mining districts should not be ignored. Trial and error has always played an essential role in all types of engineering, and certain designs have performed adequately under certain conditions usually for sound, although sometimes obscure, reasons. Nevertheless, unless the behavior of embankments in a particular setting is clearly understood, application of precedent from one area to another or from one site to another can have disastrous consequences. Precedent should supplement but obviously not replace thorough planning and analysis in the selection of a particular embankment type for surface impoundments.

UNDERGROUND DISPOSAL

Although surface impoundments are the most widely used method of tailings disposal, return of tailings to underground mines has long been practiced. Usually the purpose of underground mine backfilling using tailings has been to assist in mining operations rather than for disposal of tailings per se, with the benefits derived from reducing surface disposal requirements perceived as secondary in nature. Increasingly, however, underground disposal is being viewed as a legitimate disposal alternative as the cost and regulatory pressures associated with surface disposal increase. For underground mines, the role of tailings backfill in ground support remains of primary importance. But other underground tailings disposal methods are being more widely considered, including disposal in open pits and disposal in underground mines purely for storage purposes. Brawner (1979) summarizes a number of subsurface disposal concepts and discusses how tailings placement and mining methods are integrated for various types of mines.

Underground Mine Backfilling

The return of tailings to underground mine workings is commonly for one or more of three basic purposes:

To provide a working floor.
To provide wall support.
To maximize ore recovery.

In the cut-and-fill mining method for vein-type deposits, mining progresses upward from the bottom of the stope. Tailings backfill is an essential element of this type of operation. Tailings are discharged into the stope as the working face rises in order to provide a convenient and stable working floor. McNay and Corson (1975) describe details of backfilling procedures in cut-and-fill underground operations.

A useful adjunct to the role of backfill in providing a working floor is realized in deeper mines subject to high in situ stresses. These stresses produce serious rockburst problems in some mines as pillar or wall rock fails violently under load with resulting hazard to miners. In such mines, the lateral support provided by tailings backfill may significantly reduce rock stresses and rockburst occurrence. Corson (1971) describes the alleviation of rockburst problems by backfilling in the Idaho Coeur d'Alene lead–silver district, while Pariseau et al. (1976) describe the interaction between backfill and rock properties and present a method for predicting the support potential of tailings backfill.

Necessary properties for tailings used in backfill applications are high permeability and low compressibility. The tailings must dewater rapidly since there is insufficient area underground for a decant pond similar to that used for surface disposal. Ordinarily, this restricts backfill tailings to the sand fraction only, which is separated by cycloning at the surface leaving the slimes for separate disposal in surface impoundments. Pettibone and Kealy (1971) summarize gradation characteristics and engineering properties for a number of cycloned sand backfill tailings. In addition to high permeability, low compressibility and high stiffness are necessary if the deposited backfill is to offer wall or pillar rock support. As discussed in Chapter 2, the removal of slimes from the tailings aids significantly in reducing the compression index. Also, achieving a high in-place density further reduces compressibility; Nicholson and Wayment (1964) describe the use of vibratory compaction to increase density and reduce compressibility. Although the use of slimes as underground backfill is relatively uncommon, Sprute and Kelsh (1976) demonstrate that consolidation accelerated by the application of electrical current (electro-osmosis) can increase the rate of drainage sufficiently to provide a stable working floor for cut-and-fill operations using slimes backfill.

The use of tailings backfill to increase ore recovery places the most demanding requirements on the properties of the material. In room-and-pillar mines, for example, backfilling of stopes can allow subsequent remining and recovery of ore in pillars, adding significantly to the overall degree of recovery of the orebody (Thomas, 1971). For backfill to function effectively in this role, however, it must be free standing and sufficiently stiff to accept appreciable load transferred from the roof as pillars are mined. Achieving these properties often requires addition of cement to the sand tailings slurry, typically in proportions ranging from about 1:20 to 1:30 by dry weight (Corson, 1970). In addition to cement content, the tailings gradation range, in-place density, and pulp density of the slurry influence the strength and stiffness of cemented tailings backfill (Weaver and Luka, 1970; Singh, 1976). Sulfate-resistant cement may be required for tailings derived from high-sulfide ores.

Disposal of tailings in underground mines purely for storage purposes, outside of any mining-related function, has not been routinely performed to date. However, many room-and-pillar type mines in such materials as coal, trona, and sometimes copper may produce large volumes of otherwise unused underground space after mining in a certain area is essentially completed. Use of this space for tailings disposal can produce major advantages by reducing the area and related disturbance required for surface impoundments. This can also have a major cost advantage in cases where, for example, impoundment lining or reclamation requirements impose a severe cost penalty on surface tailings disposal. Underground disposal below the water table may be of particular advantage for tailings high in pyrite. By keeping the tailings permanently saturated underground, oxidation that could otherwise produce severe pH and heavy metal contamination problems on the surface can be reduced.

Mine Pit Disposal

Disposal of tailings in open-pit mines has been given major impetus by relatively recent regulatory requirements. In the United States, disposal of uranium tailings in either mined-out ore pits or specially excavated pits is virtually a regulatory requirement (U.S. Nuclear Regulatory Commission, 1979). Scarano (1980) reviews a number of concepts used or proposed for in-pit uranium tailings disposal. For other types of materials, disposal of tailings in mined-out pits may be cost-effective if pit backfilling is required by reclamation regulations, or simply to reduce the area and cost of surface impoundments.

Disposal of tailings in open-pit mines is best implemented where one or more separate worked-out pits are available. The pit configuration, of course, is independent of tailings disposal requirements and depends on the nature of the orebody. Disposal procedures in worked-out pits are relatively straightforward. Highland et al. (1981) describe operating procedures for a typical mine pit disposal system. Tailings are discharged into the pit either using peripheral discharge around the pit walls or by a simple single-point discharge system. Because the surface area of most mine pits is comparatively small, the area available for the decant pond is usually restricted. In some cases where more water is discharged with the tailings than can be used as mill-water recycle, notably acid-leach uranium processes, excess water must be decanted from the pit and either pumped to surface evaporation ponds or treated prior to release.

Simultaneous tailings disposal and mining in a single pit is much more complicated. For orebodies that are essentially linear, an initial pit is often excavated to the full depth of the orebody and then advanced to follow the trend of the ore. In theory, a series of water-retention type or downstream raised embankments could be constructed in the pit behind the advancing face to retain discharged tailings. However, because of the depth of the pit, typically 150–300 ft or more, it is often the case that the embankments themselves would occupy more volume than would be available for stored tailings. Also, problems of seepage through the dam abutments, with adverse effects on dam stability, pit wall stability, and mine dewatering, can be serious. Williams (1979) describes a failure of an in-pit tailings embankment in western Australia believed to have been caused by artesian pore pressures in the pit floor. For these reasons among others, simultaneous tailings disposal and mining in the same pit has been proposed but seldom implemented in practice.

Summary of Underground Options

Underground tailings disposal, either as backfill in underground mines or in worked-out pits, can offer significant advantages by reducing surface disposal requirements. Surface disposal, however, can never be completely eliminated. In the case of underground mine backfilling using sand tailings, surface impoundments for slimes are still required. Even if both fractions, sands and slimes, could be returned underground to either underground mine workings or open pits, the difference in in-place density between the tailings and the excavated rock implies that only about one-half to two-thirds of the total tailings volume could be stored underground.

Unlike surface disposal, underground disposal is not independent of mine planning or operation. When a commitment to underground tailings disposal is made, mine planning must proceed to make the necessary space available, even if different technical or economic conditions change the desirability of extracting the ore according to the original plan. Similarly, loss of flexibility in mine operation may result from the restrictions imposed by tailings backfilling on changes in day-to-day operating needs. Finally, backfilling may preclude the recovery of material of marginal mineral value at a future date. For example, a pit, once backfilled with tailings, cannot easily be deepened to extract additional lower grade ore should a future increase in ore value make it desirable to do so. In several ways, backfilling may impose rigid restrictions and limitations on the flexibility of mine planning to respond to long-term changes in economic conditions.

These limitations notwithstanding, underground disposal generally deserves more emphasis in overall tailings disposal planning. Particularly for underground mines, backfilling should be viewed as a legitimate tailings disposal option and a beneficial means of utilizing otherwise unused space, independent of any role it may play in the mining method. Strong consideration should be given to underground disposal particularly when surface, site, and/or regulatory conditions create difficult technical or economic constraints on surface impoundments. Even though above-grade disposal problems for slimes may in some cases be more difficult than for the whole unseparated tailings, these problems are usually outweighed by the resulting reduction in area and size of the remaining surface impoundment.

OTHER DISPOSAL METHODS

Other methods for tailings disposal include the thickened discharge method, "dry" disposal by belt filtration, and offshore disposal. These methods, while not widely practiced, may be useful under some specific circumstances.

Thickened Discharge Method

The thickened discharge method of tailings disposal stems from a concept first described by Shields (1974) and further developed by Robinsky (1979). The method is illustrated in Figure 3.8. Essentially the thickened discharge method relies on thickening the whole tailings to a high pulp density, about 60%, at which the tailings–water mixture behaves more like a highly viscous fluid than the liquid-type slurry consistency at normal discharge pulp densities of about 40–45%. At high pulp densities, the thickened tailings assume a slope of about 6% upon discharge, allowing the deposit to form the shape of a conical pile. Water remaining after deposition of the tailings is collected in a small dam at the toe of the pile.

The principal advantage of the method is that impoundment dams or embankments are largely eliminated, and return-water pumping is reduced. Reclamation of the tailings pile may be simplified by the flatter and more uniformly graded pile slopes, and seepage may be reduced by essentially eliminating the decant pond. There is no risk of embankment failure under static loading conditions.

Although costs for embankment construction are largely eliminated by the thickened discharge method, balanced against this savings are higher costs for thickener construction and operation. Also, pumping of the thickened slurry may be more costly and difficult because of higher energy requirements and possibly greater pipe wear. More surface area may be disturbed than for conventional impoundments, resulting in larger areas to be reclaimed.

Runoff-handling requirements for the thickened discharge method must be carefully considered. If all flood runoff is not completely diverted around the pile, erosion and transport of tailings at the toe can result. In addition, the liquefaction resistance of the method is an unresolved issue. It is likely that the lower portions of the tailings pile will be saturated, and observed liquefaction of natural slopes indicates that liquefaction flow slides can develop even on slopes as flat as a few percent. Jeyapalan (1982) concludes that tailings deposited by the thickened discharge method are susceptible to liquefaction and flow sliding under moderate to high levels of seismic shaking.

Both potential runoff-handling and liquefaction problems become more severe if thickened discharge deposition of a conical pile is used on top of a conventional impoundment to augment its capacity, as proposed by Robinsky (1979). In this case, the conical pile may consume the reserve storage volume originally intended for flood storage in the impoundment. Also, liquefaction of the originally deposited tailings supporting the thickened discharge pile could occur in severe earthquakes. Either floods or

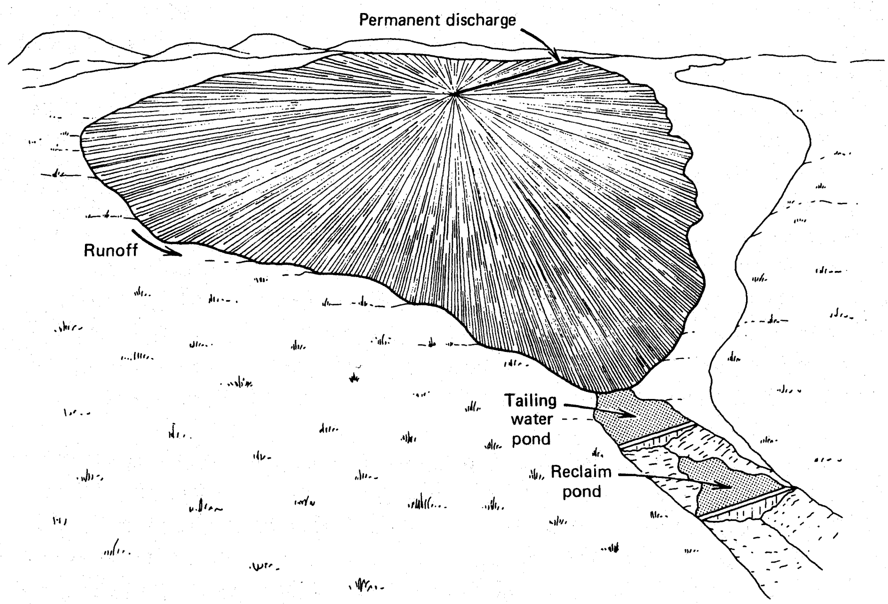


Figure 3.8 Thickened discharge disposal method. (Reprinted from Robinsky, 1979, by permission Miller Freeman Publications.)

earthquakes could therefore cause overtopping and failure of the original embankment.

The thickened discharge method appears to be best suited to disposal sites located in relatively flat topography and where concentrated runoff does not occur, at sites close to the mill where pumping costs are minimized, and in low seismic risk areas. In this regard, the thickened discharge method shares many of the siting restrictions of upstream-type embankments. Also, like upstream methods, thickened discharge disposal is only applicable for tailings containing a reasonable sand fraction and without a major proportion of clayey fines.

"Dry" Disposal

The use of belt filtration to produce "dry" tailings that can be handled in essentially solid form has recently been advocated as a method to reduce seepage from tailings disposal areas by removing water from the tailings before they are deposited.

Belt filtration is an integral part of some European uranium milling processes that have been used extensively in France and South Africa. The operation of the belt filter device is simple in principle, with liquid drawn from the tailings by a vacuum box as they move on an elastomer-supported filter cloth belt. The moisture content of the tailings is reduced from about

50% to 20–30%. Tailings come off the belt as an easily handled cake, often referred to as "dry cake," leading to the commonly used (but incorrect) nomenclature of "dry" tailings disposal.

There remains considerable controversy over the economics, feasibility, and advantages of belt filtration for tailings disposal. Factors such as ore grind and gypsum content affect the efficiency of the filtration process, and for some ores of high clay content the process may not work at all. Frankfort (1978) reports generally promising results based on operating experience, pilot tests, and process studies, while von Michaelis (1979) notes that South African operating experience has been mixed. Murray (1979) shows that both capital and operating costs for belt filtration are very high, justifying use of the method only when it is an integral part of the ore processing operation, but not as a supplementary dewatering method added to conventional thickeners.

Webb et al. (1980) describe an application of belt-filtered tailings disposal. The tailings are placed in disposal pits using a series of conveyors and radial stackers, both expensive pieces of equipment. Because the tailings are in essentially solid form as they are placed, reclamation can proceed concurrently with placement to offer significant benefits. However, claimed reduction in impoundment seepage sometimes results from an overliteral interpretation of the term "dry." Although the tailings are in solid form during handling and placement, their residual water content of 20–30% will still result in near-complete saturation at typical in-place void ratios. The reduction in seepage compared to a conventional slurry-discharged tailings impoundment, if any, depends a great deal on the permeability characteristics of the underlying natural materials. Seepage from the saturated tailings may still be significant in the absence of impoundment liners or underlying natural materials of low permeability.

A related disposal method for coal waste involves first dewatering the fine refuse from washing operations, initially by thickeners followed by filters or chemical treatment. The dewatered fine refuse is then mixed with the coarse waste to form a stable and easily handled silt–sand–gravel-sized mixture, which is transported to the disposal area by either truck or conveyor (Stewart and Atkins, 1982). Since the mixed material is not saturated, no impoundment need be constructed, and disposal follows practices ordinarily used for the coarse refuse. Eliminating the need for impoundments is a significant advantage where steep topography and high runoff complicate impoundment design.

Offshore Disposal

Offshore disposal of tailings is a much maligned concept often viewed as merely a euphemism for dumping tailings in natural bodies of water. Offshore disposal has been used at several mines in British Columbia (Brawner, 1979), in the Philippines (Salazar and Gonzales, 1973), and in Central America.

The effects of offshore disposal on water quality may be limited if the chemical composition of the mill effluent is relatively innocuous and if the tailings are relatively coarse or sufficiently flocculated to settle rapidly without excessive turbidity. It is also important that the point of tailings discharge be in water sufficiently deep and far from the shoreline to avoid the most biologically productive and sensitive shallow-water and near-shore zones. Evans et al. (1973) report the results of a water quality and biological monitoring program for one offshore disposal scheme, and Poling (1979) summarizes the results of environmental monitoring programs from several such operations. The results of these studies generally indicate minimal biological and water quality consequences for offshore disposal, and suggest that offshore disposal areas quickly become rehabilitated after discharge ceases. Other studies of offshore disposal, however, indicate unexpectedly large areas covered by the discharged tailings, as well as turbidity problems (Ripley et al., 1978).

Regardless of technical arguments for or against offshore disposal, regulatory authorities view it with a jaundiced eye. No other disposal method generates public concern more rapidly and intensely, and mines using offshore disposal can usually anticipate that control of their tailings disposal, and therefore of their entire operation, will ultimately reside in the political arena. For these reasons, offshore disposal should realistically be considered only as a last resort, after all other disposal possibilities have been exhausted. Offshore methods, however, may be the only possible option for tailings disposal in some coastal areas where the combined effects of extremely high precipitation, steep terrain, and high seismicity make surface impoundments impossible from a practical standpoint to safely design and construct.

SUMMARY OF DISPOSAL METHODS

Of the tailings disposal methods discussed in this chapter, surface impoundments are the most widely used, usually with one of the various raised embankment approaches. Among the various embankment raising methods, downstream construction is the most versatile and adaptable to areas of high seismicity and to conditions requiring storage of significant volumes of water, but it may be the most costly method. Upstream construction is the least costly procedure, but it is suitable only under a very specific combination of tailings gradation, water storage, and seismic conditions. Centerline construction offers most of the advantages of downstream construction but at a more moderate cost. The selection of an appropriate embankment type must always be based on the specific characteristics of each individual mine, mill, and site. Precedents established in one mining district or geographic area may be suited to local conditions, but application of these precedents to other mines, sites, or geographic areas may be dangerous without careful evaluation.

Underground disposal can be used to supplement, but not entirely replace, surface impoundments. It may be an essential component of some mining operations. Even when not required by mining methods, underground disposal provides a very beneficial use of available underground space to the extent that the surface disturbance and cost of surface impoundments can be significantly reduced. Underground disposal, however, must be made an integral part of the mine plan and may limit mining flexibility as a result.

Other methods, including thickened discharge and "dry" tailings disposal, may be of merit under specific circumstances. However, both methods are relatively untested, and careful evaluation of their economic and technical adequacy is required for each specific case. Finally, recognizing that political realities of tailings disposal are sometimes more significant than technical arguments, offshore disposal must be viewed as a method of last resort.

4

Control of Water in Surface Impoundments

You can feel the anger in water behind a dam.

Barry Lopez, *River Notes*

In the discussion of the various types of embankments for surface impoundments in Chapter 3, a recurring theme is the compatibility between water-handling requirements and embankment type. The need to anticipate and control surface water is essential during initial planning to properly accommodate not only the expected volume of tailings and discharged liquid mill effluent but also runoff inflow. It is an unfortunate fact that many tailings impoundments, while initially sized to contain the anticipated mill tailings output, become full to design capacity long before their time because of failure to properly account for accumulated volumes of runoff water.

Proper design of surface-water control measures is also fundamental to the safety of the embankment against floods. While some tailings embankments have survived slope failures, seepage-related failures, and even partial liquefaction, few have survived intact the effects of overtopping due to inadequate flood-control provisions. Tailings embankments usually experience rapid erosion, downcutting, and complete breaching shortly after water overtops the crest—for example, as described by Toland (1977) for an overtopping failure of an upstream embankment.

Hydrologic analysis methods and design of hydraulic structures for tailings dams are for the most part the same as for conventional water-retention structures and will not be reviewed in detail here. Conventional design procedures are summarized, for example, by the U.S. Bureau of Reclamation (1973). Flood criteria and methods for water handling may, however, be somewhat different for tailings embankments than for conventional dams. The purpose of Chapter 4 is to present a discussion of basic considerations in water handling for surface tailings impoundments.

HANDLING OF NORMAL WATER INFLOWS

Leaving temporarily aside the problem of floodwater control, initial tailings impoundment sizing must take into account handling of water under "normal" operating conditions—that is, water discharged into the impoundment during the course of ordinary mill operation, and water entering the impoundment from precipitation and runoff under usual or average climatic conditions.

Central to anticipating normal water-handling requirements is the water balance, a procedure of comparing water flows into and out of the impoundment. Table 4.1 lists the sources of impoundment water input and withdrawal.

Inflows

Sources of water input include mill water discharge, direct precipitation on the tailings and ponded water surface, and runoff derived from tributary drainage areas external to the impoundment itself. Mill water discharge can usually be easily calculated knowing the daily mill tailings tonnage and the pulp density of the discharge slurry and using phase relationships presented in Chapter 1. Mine water discharge, if added to the mill tailings stream for disposal in the impoundment, is much more variable and difficult to predict.

Average annual precipitation in the United States is shown in Figure 4.1, which is useful as a very general guide. Actual precipitation varies over wide extremes in mountainous areas according to elevation and topographic screening effects. Average annual runoff, summarized for the United States in Figure 4.2, is even more complex, influenced by soil type, vegetative cover, and slope, in addition to the same variables that affect precipitation. The best sources for site-specific precipitation and runoff data are weather stations and stream-gauging stations if they exist near the site.

In performing a water balance for normal conditions, precipitation falling on the surface of the impoundment is usually incorporated in its full amount. Runoff values are applied to undisturbed lands draining into the impoundment, together with any permanent base flows from springs or abandoned mine drainage.

Table 4.1 Water Balance Variables

Inflows	Outflows
Mill water discharge	Mill water reclaim
Direct precipitation	Evaporation
Runoff from tributary drainage areas	Seepage
	Entrainment in voids
	Direct discharge

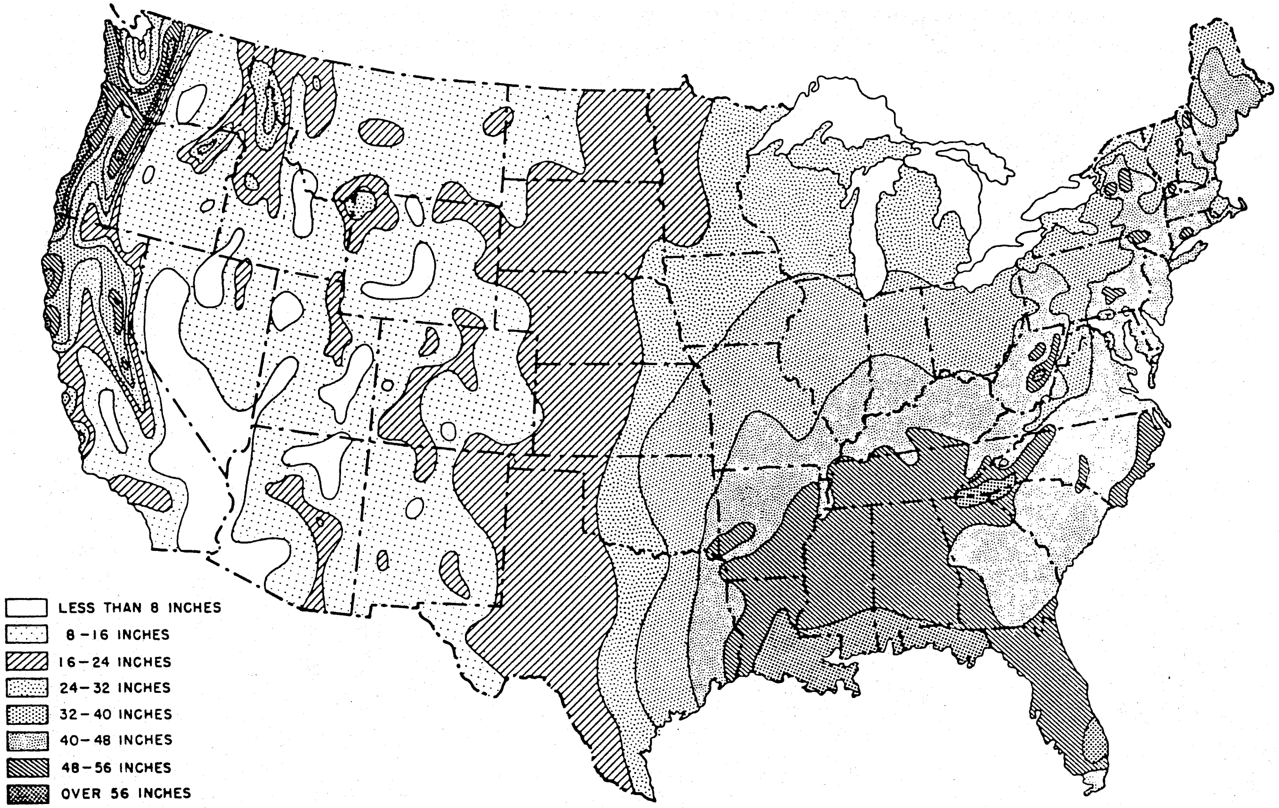


Figure 4.1 Average annual precipitation in the United States. (From *Handbook of Applied Hydrology* by V. Chow. Copyright © 1964 McGraw-Hill. Used with permission of McGraw-Hill Book Company.)

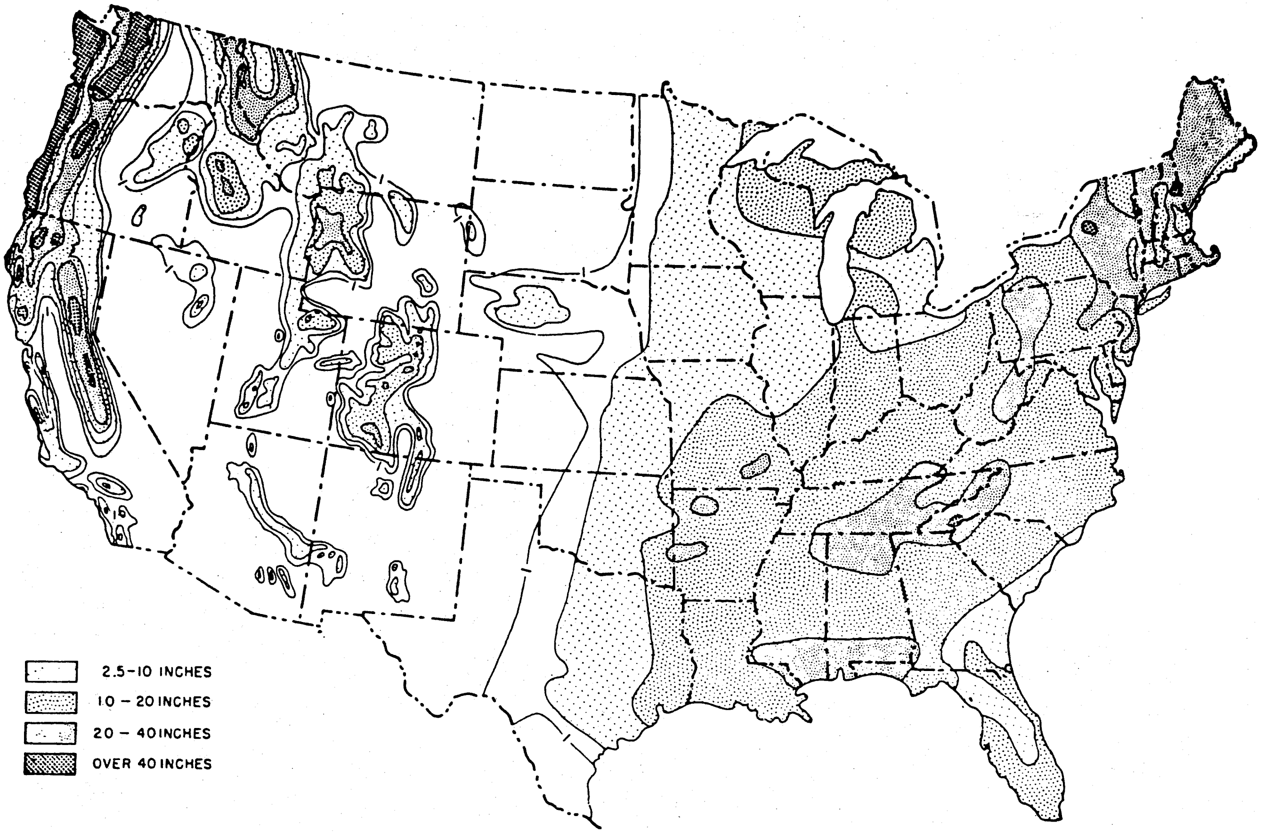


Figure 4.2 Average annual runoff in the United States. (From *Handbook of Applied Hydrology* by V. Chow. Copyright © 1964 McGraw-Hill. Used with permission of McGraw-Hill Book Company.)

Outflows

Outflows from the impoundment include mill water reclaim, evaporation, seepage, and direct discharge. Water entrained in the voids of the tailings, calculated from in-place void ratio values such as those presented in Chapter 2, can also be considered a water outflow to the extent that it represents stored water "consumed" in the disposal process.

Evaporation can be estimated using such data as those presented in Figure 4.3. Evaporation is usually assumed to occur only over the area of the ponded water surface. While some evaporation also occurs on the above-water tailings beach, its amount is difficult to determine and is often neglected.

The amount of mill water reclaim varies according to the nature of the process, but it is often approximately equal to the water discharged with the tailings, provided that tailings pond reclaim is the sole source of mill makeup water.

Methods for estimating seepage are discussed in Chapter 10. During initial impoundment planning, however, there is seldom sufficient data to justify complex seepage analyses. For water balance purposes, seepage is often estimated on the basis of experience with impoundments for comparable types of tailings of similar size, located on sites of generally similar permeability.

The final source of outflow, direct discharge, is to be avoided if possible. In most cases, water treatment to remove mill effluent contaminants is required prior to direct discharge, an expensive proposition. It is nearly always preferable to operate the impoundment as a closed system, without direct discharge.

Example

An example water balance for a typical mill using a flotation concentration process and a hypothetical impoundment site is given below. The mill is assumed to discharge 3,000 dry T/day of tailings at a pulp density of 45%, with reclaim pumpback from the impoundment equal to the mill water discharge. The tailings have a specific gravity of 2.7 and will have an average dry density of 90 pcf in the impoundment.

At its average elevation, the impoundment will have an area of 120 acres. Of this amount, it is assumed that one-half will be covered by ponded water. The drainage area external to the impoundment at its average elevation is 260 acres, and seepage is estimated to be 90 gpm. The impoundment is operated as a closed system with no direct discharge.

The example water balance calculation in Table 4.2 shows an excess flow of 93 gpm. This excess could be allowed to accumulate in the impoundment over its operating life by providing sufficient impoundment capacity, but it would have to be eventually treated and released or evaporated upon aban-

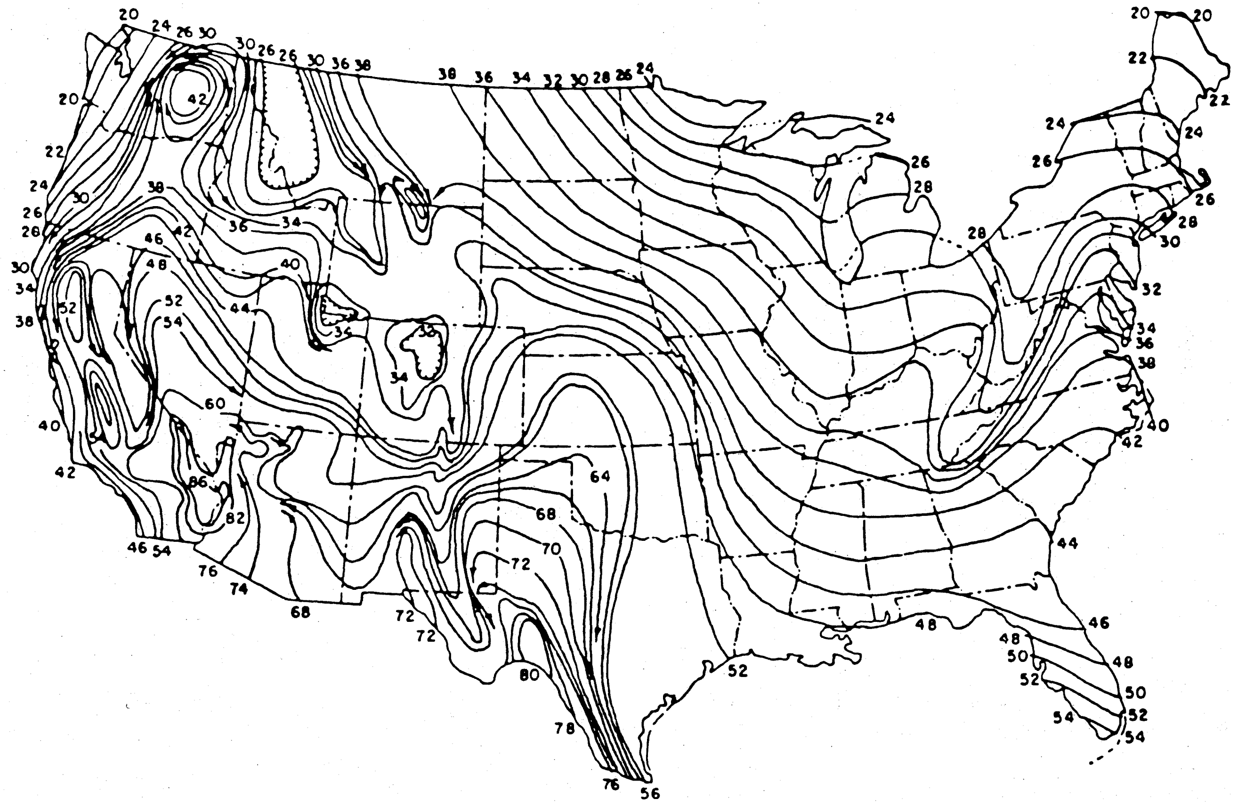


Figure 4.3 Average annual lake evaporation in the United States in inches. (From *Handbook of Applied Hydrology* by V. Chow. Copyright © 1964 McGraw-Hill. Used with permission of McGraw-Hill Book Company.)

Table 4.2 Example Impoundment Water Balance

	Area (acres)	Amount (in./yr)	Flow (gpm)
Inputs			
Mill water discharge	—	—	+608
Direct precipitation	120	34	+210
Tributary runoff	260	16	+214
		Total inflow	+1,032
Outputs			
Mill water reclaim	—	—	−608
Evaporation	60	26	−80
Seepage	—	—	−90
Entrained void water	—	—	−161
		Total outflow	−939
		Net	+93 (excess)

donment of the facility. Table 4.2 shows, however, that the excess could be eliminated by reducing the inflow from external runoff—for example, by diversion ditches. Had a negative total indicated a water deficit in the system, makeup water would have been required for mill operation, perhaps from wells. Alternatively, such measures as described in Chapter 11 could be used to reduce seepage loss.

Comments

The water balance procedure can provide only a relatively crude estimate of expected water accumulation in the impoundment, if any. As noted by Skoglund and Hanson (1979), water inputs and outputs are quite variable and highly sensitive to a number of factors that are often very difficult to estimate in the absence of actual operating experience for a particular impoundment. For example, climatic factors—including precipitation, evaporation, and runoff—undergo extreme departures from "average" conditions, both seasonally and from year to year. It is always of benefit to perform water balance calculations on a monthly basis to establish seasonal fluctuations in water accumulation, and by year using both assumed "wet" and "dry" conditions to bracket the upper and lower bounds of potential water accumulation or deficit.

In addition, it should be recognized that the areas of the impoundment surface, ponded water, and tributary drainage vary during the life of the impoundment as the tailings surface rises and covers a larger area. A complete understanding of long-term water balance variation therefore requires analyses performed for different time periods throughout the life of the impoundment. Similarly, seepage outflows vary over the life of the impoundment and are difficult to estimate at any stage, particularly considering the dearth of subsurface information ordinarily available during the planning process.

In spite of these limitations, a water balance can usually predict whether excess water will accumulate in the impoundment over the long term, and will point to the possible need for diversion ditches or other measures to minimize water inflow. In arid climates, handling of excess mill water discharge may require simply a higher embankment and larger pond surface to increase evaporation. A time-phased water balance is essential in such cases to predict the elevation at which steady-state conditions will be achieved (where net inflows balance evaporation losses) and therefore to predict the elevation at which the ponded water will stabilize. As pointed out in Chapter 3, if long-term accumulation of water is indicated, the use of certain types of raised embankments may be precluded because of their unsuitability for water-storage purposes. A water balance will also serve to quickly identify certain hopeless cases where, even with complete diversion of tributary runoff, precipitation so far exceeds outflows that direct discharge is unavoidable. This will usually require treatment to applicable water quality standards prior to discharge.

FLOOD HANDLING

In contrast to handling normally anticipated impoundment inflows, planning and sizing for floods requires consideration of extreme events derived from precipitation, snowmelt, or both. Floods can endanger impoundments in two ways: by providing sufficient inflow into the impoundment to cause dam failure by overtopping, or by eroding the dam at its toe causing damage or possibly failure.

Design Criteria

Flood design criteria in common use for tailings impoundments include statistically derived return-period floods, and extreme floods determined by meteorological and climatic conditions without regard to likelihood of occurrence. The conservatism used in selecting the design flood usually reflects the consequences of flood-related embankment failure as determined by the size of the impoundment, the extent of downstream development, and downstream land use.

Statistical Approach

Return-period floods can be derived statistically from either stream-gauging or precipitation records, together with hydrologic characteristics of the impoundment drainage basin. However, in determining the appropriate flood for design, level of risk must be considered. The annual probability of occurrence of a flood having a given magnitude is equal to the reciprocal of its return period. For example, the 100-yr return-period flood has a probability of 0.01 of being equaled or exceeded in any given year. The following relationship can be used in establishing the probability of exceedance for a given level flood over the life of an impoundment:

$$p[f]_i = 1 - (1 - p_0)^i$$

where $p[f]_i$ = probability of failure in i years
= probability that flood of a given design magnitude is equaled or exceeded in i years
p_0 = probability of occurrence of a given design magnitude flood in any year
= reciprocal of return period
i = impoundment life in years

Table 4.3 illustrates typical failure risk calculations for impoundments with various operational lives. It is assumed that, upon abandonment, the impoundment would be reclaimed in such a way as to prevent failure by overtopping. Table 4.3 shows that the probability of failure decreases as the return period of the flood becomes greater. Also, the probability of failure increases for longer facility life even for the same flood return period. It is also important to note, for example, that the 10-yr flood is not appropriate for design of an impoundment with a 10-yr life. Table 4.3 shows that a facility designed for an event having a return period equal to its life will have a better than even chance of failing, with a failure probability of about 64%. Even the 100-yr event will have a significant exceedance probability over a typical impoundment life.

There are no definite guidelines for determining acceptable levels of fail-

Table 4.3 Example Tabulation of Overtopping Failure Probabilities

Impoundment Life, i (yr)	Flood Return Period (yr)	Annual Probability of Occurrence, p_0	Failure Probability, $p[f]_i$ (%)
10	10	0.10	64
	100	0.01	10
	1,000	0.001	1
30	30	0.033	64
	100	0.01	26
	1,000	0.001	3

ure risk for most types of structures, and tailings embankments are no exception. In general, failure probabilities should not exceed a few percent, at most, depending on the consequences of failure to downstream inhabitants, downstream land users, and the mine or mill itself. Environmental consequences of failure are also very significant. While the long-term effects of tailings release on biota, air, and groundwater may be difficult to assess directly, the costs of cleaning up many thousands of tons of spilled tailings have economic consequences whose significance is readily apparent.

Deterministic Approach

On the other end of the scale from the return-period approach are floods established on a deterministic basis—that is, without regard to their likelihood of occurrence. For example, the Probable Maximum Flood (PMF) is defined as that flood which may be expected from the most severe combination of meteorologic and hydrologic conditions reasonably possible in the region. The PMF is derived from probable maximum precipitation (PMP), which in turn is based on the most severe combination of meteorological and terrain factors as they pertain to precipitation. The PMP is often on the order of five times greater than the 100-yr return-period precipitation.

The Corps of Engineers has developed guidelines for flood criteria pertaining to water-storage dams, and these or similar criteria are often applied to tailings dams. As shown in Table 4.4, the magnitude of the design flood depends on impoundment size, dam height, and the economic and mortality consequences of failure. Added to this for tailings dams should be considerations relating to the environmental failure consequences.

According to the criteria of Table 4.4, most typical size tailings dams and impoundments will require design for the PMF or a major fraction thereof. The exception may be for initial impoundment discharge behind a small starter dike. For the "low" to "significant" hazard classification, an appropriate-level return-period flood may be acceptable for initial starter dike flood criteria, provided that additional flood-handling capacity is provided as the embankment is raised and the impoundment increases in size.

In deriving the total quantity of water generated by the PMF, the PMP can be summed over the impoundment drainage area, with appropriate allowances for infiltration, using procedures such as those summarized by the U.S. Bureau of Reclamation (1973). Values of the 6-hour PMP for the eastern United States are shown in Figure 4.4.

For the western United States, two distinct types of storms are considered: general-type storms and thunderstorms. General-type storms may yield the maximum total quantity of inflow, the major factor in determining flood storage requirements for closed-system tailings impoundments. Thunderstorms, on the other hand, usually generate higher peak flow rates, a factor that may control the design of spillways and diversion channels. Selecting the proper PMP value therefore requires a knowledge of the

Table 4.4 Hydrologic Evaluation Guidelines

Size Classification

Category	Impoundment Storage (acre-feet)	Dam Height (ft)
Small	50–1,000	25–40
Intermediate	1,000–50,000	40–100
Large	>50,000	>100

Hazard Potential

Category	Loss of Life	Economic Loss
Low	None expected (no permanent structures for human habitation)	Minimal (undeveloped to occasional structures or agriculture)
Significant	Few (no urban development and no more than a small number of inhabitable structures)	Appreciable (notable agriculture, industry, or structures)
High	More than a few	Excessive (extensive community, industry, or agriculture)

Design Flood

Hazard	Size	Design Flood
Low	Small	50–100 yr frequency
	Intermediate	100 yr to ½ PMF
	Large	½ PMF to PMF
Significant	Small	100 yr to ½ PMF
	Intermediate	½ PMF to PMF
	Large	PMF
High	Small	½ PMF to PMF
	Intermediate	PMF
	Large	PMF

Source: Corps of Engineers, 1974.

value's ultimate use in impoundment design and the type of impoundment under consideration. Six-hour PMP values for general-type storms in the western United States are shown in Figure 4.5. The 6-hour general storm PMP can be extended to longer durations by multiplying by constants given in Table 4.5.

Six-hour PMP values derived by the U.S. Weather Bureau for the entire United States are shown in Figure 4.6. These values are generally considered to represent the larger of either thunderstorm or general-type storm in regions where the distinction is appropriate.

Considerable caution must be exercised in using large-scale mapping to determine PMP values. Precipitation is very sensitive to such site-specific

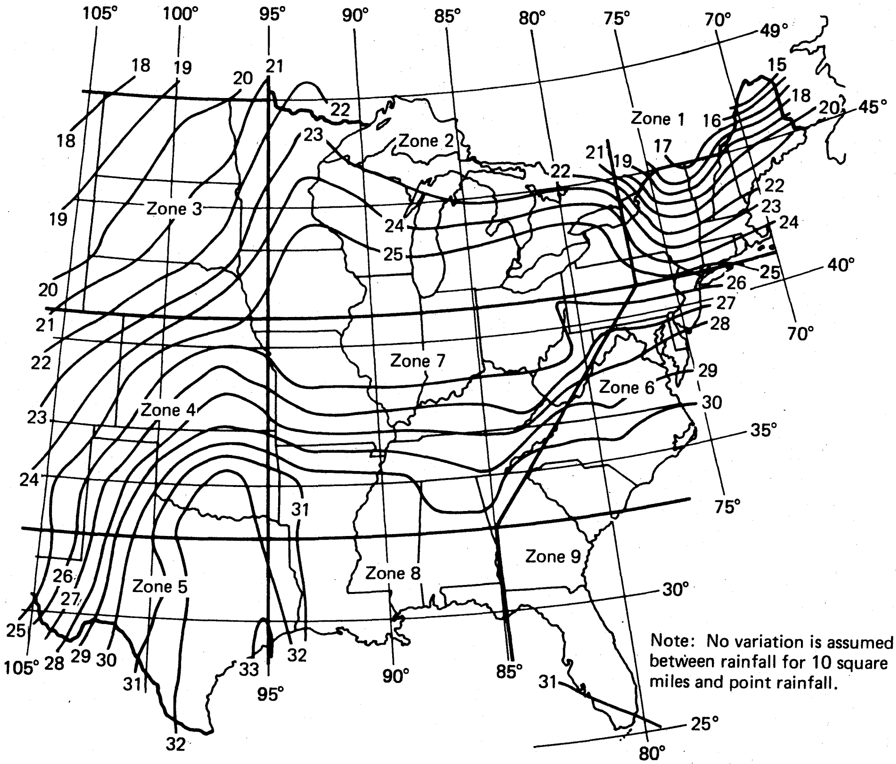


Figure 4.4 Six-hour PMP point values in inches for eastern United States. (Reprinted from U.S. Bureau of Reclamation, 1973.)

factors as elevation, wind direction, and topographic barriers, and use of detailed methods for determining PMP for individual geographic areas, such as those presented by Hansen et al. (1977) and the U.S. Weather Bureau (1966), are to be preferred whenever possible.

Where the impoundment is operated as a closed system, making it necessary to retain all flood water, the PMF may not produce the controlling design flood inflow quantity. Even higher volumes of runoff may enter the impoundment over longer periods from extreme snowmelt runoff or simply from exceptionally rainy seasons or years. In such cases, it is necessary to ensure that the impoundment retains adequate volume allowance to accommodate, say, the 1,000-yr rainy season or wet spring. Estimates of water inflow from extreme snowmelt or wet seasons are sometimes derived from statistical analysis of stream-gauging data, with appropriate adjustments for hydrologic characteristics of the tailings impoundment watershed. Methods to calculate snowmelt rates under various conditions of terrain, vegetation, and weather are summarized by Coates and Yu (1977).

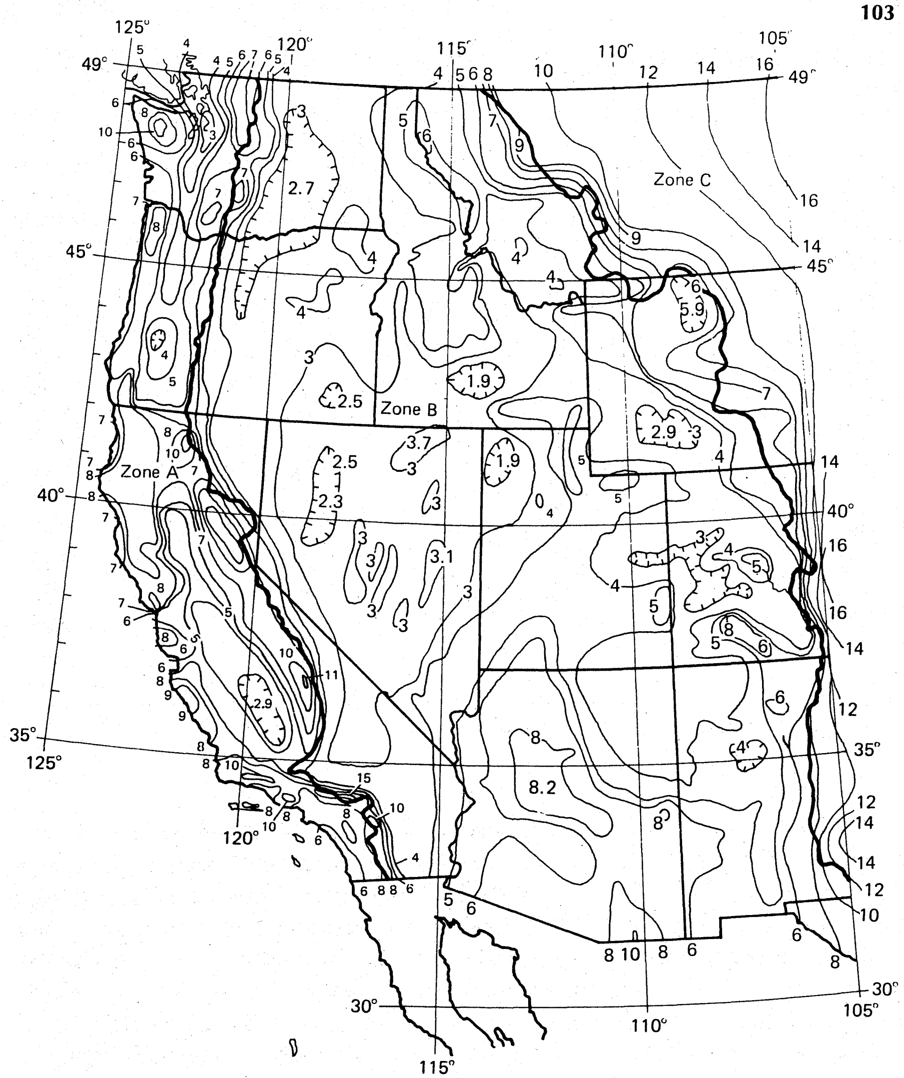


Figure 4.5 Six-hour PMP point values in inches for general-type storms in the western United States. (Reprinted from U.S. Bureau of Reclamation, 1973.)

Inflow Control Methods

As previously indicated, the major hazard associated with surface-water control is the danger of embankment flood overtopping. The preferred situation is to limit water inflow by proper siting of the impoundment, as discussed subsequently in Chapter 5. There are several methods available for handling remaining flood inflows.

Table 4.5 Constants for Extending General-Storm PMP in the Western United States

Duration (hr)	Constants Zone A	Zone B	Zone C
8	1.20	1.18	1.14
10	1.39	1.36	1.26
12	1.58	1.53	1.36
14	1.76	1.66	1.43
16	1.93	1.77	1.50
18	2.10	1.87	1.57
20	2.26	1.95	1.64
22	2.42	2.03	1.71
24	2.57	2.10	1.78
30	2.95	2.28	1.97
36	3.26	2.38	2.15
42	3.55	2.40	2.25
48	3.79	2.41	2.28
60	4.14	—	—
72	4.34	—	—

Source: U.S. Bureau of Reclamation, 1973.

A primary method of inflow control is storage. Adequate dam freeboard or surcharge is maintained at all times by raising the embankment at a rate such that sufficient volume is always available for storage of the design inflow. If the design flood inflow has been determined with an appropriate degree of conservatism, it is unlikely to be experienced during the life of the impoundment. In the improbable event that it should occur, the impounded runoff could be stored for eventual evaporation in regions of arid climate. In other areas, the water, if contaminated by mixing with the mill effluent, could be treated and released at a gradual rate. In general, however, control of flood inflow by storage avoids the recurring necessity of expensive and sometimes difficult treatment of contaminated flood inflow prior to release into surface watercourses.

In some areas, topographic constraints on practical embankment height and impoundment volume, combined with high precipitation or high rates of net mill water discharge, may make storage of flood inflows unfeasible. In such cases, the only option may be to provide treatment of all mill effluent prior to its discharge into the impoundment. Flood inflows entering the impoundment, presumably no longer at risk of contamination by mixing with the treated mill effluent, can be passed through the impoundment and discharged through a spillway designed according to conventional methods. In applicable areas, the thunderstorm rather than general storm PMP may con-

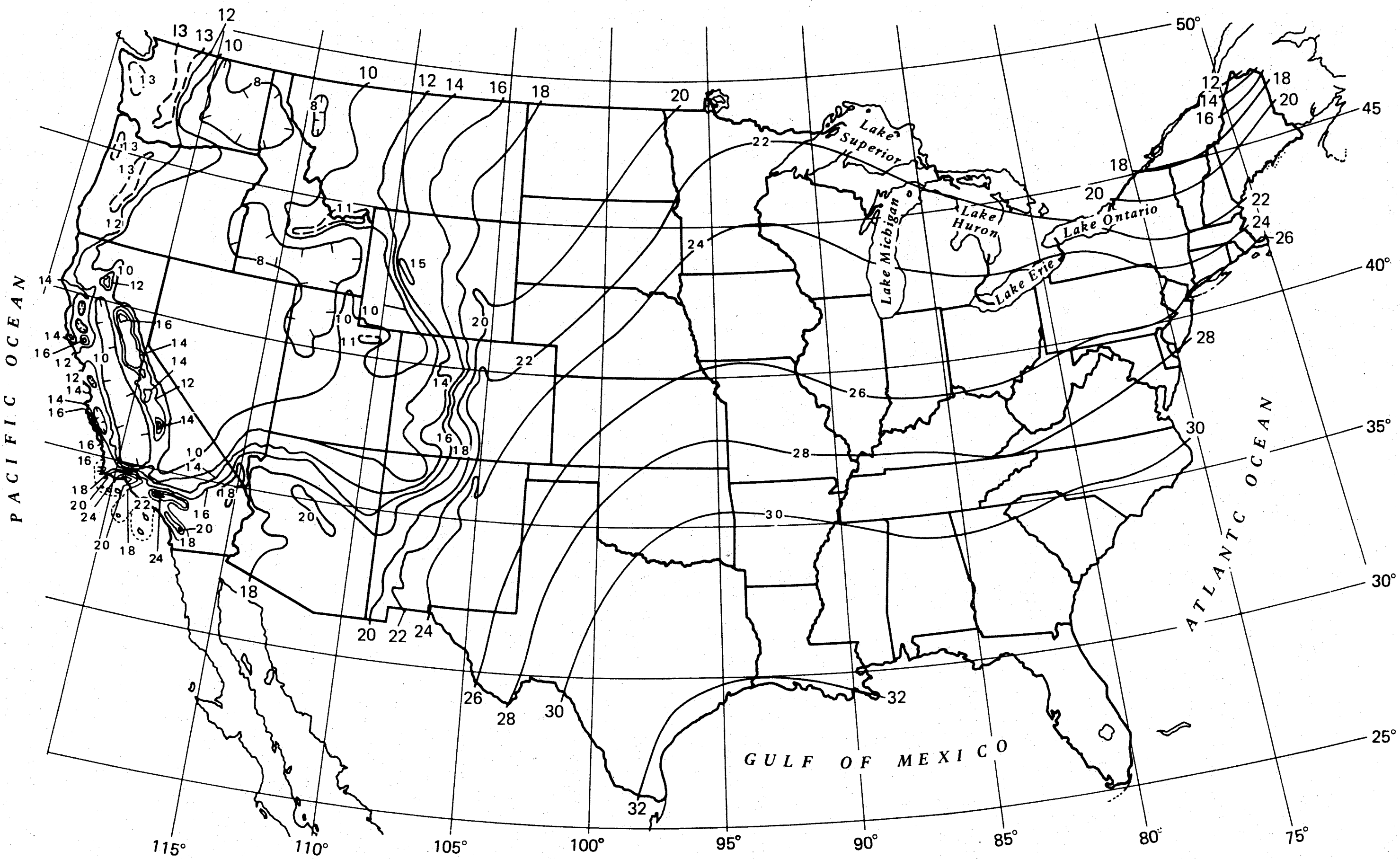


Figure 4.6 Six-hour PMP values in inches for 10-square mile area. (Reprinted from Herschfield, 1961.)

trol the design of spillways where peak flow rate rather than total inflow quantity is of primary importance. The use of spillways with raised embankments, however, is awkward at best. With each embankment raise, a new spillway must be constructed at the new crest elevation. This adds appreciably to cost and difficulty of construction. In extreme cases, water handling by spillways may preclude the use of raised embankments, instead requiring water-retention type structures built initially to full height.

Diversion channels are desirable in most cases and essential in some for diversion of normal runoff flows. Although diversion channels can also be used to divert flood flows around the impoundment, major design floods often produce flows that require large channels (channel widths for PMF diversion in excess of 100 ft are not uncommon) and heavy riprap for protection of the channel banks against high flow velocities. As a result, diversion channels designed for peak flood flow are sometimes impractical, depending on the channel length required. For raised embankments, however, excavation of even relatively large flood diversion channels may provide a convenient source of starter dike fill material.

For open-pit mining operations, creative planning of waste rock dumps and even the pits themselves can provide useful water-control benefits for the tailings impoundment. Process flow considerations in mine facility layout usually dictate that the tailings impoundment and mill be located at lower elevations than the pit and associated waste rock dumps. If the pit is located within the drainage basin of the tailings impoundment, the pit volume itself can be credited with flood water storage for extreme PMF-type floods. If the waste rock dumps can be extended across the tailings impoundment drainage area without excessive haul distance, diversion of extreme floods by the mass of the waste rock can be realized at essentially no cost. This requires that waste dump and tailings impoundment planning be integrated and carried out in conjunction with each other.

A related method of runoff diversion is to construct nonimpounding diversion dikes across the impoundment drainage area. To be effective, such dikes should be located as close as possible to the maximum upstream extent of the impoundment. This method of surface-water control may be of particular advantage if the impoundment is located over relatively shallow bedrock where construction of diversion channels would require expensive rock excavation. Flow velocities against the diversion dike may be high, however, possibly requiring the use of riprap if the dikes are constructed of native soils. The use of open-pit waste rock for diversion dike construction, usually relatively coarse and erosion resistant, may be attractive for this reason.

In extreme cases where the tailings impoundment is located in a narrow, constricted valley with a large upstream drainage area, steep valley sidewalls may make it impractical to divert flood runoff around the tailings impoundment by either diversion channels or nonimpounding dikes. Here it may be necessary to construct an entirely separate flood-control dam upstream from the tailings impoundment. The flood-control dam can provide full storage of expected flood runoff from the upstream drainage area, with gradual evacuation of the stored water by a culvert passing through the water-control dam and around the tailings deposit. This arrangement is to be avoided if possible, since fill requirements for the water-control dam may be large, in some cases even exceeding fill required for construction of the tailings embankment itself. Moreover, construction of the flood-control dam cannot be staged; it must be completed prior to tailings impoundment operation in order to serve its intended flood-protection function. Finally, maintenance of buried culverts is problematic, and the limited life of the culvert structure may make it necessary to provide more permanent water-control measures after impoundment abandonment and reclamation.

Figure 4.7 summarizes the various methods for handling extreme flood inflows from external drainage areas. It is important to note that even with complete diversion of flood runoff from tributary drainage areas, that portion of the extreme flood precipitation which falls directly on the impoundment surface must still be accounted for. Depending on impoundment area, direct precipitation may be a significant source of inflow.

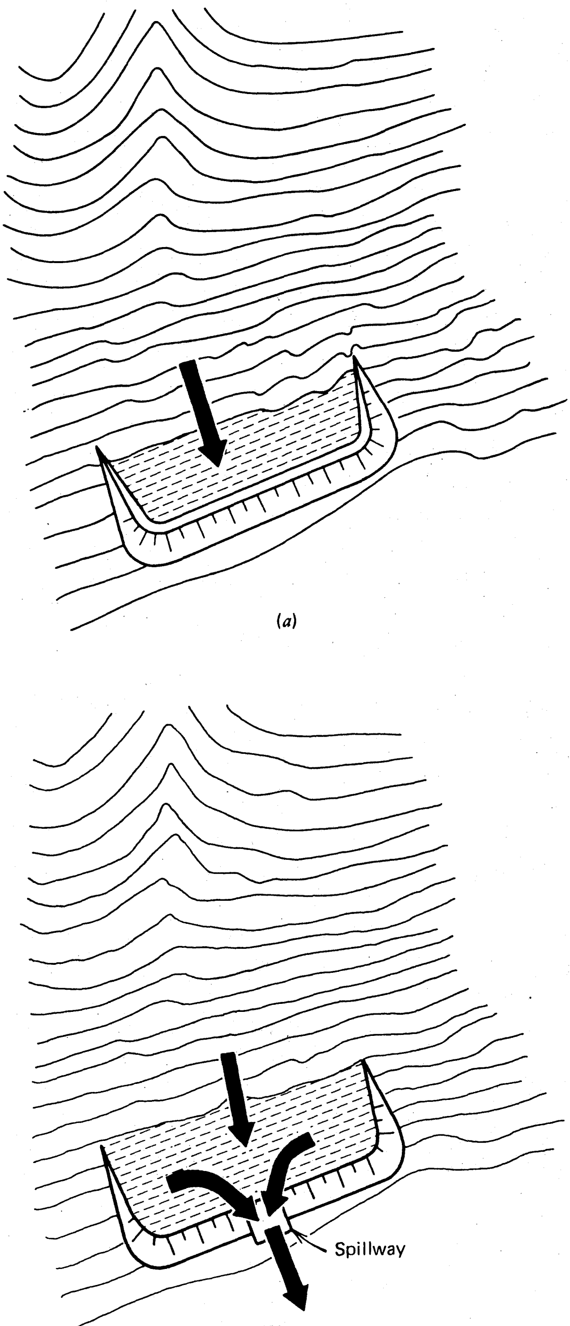
Effects of Floods on Impoundment Structures

A completely separate consideration from control of inflow into the impoundment is the need to account for floods that may pass at the toe of the embankment. This situation may result in erosion, undercutting, and eventual failure of the exterior face of the embankment, and it is especially critical for impoundments located in low-lying river floodplain areas or narrow canyons. The same general flood criteria as for impoundment inflow apply, only here the concern is with flow velocities of the water at the embankment toe. Riprap of affected portions of the embankment face provides one solution using design procedures such as those presented by the U.S. Bureau of Public Roads (1967). The method of choice is to avoid impoundment siting in floodplains, particularly in areas of predominantly soft, sedimentary rocks where riprap may be unavailable locally and difficult or expensive to import. For some Southeastern U.S. phosphate tailings impoundments located in low-lying coastal areas, imported riprap for wave protection during hurricanes constitutes the major expense in impoundment construction.

SUMMARY

Water control for surface tailings impoundments requires consideration of two separate circumstances: "normal" inflows and flood conditions.

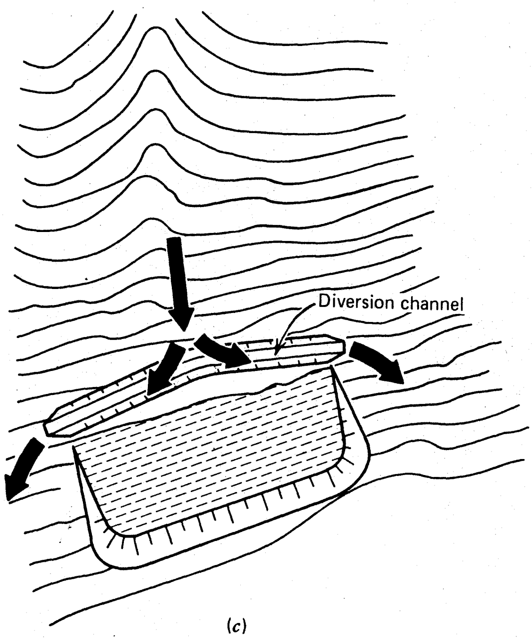
Water balance procedures can predict in an approximate way whether water accumulation will result from normal operation under "average"



(*a*)

(*b*)

Figure 4.7 Methods for handling external extreme runoff. (*a*) Storage. (*b*) Passage through spillway. (c) Diversion by channels. (*d*) Storage and diversion by mine pits and waste dumps. (e) Diversion by dikes. (*f*) Storage and diversion by upstream water dam and conduit.



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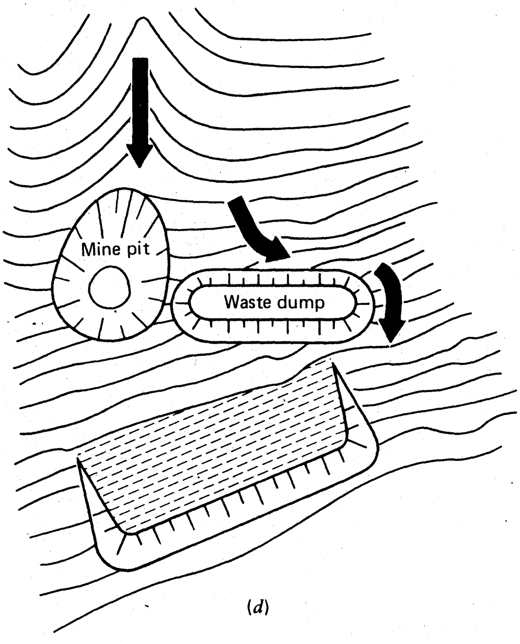

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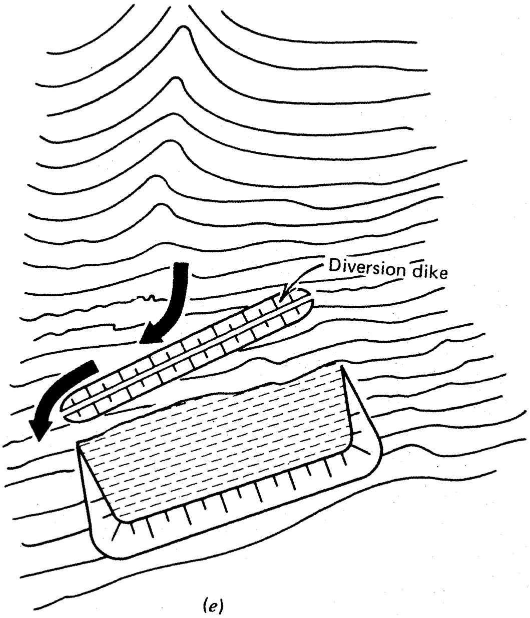

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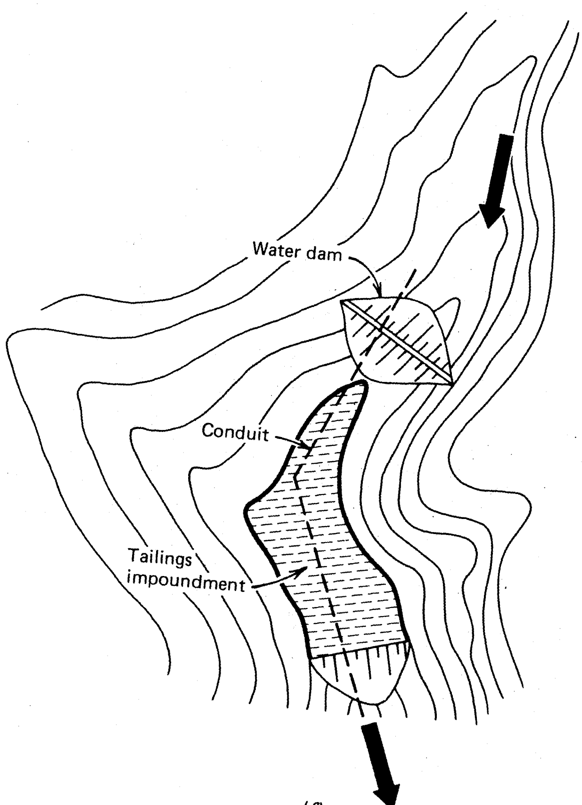


(f)

climatic conditions. The need for diversion ditches to reduce normal runoff inflow into the impoundment and resulting long-term water accumulation will be indicated by the results of the water balance. It is essential that the type of embankment selected from among the options discussed in Chapter 3 be compatible with any anticipated water storage or water accumulation. To reduce seepage and safety hazards, it is desirable to minimize water storage in the tailings impoundment, unless collection of water for mill process requirements is an intended objective of impoundment operation and has been specifically accounted for in embankment design.

Flood handling requires that both flood inflows into the impoundment and the effects of floodwaters on embankment structures be anticipated. Design requirements vary from return-period type floods to "Probable Maximum" events determined without regard to occurrence probability, depending on the consequences of impoundment failure and the overall degree of hazard posed. In some regions, PMF inflows may not control, being exceeded by snowmelt or seasonal runoff in exceptionally wet years.

The most common method of flood inflow handling is by providing adequate flood storage volume and by operating the impoundment as a closed system. As an alternative, release of accumulated floodwater through spillways may be unavoidable in regions of high precipitation, but this may require costly treatment of mill effluent prior to discharge. While diversion ditches are useful for diverting normal runoff around the impoundment, high peak flows for PMF-type floods often make design and construction of flood diversion channels difficult. An integrated approach to mine planning and tailings impoundment planning may present the opportunity for innovative diversion of floodwater by judicious layout of open-pit mine waste dumps.

5

Surface Impoundment Siting and Layout

Looking up from time to time and across the valley, he offhandedly indicated moraines that had been bulldozed by advancing ice, silt deposited in what had been lakes, the pitted outwash plains of melted glaciers. He was like a radiologist reading a picture. There were in this area some beautiful sites, he said, for vegetation and topography, but there were some problems, too: an uncomfortable level of seismic risk, for one thing, and a few too many bogs, and the possibilities of permafrost.

John McPhee, *Coming into the Country*

Once the type of tailings, production rate, and impoundment life are known, data provided in Chapters 1 and 2 can be used for preliminary estimates of the impoundment volume required for tailings disposal. On the most basic level, impoundment volume requirements will dictate the adequacy of possible disposal sites. In addition, the general chemical composition of the mill effluent may provide an indicator of the potential groundwater contamination hazard, and this may have important consequences on siting decisions.

After establishing volume requirements and the nature of the materials to be contained, the stage will be set for selecting an impoundment site and for identifying the type of impoundment layout best suited to the site characteristics. An essential part of site and layout selection are water balance and flood-control factors discussed in Chapter 4, as well as selection of embankment type from among the various options defined in Chapter 3. To a large degree, water-control planning and embankment selection are site specific. Therefore, siting and layout of an impoundment are not isolated processes, but rather ones that must be integrated with several other factors.

Unique among the processes involved in tailings disposal planning, layout and siting of impoundments involve a considerable degree of intuitive insight and creativity sometimes not amenable to rational analysis in the conventional sense. Because impoundment layout and siting is not a highly systematic exercise, it follows that the process cannot be treated and discussed in a highly structured setting. The aim of Chapter 5 is to present in a broadly generalized way some of the factors that influence siting and to present and compare various layout options. Developing sites and layouts for actual impoundments is at the core of creative engineering, and the actual process involves defining several alternatives for a given case, together with the exercise of a great deal of skill and judgment in making comparisons between the options defined.

SITING CONSIDERATIONS

Siting is not independent of impoundment layout; some sites discarded as unsuitable for one type of impoundment layout may be compatible with others. However, to avoid undue confusion in presentation, siting is discussed separately from layout in this section. Impoundment layout options will be treated in subsequent portions of Chapter 5.

Siting is essentially a screening process. Presented with a mine and mill location, there is initially an infinite array of possible tailings impoundment sites. Screening involves the application of various constraints to the initial array, which ultimately results in a much smaller subset of reasonable siting possibilities. Constraints involve factors related to the mill location, topography, hydrology, geology, and groundwater factors, which are discussed individually below.

Distance and Elevation Relative to Mill

The first constraint applied to the range of possible sites is distance from the mill and relative elevation with respect to it. This arises from the construction and operating costs for transportation of tailings slurry and reclaim water. Considering that the initial cost alone of tailings and reclaim water pipelines may be as much as \$0.5 million per mile, a significant cost penalty quickly accrues to more distant sites. Consequently, it is ordinarily desirable to locate the tailings impoundment as close as possible to the mill. As a general guideline, initial site screening should consider an area up to about 5 mi from the mill, except in unusual cases involving very large tailings storage requirements or those that may require unique geologic conditions. Even for large storage requirements, it may be preferable to develop several individual impoundments at sites near the mill rather than a single, large impoundment at a more distant location.

While sites located at elevations moderately higher than that of the mill should not be ruled out, it is desirable that the impoundment be located downhill from the mill to allow for gravity flow of the tailings slurry, or at least to minimize slurry pumping costs. Average downgrades of several percent are usually optimal. Steeper gradients may require drop-boxes in the tailings pipeline for energy dissipation and will also increase pumping costs for return of reclaim water to the mill. In most cases, screening for sites on the basis of proximity to the mill and relative elevation will considerably narrow the initial range of siting possibilities.

Topography

Following screening on the basis of distance and elevation, natural topography becomes the dominant factor. The general aim is to identify sites where the maximum storage capacity can be achieved with the least amount of embankment fill material, within the limits of the storage volume required. This usually involves identifying natural valleys, basins, or other topographic depressions on suitably scaled maps, and sketching preliminary trial embankment–impoundment configurations for each potential site. To identify topographically suitable sites often requires a considerable amount of both experience and trial-and-error iteration.

Topographic features may suggest possible sites involving very high embankments or very large impoundment areas. There are practical limits, however, to the total potential volume of such sites that can or should be developed for tailings disposal. On the basis of stability, hazard, and economic factors, embankments less than about 100–200 ft high usually prove to be optimal, and very high embankments (greater than 400–500 ft) almost always pose design and construction problems that would better have been avoided during siting. On the other hand, very shallow and large impoundments may result in excessive seepage, land disturbance, and land acquisition costs by virtue of their area. Obtaining necessary storage volumes may require use of high embankments and/or large, shallow impoundments in specific situations, however, depending on the topographic character of the site vicinity.

Hydrology

Closely related to topography are surface-water hydrology considerations. As discussed in Chapter 4, the objective is to site the impoundment so as to minimize inflow or diversion requirements under both normal and flood conditions, except in circumstances where the impoundment is specifically intended to collect water for mill operation.

To the extent that surface-water inflow is directly proportional to drainage catchment area, minimizing inflow requires that the impoundment be sited as close as possible to the head of a drainage basin near the drainage divide, as illustrated in Figure 5.1. As a general rule, the total drainage catchment area should be less than 5 to 10 times the impoundment surface area in order to avoid excessive water-handling requirements. Minimizing impoundment drainage area often involves trade-offs with other siting constraints, since, for example, drainage divides may be distant from and higher than the mill.

The effect of the screening process becomes evident when the three siting criteria of location with respect to the mill, topography, and hydrology are

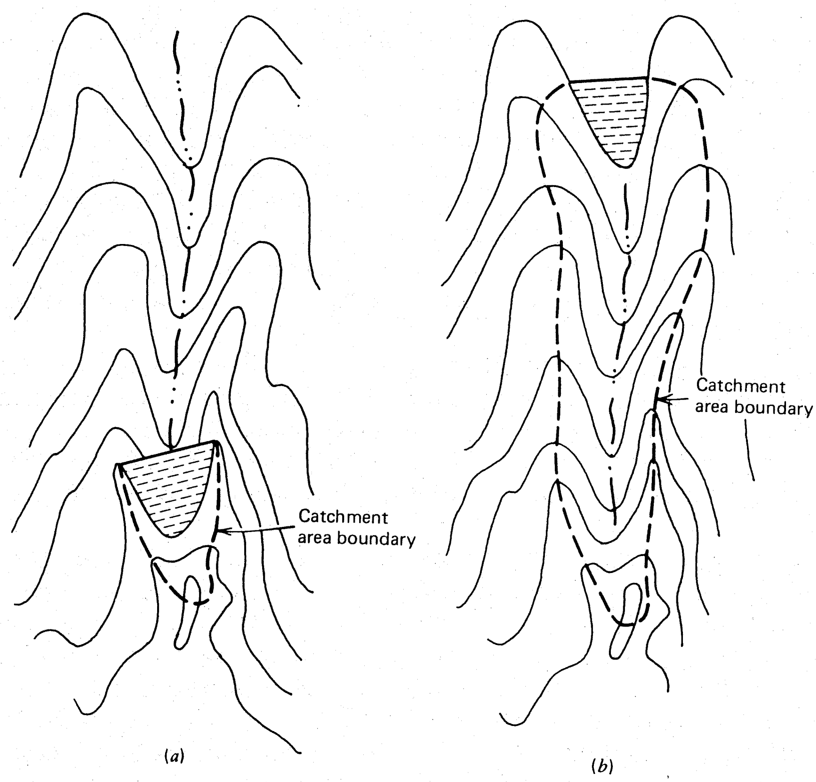


Figure 5.1 Effects of impoundment location on catchment area. (*a*) Impoundment near head of drainage basin with limited catchment area. (*b*) Downstream location with extensive catchment area.

superimposed. To satisfy these criteria simultaneously requires that a potential site be located within several miles from the mill, downhill or only slightly uphill from it, within a suitable topographic depression, and near the head of a drainage basin. The site must also be able to accommodate the required tailings storage volume with a reasonable amount of embankment fill. It is not difficult to appreciate that these criteria applied together will drastically restrict the range of feasible options, and it is a rare and fortuitous combination of circumstances that results in more than a few possible sites surviving the screening process to this point.

Geology

Geologic factors influence embankment foundation conditions, potential seepage rates, and fill availability. Soft foundations can limit the allowable rate of embankment height increase by inadequate pore pressure dissipation in much the same way as raising rates may be limited by slimes tailings as discussed in Chapter 3. In addition, impoundment seepage is often controlled by the permeability of underlying natural soil or rock formations. Geology in the broad sense also governs the local availability of borrow soils. Natural soil borrow must be present in sufficient quantity for construction of at least the starter dike and also in sufficient variety to provide a range of material types, such as clay for dam cores, gravel for internal drainage zones, and sand for filter transitions. The local availability of adequate borrow is extremely important for tailings dam siting, since the economics of tailings disposal seldom justify importing or processing large volumes of fill, as may be the case for conventional earth dams constructed for other purposes.

It is by now almost a geotechnical cliché to call for heavy emphasis on the role of geologic factors in tailings impoundment siting. However, as a practical matter the previously described siting constraints of distance and elevation, topography, and hydrology usually restrict the range of possible sites to the degree that geologic factors often must of necessity play a secondary role.

In addition, it is often critical geologic details rather than broad geologic conditions that are crucial in impoundment performance, and there are usually insufficient time and resources at the siting stage to obtain information in sufficient detail to warrant conclusive comparisons between sites on the basis of geologic details. For example, on a general basis it might be assumed that siting on shale formations would be preferred to other geologic strata because of the lower permeability of the shale and reduced seepage. However, shales are often susceptible to weathering and fracturing at the surface, which may extend to great depth and produce highly permeable conditions. Thorough evaluation of these conditions requires a comprehensive exploration program. Without detailed subsurface investigation normally not possible within the context of siting studies, definitive geologic evaluation of sites may be unwarranted and even misleading.

While the importance of geologic conditions is not to be neglected, the fact remains that other siting factors are often of overriding importance and that tailings impoundments must often be built on sites that are geologically less than ideal. The full importance of geologic conditions is often realized only after detailed subsurface exploration. Discovery of "fatal flaws" related to geology at this stage may force a previously identified site to be abandoned. In some cases, however, it is essential that comprehensive and detailed subsurface programs be performed for several sites from the start of the planning and siting process. For example, if multimillion dollar expenditures for impoundment lining are likely to be required by the nature of the effluent, a comprehensive search for sites on natural formations of verified low permeability may be justified. Also, in such cases it may be prudent to extend the distance from the mill at which sites can be located, since the economic advantage of avoiding the need for a liner could outweigh the penalty of higher pipeline or pumping costs.

Groundwater

Existing groundwater conditions also play an important role in impoundment siting. From a construction-related standpoint, high groundwater conditions and resulting soil saturation will limit the quantity of dry fill available for starter dike or embankment construction. From a groundwater contamination standpoint, high groundwater levels will allow seepage to quickly enter the groundwater regime, while a considerable unsaturated zone between the impoundment and the groundwater level may reduce the effects of seepage. The gradients and directions of groundwater flow will determine both the velocity of any contaminant migration as well as the possible adverse effects on groundwater users. Finally, the preexisting quality of the groundwater will determine the effect of seepage on it, as well as any resulting limitations on groundwater use.

Unfortunately, much like geologic conditions, groundwater-related factors usually require investigation at a level not ordinarily feasible within the context of siting studies. Exceptions do arise, however, when siting tailings impoundments in areas previously affected by older mining operations. Here the effects of acid mine drainage and related problems may have produced a severe impact on some groundwater basins while leaving other nearby basins virtually untouched. In such cases it would generally be preferred to site the tailings impoundment in the already degraded area. Also, if it is possible to site the tailings impoundment up-gradient from a large mine pit, gradients induced by pit dewatering may direct impoundment seepage into the pit itself, thereby restricting wider migration of contaminants in the groundwater and allowing the collected contaminated water to be appropriately treated or returned to the tailings impoundment.

Summary of Siting Factors

The essence of impoundment siting usually involves a screening process subject to several constraints, the most important of which typically include proximity to the mill, topographic suitability, and hydrologic considerations. Additional constraints may include geologic and groundwater factors if sufficient information is available at the siting stage to warrant reasonable inferences. Table 5.1 summarizes the influences of various siting constraints.

From what is initially an infinite array of possible sites, application of siting constraints one by one produces increasingly smaller subsets of reasonable possibilities. Usually only a few sites will satisfy all constraints simultaneously. On the other hand, if no site can satisfy the initial set of constraints, then one or more must be relaxed and the screening process repeated. For example, if no satisfactory sites can be found within 5 mi, then the search can be extended to wider areas. Or if no site is located at the head of a drainage basin, then an otherwise topographically suitable site can be

Table 5.1 Factors Influencing Impoundment Siting

Parameter	Effects
(1) Location and elevation relative to mill	Length of tailings and return-water pipelines Capital and operating cost for pumps
(2) Topography	Embankment layout Embankment fill requirements Diversion feasibility
(3) Hydrology and catchment area	Long-term water accumulation Flood-handling requirements
(4) Geology	Availability of natural borrow types and quantities Seepage losses Foundation stability
(5) Groundwater	Rate and direction of seepage movement Contamination potential Moisture content of borrow materials

selected, recognizing that one or more of the methods discussed in Chapter 4 may be required to mitigate water-handling problems.

Siting decisions are not always fixed; they may change at any subsequent stage of the more detailed design process. The discovery of sufficiently adverse geologic or groundwater conditions at a later stage of design may make it necessary to reopen the siting issue for reevaluation. By its nature, site selection must also interact with a host of other elements in the disposal planning process. For example, inability to identify sites for surface impoundments of suitable economic or technical feasibility may point the way to evaluation of other disposal methods, such as those presented in Chapter 3. Sites that may not lend themselves well to a certain class of embankment may be adequate for other more versatile embankment types, raising methods, or materials. Finally, site suitability may be a function of impoundment layout, the topic of remaining portions of this chapter.

IMPOUNDMENT LAYOUT

Impoundment layout is an integral part of the siting process, since the suitability of a particular site cannot be fully established without confirming that the site will accommodate a particular impoundment configuration. Like impoundment sites, impoundment layouts exist in infinite variety. Nonetheless, categories of impoundment layouts can be defined that are generally compatible with various topographic settings. Impoundment layout types considered in this chapter include:

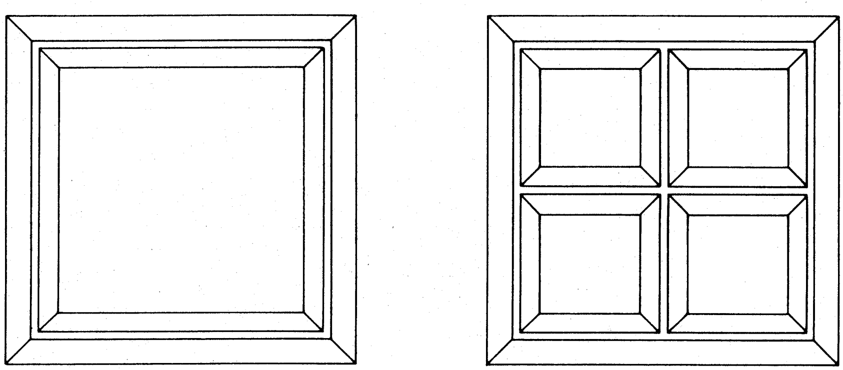
1. Ring dikes.
2. Cross-valley impoundments.
3. Sidehill impoundments.
4. Valley-bottom impoundments.

To some degree, layout of the impoundment is independent of the type of embankment used to confine it. Any of the embankment types or raising methods described in Chapter 3 can be used, provided that the embankment type is compatible with the specific mill tailings and effluent parameters as well as the specific conditions of the particular site.

Ring Dikes

The ring dike impoundment layout is shown schematically in Figure 5.2. Best suited for flat terrain in the absence of natural topographic depressions, the ring dike layout requires a relatively high quantity of embankment fill in relation to the storage volume produced. Since all sides of the impoundment are enclosed, runoff from external drainage areas is eliminated, and accumulated water results only from that which falls directly on the impoundment surface. Ring-type impoundments are usually laid out with a regular geometry, resulting in a uniform configuration easily adapted to various types of liners.

As indicated in Figure 5.2b, the impoundment can be segmented, with each segment constructed sequentially as the previous segment is filled with tailings. While this may produce a number of benefits, including seepage



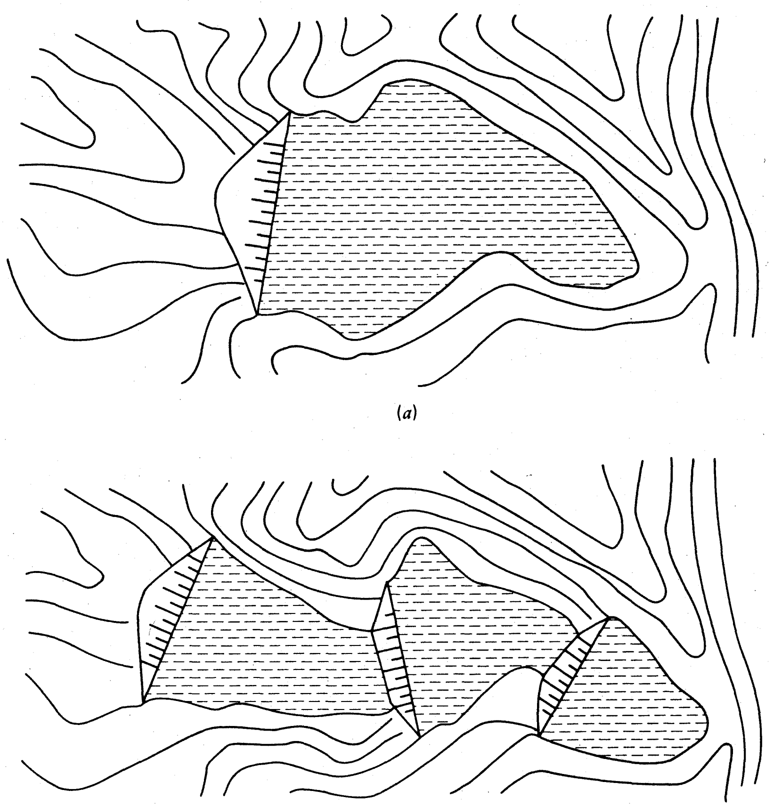
(*a*) (*b*)

Figure 5.2 Ring dike configuration. (*a*) Single impoundment. (*b*) Segmented impoundment.

reduction, concurrent reclamation, and deferral of construction cost, segmented-type impoundments require greater embankment fill volumes—in the extreme case of the example shown in Figure 5.2, 50 percent more fill than required for a single impoundment.

Cross-Valley Impoundments

Cross-valley impoundments, illustrated in Figure 5.3, differ little in layout from a conventional water-storage reservoir. As the name implies, the cross-valley impoundment is confined by a dam extending from one valley wall to another. Cross-valley type layouts can be nearly universally applied to almost any natural topographic depression, in either single- or multiple-impoundment form, as shown in Figure 5.3b, accounting for the prevalence of this layout for tailings disposal. Paramount in the use of the cross-valley layout is that the impoundment be located near the head of the drainage



(*a*)

(*b*)

Figure 5.3 Cross-valley impoundment. (*a*) Single. (*b*) Multiple.

basin to minimize flood inflows. While sidehill diversion ditches can be used to reduce normal runoff accumulation in cross-valley impoundments, larger diversion channels to pass peak flood flows around the impoundment are often not feasible because of steep valley sidewalls. Flood runoff from large drainage catchment areas can often be handled for cross-valley impoundments only by storage, spillways, or separate water-control dams upstream from the tailings impoundment.

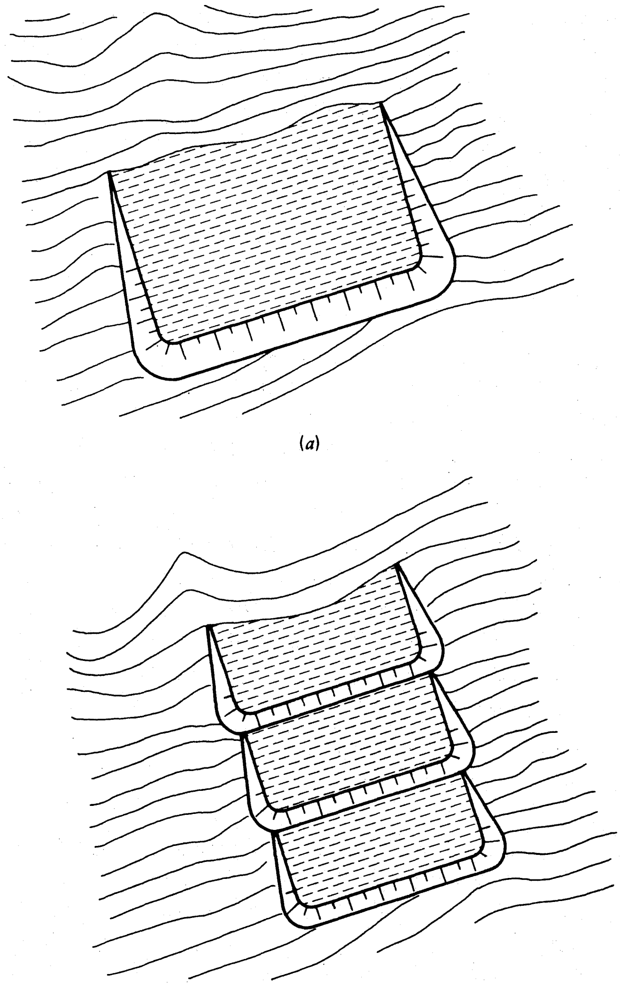
Sidehill Impoundments

The sidehill impoundment layout is shown in Figure 5.4. This layout type encloses the impoundment by embankments on three sides and therefore generally requires more fill than the cross-valley option. This type of impoundment, however, can be used where no incised drainages suitable for cross-valley impoundments are available—for example, on mountain-front alluvial pediment deposits or where the available incised drainages would have an excessive catchment area. This type of layout is best suited for sidehill slopes of less than about 10% grade; on steeper slopes, fill volumes may become excessive in relation to storage volume achieved, and for downstream-type embankments, the upstream portion of the embankment itself may occupy a significant proportion of what would otherwise be impoundment storage volume.

Valley-Bottom Impoundments

Valley-bottom impoundments, depicted in Figure 5.5, represent a compromise between cross-valley and sidehill layouts. The valley-bottom option is well suited for cases where the drainage catchment area would be too large for cross-valley layouts, but hillside slopes are too steep for practical application of the sidehill option. Since the impoundment is enclosed by embankments on two sides, fill requirements are generally intermediate between those for cross-valley and sidehill layouts. Valley-bottom impoundments are often laid out in multiple form, as shown on Figure 5.5b, in order to "stack" the impoundments one above the other as the valley floor rises, thereby achieving greater total storage volume.

Central to the use of the valley-bottom layout is a diversion channel to carry the full peak flood flow around the impoundment. Diversion is usually necessary since these impoundments, commonly located in relatively narrow valleys, are often constructed across the original stream channel. The diversion channel usually corresponds to the gradient of the original stream channel but is constructed tight against the opposing valley wall. During initial layout, if sufficient space is not allocated for the diversion channel, costly excavation in valley sidewall rock may be required to achieve necessary channel widths.



(*a*)

(*b*)

Figure 5.4 Sidehill impoundment. (*a*) Single. (*b*) Multiple.

Because peak flows under PMF or similar flood conditions are usually large, widths for diversion channels associated with valley-bottom impoundments are often considerable. Excavated material, however, can often be conveniently used as starter dike fill. In addition, it is frequently the case that high-velocity flow will occur against the outer embankment face under design flood conditions, requiring that lower portions of the embankment be protected by riprap. This can make the use of centerline or downstream embankment raising methods awkward because of the need to continually

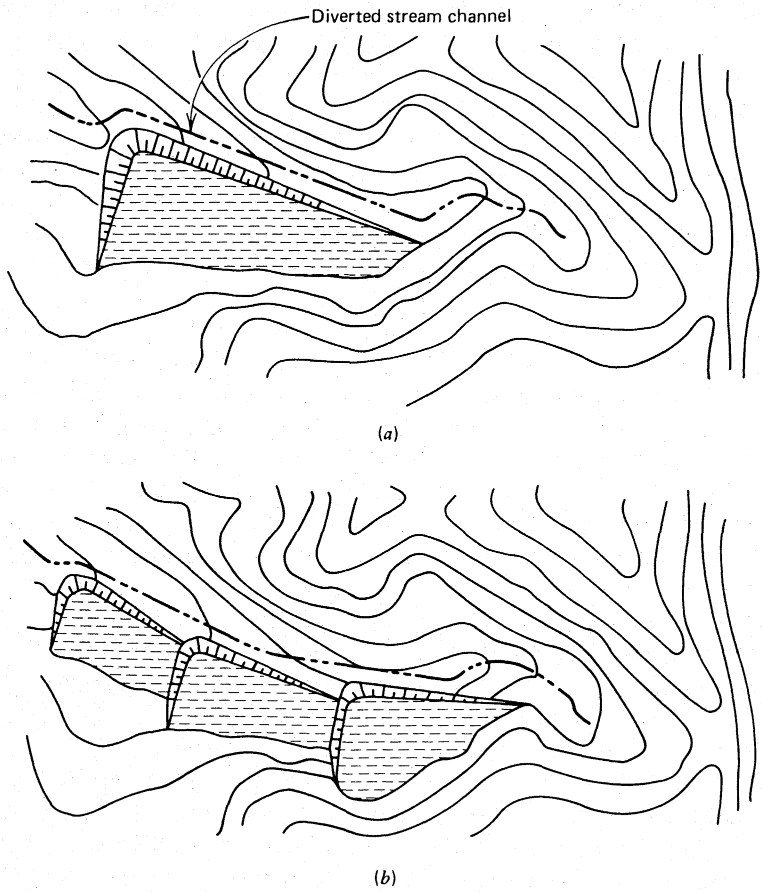


Figure 5.5 Valley-bottom impoundment. (*a*) Single. (*b*) Multiple.

replace the riprap as the embankment face moves outward with progressive raises.

Single versus Multiple Impoundments

All four impoundment layout options described above can be implemented in either single- or multiple-impoundment form. While the best choice depends on specific site conditions, some general advantages and disadvantages apply.

Multiple impoundments usually require a greater total quantity of embankment fill. In the extreme case for ring dikes illustrated in Figure 5.2, 1.5 times as much fill is required as for a single impoundment to achieve slightly less total storage volume. In other cases, however, particularly for constricted sites such as illustrated in Figure 5.5 for valley-bottom impoundments, the fill penalty is not so severe and multiple impoundments may significantly aid in achieving the required storage volume in a limited available space.

Also, for multiple cross-valley and sidehill impoundments shown in Figure 5.3b and 5.4b, the uppermost impoundment bears the full burden of flood runoff inflows. Since the size of the individual impoundment segment is much less than for a single large impoundment, excessive flood storage requirements for the uppermost segment may result, and careful planning for control of surface water is required.

These disadvantages notwithstanding, the benefits from multiple impoundments can be considerable, again depending on individual site conditions. In general, multiple impoundments are constructed sequentially, allowing for smaller initial capital expenditures and producing cash-flow benefits much the same as those realized for raised embankments. Multiple impoundments also offer considerable operational flexibility. Impoundment segments can be constructed either strictly on an as-needed basis or in advance of actual tailings storage requirements as fill material or construction equipment become available. When more than one segment has been constructed, discharge of tailings can be alternated between the impoundments to provide beneficial flexibility in impoundment operation.

Environmental benefits for multiple impoundments compared to single impoundments of equivalent capacity can be major. Generally, multiple impoundments are constructed and filled sequentially. Thus, only a small portion of the eventual total impoundment area is covered with water at any given time. To the extent that seepage is directly proportional to the area over which flow occurs, seepage rates may be considerably reduced. At least as significant is the fact that reclamation can proceed concurrently with ongoing tailings disposal. Following filling of one multiple-impoundment segment, reclamation can begin as discharge is shifted to the next segment, thus minimizing the area disturbed at any one time and reducing problems related to blowing dust.

OPTIMIZATION OF IMPOUNDMENT LAYOUT

For a given impoundment layout, site, and embankment type, there is often an optimum combination of embankment height and impoundment area that will give the lowest fill volume for the required storage capacity. The concept of fill efficiency ratio, originally defined by Coates and Yu (1977), is useful in this regard. The fill efficiency ratio is defined as the ratio of impoundment tailings storage volume to the volume of fill required to achieve that storage. Since fill volume is usually closely related to the cost of the impoundment, the fill efficiency ratio provides an indirect indicator of relative impoundment cost and is useful, not only in optimizing embankment

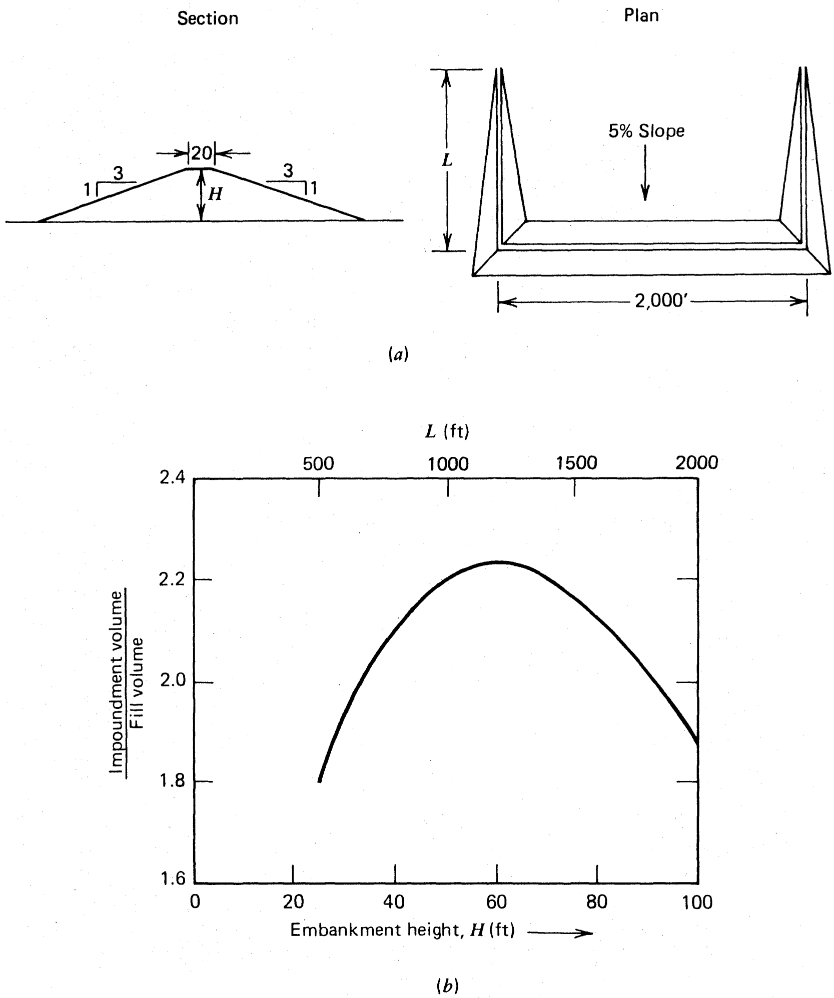


Figure 5.6 Variation in fill efficiency for idealized sidehill impoundment. (*a*) Embankment and impoundment configuration. (*b*) Fill efficiency ratio versus embankment height.

height and impoundment area for a given storage volume, but also in comparing the relative costs of different impoundments with dissimilar capacities. When applied to compare impoundments at different sites, the fill efficiency ratio properly penalizes those sites where higher embankments are required for storage of flood runoff, since the ratio is defined in terms of available tailings storage volume rather than total reservoir volume.

To illustrate the use of the fill efficiency concept, consider the simplified example shown in Figure 5.6. The assumed embankment and impoundment configurations are shown in Figure 5.6a. A sidehill-type layout is being planned on ground sloping at a uniform 5%. The impoundment width is fixed at 2,000 ft by site boundaries, but impoundment length and height can vary.

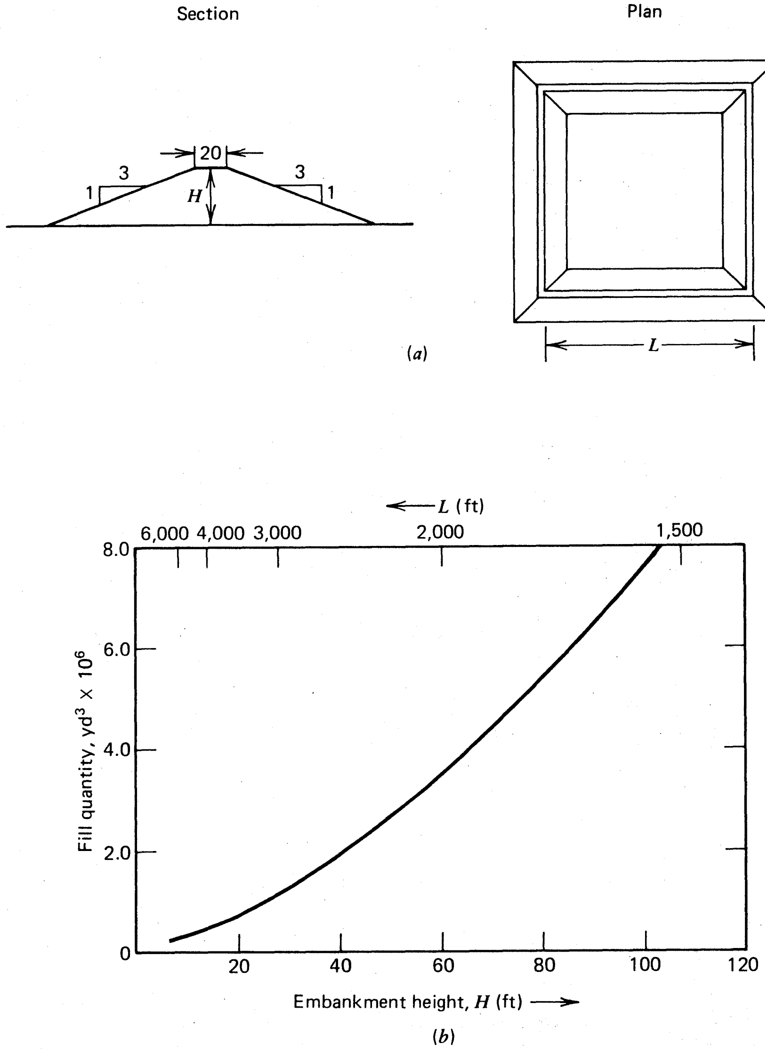


Figure 5.7 Embankment fill quantities for square, unlined impoundment. (*a*) Embankment and impoundment configuration. (*b*) Variation in fill quantity as a function of embankment height.

The problem thus becomes to select the most efficient height and length of the embankment.

Figure 5.6b shows the fill efficiency ratio plotted against embankment height and impoundment length. Small impoundments of low height produce less storage volume per unit fill volume, as do large impoundments with high embankments. For this particular example, the maximum fill efficiency ratio yields an optimum embankment height of about 50 ft and a corresponding length of about 1,200 ft. The resulting storage volume might or might not be compatible with volume requirements dictated by the mill tailings output,

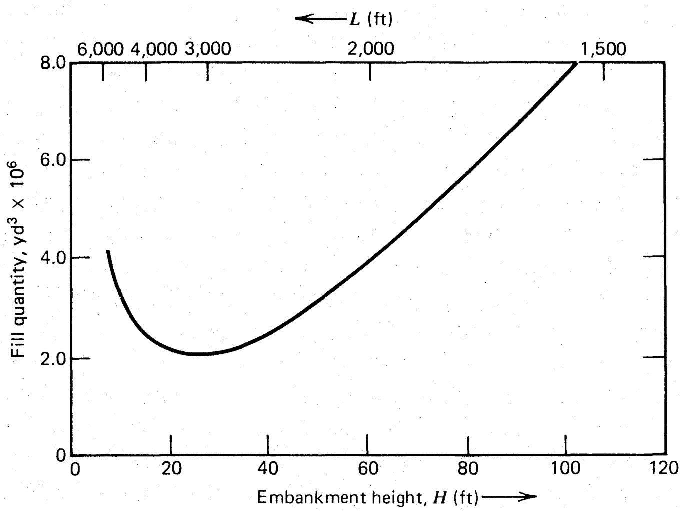


Figure 5.8 Fill quantity versus embankment height for lined impoundment.

but for large storage requirements this example would suggest that a series of multiple sidehill impoundments of the optimum dimensions would be most efficient.

While the geometry of natural drainage basins is much more complex than indicated by this example, it is often the case that fill efficiency decreases for very high embankments. Minimum embankment height is dictated by tailings and flood storage requirements, but there often comes a point of diminishing returns where new impoundments at different sites would be more efficient than excessive raises of a single large impoundment.

Another example illustrating fill efficiency is shown in Figure 5.7. In this case, suppose that a 15-yr tailings storage volume is required for a mill output of 2,000 T/day. The tailings will have an in-place dry density of 90 pcf, resulting in a tailings storage volume requirement of about 2.4×10^8 ft^3. The assumed impoundment is a ring dike layout, as shown in Figure 5.7a, with square sides of length L. The ground surface is assumed to be flat, and runoff water inflow is negligible.

In this example, the problem becomes to select a combination of impoundment height and area to achieve the given storage volume. Obviously, this can be accomplished by either a very low, large impoundment or a high small one. The graph in Figure 5.7b illustrates the diminishing returns for high embankments in terms of their large fill volume and indicates that the given storage volume can be achieved with minimum embankment fill quantity for low dikes and a large impoundment.

Both of the preceding examples have defined fill quantity in terms of embankment fill only, but in some cases, this can produce misleading results. For instance, the example in Figure 5.7 is repeated in Figure 5.8, only this time assuming that a 3-ft thick compacted clay liner is required over the impoundment bottom to minimize seepage. A similar situation might result from requirements for impoundment topsoil stripping or replacing topsoil over the impoundment surface as a part of reclamation. Comparison of Figure 5.8 with Figure 5.7b shows that total fill volumes are substantially increased by the clay liner. More significant is that the liner fill, being a function of impoundment area, penalizes larger impoundments. For the example in Figure 5.8, storage requirements can be met with a minimum total fill volume for an embankment height of about 30 ft and impoundment width of 3,000 ft. This serves to emphasize the point that optimizing impoundment layout must often account for earthwork requirements in the larger sense, including reclamation and seepage-control measures, rather than being based strictly on embankment fill requirements.

Although the above examples are highly simplified, the same general principles apply to more realistic impoundment topography and to all types of impoundment layouts. Establishing the optimum layout is usually a trial-and-error procedure. While experience is of considerable assistance, an optimum layout can usually be arrived at using the concept of fill efficiency.

6

Evaluation of Tailings Disposal Alternatives

In the view of conservationists, there is something special about dams, something—as conservation problems go—that is disproportionately and metaphysically sinister. The outermost circle of the Devil's world seems to be a moat filled mainly with DDT. Next to it is a moat of burning gasoline. Within that is a ring of pinheads each covered with a million people—and so on past phalanxed bulldozers and bicuspid chain saws into the absolute center of hell on earth, where stands a dam. The implications of the dam exceed its true level in the scale of environmental catastrophes. Conservationists who can hold themselves in reasonable check before new oil spills and fresh megalopolises mysteriously go insane at even the thought of a dam.

John McPhee, *Encounters with the Archdruid*

Chapter 3 has discussed various options available for tailings disposal, including surface disposal, underground mine backfilling, and open-pit backfilling, among others. Chapter 5 has presented considerations pertaining to various impoundment sites and layouts that may be identified for a particular project. Common to both tailings disposal methods and siting–layout procedures is the need to somehow select a preferred method or site from among what is usually a set of several possible options defined during the course of initial tailings disposal planning studies. Whether applied with regard to disposal methods or impoundment sites, the selection process involves generating a number of feasible alternatives and comparing their relative advantages and disadvantages.

In the past, selecting a preferred alternative disposal method or site was a relatively simple procedure. Cost estimates could be generated for each option, and the lowest-cost option would ordinarily be the hands-down winner. More recently, however, environmental considerations have gained increasing importance, and perhaps nowhere else in a mining operation are these environmental issues of more significance than in tailings disposal. Environmental factors are often of equal or greater importance than economic issues in tailings disposal planning, at least in the eyes of regulatory agencies with overall authority for approval of the mining operation and citizens' groups having considerable influence in the political process.

The selection of a preferred tailings disposal method or site now becomes much more complicated. Both economic and environmental merits of different alternative disposal methods or sites must by some means be compared. In addition, the selection process is now more visible to both regulatory agencies and the public, and accountability in justifying tailings disposal decisions is required.

At present, tailings disposal alternatives having different combinations of economic and environmental attributes are often compared and selected on an informal ad hoc basis. Since few formal guidelines exist for conduct of the evaluation procedure, disposal decisions made by mining companies are sometimes viewed by regulatory agencies as lacking objectivity, and similar evaluation of alternatives by regulatory agencies may appear arbitrary to the mining company. At the core of the conflict is often the way in which alternatives are evaluated and selected. The purpose of Chapter 6 is to present a more structured approach to the evaluation and selection of tailings disposal methods and sites. The aim of this approach is to clarify and rationalize the decision-making process.

To the extent that mining companies and regulatory agencies have fundamentally different objectives, differences of opinion in the selection of tailings disposal strategies will always arise. Central to these differences are sometimes substantive questions of fact, but more often matters of values. By their respective natures and organizational goals, mining companies will generally place higher values on cost and production, whereas regulatory agencies will place higher values on environmental protection and public health. Resolution of questions of values is always subjective, and to attempt to rationalize the procedure by which decisions are made does not imply the invention of a "magic formula" that, properly applied, produces a single, unique "objective" decision. Rather, by establishing a process for systematic decision making, differences in values and subjective biases can be quickly identified, defined, and discussed by the concerned parties. Trade-offs and conflicting values become more apparent, and the logic behind a particular decision is clarified. Conflicting values on the part of the mining company and the regulatory agency will not be resolved, nor will the subjective element in decision making be eliminated. The process may, however, more clearly identify exactly where differences arise and reduce the mutual misunderstanding that sometimes results when informal, ad hoc procedures produce decisions that may appear to be poorly documented at best and arbitrary at worst. A more rational framework for discussions between the mining company and other parties concerned with tailings disposal decisions may thus be established.

The evaluation methodology presented in Chapter 6, a systematic approach to the definition and evaluation of alternative tailings disposal strategies, consists of the following elements (de Neufville and Stafford, 1971):

1. *Definition of Objectives.* This step establishes in clear, unambi-

guous terms the goals by which various possible disposal methods or sites are to be judged.

2. *Formulation of Measures of Effectiveness.* This step attempts to define quantitative units to measure how well an alternative strategy meets a particular objective.
3. *Definition of Alternatives.* In this step, a manageable number of feasible alternative tailings disposal strategies is defined, each consisting of a unique combination of possible solutions for problems related to tailings disposal methods, disposal sites, seepage control, reclamation, and so on.
4. *Evaluation of Alternatives.* In this step, a matrix evaluation procedure is used for evaluation of alternative tailings disposal strategies. Elements of the evaluation procedure also include sensitivity studies and analysis of uncertainty.
5. *Selection of Preferred Alternative.*

EVALUATION METHODOLOGY

The five steps listed above comprise the core of evaluation methodology and, taken together, provide a systematic approach to the evaluation and selection of alternative tailings disposal methods or sites. The individual components of the approach are explained below.

Definition of Objectives

Clear definition of the objectives that a tailings disposal strategy should achieve may be the single most important aspect of systematic evaluation. No logical evaluation can proceed far without precise, explicit statements of purpose.

Objectives for selection of a tailings disposal method, such as surface disposal, underground mine backfilling, or open-pit disposal, might include:

Maximize integrity of the impoundment to ensure against release of tailings.

Minimize disposal costs.

Minimize seepage.

Maximize the restoration of the disposal area to its former condition during reclamation.

Minimize disruption to ongoing mining operations.

For comparison of surface impoundment sites, many of the siting considerations discussed in Chapter 5 could become objectives, such as:

Minimize distance of site from mill.
Minimize drainage catchment area.
Minimize total fill quantities.

In addition, environmentally related objectives could include:

Minimize seepage effects on groundwater.
Minimize disturbed surface area.
Minimize disturbance to critical species or habitats.

Many environmentally related objectives are often prescribed by a regulatory agency. In the case of uranium tailings disposal in the United States, the U.S. Nuclear Regulatory Commission (1979) has provided clear objectives for uranium tailings management. Other countries, such as Canada and Australia, have similar objectives (Coady and Henry, 1978). Added to these criteria are objectives of state, provincial, and/or local government units that, unfortunately, are sometimes not clearly stated in unambiguous terms and that may be in conflict with objectives of a federal agency. Although these conflicts cannot be resolved within the context of a particular tailings disposal evaluation, it is important that the evaluation process define the objectives that have actually been used.

Formulation of Measures of Effectiveness

Defining an objective may be meaningless if there is no way to determine whether or not it has been met. Defining appropriate measures of effectiveness for each objective, therefore, is also an essential element in the evaluation process. An ideal measure of effectiveness satisfies two criteria:

1. It is a quantitative numerical unit.
2. It can be documented by calculations or verified by field measurement.

Defining measures of effectiveness for many objectives is relatively straightforward. For example, distance and elevation from the mill measure the mill proximity objective, and the fill efficiency ratio defined in Chapter 5 provides a good measure of the extent to which fill quantity is minimized. Certain environmental effects may be correlated with the area disturbed by the disposal impoundment, making impoundment area an indirect but useful measure of effectiveness.

Measures of effectiveness may be of tremendous importance in the ultimate selection of alternatives, and the precise definition of some measures of effectiveness requires careful assessment in some cases. For example, consider the objective: Minimize the adverse effects of seepage from the impoundment. Behind this simple statement lurks a bewildering range of possible consequences, depending on the measure of effectiveness intended. Possible interpretations of the above statement could include:

1. No degradation in water quality at the nearest down-gradient well.
2. No degradation in groundwater quality that would restrict its postmining use to a lower level than it had initially.
3. No migration of contaminants beyond the property boundary.
4. No introduction of contaminants into the groundwater.
5. No release of seepage from the impoundment.

Although these measures of effectiveness appear to have similar intents and purposes, the above list is arranged in increasing order of severity and restrictiveness. To meet the first measure of effectiveness might require only that the impoundment be located a sufficient distance from the nearest well. The second measure of effectiveness—limiting postmining restrictions on groundwater use—would depend on existing groundwater quality but might have minimal effects on siting. Preventing contaminant migration beyond the property boundary, the third measure of effectiveness listed above, could require a variety of measures ranging from no control of seepage to impermeable liners, depending on the hydrogeologic and geochemical characteristics of the site. On the other hand, preventing introduction of contaminants into the groundwater could in fact allow some seepage, depending on the depth to the groundwater table and the ability of natural soil to absorb contaminants by pH buffering and ion exchange. However, the final measure of effectiveness—preventing release of seepage from the impoundment—will restrict impoundment siting to impervious natural formations or will require structural measures, such as liners.

Ideally, the measure of effectiveness would provide a specific numerical limit on the rate of allowable seepage from the impoundment, or would identify specific contaminants and concentrations that cannot be exceeded at a specified location and time. It is clear from the above example that considerable thought must be given to defining a precise measure of effectiveness in conjunction with each objective.

A related situation may arise when defining measures of effectiveness for tailings disposal cost. It is necessary to define whether the cost to be minimized is measured as total or initial construction cost, in addition to whether operational costs are to be considered. This becomes significant, for example, in comparing single versus multiple impoundments, where initial costs for an individual impoundment segment may be lower, but with a greater eventual total cost. Subsequent evaluation of alternatives may be distorted if the measure of cost-effectiveness is not appropriately defined.

Definition of Alternatives

To use any evaluation methodology requires that a reasonable number of discrete alternatives be defined, whether in relation to tailings disposal methods or tailings impoundment sites. Defining alternative methods or sites will be based on factors discussed in Chapters 3 and 5. Screening procedures will usually eliminate methods or sites that are obviously unfeasible, resulting in a smaller subset of feasible options for evaluation. Experience, judgment, and familiarity with the positions of regulatory agencies are important factors in defining alternatives.

A possible pitfall in defining alternatives may stem from failure to include all components of the disposal system. For example, consider an evaluation of tailings disposal methods, one of which might be in-pit disposal. Because of the usually small pit area, decanting of discharged effluent to evaporation ponds may be required to dispose of excess water produced by the mill. If the definition of this alternative tailings disposal method does not include consideration of the evaporation ponds, a misleading picture could result. This method might appear to be very favorable from a seepage loss standpoint when in fact the effect might be merely to shift the location of potential seepage problems from the tailings disposal area to the evaporation pond site. Another example might be a case where hydraulic backfilling with sand tailings is being considered. In this case, the backfilling alternative would also have to take into account supplementary surface impoundments required for disposal of the remaining slimes tailings. Unbiased evaluation requires that the definition of each alternative include all facilities necessary for disposal or treatment of all solid and liquid wastes produced by the mill.

Evaluation of Alternatives

Once a set of feasible alternatives has been defined, the alternatives must be individually evaluated and compared to determine the degree to which they meet the defined objectives according to the established measures of effectiveness. One approach for evaluating tailings disposal alternatives involves a matrix evaluation procedure consisting of the following elements:

1. A matrix consisting of a two-dimensional array is constructed. One side of the matrix is the set of alternatives, and the other side is the set of objectives by which the alternatives are judged. Each objective is assigned a weighting factor that reflects its relative importance. Each alternative is then assigned a set of numbers reflecting the degree to which it satisfies each objective. An overall *ranking* for each alternative is computed by summing the products of each individual objective score and the associated weighting factor.

This procedure follows a methodology Hill (1968) originally proposed for highway planning.

2. A sensitivity analysis is performed to investigate the variation in the alternative rankings brought about by changes in weighting factors. This provides a measure of the importance of the various objectives, identifies those objectives that are crucial to the overall evaluation, and accounts for differences in values that may arise among various evaluators.
3. The effects of uncertainty are accounted for, if required, by the use of subjective probability assessment and the use of expected value in assigning scores to the individual objectives for each alternative.

The procedure for matrix evaluation requires more detailed evaluation and is the topic of subsequent sections in this chapter.

Selection of Preferred Alternative

The matrix evaluation procedure described above provides information on the degree to which each alternative satisfies the defined objectives according to its rank or score. Generally, the result is a group of two or three alternatives that have relatively similar ranks, with the remaining alternatives being eliminated from consideration because of substantially lower ratings. A sensitivity analysis is then performed to investigate the effects of different weighting factors on alternative ratings.

The results of the sensitivity analysis may offer guidance on which of the high-scoring alternatives is preferable. As indicated in the introductory sections of this chapter, a given objective may be valued quite differently by the mining company and by the regulatory agency, and this difference in values is reflected by the changes in ranking resulting from changes in the weighting factors assigned to various objectives in matrix. If a given alternative proves to be very sensitive to the weighting factor assigned to a particular objective, its selection could prove to be controversial. If, on the other hand, major changes can be made in critical weighting factors with little effect on relative score for a given alternative, this alternative might be most preferable to both the mining company and the regulatory agency.

MATRIX EVALUATION PROCEDURES

Matrix Construction

Matrix evaluation procedures have been used in various forms for evaluation of diverse types of multiple-objective project planning, including power plant siting (Hobbs, 1980; Motayed, 1980), highway planning (Hill, 1968), and water resource project evaluation (Znotinas and Hipel, 1979b). While application of the techniques to tailings disposal is new, the basic methodology is fairly well established.

Impoundment Siting Example

Application of matrix evaluation procedures is best illustrated by several examples. The first illustrates evaluation of alternative impoundment sites for an operating mine in southern Wyoming. At least 5 yr of additional tailings storage capacity is required, but longer impoundment lifetimes are not precluded. It is of importance that seepage be minimized, and stripping of topsoil over the impoundment surface is required with subsequent replacement during reclamation.

Three sites have been identified, designated A, B, and C. All are essentially equidistant from the mill and at similar elevations, so mill proximity is not a factor in comparison of the sites. Flood runoff control will be by storage methods. The impoundment site characteristics are provided on Table 6.1.

Site A provides 10 yr of storage capacity and covers 220 acres. The fill efficiency ratio, including topsoil stripping, is 5.7. Site A is judged to have the lowest potential seepage of the three sites because it has a relatively thick deposit of natural silt.

Site B has a larger area of 266 acres and provides 15 yr of storage capacity. Fill efficiency is the highest for the three sites identified. Although similar silt deposits as for Site A are present, seepage potential for Site B is judged to be somewhat higher because of the larger impoundment area.

Site C is the smallest impoundment alternative, covering 105 acres and providing 6 yr of tailings storage capacity. The fill efficiency ratio of 7.8 is only slightly less than for Site B. However, Site C would require a higher dam on steep abutments of highly fractured shale that has proven troublesome from a seepage standpoint for similar impoundments in the area. Site C is also located near a river, presenting the possibility that seepage escaping the impoundment could directly enter surface waters. Site C is judged to have the highest seepage potential.

Three objectives have been defined for the impoundment: to minimize land disturbance, to minimize cost, and to minimize detrimental effects of seepage. The measure of effectiveness for land disturbance is impoundment area. Since tailings pipeline and pumping costs would be similar for the three sites, the measure of effectiveness for cost can be taken as fill quantity, expressed in terms of fill efficiency ratio to compensate for the varying impoundment sizes. Since available information on seepage is limited, the measure of effectiveness for the seepage objective cannot be realistically quantified and is established by judgmentally based estimates of relative seepage loss potential.

Table 6.1 Summary of Alternative Impoundment Characteristics

Site	Area (acres)	Maximum Dam Height (ft)	Tailings Storage Volume ($yd^3 \times 10^6$)	Impoundment Life (yr)	Dam Fill ($yd^3 \times 10^6$)	Topsoil Stripping ($yd^3 \times 10^6$)	Total Earthwork ($yd^3 \times 10^6$)	Fill Efficiency Ratio	Relative Seepage Potential
A	220	35	6.0	10	0.35	0.71	1.06	5.7	Lowest
B	266	40	9.2	15	0.25	0.86	1.11	8.3	Intermediate
C	105	50	3.6	6	0.12	0.34	0.46	7.8	Highest

Table 6.2 Evaluation Matrix for Impoundment Siting Example

Objective	Land Disturbance (weighting factor, 0.1)	Cost (weighting factor, 0.5)	Seepage (weighting factor, 0.4)
Site A	2	1	3
Site B	1	3	2
Site C	3	2	1

Overall rankings

Site A:	$0.1(2) + 0.5(1) + 0.4(3) = 1.9$	intermediate
Site B:	$0.1(1) + 0.5(3) + 0.4(2) = 2.4$	most favorable
Site C:	$0.1(3) + 0.5(2) + 0.4(1) = 1.7$	least favorable

Within this framework, the matrix shown in Table 6.2 can be constructed. The three objectives form one axis of the array, and the three sites form the other. Also indicated on Table 6.2 are weighting factors assigned to each objective and summarized below:

Objective	Weighting Factor
Minimize land disturbance	0.1
Minimize cost	0.5
Minimize seepage	0.4

These weighting factors attempt to account for the perceived relative importance of each objective. Minimizing land disturbance is seen as of little long-term importance, principally because the impoundment will be reclaimed, topsoiled, and revegetated after abandonment, so it is assigned a low weighting of 0.1. Of the remaining two objectives, minimizing seepage is viewed as only slightly less significant than minimizing cost, resulting in respective weighting factors of 0.4 and 0.5.

In Table 6.2 the matrix is constructed by assigning an integer rank for each site according to its effectiveness in meeting each objective: 3 is the highest ranking and 1 is the lowest. It can be seen, for example, that rankings for land disturbance follow directly from the impoundment surface areas given in Table 6.1. Similarly, cost rankings follow directly from fill efficiency ratios for the various sites.

Overall site rankings are computed as shown in Table 6.2 by summing the products of the weighting factors and the ranking for the corresponding objective for each site. The highest overall ranking indicates the most favorable site. In this particular example, Site B is most favorable, while Site C is least favorable. Site A is intermediate.

These results are intuitively satisfying on a more general level from review of Table 6.1, keeping in mind the weighting factors assigned to the various objectives. Site A is heavily penalized by its low fill efficiency and high cost, whereas Site C, although of favorable cost, has high demerits due to seepage potential. While it is possible that the same conclusions could have been obtained informally by simple inspection of Table 6.1, application of the matrix evaluation provided the opportunity to formally state assumptions regarding weighting factors, definitions of objectives, and measures of effectiveness. Certainly selection of a preferred alternative by nothing more than inspection of such information as that provided in Table 6.1 would be difficult to justify had there been a greater number of alternatives or objectives than in this simple example, which is usually the case for actual site-selection decisions.

Disposal Method Example

Although preferred disposal methods for most types of tailings are usually fairly obvious and seldom require formal investigation and justification, uranium tailings disposal is quite different. The following provides an example of the application of matrix evaluation techniques to a hypothetical uranium mine where thorough evaluation of various disposal methods is necessary. In this case, a prime objective of the disposal method is that it provide a high degree of integrity against dispersion of the tailings by erosional processes for many thousands of years. The selection of a disposal method is often heavily weighted toward meeting this objective. Also of major importance is the ability of the disposal method to minimize seepage losses.

Suppose that three objectives for the disposal method have been defined:

1. The disposal method should be such that the tailings are isolated from long-term disruption by natural factors.
2. The disposal method should be such that seepage is minimized.
3. The initial construction cost of the disposal facility should be minimized.

Suppose also that the uranium deposit under consideration is at a depth of 3,000 ft in relatively competent rock, so that an underground operation is the proposed mining method. The mill site is located on a gently sloping alluvial fan, and sites for both conventional surface impoundments and specially excavated pits for below-grade tailings disposal are available. Given these factors, the following three disposal method alternatives have been defined:

Alternative I. After initial mine development, tailings from the mill would be pumped underground and deposited in mined-out stopes. The nature of the stopes would prevent construction of artificial impermeable liners; therefore the stopes would remain unlined. It is assumed for this example that all tailings produced by the mill could be returned underground.

Alternative II. During mill construction, a large pit would be excavated at the surface especially for tailings disposal. The pit would be lined with a synthetic liner material and would provide below-grade tailings disposal for the life of the mill.

Alternative III. During mill construction, a nearby drainage would be dammed to form a conventional surface tailings impoundment. Since no natural soils with sufficiently low permeability are available on site, the impoundment would be lined with natural soil containing an admixture of commercial bentonite imported from off-site sources.

Given a set of objectives and a set of alternatives, an evaluation matrix can be constructed according to either of two methods, designated *rank-ordering* procedures and *scoring* procedures. The two methods, which are identical in concept but slightly different in application, are both applied below to the uranium tailings disposal method example.

Rank-Ordering Procedure

Rank-ordering procedures correspond to the general method used for the previous impoundment siting example. After defining alternatives, objectives, and measures of effectiveness, the alternatives are rated on a rank-order, 3–2–1 basis according to their relative abilities to meet each objective.

For the present example, suppose that the evaluator has concluded that the objective of achieving long-term isolation of the tailings is critical and roughly three times as important in the long run as minimizing seepage or minimizing cost. Suppose also that it has been concluded that the objectives of minimizing seepage and minimizing cost are roughly equivalent in importance. The evaluator then assigns the following weighting factors, which reflect these subjective views of relative emphasis:

Objective	Weighting Factor
Long-term isolation	0.6
Minimize seepage	0.2
Minimize cost	0.2

After weighting factors are assigned to the objectives, the next step in matrix construction is to rank the three alternatives to reflect their relative ability to meet each objective. In this case, a ranking of 1–3 is established, with 1 being the lowest ranking and 3 the highest.

Table 6.3 Example of Matrix Evaluation Procedure, Rank Order of Alternatives

Objective	Long-Term Isolation (weighting factor, 0.6)	Minimize Seepage (weighting factor, 0.2)	Minimize Cost (weighting factor, 0.2)
Alternative I: Underground mine disposal	3	1	2
Alternative II: Disposal in special pits	2	3	1
Alternative III: Surface disposal (dams)	1	2	3
Weighted ranking			
Alternative I:	(0.6)(3) + (0.2)(1) + (0.2)(2) = 2.4		most favorable
Alternative II:	(0.6)(2) + (0.2)(3) + (0.2)(1) = 2.0		intermediate
Alternative III:	(0.6)(1) + (0.2)(2) + (0.2)(3) = 1.6		least favorable

For the objective of achieving long-term isolation, Alternative I, mine disposal, provides deep burial of tailings in mine stopes and provides the greatest degree of isolation. Alternative I is assigned a 3 ranking for this objective. Alternative II, pit disposal, provides slightly less isolation since burial depths are shallower, so is assigned a rank of 2. Alternative III, disposal in a surface impoundment, might be subject to erosion in the long term by wind or water and is assigned the lowest ranking of 1.

For the objective of minimizing seepage, the unlined mine rock of Alternative I contains joints that could permit more seepage than Alternatives II or III; Alternative I is therefore assigned a ranking of 1 for this objective. Since the synthetic liner for Alternative II would virtually eliminate seepage (provided no leaks develop), Alternative II is assigned a ranking of 3. Because of the possible difficulty in properly constructing a soil-bentonite liner, Alternative II is considered slightly less effective than an intact synthetic liner but better than no liner at all and is therefore ranked 2.

In terms of initial construction cost, Alternative I will be moderately costly because of lengthy underground tailings pipelines, so it is assigned a 2 ranking. Excavation of special disposal pits for Alternative II is clearly the most costly and construction of Alternative III tailings dams is the least costly, accounting for their respective rankings of 1 and 3 for the cost minimization objective.

Table 6.3 illustrates the matrix constructed from the above information. The objectives and weighting factors describe one dimension of the array, and the alternatives the other dimension. Rankings of the alternatives for each objective constitute the elements of the matrix.

The final step in matrix construction is the computation of weighted ranking. As illustrated in Table 6.3, this is accomplished by summing the products of each weighting factor and the associated ranking for each alternative. The highest weighted ranking indicates the most favorable alternative. In this example, Alternative I, deep mine disposal, is most favorable; Alternative II, in-pit disposal, is moderately favorable; and Alternative III, surface impoundment disposal, is least favorable.

This rank-ordering procedure for matrix evaluation is often consistent with preliminary or feasibility-level investigations where relatively little factual information or few confirmed data are available. For example, data may not be sufficient to estimate costs reliably, but an evaluator who is experienced in tailings disposal may nevertheless be reasonably able to conclude that certain alternatives will be more costly than others. The rank-ordering procedure also has an advantage in its simplicity; this may become an important consideration when, say, nine alternatives and six objectives comprise the matrix.

The rank-ordering procedure, however, suffers from several disadvantages. First, the rank-ordering of alternatives "forces" a uniform variation in the ability of each option to meet a certain objective, even though the options may vary widely in their effectiveness. For example, if the costs for Alternatives I, II, and III were \$1 million, \$19 million, and \$20 million, respectively, assigning a 3–2–1 ranking to the alternatives would not reflect the similarity in costs between Alternatives II and III, or the wide range between them and Alternative I. Another disadvantage is that the numerical value of the weighted ranking has no meaning; it cannot be concluded from Table 6.3, for example that since the ratio of weighted rankings for Alternatives I and III is 1.5, Alternative I is 50% "better" than Alternative III. A weighted ranking is a qualitative index indicating which alternatives are more favorable than others, but the quantitative "spread" between alternatives cannot be ascertained.

Scoring Procedure

Some of the disadvantages of rank-ordering can be overcome by the use of a *scoring* procedure, but at the expense of some simplicity. The scoring procedure requires more data and a greater degree of judgment, but it may yield more precise and useful information for the evaluation of alternatives.

To illustrate the use of the scoring procedure, consider the same uranium tailings disposal example. The three alternatives, the objectives, and the weighting factors are the same as previously discussed. A matrix constructed using the scoring procedure is shown in Table 6.4.

Scores are established by either a subjective approach or a quantitative approach where well-defined measures of effectiveness are available. In the

Table 6.4 Example of Matrix Evaluation Procedure, Scoring of Alternatives

Objective	Isolation (weighting factor, 0.6)	Seepage (weighting factor, 0.2)	Cost (weighting factor, 0.2)
Alternative I:			
Underground mine disposal	3.0	1.0	1.8
Alternative II:			
Disposal in pits	2.8	2.8	1.5
Alternative III:			
Surface disposal (dams)	1.5	2.8	2.0
Weighted scores			
Alternative I:	(0.6)(3.0) + (0.2)(1.0) + (0.2)(1.8) = 2.36		favorable
Alternative II:	(0.6)(2.8) + (0.2)(2.8) + (0.2)(1.5) = 2.54		most favorable
Alternative III:	(0.6)(1.5) + (0.2)(2.8) + (0.2)(2.0) = 1.86		unfavorable

case of scores assigned to the long-term isolation objective, suppose that the evaluator has established an effectiveness scale from 0 to 3.0. Referring to Table 6.4, the evaluator believes that underground mine disposal (Alternative I) will be totally effective in providing permanent isolation, so the highest score of 3.0 is assigned. Disposal in specially excavated pits (Alternative II) is believed to be only slightly less effective, justifying a score of 2.8. Tailings dams are believed to be twice as susceptible as deep mine disposal to long-term disruption, so the score for Alternative III is one-half of that for Alternative I, or 1.5.

A quantitative approach to scoring is illustrated in Table 6.4 by scores for the seepage and cost objectives. Again using a scale of 0–3.0, and assigning corresponding upper and lower limits for seepage quantity and cost, the scores would be directly proportional to computed seepage quantities in gallons per minute, or estimated cost in dollars, assuming that dollar cost and computed seepage quantities are the appropriate measures of effectiveness. It should be noted that the scoring procedure, unlike the rank-order procedure, permits alternatives that satisfy a given objective to the same degree to be assigned equal scores. This is illustrated in Table 6.4 by equal scores for Alternatives II and III with respect to the seepage objective.

Weighted scores are computed in the same way as weighted ranking, as shown in Table 6.4. From the weighted scores, it can be concluded that Alternative II is most favorable, followed closely by Alternative I with a score only slightly lower. Alternative III has a significantly lower score and probably should not be considered further. The difference between Alternatives I and II is so comparatively small, however, that this difference could be influenced by many second-order factors not considered in the matrix, and a decision between the two would not be warranted without further analysis. For alternatives with such similar scores, sensitivity analysis would be essential prior to a final decision.

Sensitivity Analysis

Sensitivity analysis attempts to identify the subjective element in the matrix evaluation and to account for differences in values among various parties to the selection process. The assumption underlying the sensitivity analysis is that, all things being equal, alternatives favorable under a wide range of different values are preferable to those favorable under only a narrow set of circumstances. That alternative should be preferred which is found favorable for a wide range of weighting factors reflecting differing values of different evaluators.

To illustrate the procedures involved, a simplified sensitivity analysis can be performed on the matrix presented in Table 6.4. From the preceding discussion of the uranium tailings disposal example, it can be recalled that weighting factors for the example matrix stressed the importance of long-term isolation, with lesser emphasis placed on seepage and cost. This might represent values placed on the various objectives by a regulatory authority. If the evaluator were a mining company, however, values might be somewhat different, with greater emphasis on cost. For purposes of sensitivity analysis, this could be reflected by generating a new set of weighting factors, such as those shown below:

Objective	Weighting Factor Used in Original Analysis	New Weighting Factor Used in Sensitivity Analysis
Long-term isolation	0.6	0.3
Minimize seepage	0.2	0.3
Minimize cost	0.2	0.4

To perform the example sensitivity analysis, only the weighting factors for objectives are changed; the scores with respect to the various objectives remain as before. The weighted score for each alternative is recomputed using the new weighting factors, as illustrated in Table 6.5.

The resulting weighted scores can be compared to those from the previous matrix shown on Table 6.4. Of the two options that scored highly in the original analysis, Alternatives I and II, Alternative II remains most favorable. Because of the increased emphasis on minimizing cost in the sensitivity analysis, the less costly Alternative II scores higher. The score for Alternative I drops because of the reduced emphasis on long-term isolation.

Table 6.5 Sensitivity Analysis for Example Matrix Evaluation

Objective	Long-Term Isolation (weighting factor, 0.3 [0.6][a])	Minimize Seepage (weighting factor, 0.3 [0.2][a])	Minimize Cost (weighting factor, 0.4 [0.2][a])
Alternative I:			
Underground mine disposal	3.0	1.0	1.8
Alternative II:			
Disposal in pits	2.8	2.8	2.5
Alternative III:			
Surface disposal (dams)	1.5	2.8	2.0
Weighted scores			
Alternative I:	$(0.3)(3.0)+(0.3)(1.0)+(0.4)(1.8)$	$= 1.92$	least favorable
Alternative II:	$(0.3)(2.8)+(0.3)(2.8)+(0.4)(1.5)$	$= 2.28$	most favorable
Alternative III:	$(0.3)(1.5)+(0.3)(2.8)+(0.4)(2.0)$	$= 2.09$	intermediate

[a]Original weighting factor applied in Table 6.4.

The result of the sensitivity analysis for the particular example problem would probably be the selection of Alternative II as the preferred uranium tailings disposal method. Not only does Alternative II score highly in the original matrix evaluation, but it also proves to be relatively insensitive to different values that might be brought to bear on the analysis by different evaluators.

More sophisticated approaches are also available for attempting to reconcile different weighting factors and values that may be held by competing interest groups. One such technique is the so-called *Fuzzy Set* approach, which uses a set of formal matrix operators to identify areas of common agreement among various groups to provide a basis for decision making. These techniques have been used for evaluation of complex and controversial water resources projects, such as the Garrison Diversion Unit (Znotinas and Hipel, 1979a, 1979b).

Effects of Uncertainties

Major uncertainties or risks may sometimes complicate the process of assigning scores to objectives for individual alternatives. Significant uncertainties might arise, for example, in assessing the susceptibility of an impoundment to long-term disruption, in evaluating the likelihood of successful revegetation for a particular reclamation scheme, or in evaluating the effects of tailings disposal on biota. The effects of uncertainty can be accounted for in the matrix evaluation by substituting a probabilistically derived expected value in the matrix score.

Expected value accounts for the outcome of uncertain events by weighting each possible outcome according to its probability of occurrence. Mathematically, expected value can be defined as follows:

For a random event, x, with n possible outcomes,

$$E(x) = \sum_{i=1}^{n} p_i C_i$$

where $E(x)$ = expected value of a random event with n possible outcomes that are mutually exclusive and form a collectively exhaustive set

p_i = probability of outcome i

C_i = value associated with outcome i

and also, from elementary probability theory,

$$\sum_{i=1}^{n} p_i = 1$$

Although the concept of expected value is a simple one, the proper interpretation and use of probability in determining expected value require some clarification.

There are two philosophical schools regarding the interpretation of probability. These philosophies are by no means mutually exclusive, but application of probabilistic techniques requires an understanding of some underlying assumptions. One interpretation is relative frequency type probability, in which the probability of a random event is determined by repeated trials. Relative-frequency probability is used in statistical determination of flood frequency and earthquake recurrence intervals, as discussed in Chapters 4 and 9. The other interpretation is subjective, degree-of-belief probability, which holds that the probability assigned to a random event is simply a numerical measure of confidence in the evaluator's prediction of a given outcome.

The difference between the two approaches can be illustrated by a weather forecaster's prediction of rain on a particular day, say April 1. The forecaster could assess the probability of rain according to the relative frequency approach by determining from climatological records that rain fell on April 1 during 30 out of the past 100 yr. The resulting rainfall probability of 30% would be a theoretically valid, but not particularly useful, interpretation. Alternatively, the forecaster might predict a 70% chance of rain on April 1 by examining all the available data and interpreting it according to experience and judgment. This subjective, degree-of-belief probability would be a numerical measure of the forecaster's confidence in the prediction of rain.

Subjective, degree-of-belief probability is most useful for the matrix evaluation approach. A full discussion of probability assessment and statistical decision theory is beyond the scope of this chapter, but Raiffa (1968), Schlaifer (1959), and Grayson (1960) provide excellent discussions of the use of subjective probability in business decisions. In addition, Baecher (1972) and Vick (1974) explore the application of subjective probability to subsurface exploration and geotechnical engineering decisions.

The application of subjective probability and expected value to the matrix evaluation technique can be illustrated by an example that addresses the effectiveness of synthetic liners in seepage control. An actual case involves a proposal to use a synthetic liner in a 90-ft deep tailings impoundment with a projected operating life of 50 yr. The design is unprecedented to the extent that liners have seldom been used under such great depths of tailings, and satisfactory liner aging characteristics have been documented only for periods of less than about 20 yr. Some uncertainty therefore exists concerning the long-term integrity of the liner, in addition to unknown short-term integrity related to design and quality of construction.

For purposes of matrix evaluation, the integrity of the liner could be treated as an uncertain random event with three possible outcomes. If the liner is completely effective for the full 50-yr period, there will be virtually no seepage. A matrix score of 3.0 would be assigned to this outcome. Assuming that the evaluator is knowledgeable and experienced in the factors affecting liner performance, the evaluator might estimate, based on his or her judgment, a probability of 0.5 that the liner would be completely effective.

Recognizing, however, that perfect seaming is difficult to achieve in the field, the evaluator assigns a probability of 0.3 to the event that leaks could occur because of imperfect construction. Should this occur, seepage would be estimated at 50 gpm. A score of 2.0 is assigned to this seepage value. Because of unknowns related to long-term liner integrity, the evaluator believes there is a 20% chance that the liner may degrade before the end of its 50-yr service life. Should this happen, seepage up to 200 gpm would occur, to which a score of 0.5 would be assigned.

The expected score for the seepage control objective for this alternative would be computed by summing the products of the probability and the score assigned to each of the three possible outcomes:

$$E(x) = 0.5(3.0) + 0.3(2.0) = 0.2(0.5) = 2.2$$

This calculation is illustrated in decision-tree format in Figure 6.1. If, for example, Alternative II were the one under consideration, the original score of 2.8 for seepage control shown on Table 6.4 would be replaced by the

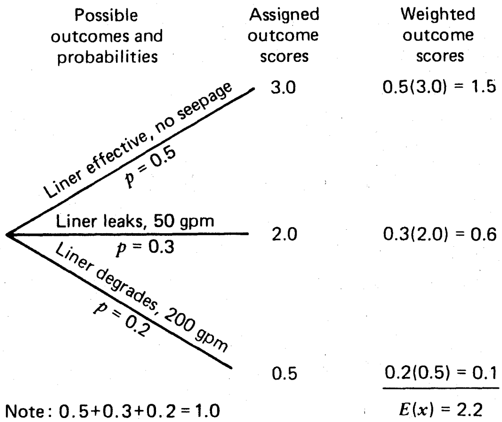


Figure 6.1 Calculation of expected score for liner-integrity example.

expected score of 2.2 computed as shown above. Expected scores could be computed in a similar way for other matrix elements subject to risk or uncertainty. The technique, however, is generally not appropriate for use with the rank-order procedure.

SUMMARY

Establishing a tailings disposal plan for a particular operation involves identifying both a disposal method as well as a site for the disposal area. Application of principles discussed in Chapters 3 and 5 will usually result in a set of alternative disposal methods or sites. When the number of alternatives exceeds more than a few, rational selection of a preferred alternative becomes difficult unless a systematic evaluation methodology is applied. The need for such a systematic method can be even more acute when different evaluators with different sets of values, such as a mining company and a regulatory authority, are both parties to the selection process.

The alternative evaluation methodology presented in this chapter brings to bear a systematic approach to the selection of disposal method or site alternatives. The methodology does not eliminate the subjective element in decision making, nor does it reconcile conflicting values among different evaluators. By clearly identifying assumptions and the logic process by which a decision has been made, the method does, however, reduce the arbitrary quality of selections made on an informal or ad hoc basis. Such a systematic approach can establish a common framework for discussion between the mining company and other concerned parties, thereby facilitating clear understanding and cooperative interaction in the decision-making process.

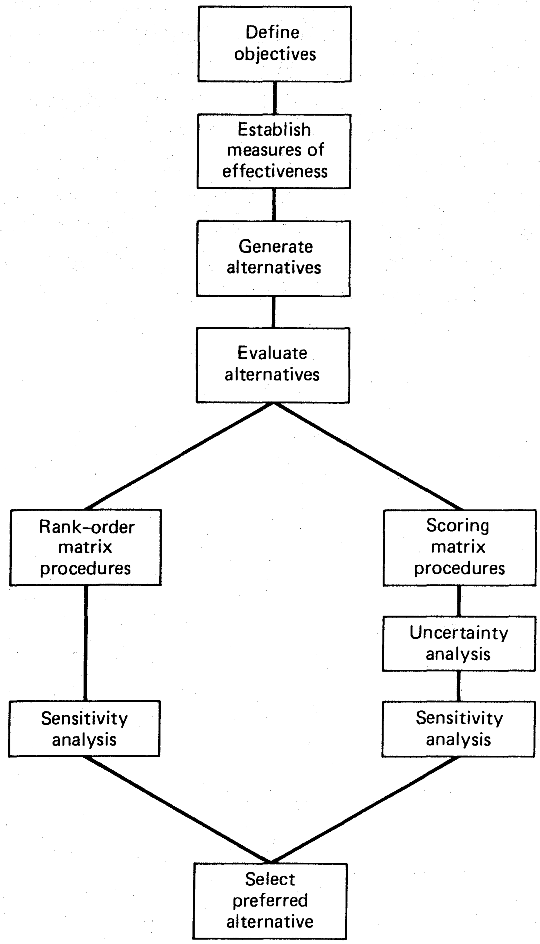


Figure 6.2 Flowchart for tailings disposal alternative assessment methodology.

The alternative assessment approach presented in Chapter 6 consists of several basic steps:

1. Define the objectives of the disposal plan.
2. Establish measures of effectiveness.
3. Generate alternative schemes.
4. Evaluate the alternatives.
5. Select a preferred alternative.

A matrix evaluation procedure, performed using either ranking or scoring techniques, can be used for evaluating alternatives. Expected-value concepts can be used in the matrix to account for the effects of uncertainty.

The result of the matrix evaluation will be to identify one or more favorable alternatives, given the objectives established for the analysis and the weighting factors applied to each objective. An essential element of the matrix evaluation is sensitivity analysis, whereby objective weighting factors are varied to reflect possible differences in values among various parties to the selection process. The results of the sensitivity analysis may identify which from among a group of high-scoring alternatives remain favorable even for significant differences in values or weighting factors.

A flowchart summarizing the sequential steps of the evaluation approach discussed in Chapter 6 is shown in Figure 6.2.

7

Tailings Embankment Design

Engineers are thought to be practical people, and yet the core of engineering contains elements of idealism that are absent in other professions, medicine and law. I mean idealism in the original platonic sense, the belief that somewhere is the perfect design, the perfect dam. We'll never see it, and we'll certainly not come close to building it, but we know enough about it to be forever seeking it. . . . I learned the pleasure in it, in this design. For in the end it does not differ from any other art, the satisfaction in making a clay bowl or a painting or writing a sentence or a symphony. First the concept, the trial efforts, the crude shape of a good solution, then refinements, balance, and polish until the final arrangement sings with deceiving simplicity and stuns with accuracy of effect.

Richard Meehan, *Getting Sued and Other Tales of the Engineering Life*

Methods such as those presented in Chapter 6 will ordinarily result in the selection of a site for tailings disposal, as well as a disposal method. Assuming that surface disposal has been identified as the preferred method, one of the embankment types discussed in Chapter 3 will also have been identified on a preliminary basis as being compatible with the tailings characteristics, mill parameters, and site conditions.

The embankment type under consideration may be of the water-retention variety or one of the raised embankments, including upstream, downstream, or centerline types. Determination of embankment type, governed by principles outlined in Chapters 3 and 4, incorporates consideration of the following key issues:

Mill-related factors.

- Type of tailings and their engineering characteristics.
- Mill output of tailings and liquid effluent.

Site-related factors.

- Expected level of seismicity.
- Water-handling requirements.
- Available construction materials.

The type of embankment selected must be compatible with constraints imposed by these factors.

Integrally tied to the selection of embankment type are siting and layout considerations. As discussed at length in Chapter 5, site topography, hydrology, and geology dictate the configuration of both the impoundment and the embankment required to confine it. Layout of the embankment may be in ring dike, cross-valley, sidehill, or valley-bottom configuration. For any of these options, the impoundment may be developed as a single unit or in multiple-segment form.

Having established the preferred disposal method, embankment type, site, and layout, the stage is set for consideration of specific factors that affect design of the embankment itself. The purpose of this chapter is to explore the various issues in tailings embankment design. "Design" in the context of this chapter refers primarily to selection of materials and their internal arrangement or zoning within the embankment section, as well as accounting for special foundation conditions that may influence the performance of the structure. Principles developed in this chapter will result in development of a detailed internal configuration and raising plan for the embankment. They stop short of embankment analysis, various forms of which are considered in subsequent chapters.

CONTROL OF PHREATIC SURFACE

The location of the phreatic surface, or internal water level, within an embankment exerts a fundamental influence on its behavior, and control of the phreatic surface is of primary importance in embankment design. The phreatic level governs to a large degree the overall stability of the embankment under both static and seismic loading conditions, in addition to influencing the susceptibility of the embankment to seepage-induced failure.

The objective of prime importance is to keep the phreatic surface as low as possible in the vicinity of the embankment face. To the extent that the arrangement of materials of differing permeability within the embankment governs internal seepage patterns, control of the phreatic surface dictates the types of materials required for construction and their configuration in internal zones. A general principle that guides embankment design in relation to phreatic surface control is that permeability of various internal zones should increase in the direction of seepage flow. As permeability increases, the phreatic surface is progressively lowered, and ideally the most pervious available material should be located at or beneath the embankment face.

This principle is illustrated in Figure 7.1. Figure 7.1a shows an idealized upstream embankment in which permeability increases in successive zones in the direction of seepage flow, from low-permeability slimes near the decant pond to high-permeability sands at the embankment face. In this case, the phreatic surface is reasonably low near the face, and seepage breakout

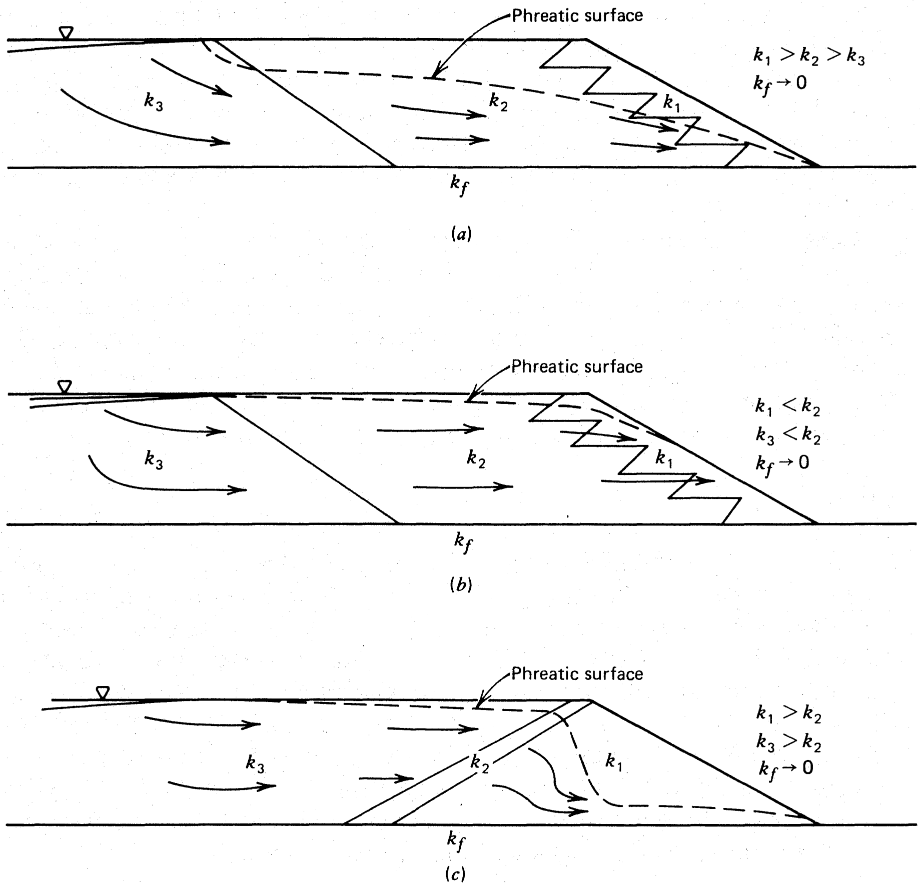


Figure 7.1 Effect of internal zoning on phreatic surface. (*a*) Proper internal permeability configuration for control of phreatic surface. Arrows indicate flow direction. (*b*) Seepage blocked by low-permeability material at embankment face, producing high phreatic surface. (*c*) Seepage restricted by upstream core and drained by downstream pervious zone to produce good phreatic surface control.

on the face itself, which could induce dangerous erosion and slumping, is avoided.

Figure 7.1b shows the same case, except with a low-permeability zone at the face, such as might result from perimeter dikes constructed of clayey natural soils. Here the low-permeability zone impedes drainage and results in an elevated phreatic surface that breaks out high on the embankment face, producing conditions conducive to both mass instability and such seepage-related problems as piping and erosional sloughing.

The principle of increasing permeability in the direction of seepage flow applies in a strict sense only to materials near the embankment face. Figure

7.1c shows a downstream or water-retention type embankment with an upstream core and a pervious downstream shell. In this case, the permeability of the retained tailings can be higher than that of the core with no appreciable effect on the phreatic surface. Thus, it is possible for a properly designed downstream or water-retention dam to function entirely independently from the nature of the retained material, either tailings, water, or both.

Control of the phreatic surface by internal zoning does not depend on the absolute value of permeabilities of various zones, but rather on their relative magnitudes with respect to each other. Consequently, a relative increase in permeability in the direction of seepage flow can be achieved by introducing either lower-permeability zones in upstream regions of the embankment or higher-permeability zones downstream. The former method is accomplished using cores of various types, and the latter by internal drainage zones.

Cores

The use of low-permeability cores is adaptable to downstream or centerline raising methods. As shown in Figure 7.2, inclined cores are well suited to incremental downstream raises, whereas central cores are best adapted to centerline methods. Good control of the phreatic surface in the vicinity of the embankment face is achieved by either arrangement, provided that the downstream shell materials are sufficiently pervious with respect to the core. Prerequisite to the use of cores as a phreatic surface control method is, of course, that suitable low-permeability natural soils be locally available. Cores of some type are usually mandatory where water will stand directly against the upstream face of a pervious embankment.

Drainage Zones

As a supplement to cores, or as an alternative when only permeable soils are available at the site, internal drainage zones also provide a means for phreatic surface control. Internal drainage zones may be of either chimney or blanket type. In common nomenclature, *chimney drains* are those that rise upward within the embankment, either vertically or inclined, and intercept lateral seepage. Horizontal *blanket drains* may be used alone at the base of the structure or in combination with chimney drains. A variation of blanket drains are *finger* or *strip* drains, essentially individual blanket drains that are discontinuous in the direction parallel to the dam centerline. Soderberg and Busch (1977) describe several variations of blanket and finger drains in tailings embankment applications. They also note that perforated pipes are sometimes incorporated into blanket or finger drains to increase discharge capacity. However, the use of pipes is to be avoided where foundation settlement could cause separation of pipe segments at joints or where

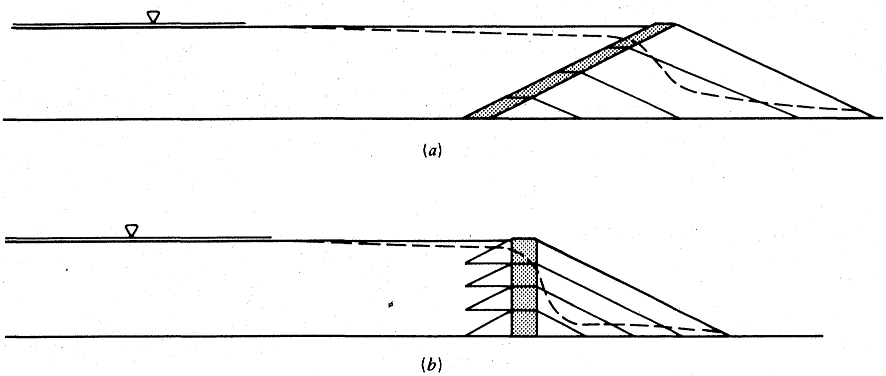


Figure 7.2 Use of low-permeability cores in raised embankments. (a) Downstream embankment. Phreatic surface indicated by dashed line. (b) Centerline embankment.

either corrosion or plugging by precipitates could occur in the chemical environment of the seepage effluent.

Continuous drains are not used in conjunction with upstream embankments, since chimney drains within the perimeter dikes would be too close to the embankment face to be of any value. As shown in Figure 7.3a, however, blanket drains extending well upstream from the starter dike and constructed prior to impoundment operations may have a beneficial effect on the phreatic surface, provided that great care is exercised during operation to prevent drain plugging by tailings fines. Methods for sizing of blanket drains beneath upstream embankments are reviewed by van Zyl and Harr (1977).

A related consideration illustrated in Figure 7.3a is the importance of providing a starter dike for upstream embankments that is more pervious than the tailings. Otherwise, the phreatic surface may be forced to exit on the embankment face at or above the starter dike crest elevation. Based on the results of parametric seepage analyses, Nelson et al. (1977) show that starter dike permeability is the most important factor influencing phreatic surface location for upstream embankments, with the presence of a drainage blanket having a lesser effect. As an alternative to an entirely pervious starter dike, an upstream drainage zone connecting to a blanket drain underlayment may be provided in the starter dike as a means to conserve scarce drainage material. Water cannot be allowed to pond directly against such a pervious starter dike, and a spigotted tailings beach is mandatory.

Figures 7.3b and 7.3c illustrate the use of chimney–blanket (or chimney–finger) drains in downstream and centerline-type embankments. Sizing of such drains can usually be performed using methods established for water-retention dams, as described by Cedergren (1967). The use of combined chimney–blanket drain arrangements allows maximum flexibility in selection of materials for the remainder of the embankment. Since seepage is

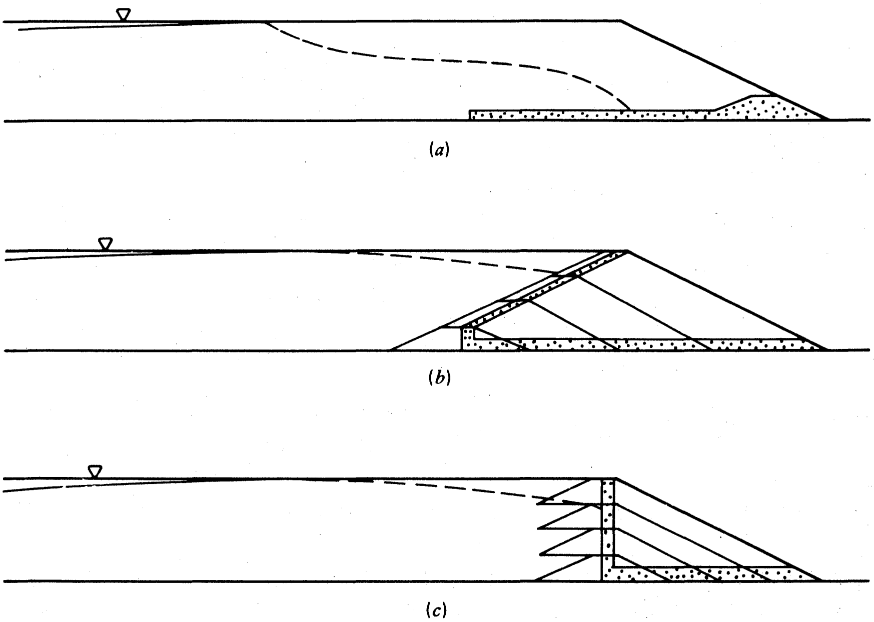


Figure 7.3 **Use of internal drainage zones in raised embankments. (*a*) Upstream embankment using pervious starter dike with upstream blanket drain. (*b*) Downstream embankment using inclined chimney drain and blanket drain. (c) Centerline embankment with vertical chimney drain and blanket drain.**

intercepted and carried along the base of the structure by the drainage system, restrictions on permeability of the remaining fill zone in the downstream shell are not necessary. If the internal drains prevent saturation of the downstream shell, then the downstream shell materials will have no effect on the phreatic surface and their permeability is of no consequence. As a result, the downstream shell can be composed of a random material zone using almost any readily available material, subject only to specifications regarding uniformity and strength.

The use of drainage as a primary means of phreatic surface control requires that sands and gravels be locally present and readily obtainable, ideal conditions that are not often realized in actual practice. Even where natural sand and gravel deposits are available, washing and/or processing are often required to obtain suitably graded materials. These operations are ordinarily quite costly, with the result that a drainage zone, even though comprising a small percentage of the embankment fill volume, may sometimes account for a disproportionately large fraction of total fill costs. In addition, environmental restrictions sometimes prohibit excavation of sands and gravels within floodplains, where these alluvial materials are most often found.

The effectiveness of drainage zones may be seriously impaired if the tailings effluent is high in dissolved solids susceptible to precipitation in response to small changes in pH or temperature. These conditions are typified by gypsum tailings embankments, where plugging of drainage zones and drain pipes by precipitate is common, rendering drainage all but ineffective as a phreatic surface control measure. Also, it is prudent to avoid the use of drainage aggregates high in carbonates to avoid solutioning and breakdown of the aggregate particles if the mill effluent is of low pH.

Use of Tailings

The designer is faced with a dilemma in the all-too-frequent case when neither low-permeability materials for cores nor high-permeability soils for drainage zones are locally available at a particular site. Under such circumstances an option for control of the phreatic surface may be to make use of the tailings themselves. Major reduction in the phreatic surface generally requires a permeability difference between adjacent zones of about two orders of magnitude. To achieve this difference using tailings usually requires that the sands and slimes be separated from the whole tailings by cycloning.

The use of slimes and sands for control of the phreatic surface is analogous to the function of cores and drains, as shown by examples in Figure 7.4 for various embankment types. For upstream embankments, sands are usually cycloned on the embankment crest, and slimes are discharged through

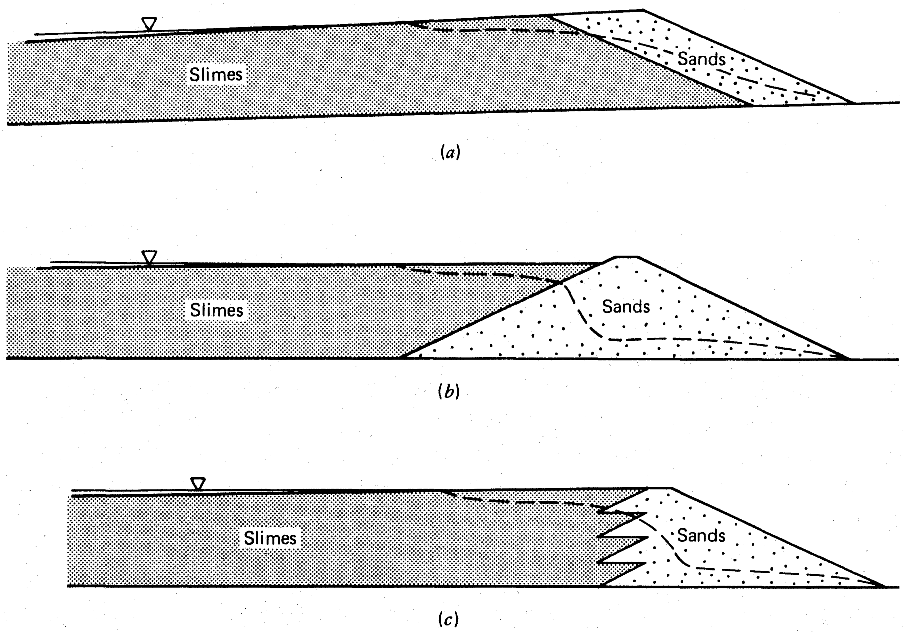


Figure 7.4 Use of tailings for internal drainage. (*a*) Upstream embankment. (*b*) Downstream embankment. (*c*) Centerline embankment.

pipes extending farther out onto the beach to form two separate permeability zones, albeit with an irregular contact. Similar procedures result in phreatic surface control for downstream and centerline dams as shown in Figures 7.4b and 7.4c. Soderberg and Busch (1977) and Coates and Yu (1977) summarize practical aspects of cyclone setup and arrangement for the various embankment types shown in Figure 7.4.

Central to the success of using cycloned tailings zones to control the phreatic surface is that runoff under both normal and flood conditions be minimal. Storage of any significant quantity of water is precluded (unless a core is present), and a wide beach is necessary at all times. Should large water inflows cause the decant pond to come into direct contact with the pervious sand zone for any of the embankments shown in Figure 7.4, failure could quickly result.

FILTER REQUIREMENTS

The arrangement and types of materials within the embankment are governed not only by control of the phreatic surface but also by filter requirements to prevent migration of soil or tailings into adjacent coarser fill zones. Filter requirements to prevent piping are well established from conventional water dam design practice and are summarized briefly below (Cedergren, 1967):

$$\frac{d_{15} \text{ of filter}}{d_{85} \text{ of protected soil}} < 5$$

$$\frac{d_{50} \text{ of filter}}{d_{50} \text{ of protected soil}} < 25$$

Problems related to piping and improper filter zones in tailings embankments are most often encountered where coarse mine waste is used as a construction material directly in contact with the tailings, especially where end-dumping of the mine waste in high lifts has produced severe size segregation. Klohn (1980) describes two piping-related failures of tailings dams and remedial measures used for their repair. Although obtaining properly graded filter materials is often difficult in the absence of expensive processing, they may be essential to the proper functioning of the structure.

The use of synthetic filter fabrics to replace conventional graded sand filters may be particularly attractive where natural materials are difficult or expensive to obtain. Available in either woven or nonwoven varieties, the longevity of synthetic filter fabrics is usually considered adequate for the relatively short active life of many tailings embankments. Koerner and Welsh (1980) note that the use of filter fabric to protect chimney and blanket drains of conventional water dams was pioneered in Europe in the early

1970s. More recent filter fabric applications have increasingly involved tailings embankments. Haas (1982), Bentel et al. (1982) and Scheurnberg (1982) provide examples of successful filter fabric use in tailings embankments but caution that the fabric may become clogged by precipitates for some effluent types.

Considerable care with regard to filters is required during impoundment operation. For example, filters to protect blanket drains for upstream embankments, such as that depicted in Figure 7.3a, have often been designed on the assumption that cycloned sand tailings will be the overlying material. In more than one case, improper cycloning or spigotting has allowed slimes rather than sand, as intended, to come into direct contact with the filter, plugging it and the drain and rendering both useless.

In conventional earth dams constructed for water storage, filter zones of cohesionless sand and gravel are sometimes constructed immediately upstream from an internal core in cases where cracking of the core is of concern. This may be due to brittleness of the core material, high anticipated differential settlement, or seismic forces. These upstream filter zones are intended to provide an automatic crack-plugging mechanism whereby the cohesionless filter material would be carried by seepage forces into any open core cracks that might develop. In the case of upstream-core tailings embankments, the tailings themselves may provide a degree of crack-plugging protection similar to that provided by upstream filter zones, particularly if the tailings in contact with the core are coarse, sand-sized materials. Because of their relatively uniform and relatively fine grain size, however, tailings cannot be relied upon to plug large cracks, and sand and gravel upstream filters should still be provided if major core cracking is anticipated.

MATERIALS SELECTION

The governing factor in selection of materials for use in embankment construction is control of the phreatic surface. Additionally, the need for filter zones follows from the gradations of the basic fill materials, both with respect to each other and relative to the impounded tailings. Materials selection is critical to the success of the design, from both safety and economic standpoints. Three basic material options are available: natural soils, mine waste, and cycloned tailings.

Natural Soils

The best use of readily available natural soils to meet the design objectives of phreatic surface control often requires a degree of creativity and innovative thought on the part of the designer. Unlike dams constructed by government agencies for water-retention purposes, tailings dams are subject to rigid economic constraints defined in the context of the mining project as a whole. While water-retention dams produce economic benefits that presumably outweigh their cost, tailings dams are economic liabilities to the mining operation from start to finish. As a result, it is not often economically feasible to go to the lengths sometimes taken to obtain fill for conventional water dams, such as quarrying rockfill or importing great quantities of fill from distant borrow pits. Economic necessity dictates that the design make maximum use of those natural soils that are readily available in the immediate site vicinity. In fact, wherever possible, obtaining borrow from within the impoundment area itself is preferred, since impoundment capacity is thereby increased and disturbance due to off-site borrow areas is minimized. Where natural soils are available in suitable types and sufficient quantity, it may be desirable to use them either for construction of the entire embankment or in combination with mine waste or cycloned tailings. As a minimum, however, enough suitable natural material must be present for construction of the initial starter dike.

The design of internal embankment zones will be dictated to a large extent by the types and variety of natural soils present on or near the impoundment site. Obviously, if clays are plentiful, control of the phreatic surface using internal cores will be strongly considered. For example, Brawner (1979) illustrates the common design practice in the Missouri lead district of using readily available clays as cores in downstream-type embankments constructed using cycloned tailings sand. Conversely, easily obtained sands and gravels will argue for phreatic surface control by internal drainage zones. Klohn and Maartman (1973) describe examples of cycloned sand dams in British Columbia drained by extensive finger and blanket drain systems.

A particularly difficult design situation presents itself where, for various reasons, natural soils are the only feasible material for embankment construction and where these materials are neither fine enough to provide ideal core zones nor clean enough to provide ideal drainage materials. An example might be where only silty sands are available on or near the site. Here it is necessary to recognize that cores need not necessarily be completely impervious, nor must the permeability of drainage zones necessarily approach infinity. It is required only that there be a sufficient difference in relative permeability between the various internal zones to control the phreatic surface and to prevent breakout of seepage on the embankment face. Less than ideal materials can sometimes be used as cores if their permeability can be reduced sufficiently by compaction at high moisture contents or to high densities. The former case will require that possible reductions in strength be accounted for, and the latter condition may make it necessary to provide design features to protect against cracking of the possibly brittle, overcompacted material. On the other hand, marginal materials are sometimes used as drainage zones by compensating for their lower-than-desired permeability by increasing the thickness of the zone. Careful permeability measurements on the material are required, and thorough analysis of flow capacity is necessary by procedures such as those described by Cedergren (1967). Another possible design solution would be to import or process material for high-quality chimney–blanket drains according to the zoning shown in Figures 7.3b and 7.3c. This would allow maximum use of marginal available soils as a random zone in the downstream shell of the embankment protected from saturation by the internal drainage system.

Another limitation to the use of natural soils is moisture content. When available natural soils are at or near saturation because of high groundwater levels, material handling may be difficult, and compaction to densities required for adequate strength may be impossible. While drying of borrow soils to reduce moisture content may be considered, this is a costly proposition and one that makes it difficult to ensure that adequate quantities of dry fill will be available as demanded by the embankment raising schedule.

Fortunately, the use of natural soils is usually mandatory only for the embankment starter dike. Where the site conditions impose inherent and severe limitations on the types of soils, available quantities, or moisture content, raised embankment design using either mine waste or cycloned tailings may present other possibilities.

Mine Waste

As previously discussed in Chapter 3, the use of open-pit mine waste can provide an attractive alternative to use of natural soils where their availability is limited. Even as a substitute for readily available soils, mine waste may present a cost advantage if haul distances from the pit to the waste dump and to the embankment are essentially similar. In this case, mine waste material provided for embankment construction is essentially "free," since it must be disposed of in any case. The schedule of waste production by the mine must, however, be compatible with fill quantities required by the embankment raising schedule, as illustrated in Figure 3.5.

In the initial stages of open-pit mining, overburden prestripping may be conducted while the mill is still under construction. Stripping wastes will often consist primarily of soil or weathered rock, which may be useful for construction of the embankment starter dike prior to mill startup. In general, however, pit mine waste during later stages consists of relatively coarse shot rock. This material is most often useful as the pervious or random zones for centerline or downstream raised embankments. A core of some type is often necessary to control seepage through the highly pervious rockfill.

Any use of mine waste in embankment construction requires careful attention to filter design. Because of the operating characteristics and size of large, high-capacity hauling units used for transportation of mine waste, mine waste is usually end-dumped in lifts on the embankment varying from 10 to 50 ft in thickness. Not only does this make controlled compaction difficult, but it also promotes segregation of the fill. Larger boulders roll to the bottom of the end-dumped lifts, with finer fragments remaining near the top. Without filters designed with due consideration for particle-size segregation, piping—a major cause of failure of tailings embankments constructed from mine waste—can result.

Unlike specially quarried materials for conventional rockfill dams, the properties of mine waste are dictated by the nature of the mine rock rather than by the requirements of the dam design. Mine waste may vary from shale to hard, igneous rock. The grading, fines content, and compaction of the material are difficult or impossible to control. Under these circumstances, determining mine waste properties by large-scale laboratory tests is usually difficult to justify. The engineering properties of mine waste can, however, be estimated by making conservative comparisons with quarried rockfill of similar mineralogical character using such data as that presented by Marsal (1973), Marachi et al. (1972), and Donaghe and Cohen (1978). The properties of shale wastes from coal mines are summarized by Busch et al. (1974).

Cycloned Tailings

Like mine waste, the use of cycloned tailings in embankment design presents an attractive design option when natural soils of suitable type or quantity are unavailable. Sand tailings can be produced by cycloning at small cost, usually less than about \$0.05–0.15 per ton. A significant cost advantage can arise from substituting cycloned sands for natural soils by virtue of the fact that the sands are produced on or very near the embankment itself, substantially reducing or eliminating fill hauling costs. Moreover, the characteristics of the material produced can be adjusted to fit the design requirements. Reasonably clean sand tailings can be produced from most mill tailings having less than about 60% passing the No. 200 sieve, provided the tailings are essentially nonplastic and free of clay minerals. The resulting cycloned sand has high effective strength and permeability, making it an ideal material for providing drainage and phreatic surface reduction in downstream zones of the embankment.

Cycloning has other sometimes overlooked advantages. By removing sand from the whole tailings feed for use in embankment construction, the remaining volume of tailings discharged to the impoundment is reduced, typically by 10–30% depending on impoundment configuration and embankment type. With reduced impoundment storage requirements, the height of the embankment, its volume, and its cost are lower.

Tailings discharged to the impoundment after removal of the cycloned sands are finer than the whole mill tailings that would otherwise have been deposited and typically have an overall permeability about one to two orders of magnitude less. In certain cases, particularly when the impoundment is underlain by highly pervious soils and seepage is governed by vertical flow through the tailings deposit, the effect of cycloning can be to considerably reduce impoundment seepage losses by virtue of the lower permeability of the tailings in the impoundment.

Cycloning Principles

Cyclones are simple devices that function on a centrifugal separation principle with no moving parts. As illustrated in Figure 7.5, whole tailings feed slurry under pressure enters a cylindrical feed chamber. Coarser particles in the slurry spiral downward through the conical apex at the bottom, giving rise to the term *underflow* to refer to the coarse separated sand fraction. The finer fraction and most of the slurry water rise to the outlet as *overflow*, principally slimes. The performance of cyclones is a function of the size and design of the device, and is influenced by such operating parameters as pressure, solids concentration (pulp density), and specific gravity of the tailings feed slurry. Performance characteristics are influenced by complex interactions between these variables and are best estimated by cyclone manufacturers (Arterburn, 1976).

The use of cycloned tailings in embankment design requires that criteria for the cycloned sand be established. These criteria are based primarily on two factors: desired permeability and drainage–handling characteristics. In many cases, permeability considerations limit the percent fines of the underflow sand to about 5–12%, although Mittal and Morgenstern (1977) suggest that an adequate difference in permeability between the separated sands and slimes can be maintained with as much as 20–25% allowable fines content in the sand underflow. Even when permeability is not the governing factor, it is desirable to minimize fines content within practical limits in order to produce a sand underflow product that will drain quickly upon discharge from the cyclone, simplifying handling and spreading of the material.

On the other hand, the permeability of the underflow sand and the percent sand recovery from the whole tailings feed are inversely related: the cleaner the sand product, the smaller the volume of sand produced. Sand recovery is very sensitive to small differences in percent fines in the underflow, and obtaining maximum recovery becomes important when large quantities of sand are required to meet the embankment raising schedule. This factor may mitigate against overly rigid restrictions on fines content.

Typical grain-size separations for a relatively coarse mill tailings feed with about 40% passing the 200 sieve are shown in Figure 7.6. Lower capacity 10-in. cyclone units provide 67% recovery of sand underflow having 16% fines. Higher capacity 26-in. cyclones would reduce the fines content to 13%, but at the cost of reducing recovery to 48% of the total tailings feed by weight. Even further reductions in underflow percent fines can be achieved, but this requires two-stage cycloning. Here the underflow from the first cyclone is diluted and introduced as feed to the second cyclone. Material with about 3% fines is produced, but sand recovery is reduced to 37%.

Figure 7.7 shows a similar example but with a finer whole tailings feed initially having almost 60% passing the 200 sieve. Because of the finer feed gradation, two-stage cycloning is required to produce an underflow sand with 11% fines and 35% recovery. Typically, single-stage cycloning reduces

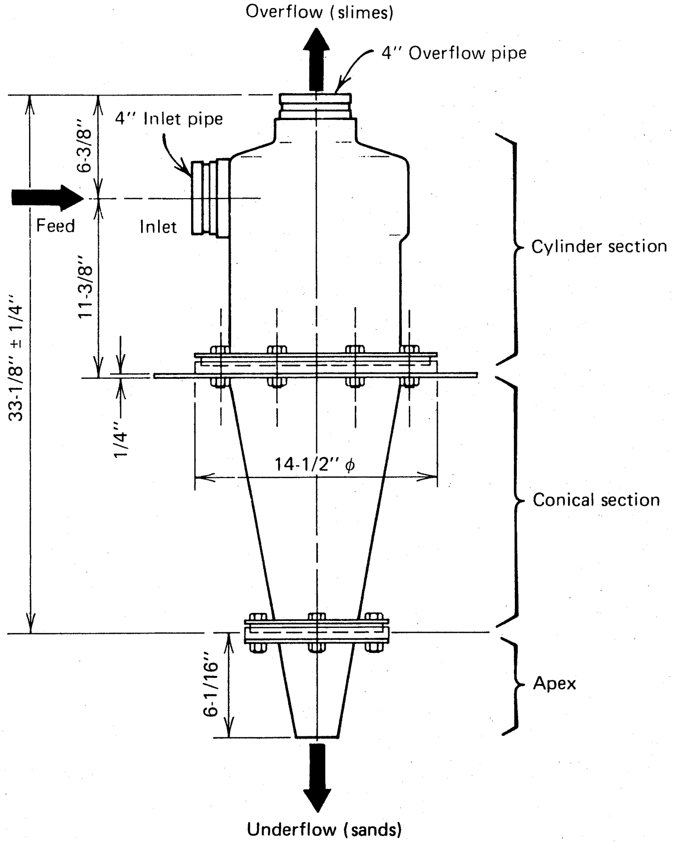


Figure 7.5 Typical cyclone configuration. (Courtesy Krebs Engineers.)

the minus 200 fraction of the whole tailings feed by about 30–40%. Significant further reductions in underflow fines content usually require multiple-stage cycloning.

Sand yield is sensitive to even small changes in the gradation of the feed tailings. Comparison of data in Figures 7.6 and 7.7 shows that, for underflow having essentially the same fines content, increasing the percent fines in the feed tailings from 40% to 60% reduces the sand yield by one-fourth, from 48% to 35%.

Cycloning Methods

Cycloning is adapted to embankment raising according to one of three general methods:

Stationary cyclone plant.
On-dam cycloning.
Hydraulic cell method.

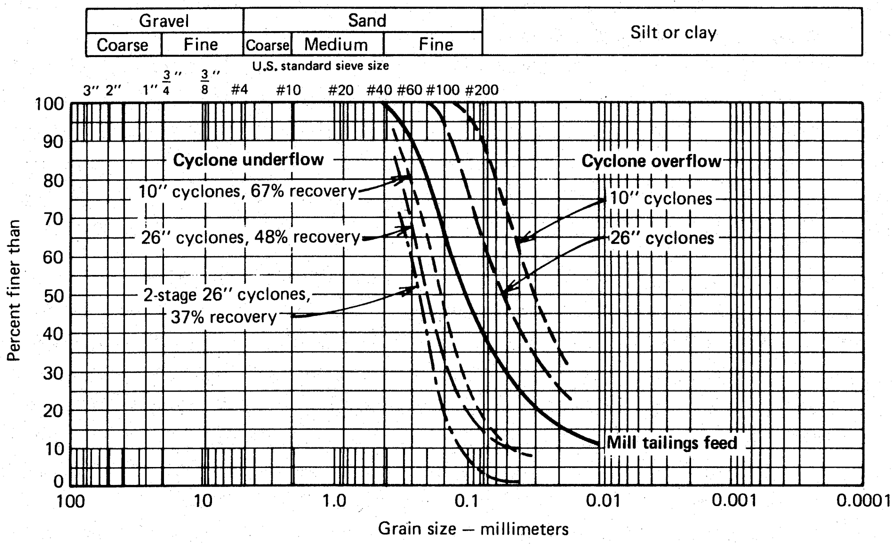


Figure 7.6 Cyclone split and recovery, coarse tailings feed.

Stationary Cycloning. For the stationary cycloning method, sands and slimes are separated before they reach the embankment. A single, high-capacity cyclone station is established, often near an abutment of the embankment, and the underflow is often allowed to form a large stockpile. Cycloned sands are placed and compacted in the embankment by conventional earth-moving equipment, and overflow is discharged to the impoundment through a separate slimes line. Stationary cycloning methods are well

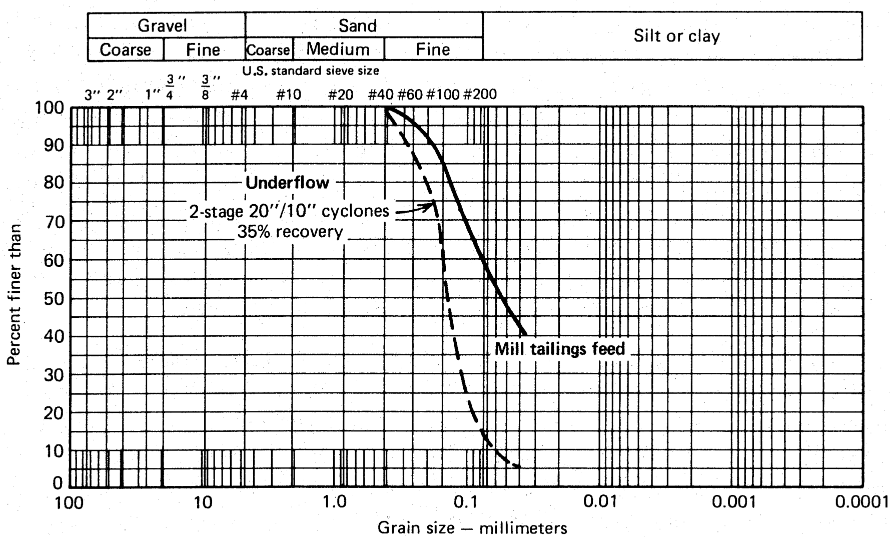


Figure 7.7 Cyclone split and recovery, fine tailings feed.

suited to sites where high seismicity requires mechanical compaction of cycloned sands to high relative densities.

Where compaction of the materials is not required, stationary-cycloned sands may be spigotted directly from the embankment crest without stockpiling. Brawner (1979) describes the use of stationary cycloning and spigotting of cycloned sands for an upstream-type molybdenum tailings embankment in Colorado. In this case, spigotting of the cycloned sands ensures high permeability near the embankment face and good phreatic surface control.

On-dam Cycloning. On-dam cycloning is probably the most prevalent cycloning method. Ordinarily, many small cyclones are set up on the embankment crest on small towers or scaffolds. Underflow sand from each cyclone is discharged toward the embankment face, and overflow slimes are discharged into the impoundment. The cyclones are periodically raised as the embankment height increases. The use of on-dam cycloning in conventional form is described by Klohn and Maartman (1973) for a centerline embankment in British Columbia; by Dopson and McGregor (1973) for an upstream embankment in Arizona; by Brawner (1979) for a downstream embankment in Missouri; and by Watermeyer and Williamson (1979) for a composite embankment using exceptionally fine underflow. An adaptation of on-dam cycloning described by Sandic (1979) and Girucky (1973) involves the use of a mobile high-capacity cycloning unit that travels back and forth along the embankment crest, eliminating the labor-intensive relocation of numerous small cyclone units.

Ordinarily, cyclone sand underflow from on-dam cycloning methods is allowed to flow as a thick, ropy discharge (about 70–75% solids) directly onto the embankment. At this pulp density, the sands assume an angle of repose of about 3:1 to 4:1. A major advantage of on-dam cycloning is that the sands are discharged from the cyclone at what is more or less their final resting place in the embankment. Since the sand is not handled mechanically, considerable cost savings are achieved. However, compaction of the sands produced by on-dam cycloning is usually difficult or impossible because they are not spread in layers of controlled thickness. Mittal and Morgenstern (1977) indicate that on-dam cycloned sands achieve average relative densities of about 45–55% without mechanical compaction. Sandic (1979) reports relative densities of about 30%. These relative densities may be acceptable for well-drained embankments in areas of moderate seismicity but may preclude on-dam cycloning in areas of high seismicity where high relative densities are required to prevent seismic liquefaction.

Hydraulic Cell Method. The final cycloning procedure, the hydraulic cell method, is similar to stationary cycloning to the extent that cycloning is performed at a single large installation. Sand underflow is diluted and pumped to the embankment, where it is discharged into small ponds or cells constructed on the embankment itself where the clean sands settle from suspension. Excess water is then decanted from the cells or allowed to percolate downward through the fill. During placement of sands in the cells, compaction can be provided with wide-tracked dozers. Mittal and Hardy (1977) report relative densities in excess of 75% by this procedure, but relative densities generally in excess of 50% may result even without mechanical compaction (Mittal and Morgenstern, 1977).

Hydraulic cell cycloning is described by Klohn and Maartman (1973) for copper tailings and by Mittal and Hardy (1977) for oil sands. Nyren et al. (1979) report the planned use of the method for other oil sands projects, and Brawner (1979) describes use of hydraulic cell cycloning in Peru.

The hydraulic cell method has the major advantage of allowing for compaction to high relative densities while avoiding mechanical handling and placement of the cycloned sands. However, the method requires a relatively wide, flat area on the embankment for cell layout. This may be a disadvantage if the width of incremental embankment raises is small. Also, for the hydraulic cell method, it is often necessary to limit the fines in the cyclone underflow to about 5–10% to achieve good drainage of discharged cell water. Corresponding reductions in the overall percent sand recovery may restrict the amount of sand available.

Compaction

Compaction of cycloned sand is a significant issue in embankment design, and, as explained above, the various cycloning methods vary in their ability to accommodate compaction procedures. Compaction of embankment sands is often desirable to reduce pore pressure buildup during shear and the possibility of flow slides, as discussed further in Chapter 8. But compaction becomes of critical design importance in determining the susceptibility of saturated embankment sands to seismic liquefaction.

The site seismicity, internal drainage provisions in the embankment design, and compaction requirements are all interrelated. Liquefaction is most likely to occur for loose, saturated sands under high levels of seismic shaking. The main design measures against liquefaction are good internal drainage to prevent saturation, and densification of the cycloned tailings sands by compaction. Klohn (1980) notes that, in the extreme, these design measures tend to be mutually exclusive: when the sand is unsaturated, liquefaction is unlikely regardless of density, while if the sand is sufficiently dense, liquefaction is improbable even under complete saturation. Since compaction usually requires mechanical hauling and placement at considerable expense, it is often less costly in areas of lower seismicity to provide for thorough internal drainage rather than high densities—provided, of course, that drainage sands and gravels are available at moderate expense. In areas of high seismicity, Klohn (1980) recommends both compaction to high densities and good internal drainage.

Where high seismicity indicates the need for compaction, a remaining design consideration is the relative density requirement. Mittal and

Morgenstern (1977) review various liquefaction-related relative density criteria and report that compaction to relative densities of 50–60% is appropriate for areas of moderate seismicity with expected accelerations up to 0.10 g. Higher relative densities, in the range of 75%, may be necessary for higher seismicity. Usually, reasonably high relative densities can be obtained for clean sand tailings with only modest compactive effort. The major cost associated with compaction results from the need to haul, spread, and place the sands in thin lifts rather than from the compaction operation itself. Use of the hydraulic cell method, however, allows for compaction without mechanical fill handling and at relatively little cost.

INFLUENCE OF FOUNDATION CONDITIONS

Although much of the preceding discussion of embankment design has centered around fill materials and internal zoning of the embankment, the design must also account for and be compatible with foundation conditions. The evaluation and analysis of tailings embankment foundations follows the same geotechnical principles applied in conventional water dam technology. The application of these principles, however, must sometimes be modified to account for unique features of tailings embankments.

Strength

Foundations composed of weak materials, such as peat or normally consolidated clay, present an obvious problem. Classic design measures for weak foundations include flattening embankment slopes to an inclination consistent with the initial undrained shear strength of the foundation material or excavating and removing low-strength soils prior to embankment construction.

Raised tailings embankments may provide yet another design option. Since the embankment is increased to its ultimate height usually over a period of many years, significant pore pressure dissipation and corresponding increase in undrained shear strength of the foundation materials will usually occur during the embankment life. By thorough laboratory and theoretical analysis of pore pressure dissipation and strength gain characteristics, it may be possible to demonstrate the adequacy of foundation strength increase over the embankment life. Taylor and D'Appolonia (1977) describe an analysis of this type for a taconite tailings embankment on a weak peat foundation. If embankment buildup rates are too rapid to allow for the necessary degree of pore pressure dissipation and strength gain, it may be possible to correct the problem by modifying the impoundment layout. A larger impoundment area could allow for retention of the same total tailings volume but with a lower ultimate embankment raised at a slower rate.

Compressibility

Foundation compressibility may cause considerable embankment settlement. The resulting reduction in crest elevation is not normally a problem for raised embankments, since settlement compensation can be made with successive raises. But cracking of fill in response to severe settlement may be damaging, particularly to embankment designs incorporating cores or impervious zones of cohesive natural soils. Another major design problem is associated with decant conduits. Decant conduits passing under the embankment can be severely damaged by excessive foundation settlement, and collapse of the decant pipe can cause embankment failure. As discussed in Chapter 1, other options for decant systems are available and usually preferable.

In arid climates, collapsible foundation soils are fairly common. Foundation settlement upon saturation can be severe, resulting in cracking and piping failure of the embankment. Nelson and Kane (1980) describe one such failure. In anticipation of potential embankment cracking in response to collapse of foundation soils, the design incorporated a spigotted tailings beach along the upstream face of the embankment. The purpose of the beach was to provide a supply of tailings sand to plug any cracks developing in the embankment, and postfailure investigations revealed that the sand functioned in the intended manner. But during operation, the pond water level rose, submerging the beach and allowing water to contact the embankment directly. Piping failure through cracks unprotected by the tailings resulted.

A related problem that applies to clayey fill or foundation soils is dispersive piping (Sherard et al., 1972). Laboratory procedures for evaluating dispersive piping potential ordinarily use the pinhole test with distilled water (Sherard et al., 1976). For tailings embankments, however, the pinhole test sometimes fails to predict dispersive piping accurately unless tailings effluent having representative pH and chemical composition is used. Nelson and Kane (1980) note that in one case nondispersive conditions were indicated for an acid tailings effluent buffered to pH values of 2, 4, and 7. Only from testing performed at the actual effluent pH of 1.2 was the potential for dispersive piping correctly indicated.

DESIGN IN COLD REGIONS

Many of the world's richest mineral deposits are located in arctic or high alpine areas where permafrost conditions are encountered either locally or regionally. The thermal regime of the ground will be disrupted by the presence of a tailings impoundment constructed upon it, and embankment design must account for settlement, strength, seepage, and piping-related problems that can develop in response to thawing or freezing of the embankment or its foundation. The thermal regime may either reequilibrate under the influence of the impoundment or, once disturbed, continue in a changing and unstable state over the life of the facility and beyond. The effect of these changes may be either to lower the permafrost table, resulting in foundation thawing, or to raise it, producing frozen conditions within regions of the embankment (Coates and Yu, 1977). Although thermal profiles can be predicted as a function of time using either closed-form or numerical solutions, these analyses are highly complex and exceed the scope of conventional soil mechanics (Andersland and Anderson, 1978).

Biyanov (1976) and Sheynfeldt (1977) discuss the peculiarities of tailings embankment design and construction in permafrost regions. In ice-rich permafrost, severe settlement and gross subsidence can result from foundation thawing. Ice-rich permafrost beneath starter dikes can be excavated and removed, usually at great difficulty and expense, using prethawing if sufficient time is available. Otherwise the foundation may be allowed to thaw during impoundment operation, with due consideration for reduction in foundation strength and settlement over time. Such measures as vertical sand drains may be necessary to accelerate consolidation of soils containing heavy concentrations of ice lenses that would otherwise produce excessively wet and soft foundation conditions upon thawing.

Control of foundation underseepage by cutoffs or other means and proper protection of embankment soils by filters is especially important for dams constructed on permafrost, in order to avoid high pore pressures or piping in the thawing foundation. Biyanov (1976) describes seepage and piping problems in a tailings dam foundation in the Soviet Union that were partly solved by spigotting tailings over portions of the impoundment bottom where foundation seepage was believed to originate.

Permafrost conditions affect not only the foundation but also the design of the embankment. Biyanov (1976) reports that in especially severe climates, starter dike fill freezes to depths up to 12–15 ft below the face, rendering the fill impervious and making it impossible for the starter dike to function as a pervious drainage zone for upstream-type embankments. Drainage zones must be located within the central portion of the embankment, where freezing is unlikely, and collected drain seepage must be conveyed through frozen portions of the embankment in heated pipes or drainage galleries. Alternatively, depending on the thermal regime within the embankment, it may also be possible to locate drains in the embankment and foundation below the zone of seasonal freezing (Coates and Yu, 1977).

Restrictions on operating procedures in severely cold climates may also impose indirect constraints on embankment design. According to Coates and Yu (1977), practice in Canada is to discontinue peripheral spigotting during freezing months, with winter tailings discharge into the rear portions of the impoundment. Limiting beach deposition to summer months in this way may make upstream embankments impractical to construct, although upstream techniques are used in some cases in both northern Canada and Siberia. Similarly, embankment construction using cycloned tailings must often be curtailed during winter months to avoid freezing of underflow water and resulting formation of ice lenses within the deposited sands.

A related consideration is that the depth of the decant pond must be increased in cold regions to provide sufficient storage for winter ice formation while at the same time maintaining sufficient additional unfrozen depth in the pond necessary for clarification and sedimentation. This increased decant pond depth may restrict the embankment type and design to those methods compatible with considerable water-storage requirements.

For sloping sites, it is also necessary to account for creep of frozen foundation material in permafrost regions. For example, Brawner (1979) cites a tungsten tailings dam site in Canada's Northwest Territories that exhibited several classic creep features. In high alpine areas, tailings embankments have been constructed on or near rock glaciers, lobate features of talus-like rock fragments containing interstitial ice. In such terrain, it may be necessary to address the degree of possible movement of the rock glacier—for example, by methods presented by Wahrhaftig and Cox (1959)—and to account for the effects of possible movement on the structure. Cracking resistance becomes of primary importance in the design if creep movements are anticipated, including the use of well-graded materials for cores, upstream crack-plugging filters, and extensive zones of crack-resistant soil or rockfill in the downstream portion of the embankment.

SUMMARY

In a strictly defined sense, *design* of tailings embankments can be used to refer to the process of determining the internal configuration of materials within the embankment. Of primary importance are two factors: arrangement of fill zones to achieve control of the phreatic surface, and maximum use of readily available materials.

Phreatic surface control can be achieved by providing either cores to limit seepage or drainage zones to collect it, the method of choice depending on available materials. In either case, permeability of the various zones must generally increase in the direction of seepage flow, resulting in lower-permeability zones in the upstream portion of the embankment and higher-permeability zones in the downstream portion.

Internal zoning of the embankment and the means of phreatic surface control depend on the range of materials available on or near the impoundment site. For predominantly clayey natural soils, cores are easily provided, whereas readily available sands and gravels suggest greater emphasis on internal drainage zones. The use of mine waste or cycloned sand tailings as a substitute for natural soils can be economically attractive in some cases and essential in others where suitable natural soils are not available. Proper filters are necessary to prevent migration of finer soils or tailings into zones of coarser material.

In essence, then, the process of design involves achieving a suitable internal arrangement of embankment soils using the materials at hand, whether these are produced by nature or by the mining operation. Because materials vary from site to site and from mine to mine, each case requires a unique design solution. Not only must fill materials be considered, but design of the embankment must also be compatible with its foundation conditions.

Even after determining internal embankment design on a conceptual basis, a great deal of analytical work remains to confirm the adequacy of the design on a detailed level. These analytical considerations in the assessment of embankment stability will be the topics of Chapters 8 and 9.

8

Stability Analysis of Tailings Embankments

So we embellish our professional activities with magic, disguised in one way or another as rational.

Richard Meehan, *Getting Sued and Other Tales of the Engineering Life*

Much of Chapter 7 deals with design of tailings embankments in the context of determining materials, internal zoning, and phreatic surface control methods. The result of this design process is to establish a general embankment cross-sectional arrangement. This arrangement, however, can only be considered a tentative or trial configuration until appropriate analysis has established the stability of the embankment slopes under a variety of loading conditions.

This chapter treats the application of classic geotechnical principles of slope stability analysis to tailings embankments under static loading conditions. The end result of the stability analyses will be either to confirm the stability of a given trial embankment internal zoning and slope inclination or to indicate the need for redesign in a different trial configuration. Stability conditions discussed in this chapter refer to static loading conditions only. Analysis of embankment stability under seismic loading conditions is reserved for Chapter 9.

Considerable discussion in Chapter 7 has been devoted to phreatic surface control in design, and this topic must be developed still further in the context of stability analysis. The first portion of this chapter deals with procedures for estimating phreatic surface location within various types of tailings embankments for use as input to stability analysis. Subsequent discussions center around the application and interpretation of classical analytical techniques for determination of tailings embankment stability.

PHREATIC SURFACE DETERMINATION

Seepage conditions within the embankment exert a controlling influence on its stability, and a primary purpose of seepage evaluation is to assess pore pressures for input to stability analyses. Seepage in tailings embankments is commonly assumed to occur under gravity flow and, for purposes of pore pressure evaluation, is usually determined for steady-state conditions.

It is important to note that the Darcy assumptions of steady seepage conditions and gravity flow are useful in the context of stability analysis because they usually yield conservative estimates of pore pressures. Seepage evaluation for purposes of determining impoundment seepage loss, on the other hand, is quite another matter. Here it may be necessary to assess nonsteady, transient, or unsaturated flow that occurs under capillary rather than gravity gradients. Seepage evaluation for the purpose of determining impoundment seepage loss is a completely separate topic, which is treated in detail in Chapter 10.

Given the assumptions of steady gravity seepage flow, determining the location of the phreatic surface (or top flow line) within the embankment yields the pore pressure distribution necessary for stability analysis input. The most common procedure for determining phreatic surface location is by flow net analyses that follow from well-established procedures in geotechnical engineering. Cedergren (1967) provides a fundamental and comprehensive treatment of flow net construction for conventional water-retention dams. These basic flow net principles derived from Darcy's Law and the Laplace equation can be applied in a fairly routine way to conventional water-retention dams, and their extension to tailings embankments is not difficult if differences in boundary conditions are taken into account.

Kealy and Busch (1971) discuss differences in boundary conditions for conventional water dams and upstream-type tailings embankments. Figure 8.1a shows that, for a conventional water dam, the equipotential line for the initial seepage entry surface approaches a vertical orientation corresponding to the upstream dam slope. (While this is obviously an exaggeration, a vertical approximation for the upstream slope is conceptually reasonable for flow nets drawn to a transformed scale to account for anisotropy.) The corresponding flow lines are essentially horizontal.

Boundary flow conditions for an upstream-type tailings embankment are shown for comparison in Figure 8.1b. Here, the initial entry equipotential line is essentially horizontal, corresponding to the flat bottom of the decant pond. As a result, flow lines are initially directed vertically downward. Where a slimes zone is present, considerable headloss occurs within these low-permeability tailings. As seepage flow becomes directed horizontally outward toward the embankment face, the resulting phreatic surface may be considerably lower than that which would result from conventional water dam boundary conditions, as the schematic comparison in Figure 8.1b shows.

The application of simple flow net construction procedures is illustrated in Figure 8.2. The embankments are assumed to be homogeneous and anisotropic, with $k_h/k_v = 9$. A transformation factor of 3 results. Figure 8.2a shows a flow net drawn to the transformed scale for a conventional water

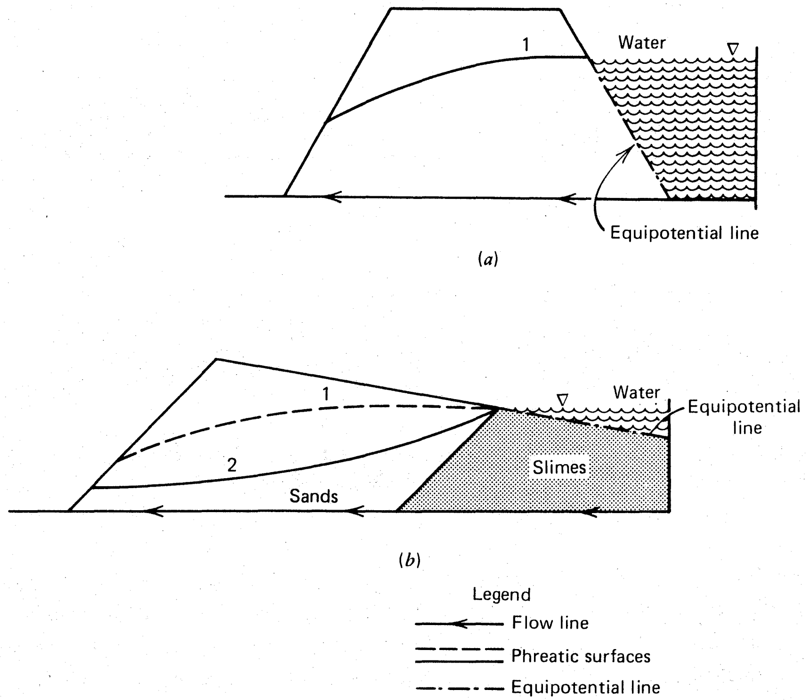


Figure 8.1 Comparison of boundary flow conditions for conventional water dams and upstream tailings embankments. (Reprinted from Kealy and Busch, 1971.) (*a*) Conventional water dam. (*b*) Upstream tailings embankment.

dam. The total head, H, is divided into equal increments of dimension Δh. The intersections of flow lines and equipotential lines occur at right angles and at elevation increments equal to Δh for the top flow line. The transformed section expanded to natural scale is also shown in Figure 8.2a. The top flow line on this natural section would ordinarily be used as phreatic surface input to stability analyses of the downstream dam slope.

Figure 8.2b shows flow nets for an upstream-type tailings embankment incorporating boundary conditions previously discussed. On the transformed section, intersections of equipotentials with the top flow line are at equal head increments, as before. Flow lines near the seepage entry surface, however, are initially vertical. The resulting top flow line on the expanded natural section would be used as stability analysis input.

For downstream- and centerline-type tailings embankments, internal zoning and boundary conditions are often sufficiently simple and similar to those for conventional water dams so that published water dam flow nets, such as those presented by Cedergren (1967, 1973), can be adapted to estimate the phreatic surface. Upstream-type embankments, however, cannot often be realistically modeled by such simple flow nets as those shown in Figure 8.2b

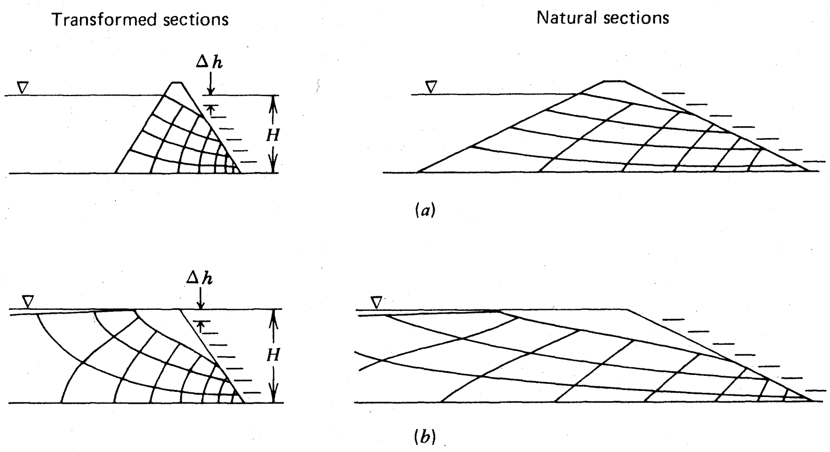


Figure 8.2 Typical flow nets, homogeneous, anisotropic embankments, $k_h/k_v = 9$. (*a*) Conventional water-retention dam. (*b*) Upstream-type tailings embankment.

that neglect the permeability variations produced by such factors as tailings grain-size segregation within the embankment. Since it is often too difficult from a practical standpoint to incorporate these factors in a flow net analysis, finite-element and related numerical methods have been used to determine the phreatic surface location for upstream-type tailings embankments, particularly those having complex permeability variations or boundary conditions (Kealy and Busch, 1971; Nelson et al., 1977; Vick, 1977).

As previously indicated, procedures for determining the phreatic surface location, whether by flow net or finite-element methods, assume that seepage flow within the embankment is governed by gravity gradients and that the source of seepage is the decant pond. These assumptions, however, require critical examination in some cases. Mittal and Morgenstern (1976) suggest that for slimes deposited at rapid rates (greater than about 15–30 ft/yr), seepage may be governed by gradients induced by consolidation of the slimes rather than those due to gravity flow. Here the distribution of pore pressures within the embankment must be determined from consolidation theory. In addition, for embankments constructed by on-dam or hydraulic cell cycloning, it is usually the cyclone underflow water, not dam through-seepage, that governs the phreatic surface location.

AVAILABLE PHREATIC SURFACE SOLUTIONS

A number of published seepage solutions are available either that apply specifically to tailings embankments or that can be adapted to tailings embankments in a relatively straightforward way. These solutions may be used to provide direct input into stability analysis in some cases, but in any event provide useful insight into the effects of anisotropy, nonhomogeneity, and boundary conditions for tailings embankments of various types.

Upstream Embankments

The determination of phreatic surface location for upstream-type embankments is more complex by far than for any other type of tailings-retention structure. Phreatic surface location is influenced by pond location, lateral and vertical permeability variations of the spigotted tailings, anisotropic permeability of the tailings deposit, boundary conditions, and other factors. Several parametric studies available in the literature are useful in addressing the significance of these variables taken individually (Kealy and Busch, 1971; Nelson et al., 1977; Vick, 1977; Abadjiev, 1976).

Beach Width

The location of the ponded water with respect to the embankment crest, or the width of the exposed tailings beach, is often the most important factor influencing phreatic surface location. Figure 8.3 illustrates approximate phreatic surface locations for a homogeneous, anisotropic embankment, assuming several values of beach width (measured from the embankment toe) normalized by embankment height. For the assumed conditions, the influence of beach width on phreatic surface location is dramatic, particularly within the region directly below the embankment slope where phreatic surface location is of greatest concern from a stability standpoint. For the particular conditions shown in Figure 8.3, a normalized beach width L/H much less than 9 might produce a troublesome phreatic surface location, and L/H less than about 5 would be undesirable.

Lateral Permeability Variation

Perhaps the second most important factor influencing phreatic surface location for upstream embankments is the degree of lateral permeability variation produced by grain-size segregation of the spigotted beach tailings. As

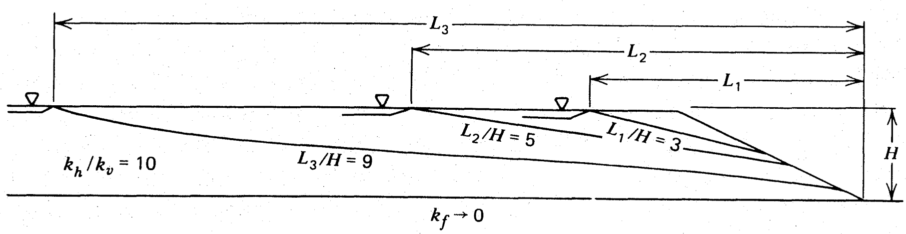


Figure 8.3 Influence of beach width on phreatic surface for homogeneous, anisotropic upstream embankment on an impermeable foundation.

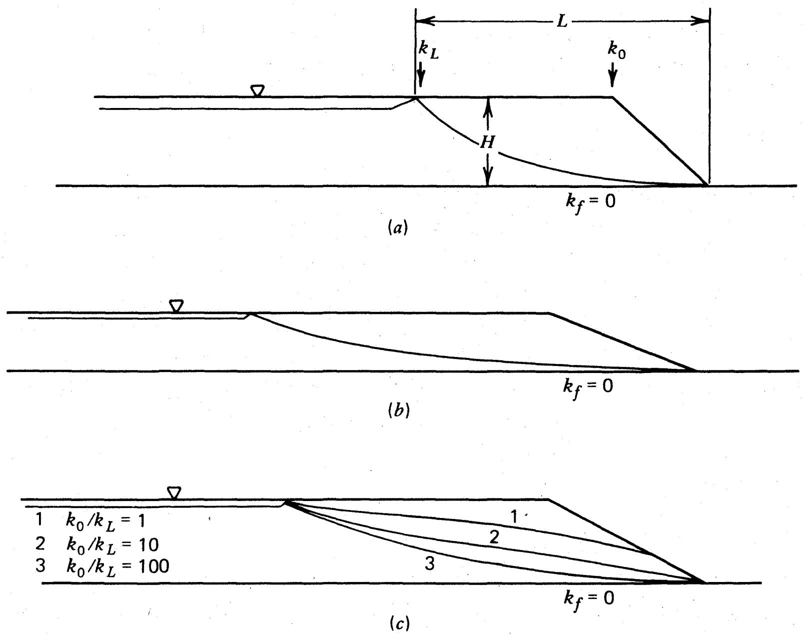
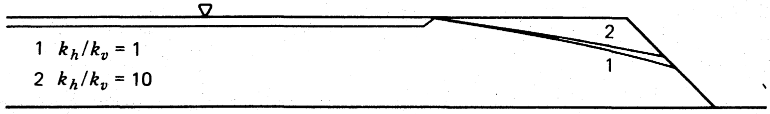


Figure 8.4 Influence of beach permeability variation for nonhomogeneous upstream embankments. (a) $k_0/k_L = 100$, $L/H \cong 3$, $k_h/k_v = 10$. (From Kealy and Busch, 1971.) (b) $k_0/k_L \cong 5$, $L/H \cong 7$, $k_h/k_v = 2.5$. (From Nelson et al., 1977.) (c) Variable k_0/k_L, $L/H \cong 5$, $k_h/k_v = 1$. (From Abadjiev, 1976.)

discussed in Chapter 2, the degree of permeability variation may range from significant to almost nil, depending on the gradation of the mill tailings, the pulp density of the discharge, and the care exercised in peripheral spigotting and pond water control.

Figure 8.4 shows the effect of lateral permeability variation for upstream embankments having beach width ratios ranging from 3 to 7 and anisotropy ratios (k_h/k_v) ranging from 1 to 10. The variation in permeability is characterized by the ratio of tailings permeability at the spigot point (k_0) to the permeability at the edge of the ponded water at the slimes zone (k_L). Figures 8.4a and 8.4b show that beach permeability variations can act in combination with beach width to produce a low phreatic surface within critical portions of the embankment. If the beach permeability variation is great ($k_0/k_L = 100$), even a narrow beach may produce acceptable phreatic conditions (Figure 8.4a). On the other hand, a lesser permeability variation may be adequate if combined with a wider beach (Figure 8.4b). Figure 8.4c shows that, for any intermediate beach width and isotropic tailings permeability, the degree of beach permeability variation can be of critical importance in controlling phreatic surface location.



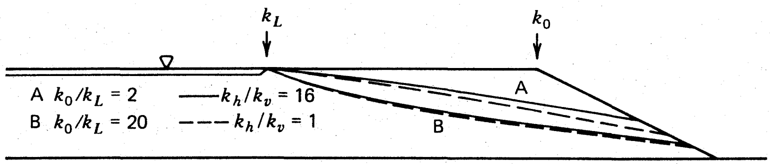


Figure 8.5 Effects of anisotropy for homogeneous and nonhomogeneous upstream embankments on impervious foundations. (*a*) Homogeneous embankment, *L/H* ≅ 3. (From Kealy and Busch, 1971.) (*b*) Nonhomogeneous embankment, *L/H* ≅ 3. (From Abadjiev, 1976.)

Anisotropy

The effects of anisotropic tailings permeability produced by moderate degrees of sand–slime interlayering are illustrated for a homogeneous embankment in Figure 8.5a and for an embankment with varying degrees of beach permeability variation in Figure 8.5b. The effects of anisotropy are to slightly elevate the phreatic surface for k_h/k_v ratios up to about 10–20. These effects are reduced for greater degrees of beach permeability variation, as shown in Figure 8.5b. In general, the influence of anisotropy on phreatic surface location for upstream embankments is minor compared to that of other variables. The fact that flow is essentially horizontal within outer portions of the embankment and therefore relatively insensitive to layering in the horizontal direction makes this observation intuitively satisfying. However, Nelson et al. (1977) note that, if the phreatic surface breaks out on or near the embankment face, even a small rise in the phreatic surface, say 10 ft, can result in "wet spots" migrating 30–40 ft up the embankment face for typical embankment slope inclinations. These zones of saturation on the face can cause sloughing problems and, as explained later, can impair overall embankment stability.

Boundary Conditions

Boundary flow conditions, particularly foundation permeability and starter dike permeability, exert an important influence on phreatic surface location. Figure 8.6a shows the effect of a foundation 10 times more permeable than the tailings for a homogeneous, anisotropic upstream embankment. Even for the narrow beach width shown, the phreatic surface is considerably lowered by the pervious foundation.

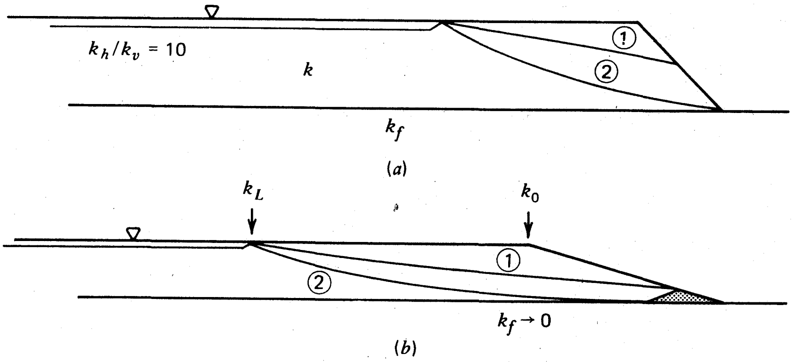


Figure 8.6 Effects of boundary conditions on phreatic surface for upstream embankments. (*a*) Effects of foundation permeability, homogeneous embankments, $L/H \cong 3$: (1) $k_f = 0$; (2) $k_f = 10k$. (From Kealy and Busch, 1971.) (*b*) Effects of starter dam permeability, nonhomogeneous embankment, $k_0/k_L \cong 5$, $L/H \cong 7$: (1) Impervious starter dam; (2) Pervious starter dam. (From Nelson et al., 1977.)

Figure 8.6b shows that the effects of starter dike permeability are pronounced. A starter dike that is only slightly less permeable than the beach tailings may act as a relatively impervious barrier, forcing the phreatic surface to rise over the starter dike crest to an exit point higher on the embankment face.

Other Factors

Several other variables may influence the location of the phreatic surface within upstream embankments. One such factor concerns the decrease in tailings permeability with depth that comes about because of a consolidation-induced decrease in void ratio. As noted in Chapter 2, this can account for a permeability decrease typically up to a factor of 5–10, depending on embankment height and type of tailings. For a moderate tailings permeability decrease with depth (up to a factor of 5) and a moderate degree of lateral beach permeability variation ($k_0/k_L = 20$), Abadjiev (1976) indicates that permeability decrease with depth can be accounted for by raising the phreatic surface by a distance equal to about 5–10% of the height of the embankment.

All foregoing discussions deal with the decant pond as the sole source of seepage water. An additional factor affecting the phreatic surface is infiltration of tailings slurry water on the beach itself during spigotting. The results of finite-element studies performed by Nelson et al. (1977) suggest that the effects of infiltration are relatively minor and produce a rise in the phreatic surface equivalent to about 2–4% of the embankment height near active spigot locations, predictions that are basically confirmed by field observations by Abadjiev (1976), who found phreatic surface elevation due to beach infiltration to be only a few percent.

In summary, the most important factors determining phreatic surface location for upstream embankments include beach width, permeability variations of the spigotted tailings, and boundary flow conditions. Beach width is an operational factor not amenable to control by the designer, and beach permeability variations cannot usually be enhanced to meet design requirements except by cycloning. The only factor available to the designer is in modification of boundary conditions by such vehicles as permeable starter dikes and underdrains. The fact that so many variables cannot be controlled or easily predicted in advance of operation cannot help but inspire a certain feeling of helplessness among those who would attempt to predict the phreatic surface location within upstream embankments. This uneasiness is often manifested by a preference for other embankment types whose seepage and stability characteristics are more easily predicted and controlled.

Downstream Embankments

Prediction of phreatic surface location for downstream embankments is straightforward compared to upstream embankments. Boundary conditions and anisotropy within the embankment are important, but permeability variations within the tailings deposit itself are generally less significant. Phreatic surface control is usually accomplished by internal zoning with fill materials placed in a controlled manner, rather than by the spigotted tailings with their many variable factors.

Prediction of phreatic surface location and pore pressure conditions within downstream embankments is often aided by flow analogies with their water-retention dam cousins. For downstream embankments on impervious foundations that contain a core, neglecting the presence of the impounded tailings usually has a negligible or slightly conservative effect on phreatic surface location compared to the internal flow regime of a conventional dam retaining water only. For embankments on pervious foundations, however, the tailings have a sealing effect on seepage entrance to pervious zones, and neglecting the presence of tailings in the impoundment may lead to considerable overestimation of phreatic surface location within pervious zones of the embankment.

Figure 8.7a shows a parametric phreatic surface study for a water-retention embankment incorporating a sloping internal core. The analogous downstream-type tailings embankment is shown on Figure 8.7b, and the phreatic conditions are directly comparable to those in the conventional water dam so long as the permeability of the tailings is not significantly less than that of the core. For downstream embankments designed with a core, Figure 8.7 shows that the ratio of permeabilities between the downstream shell and the core exerts an important influence on phreatic surface location. In general, a shell–core permeability ratio of 100 or more is desirable.

For pervious downstream embankments that do not incorporate a core, the tailings deposit is relied upon to perform a seepage-retarding function in

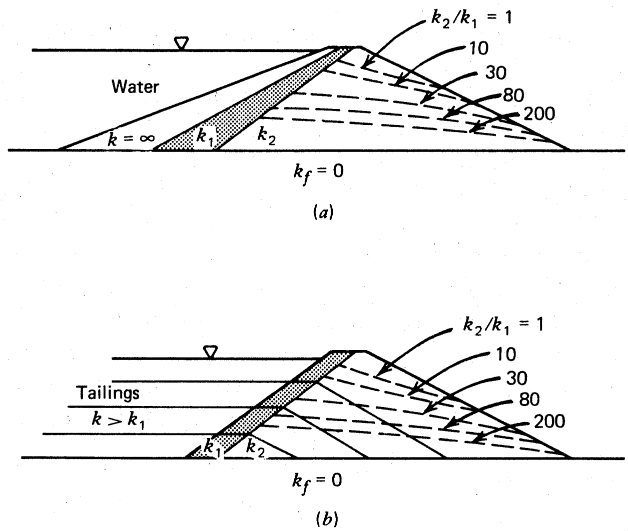


Figure 8.7 Effect of shell–core permeability ratio for embankments with upstream-sloping cores. (*a*) Conventional water dam. (From Cedergren, 1973.) (*b*) Analogy for downstream tailings embankment with core.

reducing the phreatic surface. Figure 8.8 shows the phreatic surface location for such embankments expressed in relation to the horizontal permeability of the tailings deposit (the embankment permeability is assumed to be isotropic). Again, the permeability ratio between the pervious embankment and the less pervious zone (in this case, the tailings deposit) controls the location of the phreatic surface. In addition, comparison of Figures 8.8a, 8.8b, and 8.8c illustrates the effect of horizontal layering and anisotropy of the tailings deposit.

Unlike upstream embankments, the effect of anisotropy within downstream-type structures is quite significant. Anisotropy of either the embankment fill or the tailings deposit may be important depending on whether the embankment contains a core.

It is important to keep in mind that the phreatic conditions shown in Figure 8.8 apply only where water is not allowed to contact the upstream embankment face directly. For pervious downstream embankments not incorporating a core, research by Isaacs and Hunt (1981) and discussions by Carrier (1983) and Marr (1983) indicate that ponded water in direct contact with the embankment having a depth equivalent to only a few percent of the embankment height can produce phreatic conditions in the embankment similar to those that would exist if it retained water only. Should site hydrologic conditions, tailings spigotting procedures, or mill water discharge rates make retention of significant depths of water a necessity, a core such as shown in Figure 8.7 would be mandatory to maintain acceptable phreatic levels.

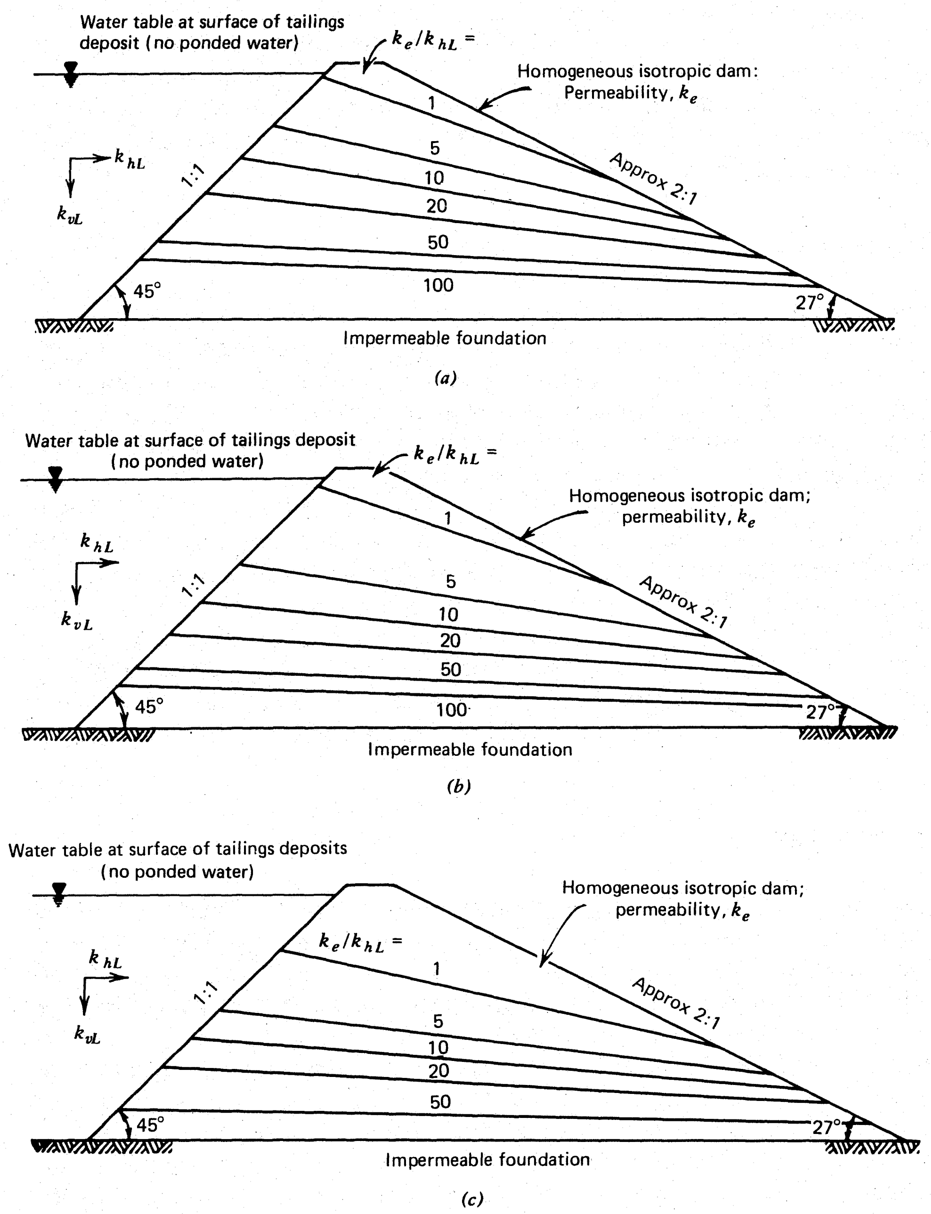


Figure 8.8 Phreatic surface for downstream embankment with no core. (Reprinted from Wimpey, 1972.) (*a*) $k_{hL} = k_{vL}$. (*b*) $k_{hL} = 10k_{vL}$. (*c*) $k_{hL} = 100k_{vL}$.

In addition, as previously mentioned, the use of cycloning procedures for downstream or centerline embankment types may introduce large quantities of water into localized portions of the embankment during cyclone operation, quantities many times greater than through-seepage from the decant pond. Cyclone operation and construction procedures will govern phreatic surface location in these cases.

Centerline Embankments

The phreatic surface within centerline tailings embankments often corresponds to that for conventional central-core water-retention dams. Figure 8.9 shows a comparison of phreatic surface locations for a water-retention dam and a similar centerline-type tailings embankment with a central core. As for downstream embankments, the phreatic surface location is governed to a large extent by the shell–core permeability ratio.

Figure 8.10 shows the effects of anisotropy of both the core and downstream shell on the phreatic surface location. Since seepage flow within the downstream shell must initially have a substantial vertical component, the presence of lower-permeability layers in the downstream shell retards vertical flow and results in an elevated phreatic surface. This effect, however, is most pronounced for shell–core permeability ratios less than about 50. In general, a shell–core permeability ratio of at least 100 results in a suitably low phreatic surface and tends to minimize the adverse effects of anisotropy.

SLOPE STABILITY

Preceding portions of this chapter describe development of phreatic surface conditions, the initial step in any evaluation of slope stability. Following assessment of phreatic conditions, it remains to evaluate the stability of the embankment slope using analytical procedures, and the remaining portion of this chapter discusses the analysis itself. Emphasis, however, is not on the details of the analytical mechanics, since this information is readily available in soil mechanics texts and is well known to the practicing geotechnical engineer. Rather, the meaning and interpretation of the analysis are the subject of considerable attention. Although these topics are complex and border at times on issues outside the normal context of routine stability analyses, their treatment can be justified by the fact that stability analyses are seldom routine when performed for tailings embankments. The complex behavior of tailings makes it perhaps more important to understand the purpose and application of a particular type of analysis rather than to understand the details of its mechanics.

With this introduction, the following portions of Chapter 8 discuss some of the philosophical issues in application of stability analyses to tailings

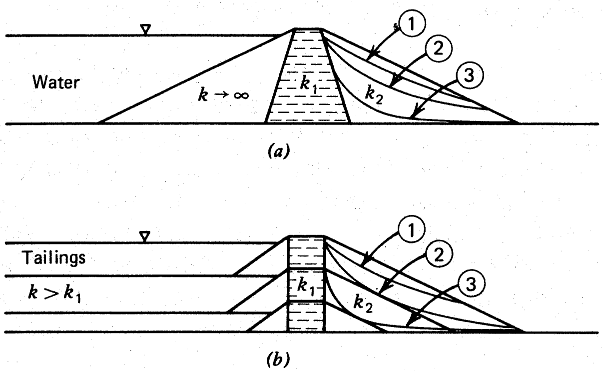


Figure 8.9 Effects of shell–core permeability ratio for centerline embankments. (*a*) Conventional central-core water-retention dam on impervious foundation: ① $k_2 = 10k_1$; ② $k_2 = 20k_1$; ③ $k_2 = 100k_1$; $k_h/k_v = 16$. (From Cedergren, 1967.) (*b*) Analogous centerline tailings dam with central core.

embankments, followed by a brief treatment of some of the available computational techniques. Attention is then devoted to strength and pore pressure behavior of tailings, perhaps the key to understanding overall embankment stability. Finally, the various types of tailings behavior under different loading conditions are translated into conditions for which stability analyses are performed.

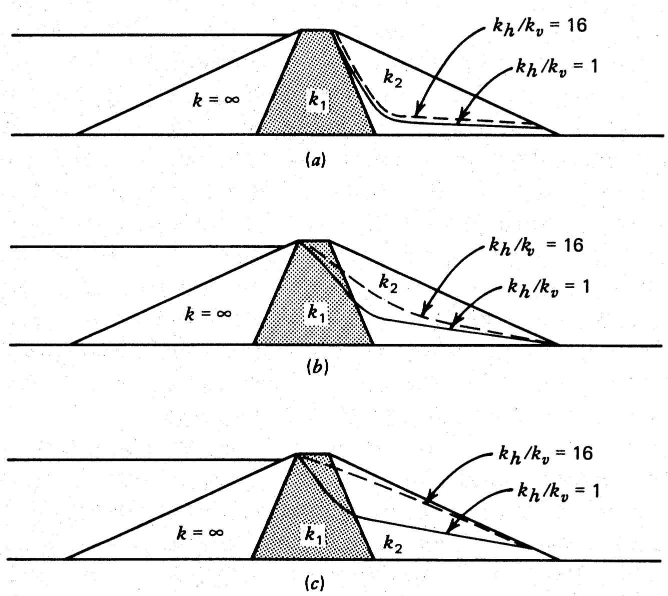


Figure 8.10 Effects of anisotropy for central-core embankments. (From Cedergren, 1967.) (*a*) $k_2 = 100k_1$. (*b*) $k_2 = 20k_1$. (*c*) $k_2 = 10k_1$.

Validity of Stability Analysis for Tailings Embankments

As described by Kealy and Soderberg (1969), stability analyses have traditionally been conducted for tailings embankments using techniques originally developed for analysis of natural slopes and conventional water-retention dam slopes. For water-retention and downstream-type tailings embankments, analytical procedures differ little in nature or validity from those used in more conventional applications. However, under conditions where hydraulically deposited tailings form all or part of the embankment (notably upstream and, to a lesser extent, centerline embankments), classical analytical procedures, while still routinely applied, must be used with more caution.

In an excellent interpretive treatment of embankment slope stability analysis, Johnson (1975) notes that the application of analytical techniques to stability of even conventional water-retention dams must be regarded as at least partially empirical. While analytical procedures ordinarily bear some relationship to actual behavior of an embankment slope, the true justification for their application involves to a significant extent the observation that dams analyzed according to certain procedural prescriptions in most cases prove to be stable, whereas the stability of dams not so analyzed is somewhat more in doubt. To a certain degree, analytical procedures become an engineering folklore of sorts whereby techniques are passed through the generations of dam designs, embellished by additional elements of sophistication from time to time but essentially unchanged in their basic nature.

This is not to imply that conventional slope stability analysis procedures have no validity for tailings embankments. To the contrary, tailings embankments find an even more pressing need for thorough stability analysis since they are not always designed with the same degree of conservatism as conventional water dams. However, the link between analytical models and actual slope behavior may be even more tenuous for tailings dams than for conventional water dams if the origins and assumptions of the classical procedures are not thoroughly evaluated. Central to the proper application of slope stability analysis to tailings embankments is an appreciation of the behavior and strength properties of tailings when they constitute a portion of the embankment. It has often been noted that errors in slope stability analysis are derived more from the input to the analysis than from approximations inherent in the computational techniques themselves, and nowhere is this more true than for analysis of tailings embankments.

All computational procedures in common use for slope stability assessment involve limiting-equilibrium procedures wherein the material circumscribed by some assumed failure surface of circular, planar, or composite geometry is assumed to slide as a rigid body. The effects of internal deformation within the sliding mass of material are neglected. While the nature of many rotational slides in natural slopes of cohesive or dense soils gives some credence to this rigid-body assumption, many if not most slides that involve hydraulically deposited tailings occur in large part as flow slides. Blight and Steffen (1979) note the occurrence of flow slides in South African gold tailings embankments, and Bishop (1973) describes several examples of flow slides in tailings embankments and mine waste piles. This mode of failure certainly violates the rigid-body assumption of limiting-equilibrium analysis.

Although most tailings embankment slides (with the exception of those induced by seismic liquefaction) probably have a rotational-type slide as their trigger mechanism, it is the flow-type behavior that is so destructive, as witnessed by the failures of a mine waste pile at Aberfan, Wales (Bishop, 1973), and a coal refuse embankment at Buffalo Creek, West Virginia (Wahler, 1973)—both with major loss of life. Unfortunately, the mechanics for understanding and analyzing flow behavior are still in their infancy (Jeyapalan et al., 1981; Lucia et al., 1981). Thus, slope stability analysis for tailings embankments by necessity concentrates on initial rotational-type slides incorporating the rigid-body assumptions of limiting-equilibrium analysis. It is important to keep in mind that these analyses therefore represent only conditions of incipient failure and are not intended to describe behavior of the embankment after failure has been initiated.

The analysis of slope stability begins with the selection of a trial embankment slope inclination. While the embankment slope must obviously be flat enough to ensure adequate stability, it cannot be too flat or excessive quantities of embankment fill may be required. Determining the embankment slope angle therefore requires a balance between safety and economic considerations. Significant differences arise between various tailings embankment types in this regard.

As discussed in Chapter 3 and illustrated in Figure 3.7, fill quantities to achieve a given embankment height differ according to embankment type, and the sensitivity of fill requirements to embankment slope angle varies in a similar way. For upstream embankments, total fill requirements are relatively insensitive to slope angle and design to a 4:1 slope carries little economic penalty compared to, say, a 2:1 slope. There is therefore little economic excuse for oversteepened slopes on upstream embankments. For downstream embankments, on the other hand, the variation in fill quantity according to slope angle can be significant, and this significance grows for increasing embankment height. For high downstream embankments, considerable effort and refinement in the analysis of slope stability may be justified to determine a slope angle that satisfies safety constraints and at the same time achieves an economic design.

Computational Techniques

A number of computational methods are available to model the stability of embankment slopes. The end product of analysis of a given trial or potential failure surface is the factor of safety (FS), defined as the summation of driving forces tending to produce failure divided by the summation of forces tending to resist failure. The more rigorous methods include the Simplified Bishop, Spencer, Janbu Simplified, Janbu Generalized, and Morgenstern-Price methods, among others. Details of some of the various methods can be found in Whitman and Bailey (1967), Spencer (1967), Janbu (1973), and Morgenstern and Price (1965). In all the methods, the body of soil within the failure mass is divided into a number of vertical slices that interact by means of forces transmitted along the sides of the slices. The methods vary principally in the assumptions regarding the location and inclination of the side forces necessary to solve equations derived for the statically indeterminate system of forces.

In a comparison of various computational techniques, Wright et al. (1973) determined that most of the procedures presently in common use give results that usually vary only several percent from the results of a completely independent type of analysis free from the various side force and equilibrium assumptions of the different methods evaluated. Only the Ordinary Method of Slices was found to be overly conservative in some cases. The practical result is that, since none of the commonly used computational techniques involve large theoretical errors, the choice of a method can be based on convenience and preference of the user. Again, these results suggest that the proper interpretation of input data is more important than details of the computational procedure.

Although theoretical imprecision in computational procedures may be small, this does not guarantee that large errors will not occur undetected due, for example, to data input mistakes. In performing any slope stability analysis, it is essential that a hierarchy of procedures be employed, with simple techniques used to check the reasonableness of results produced by techniques of the next higher level of complexity.

Janbu (1954) and Cousins (1978), for example, present convenient charts for stability analysis of embankment slopes using dimensionless parameters. Such simplified methods may not produce a precise answer, but they are useful for preliminary sensitivity studies to assess the effect on the computed factor of safety of such variables as phreatic surface location and strength parameters. Those variables shown to have a major effect on the stability of a particular slope may then be investigated in more detail by one of the more sophisticated analytical approaches. In addition, a major value of simplified techniques is that with judicious selection of input parameters, they can quickly provide reasonable upper and lower bounds within which the results of more complex analyses should lie. In this way, gross errors can be avoided.

In many cases involving slopes in cohesionless tailings, shallow critical failure surfaces are found to approach the *infinite-slope* condition. In addition to stability charts, the following infinite-slope relationships are often useful in conjunction with simplified analyses for cohesionless materials (Corps of Engineers, 1970):

for dry slopes $FS = \dfrac{\tan \phi}{\tan i}$

where i = embankment slope angle
ϕ = friction angle

for seepage parallel to slope $FS = \dfrac{\gamma_b}{\gamma_t}\left(\dfrac{\tan \phi}{\tan i}\right)$

where γ_b = buoyant unit weight
γ_t = total unit weight

for horizontal seepage $FS = \left(\dfrac{\gamma_b - \gamma_w \tan^2 i}{\gamma_t}\right)\left(\dfrac{\tan \phi}{\tan i}\right)$

where γ_w = unit weight of water

Soil Behavior for Stability Analysis

Meaningful analysis of slope stability requires that the field loading conditions be properly accounted for. This in turn requires that the type of laboratory strength used in the analysis be compatible with both drainage conditions and pore pressures generated under field conditions.

Pore Pressures

Bishop and Bjerrum (1960) note that much confusion in pore pressure prediction, and therefore in strength behavior interpretation, results from a failure to distinguish between various phenomena that give rise to pore pressures.

Figure 8.11 summarizes three basic classes of pore pressure problems that occur in analysis of tailings embankments. The first source of pore pressure arises from seepage, a topic that has received considerable attention in preceding portions of this chapter. These initial static pore pressures are usually derived from steady-state seepage flow, and their presence is not dependent on the application of any external load to the embankment. While rigorous procedures for determining static pore pressures require examination of equipotential lines from complete flow nets, a reasonable approximation commonly used in stability analysis is that the pore pressure at a particular point corresponds to its depth below the phreatic surface, as shown in Figure 8.11a.

The second class of pore pressure is initial excess pore pressure due to rapid uniform loading, as shown in Figure 8.11b. The conditions illustrated in Figure 8.11b might occur, for example, for an upstream embankment

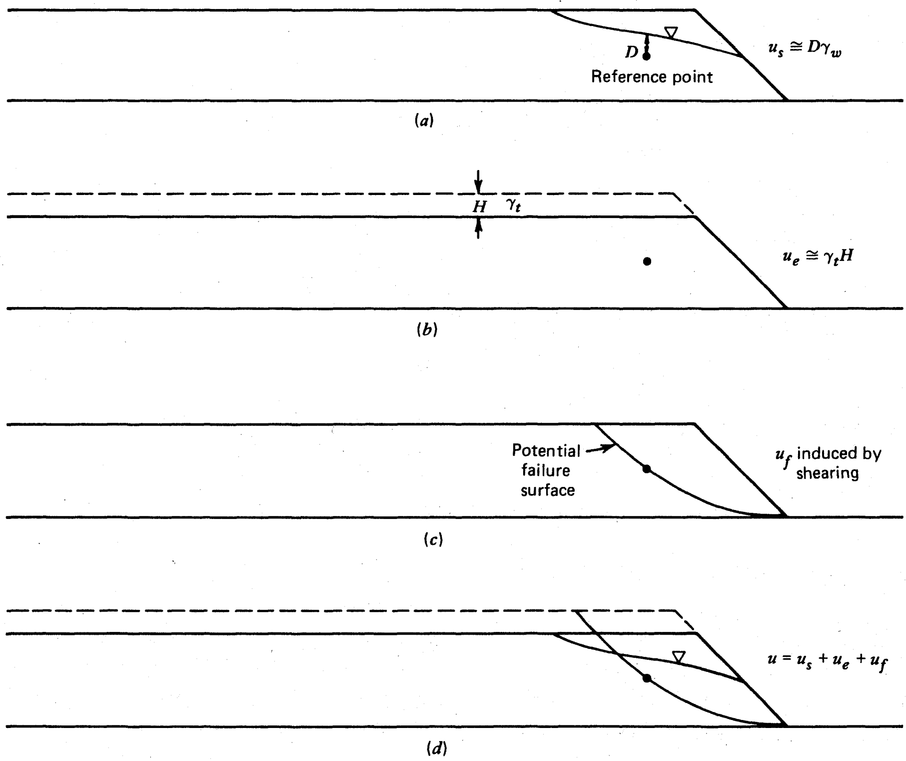


Figure 8.11 Sources of pore pressure in stability analysis. (*a*) Initial static pore pressure due to seepage. (*b*) Initial excess pore pressure due to uniform rapid loading. (c) Pore pressure due to shear. (*d*) Combined pore pressure conditions.

being raised at a rate that is very rapid in relation to the ability of the tailings to dissipate pore pressures by consolidation processes. For modeling purposes, the incremental raise might be assumed to be applied instantaneously. At the moment of load application, the total stress applied by the load increment would be carried by pore pressure (as a first approximation), giving rise to the value u_e shown in Figure 8.11b. An analogous condition generating initial excess pore pressure might be the presence of a soft clay foundation. Here initial excess pore pressures would occur within the foundation materials.

The third class of pore pressure conditions, and perhaps the one giving rise to the most confusion, is illustrated in Figure 8.11c. Pore pressures due to shearing are generated by changes in shear stress that occur rapidly in relation to the drainage and pore pressure dissipation characteristics of the material. In common terminology, this is referred to as *undrained loading*.

For essentially cohesionless soils, such as tailings, whether or not pore pressures are generated by rapidly applied shear stresses depends on both the permeability and the density (or void ratio) of the material. For soils that are very coarse and permeable, pore pressures generated by shearing may dissipate as rapidly as the load is applied. For dense compacted materials, dilatency during shear may result in negative rather than positive pore pressures, which are usually neglected for purposes of stability analysis. The most problematic conditions for analysis involve loose, fine-grained materials. In this case, rapidly applied changes in shear stress may result in the development of significant pore pressures during shear that do not have the opportunity to dissipate during rapid shearing. Changes in shear stress giving rise to this class of pore pressure need not be limited to applied loading. Unloading, such as might be produced by removal of material at the embankment toe, also produces changes in shear stress that may be conducive to pore pressure generation.

Pore pressures arising from these three sources may be additive in their application to stability analysis, as shown in Figure 8.11d. Initial pore pressures may result from both static pore pressures as well as excess pore pressures resulting from prior application of uniform load. If rotational failure of the slope is then postulated, and if the failure is assumed to occur as a result of rapidly applied changes in external loading, pore pressures generated due to shear would also occur and would be added to those existing previously from other sources.

Drained and Undrained Strength Behavior

With an understanding of pore pressure conditions, the appropriate strength behavior for use in stability analysis can be determined. It is most instructive to consider the triaxial test in connection with strength behavior, since the drainage conditions can be varied in the test apparatus and also because it is the most common type of test used in relation to embankment analysis. Two basic forms of the test can be performed, consolidated-drained (CD) and consolidated-undrained (CU). In both types, the sample is initially consolidated to a consolidation stress $\bar{\sigma}_c$ (usually isotropic) that represents the initial effective stress at a point in the embankment or foundation prior to shearing. Following consolidation, the sample is sheared either in a drained condition, where all pore pressures generated during shear are allowed to completely dissipate, or in an undrained mode, where dissipation of pore pressures generated during shear is prevented. The different drainage conditions give rise to different strength envelopes.

Drained (CD) tests, which produce only the effective-stress envelope with friction angle $\bar{\phi}$, are appropriate generally for analyses where it is not desired to account for pore pressures produced by shearing. The undrained (CU) test, on the other hand, produces total-stress strength parameters (ϕ_T and c_T) that inherently incorporate the effects of shearing-induced pore pressures. The effective-stress strength envelope can also be derived from CU tests by measurement of pore pressures in the sample during the test.

An example of an effective-stress Mohr envelope derived from CD tests is

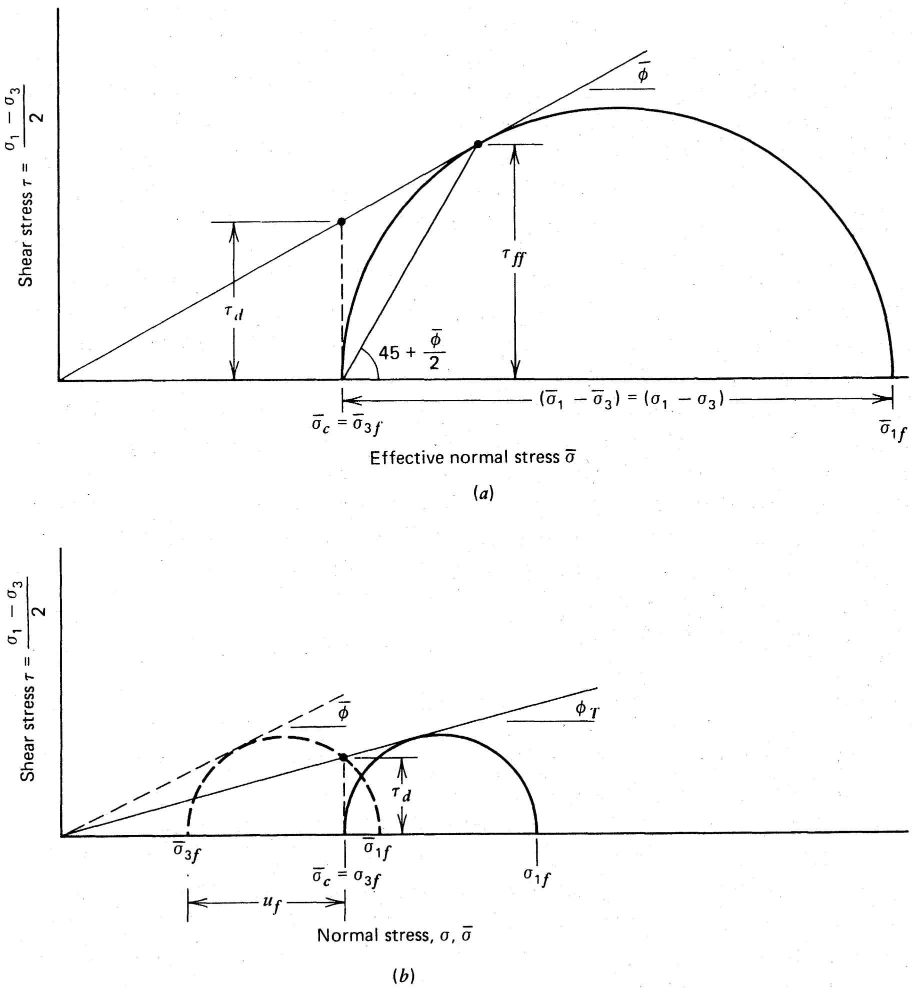


Figure 8.12 Mohr envelopes for drained and undrained conditions. (*a*) Effective-stress strength envelope from consolidated-drained (CD) triaxial test. (*b*) Effective- and total-stress strength envelopes from consolidated-undrained (CU) triaxial test.

shown in Figure 8.12a. In this example, suppose that only one test is being performed to represent the stress conditions at a particular point in the embankment. First, the total stress at the point in question would be computed, usually from the vertical overburden pressure. Then any initial pore pressures, either static pore pressures from seepage or excess pore pressures from prior loading, would be subtracted to yield the effective consolidation stress $\bar{\sigma}_c$ to be used in the test. Subsequent shearing of the sample by increasing the major principle stress would be carried out at a rate sufficiently slow to prevent any buildup of pore pressure during shear. A Mohr envelope at failure would result, as shown in Figure 8.12a. The Mohr envelope represents a relationship between the initial effective stress $\bar{\sigma}_c$ and the maximum drained shear strength that can be mobilized on the failure plane at failure.

An anomalous situation in the interpretation of the Mohr envelope for stability analyses is discussed by Lowe (1967) and by Johnson (1975). Theoretical considerations indicate that the maximum shear stress on the failure plane at failure (τ_{ff}) occurs at an angle of 45° + $\bar{\phi}/2$ from the initial effective consolidation stress, resulting in construction of the Mohr envelope from laboratory tests tangent to the Mohr's circle, as shown in Figure 8.12a. However, as commonly interpreted for stability analysis, the design shear strength, τ_d, is taken from the Mohr envelope at the point corresponding to $\bar{\sigma}_c$. Thus, the design shear strength is typically somewhat lower than the laboratory interpretation of the Mohr envelope would indicate.

Results from an example consolidated-undrained (CU) test are shown in Figure 8.12b. In this test, the sample is consolidated to the same $\bar{\sigma}_c$ as before, determined by subtracting any initial static or excess pore pressure from the overburden stress. Since drainage during shear is prevented in the CU test, a portion of the applied load is carried by pore water as pore pressures develop during shear. The resulting undrained shear strength and the corresponding total stress friction angle ϕ_T are both lower than for the CD test.

If pore pressures are measured during the CU test, an effective-stress envelope can also be constructed. As indicated on Figure 8.12b, the effective-stress Mohr's circle is shifted to the left by an amount equal to the pore pressure developed in the sample at failure, u_f.

The undrained Mohr envelope provides a relationship between initial effective stress $\bar{\sigma}_c$ and the undrained shear strength mobilized on the failure plane at failure. It differs from the effective-stress envelope in that it includes pore pressures generated during shearing and is generally appropriate for analysis of stability problems where external loading changes are imposed more rapidly than the ability of the material to drain.*

There are several theoretical objections to the use of an undrained strength envelope from triaxial CU tests to model undrained shear strength. As discussed by Lowe (1967), Johnson (1975), and Bishop and Bjerrum (1960), these objections center around the fact that isotropic consolidation in the CU test overestimates the strength of materials that are anisotropically consolidated in situ. In addition, undrained shear strength in the triaxial test

*Pore pressures generated during shear can also be accounted for by measuring Skempton's pore pressure parameter $\bar{A}$ in CU tests and incorporating these pore pressures into an effective-stress analysis with $\bar{\phi}$ strength parameters (Lambe and Whitman, 1969). While this procedure allows pore pressures during undrained shear to be accounted for in conjunction with effective-stress parameters, it is usually simpler and conceptually less confusing to use undrained strength parameters ϕ_T and c_T from CU tests where it is desired to account for pore pressures generated by shearing. These values intrinsically account for the pore pressure at failure. The end results of both procedures are the same.

is influenced by the stress system applied during shear (compression vs. extension), and unless test loading conditions are varied to approximate those that occur along the sliding surface, errors may result. These problems can be minimized by expressing undrained shear strength as a function of consolidation stress from CU direct shear tests ($S_u/\bar{\sigma}_c$), as proposed by Ladd and Foott (1974). Using this approach, computed values of undrained shear strength S_u would be used in various portions of the embankment in a $\phi = 0$ type analysis. Cumbersome at best, this approach requires dividing the embankment section used for analysis into a large number of individual elements with a unique S_u value assigned to each. An additional problem discussed in Chapter 2 is that tailings may not behave according to the normalized strength concept, with the ratio $S_u/\bar{\sigma}_c$ varying as a function of stress level.

Final mention is required concerning the unconsolidated-undrained (UU) triaxial test. In this test, the sample is sheared in an undrained mode but is not consolidated prior to shear. A flat, or $\phi = 0$, strength envelope theoretically results. Pore pressures induced by shearing are accounted for, but the test applies only when samples can be obtained that have already been subjected to consolidation in situ and when loading in the field occurs so rapidly that no dissipation of pore pressure during shear is anticipated. In practice, the use of undrained shear strength from UU tests is usually applied only to situations involving rapid starter dike construction on a soft clay foundation or to certain types of embankment construction involving rapid placement of fill over predominantly slimes tailings.

Conditions of Analysis for Tailings Embankments

The usual classes of stability analyses for conventional water-retention dams correspond to various critical loading conditions that occur during the life of the dam. These traditionally include:

1. End of construction.
2. Staged construction.
3. Long-term steady seepage.
4. Rapid drawdown.

Most of the same general conditions for analysis apply to tailings embankments. However, tailings embankments, unlike conventional water dams, in general do not experience such clear delineations between periods, such as when construction ends and when steady seepage begins. For raised embankments, construction occurs over the entire embankment life, seepage surfaces rise along with the level of impounded tailings, and when construction truly ends the embankment is abandoned and stability is no longer of major concern because the embankment becomes unsaturated. Consequently, different emphasis must be placed on the various conditions of analysis for tailings embankments.

End of Construction

Postconstruction analysis for tailings dam starter dikes is identical to that performed for conventional dams and embankments. This class of analysis applies to starter dikes constructed rapidly upon soft silts or clays. Here, significant dissipation of load-induced pore pressure may not occur during construction, and if failure occurs, the full pore pressure due to shearing is experienced by the foundation material. As illustrated in Figure 8.13, this type of loading is usually analyzed using the undrained shear strength S_u and a $\phi = 0$ approach based on the results of UU or CU direct shear tests or field vane determinations on the natural clay. For soft, normally consolidated clays, S_u usually increases with depth, and it may be necessary in the analysis to divide the foundation material into several horizontal layers to reflect this increase. Tavenas and Leroueil (1980) summarize interpretations of undrained strength behavior for embankments constructed on soft clay and analyzed using the $\phi = 0$ approach for postconstruction conditions.

A similar loading condition may occur for centerline embankment raises, as shown in Figure 8.13b. Here a $\phi = 0$ analysis may be required to determine the stability in the upstream direction for that portion of the fill which is placed over the tailings, particularly if the raise is high and constructed rapidly and if the tailings are primarily slimes. Determining initial undrained shear strength may be difficult if the actual deposit cannot be sampled and tested prior to analysis. In such cases, experience with similar deposits may provide useful information, or it may be necessary to estimate undrained

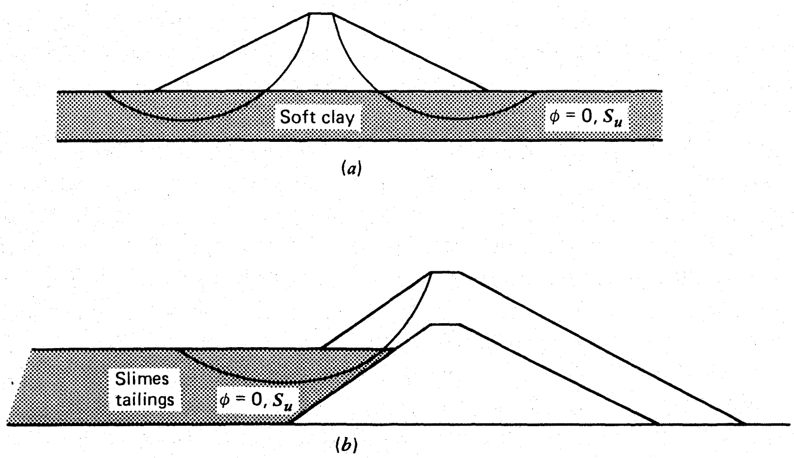


Figure 8.13 End-of-construction analysis for tailings embankments. (*a*) Starter dike constructed rapidly on soft clay foundation. (*b*) Centerline raise constructed on soft slimes.

shear strength based on artificially sedimented samples consolidated in the laboratory.

Staged Construction

The staged-construction condition for tailings embankments presents the most difficult class of stability problem. This analysis may apply to raised downstream or centerline embankments constructed on soft foundations, but it is most commonly encountered for upstream embankments that are raised very rapidly. The analysis differs from the end-of-construction case by virtue of the fact that loading occurs slowly enough so that significant dissipation of load-induced pore pressure occurs during the term of raising. However, the rate of embankment height increase is not slow enough to allow these initial excess pore pressures to be completely neglected.

For upstream embankments raised no faster than about 15–30 ft/yr, excess pore pressures are usually assumed to dissipate as rapidly as the load is applied, and staged-construction analysis is usually not performed. The classic staged-construction case, however, involves upstream embankments raised in excess of about 50 ft/yr, particularly where the spigotted beach tailings are relatively fine grained.

The staged-construction analysis requires that the embankment be modeled as a series of discrete raises and analyzed at incremental time steps. At any particular point in the embankment, excess pore pressures vary with time, as do static pore pressures from seepage. The procedural approach is illustrated in Figure 8.14. Initially at time t_1 a new raise is added instantaneously, generating excess pore pressures in the first raise (Raise I) in addition to the initial static pore pressures derived from seepage. At time t_2 another raise is added. Some dissipation of excess pore pressure from the first raise loading has occurred by now in Raise I materials, but added to this residual excess pore pressure must be the new excess pore pressure generated by the second new raise. In addition, static pore pressures increase as the impoundment level and the phreatic surface rise. A similar process occurs at time t_3.

Within each raise and at each time step, processes of excess pore pressure dissipation and generation occur simultaneously. Complex accounting procedures are required to track the development of both excess and static pore pressures with time. Mittal and Morgenstern (1976) describe an ingenious application to tailings embankments of a moving-boundary consolidation solution originally developed by Gibson (1958), who shows that the parameter governing excess pore pressure dissipation is m^2t/c_v where m is some constant rate of embankment height increase, t is time, and c_v is the coefficient of consolidation. Solutions for excess pore pressure for different rates of height increase and different boundary drainage conditions are presented in dimensionless form in Figure 8.15. These solutions can be used to derive excess pore pressures for input into staged-construction stability analyses.

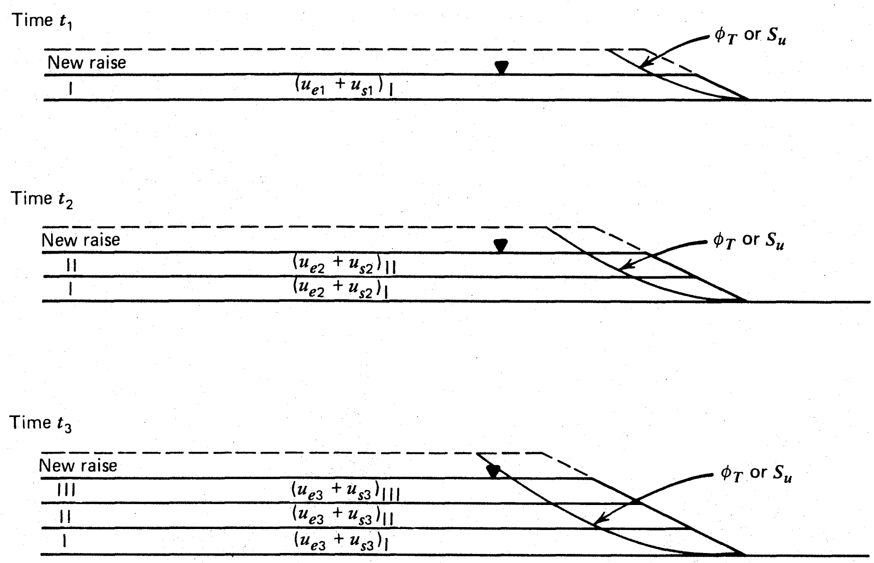
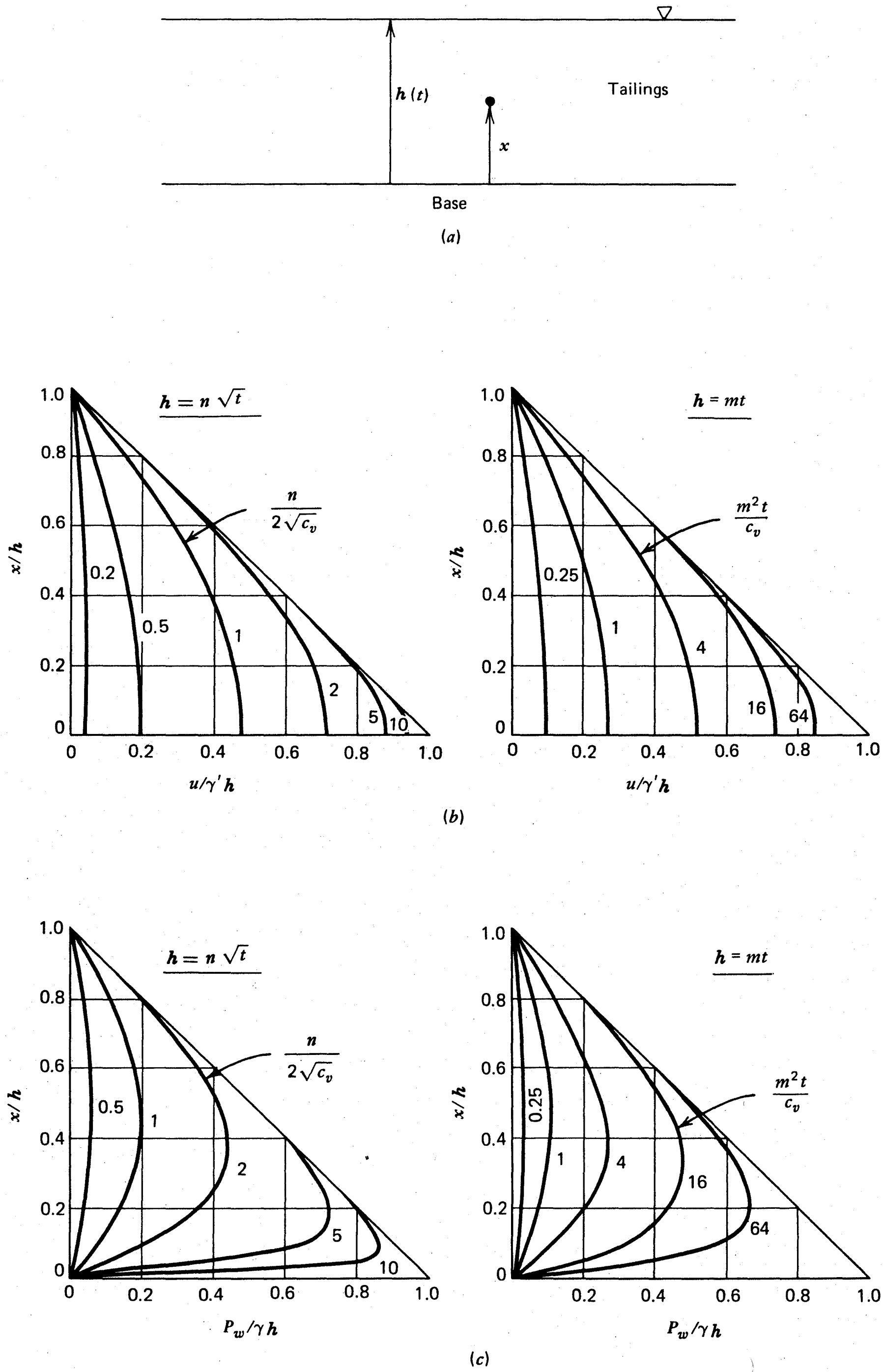


Figure 8.14 Staged construction analysis for rapid raising of upstream embankment.

In addition, the solutions are useful to check whether, in fact, excess pore pressures are sufficient to warrant a complete staged-construction analysis. Nelson et al. (1977) describe one such application.

Consistent with the assumption in staged-construction analysis that load increments are applied rapidly is that pore pressures generated during rapid shearing be included in the analysis. Consequently, undrained shear strengths usually using an undrained strength envelope are called for in the analysis of stability of each embankment raise. Residual excess pore pressures can be added to the phreatic surface for each time step to generate a piezometric surface that can be directly input into the stability analysis. A separate piezometric surface is required, however, for each individual raise to account for the varying degrees of residual excess pore pressure in each.

In an actual embankment, consolidation and dissipation of excess pore pressures will be significantly accelerated by the normally expected presence of thin sand seams in the tailings deposit and by two-dimensional consolidation horizontally outward toward the embankment face as well as vertically. These factors, however, introduce unmanageable complexities in the analysis and are usually neglected. The result is likely to be a conservative prediction of excess pore pressure with time and location in the embankment according to one-dimensional consolidation theory. Nevertheless, it is essential that pore pressure predictions for staged-construction analyses be verified by field instrumentation. An alternate or segmented impoundment is useful in the event that embankment raising must be terminated or conducted at reduced rates because of higher-than-predicted pore pressures and corresponding reduced embankment stability.


h (t)
Tailings
x
Base
(a)
h = n √t
n / 2√c_v
x/h
u/γ' h
0.2
0.5
1
2
5
10
h = mt
m²t / c_v
0.25
1
4
16
64
(b)
P_w/γh
(c)
0
0.2
0.4
0.6
0.8
1.0

Long Term

During construction and reservoir filling, stresses within conventional water dams undergo redistribution in response to external loads. Redistribution of stresses may continue for a time as foundation settlement occurs and as steady seepage develops. Long-term conditions in conventional water dam stability analysis are applied when all internal stresses and pore pressures have reached equilibrium and when steady seepage conditions under full reservoir have developed. By definition, no external loads are applied during or shortly before the time represented by the long-term conditions of analysis.

Long-term analyses are performed for "design" or "nonfailure" conditions, as discussed by Johnson (1975). In effect, this implies an a priori assumption that the embankment will be stable under long-term conditions with a suitable factor of safety. This assumption, one that seems reasonable in light of the fact that few dams are intended to fail in the long term, is important in properly interpreting pore pressure conditions to be used in long-term analysis of tailings embankments.

Long-term analyses for tailings embankments are performed under conditions similar to those defined for conventional water dams, but with some further assumptions. Long-term analyses are usually applied to raised embankments at their maximum height. For embankments that are raised slowly, it is usually assumed that excess pore pressures from all previous raises have dissipated and that internal redistribution of stresses from prior raise construction has been completed. (Cases where this assumption is not reasonable require a staged-construction analysis as previously discussed.) It is further usually assumed that, at the maximum height, steady seepage has developed since these conditions yield the most conservative estimate of long-term static pore pressures.

Two schools of thought exist on whether or not the pore pressures induced by shearing at failure should be included in long-term analysis conditions. Johnson (1975) argues that shear strength used in analysis may be too high if pore pressures due to shear are neglected. Incorporating these pore pressures in a long-term analysis would require either that total-stress parameters (ϕ_T and c_T) be used or that effective-stress parameters be supplemented by pore pressures at failure measured in consolidated-undrained tests.

However, incorporating pore pressures induced by shear at failure is not consistent with the previously described assumptions of long-term condi-

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Figure 8.15 Excess pore pressure solutions. (Reprinted from Gibson, 1958, by permission of the Institution of Civil Engineers, London.) (*a*) Tailings depth increase with time expressed as specified function $h(t)$. (*b*) $u/\gamma' h$ for deposition rates $h = n\sqrt{t}$ and $h = mt$ where u = excess pore pressure, $\gamma' = \gamma_t - \gamma_w$ (impervious base). (c) $P_w/\gamma h$ for deposition rates $h = n\sqrt{t}$ and $h = mt$ where P_w = static plus excess pore pressure (pervious base).

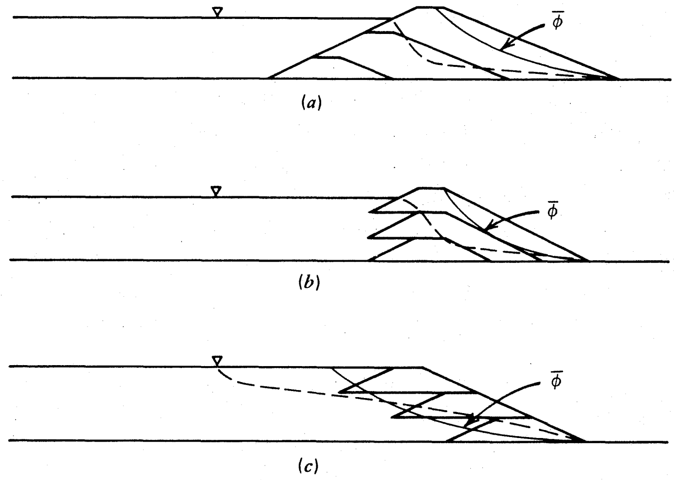


Figure 8.16 Long-term analysis for various types of tailings embankments. (*a*) Downstream. (*b*) Centerline. (*c*) Upstream.

tions—namely, that the analysis represents nonfailure (design) conditions and that no changes in external load occur. Lowe (1967) notes that effective-stress strength should be used for analysis of long-term conditions for conventional dams because by definition no load is suddenly applied. Bishop and Bjerrum (1960) also state that effective-stress parameters are appropriate for long-term stability calculations. In analysis of tailings embankment stability under long-term conditions, customary and nearly universal practice is to use effective-stress strength parameters without accounting for the pore pressures induced by shear at failure (Kealy and Soderberg, 1969; Wahler, 1974), as illustrated in Figure 8.16. For downstream and centerline-type embankments where the embankment fill is compacted, neglecting the negative pore pressures that may be induced by shear is conservative. For embankments composed largely of uncompacted material, notably the upstream type, effective-stress parameters are consistent with the assumptions of long-term conditions of analysis.

For upstream embankments or other types constructed of uncompacted material, the use of effective-stress parameters for long-term analysis, however, must be examined very critically in relation to the loading conditions that the embankment may experience. Long-term conditions are defined as those that occur with no rapid change in external loading conditions. Whether or not loading conditions will in fact remain constant may be conjectural in some cases.

As discussed in Chapter 2, beach tailings deposits at typical relative densities and void ratios do not ordinarily show a drastic decrease in strength as a function of strain in common laboratory tests. For very loose materials, however, the strength decrease may be significant. In this respect, very loose tailings deposits may share the characteristics of natural deposits of sensitive clay. Bishop and Bjerrum (1960) note that, for such soils, even a small initial slip in a slope that is otherwise stable under long-term drained (effective-stress) conditions will cause a rapid change in loading conditions at the toe of the slope, producing a series of retrogressive slides that occur under undrained (total-stress) conditions. Failure of the entire slope by flow sliding quickly results. Bjerrum (1973) further describes this mechanism of slope failure for natural slopes in sensitive clays.

In this case, the long-term analysis would correctly indicate a stable slope under the terms of its definition. However, the occurrence of a small, seemingly insignificant slide could cause a change in loading conditions that violates the assumptions of long-term analysis. Bishop (1973) suggests that this phenomenon may also occur for tailings embankments constructed of very loose material, where an initial small slump on the embankment face that occurs under drained (effective-stress) conditions causes a rapid change in loading. Although initially localized, the rapid change in loading requires that the pore pressures developed during application of the resulting shear stress be accounted for by using undrained conditions (total-stress strength) in interpreting the behavior of the embankment subsequent to the initial disturbance.

This phenomenon is illustrated by stability analyses shown in Figure 8.17. Figure 8.17b shows the analysis of an upstream embankment under assumptions of long-term conditions using effective-stress strength parameters. The computed factor of safety for the overall embankment is 1.68 for failure surface C, which would indicate a stable embankment. However, for this embankment, seepage analysis shows that seepage would break out on the face, resulting in a zone of saturation at the embankment toe. Failure surface A, with a factor of safety of 0.99, suggests that a shallow, localized sloughing-type slide could occur in saturated materials near the toe.

On initial inspection, it might be concluded that such a small, shallow, sloughing-type failure would be inconsequential to stability of the embankment as a whole. Nevertheless, if such sloughing were to occur rapidly, the nonfailure assumptions of long-term analysis conditions would be violated. Changes in stress resulting from the sloughing could generate pore pressures due to shear in remaining portions of the embankment. To model these new conditions would require an undrained analysis using total-stress strength parameters for post-sloughing embankment stability.

Figure 8.17c shows a second analysis, which accounts for pore pressures induced by the changes in shear stress by using total-stress strength parameters for the zone of saturated tailings below the phreatic surface. Effective-stress parameters are retained for the higher materials, since positive pore pressures due to shear cannot develop in the absence of saturation.

The computed factors of safety now show that failure along surface B would occur (FS = 0.95) and in addition that stability along failure surface C (FS = 1.05) would be very marginal. If further analyses were performed to

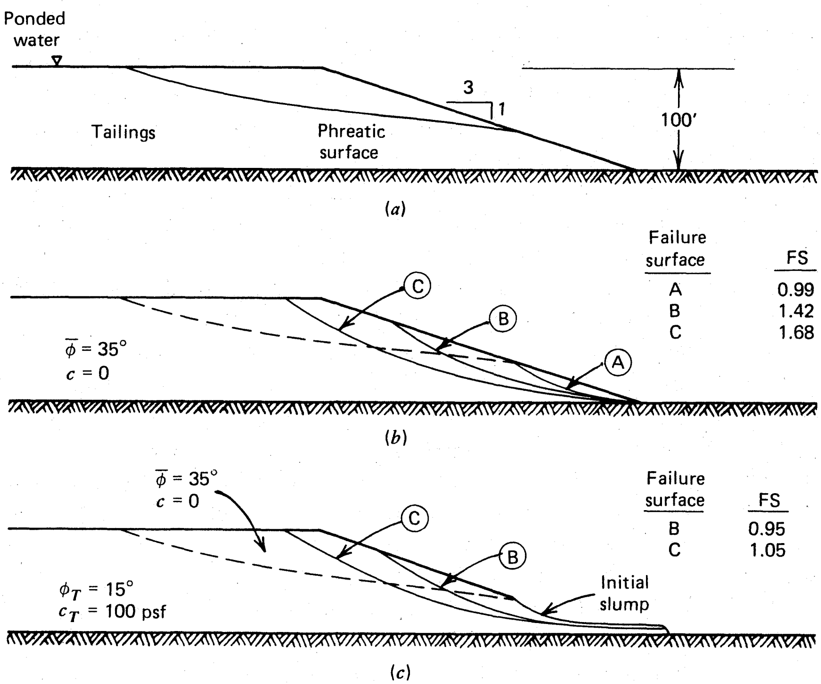


Figure 8.17 Example analysis illustrating progressive failure under drained and undrained loading conditions. (*a*) Geometry, material and phreatic conditions. (*b*) Effective-stress analysis, long-term conditions. (c) Total-stress analysis, after change in loading produced by initial slump.

reflect removal of material circumscribed by failure surface B, the entire embankment would be shown to be unstable.

The factors of safety for the post-sloughing analysis now indicate that embankment failure could occur in a progressive manner under undrained conditions, triggered by a seemingly innocuous change in external loading. In this particular example, failure under undrained conditions might be prevented by flattening the embankment slope to achieve an adequate degree of stability or by increasing the beach width to produce a lower phreatic surface and reduced saturated zone of undrained-strength material. The obvious and desirable solution, however, would be to avoid seepage breakout on the embankment face in the first place by use of a pervious compacted drainage zone at the toe, and/or underdrains.

Rapid Drawdown

A final comment concerns the rapid drawdown condition that is ordinarily required to analyze the stability of the upstream slope of conventional water dams. This analysis represents changes in loading that occur when the reservoir is emptied at a high rate. Ordinarily, the only way that a tailings impoundment can be rapidly evacuated is by breach of the embankment itself—obviously an occurrence to be avoided. Therefore, the rapid drawdown condition does not apply for stability analysis of tailings embankments, with the possible exception of those designed to temporarily retain then quickly pass significant depths of flood water through spillways.

An interesting consequence of this observation is that upstream slopes of tailings embankments can often be constructed much steeper than those governed by the rapid drawdown condition for conventional water dams. The upstream slope of the tailings embankment is buttressed by tailings in the long-term condition, so its stability is seldom of concern. The steepness of upstream starter dike slopes is limited only by stability in end-of-construction conditions where soft foundation materials are present, and otherwise by the angle of repose of the fill.

SUMMARY

Stability analyses for tailings embankments, while following the same general procedures and using the same basic computational methods as for water-retention dams, are often more complex, and a thorough understanding of pore pressure and strength behavior is required to apply conventional techniques in a rational manner. Basic to stability analysis is an appreciation for the various sources of pore pressure and the way in which they affect the interpretation of shear strength. In addition, loading conditions for tailings embankments are sometimes different from those experienced by conventional water dams, giving a different emphasis to the various conditions that must be considered in the analysis.

Table 8.1 summarizes in a general way the procedural steps required for tailings embankment stability analysis. Assuming that the basic embankment type, materials, and internal zoning have been established, the first step is to select a trial embankment slope configuration and inclination. Prediction of the phreatic surface location follows in order to estimate initial static pore pressures. In addition, an evaluation must be made to determine whether the embankment raising rate is sufficient to generate excess pore pressures in either the embankment or foundation materials. After these steps have been completed, stability computations can be performed to determine whether the trial embankment slopes are stable under all applicable conditions of analysis. If not, a new iteration of the procedural steps must be performed, or perhaps complete redesign carried out, until a stable embankment configuration is achieved.

While the general steps in embankment stability analysis are conceptually simple, the actual conditions under which the embankment must be ana-

Table 8.1 Generalized Procedures for Performing Tailings Embankment Stability Analyses

Step	Method
(1) Select trial embankment slope configuration	Experience and judgment
(2) Determine phreatic surface location based on internal zoning, material permeabilities, and boundary conditions	Flow nets Numerical models Published solutions
(3) Establish whether or not initial excess pore pressures will result from embankment raising	Compare raising rate to pore pressure dissipation rate for tailings or soft foundation soils
(4) Perform stability computations for applicable conditions	Use any of several available computational methods after defining loading conditions, cases for analysis, and appropriate strength behavior under drained and undrained conditions
(5) Return to Step 1 and revise trial configuration if factors of safety are not adequate	

lyzed are not. As summarized in Table 8.2, end-of-construction conditions using a $\phi = 0$ type analysis with undrained shear strength usually applies only to starter dikes constructed on soft soils or to centerline embankment raises constructed upon fine tailings. If the embankment is constructed on a soft foundation, or for rapidly raised upstream embankments, a staged-construction analysis is required. Difficult to perform in practice, a staged-construction analysis requires that all the various sources of pore pressure be accounted for in a time-stepped way.

Long-term analyses using effective-stress strength parameters are relatively straightforward, but application of this condition must be examined carefully to determine whether its inherent assumptions are truly reasonable. For materials susceptible to major pore pressure increase during undrained shear, the possibility of even small or localized changes in loading may make it necessary to perform analyses using undrained strength parameters.

Considering the complexities involved, it is perhaps understandable that stability analyses sometimes become imbued with an undeserved sanctity when viewed in a theoretical context without at the same time accounting for their inherent uncertainties and simplifications. In fact, it is impossible to fully account for the myriad approximations in computational techniques, the vagaries of soil behavior and pore pressure response, and the variable

Table 8.2 Summary of Stability Analysis Conditions for Tailings Embankments

Analysis Condition	Applicability	Usual Strength and Pore Pressure Conditions
(1) End of construction	Starter dike on soft foundation	Undrained strength and $\phi = 0$ analysis
	Centerline raises on fine tailings	
(2) Staged construction	Embankments of any type on soft foundation	Incremental analysis accounting for changes in excess and static pore pressures as a function of time. Use ϕ_T or $S_u/\bar{\sigma}_c$ approach to account for pore pressure during shear
	Upstream embankments raised rapidly	
(3) Long-term	Maximum-height embankment where raises are constructed slowly	Effective-stress analysis using $\bar{\phi}$ and steady-seepage phreatic surface
	Maximum-height embankment where rapid changes in loading of loose material may occur	Total-stress analysis accounting for pore pressures during shear, usually using ϕ_T
(4) Rapid drawdown	None	

properties of the materials themselves. At best, stability analyses for tailings embankments must be considered semiempirical in nature. A true assessment of tailings embankment stability cannot be made without examining past precedents for similar types of embankments, and without incorporating experience and judgment to a considerable degree. Along with many other facets of tailings embankment design, and indeed of geotechnical engineering as a whole, stability analysis in its most complex form approaches the poorly defined boundary between technology and art.

9

Seismicity and Seismic Stability Analysis

We are looking at a new seismic scar that runs as far as we can see. . . . This fault, which jumped in 1915, opened like a zipper far up the valley, and, exploding into the silence, tore along the mountain base for upward of twenty miles with a sound that suggested a runaway locomotive. . . . The lesson is that the whole thing—the whole Basin and Range, or most of it—is alive.

John McPhee, *Basin and Range*

In Chapter 8, analysis of embankment stability under static conditions is discussed. In areas of relative tectonic quiescence, these static analyses alone may suffice, but this is probably the exception rather than the rule. An unfortunate consequence of mining geology is that the same zones of alteration and structural discontinuity often responsible for ore genesis are also often associated with tectonic activity that produces earthquakes. Such is the case for many if not most major mining districts in the western United States and Canada, Mexico, and Central and South America, among others. Stability of the embankment may be controlled in these cases, not by static loading conditions, but rather by behavior under earthquake shaking.

Earthquake damage to embankments can arise from two sources: actual ground rupture beneath the embankment and seismic shaking. With regard to ground rupture, embankments built across active faults have been designed to accommodate foundation displacement produced by fault movement during earthquakes. Sherard et al. (1974) discuss appropriate design procedures for the case of anticipated foundation displacement. However, this source of earthquake damage is obviously best handled by avoiding active faults during impoundment siting and will not be treated further here. By far the more common problem is to ensure that the embankment will be stable under anticipated levels of seismic shaking. The purpose of Chapter 9 is to present an overview of procedures for assessing embankment stability under earthquake loading.

The technology for seismic analysis of earth dam stability is a young one. Only within about the last 10 yr has there developed an understanding of actual soil behavior during earthquakes, and the application of these principles to even conventional earth dams is not straightforward. As is so often the case for tailings embankments, their unique characteristics tend to introduce an additional layer of complexity. At present, seismic analysis procedures used in practice are a mix between older empirical methods and newer sophisticated ones. It can be anticipated that this mix will change along with the further development of technology in this area.

Prior to seismic analysis, it is necessary to develop an estimate of the level of seismic shaking that the tailings embankment may experience. Thus, the initial portion of Chapter 9 presents elements of seismic risk assessment, with discussion of seismic analysis methods reserved for the second portion. Seismology in its application to embankment design is a field that integrates elements of structural geology, engineering geology, geophysics, and soil dynamics. Moreover, its application is highly site specific, and philosophical differences among various geographic regions exist. Again, the field is one where knowledge is developing rapidly. The intent of Chapter 9 is to aid in developing a basic familiarity with various techniques applied to seismic evaluation for tailings embankments and to present some fundamental data sources.

SEISMICITY

Seismic Parameters

Earthquakes are characterized in severity according to several different methods. Perhaps the most well known is the Richter scale, a logarithmic index of energy release that characterizes severity according to *magnitude* (M). Magnitude is determined by seismograph measurement of the amplitude of earthquake waves and is not dependent on the perception of the event by people or its effects on structures. Magnitude may be measured and expressed as either body-wave magnitude, referring to those waves that travel through the earth, or as surface-wave magnitude, derived from those waves traveling along the earth's surface.

Another measure of earthquake severity is *intensity* and is determined by the effects of the earthquake on people and structures, usually as measured on the Modified Mercalli (MM) or less commonly the Rossi-Forel scale. Both are summarized in Table 9.1. Intensity is an empirical measurement of earthquake effects. It does not provide a unique measure of severity, since reported effects diminish with distance from the earthquake epicenter. Moreover, the maximum intensity depends also on the depth of the earthquake and on surficial geology. In general, for earthquakes releasing the same total energy, maximum intensity decreases as the focal depth increases. Loose or soft surface soils generally tend to amplify ground motions at some frequencies, causing intensity to increase at some locations but not at others. These limitations notwithstanding, intensity is a widely used re-

Table 9.1 Modified Mercalli Intensity (Damage) Scale of 1931 (Abridged)

I.	Not felt except by a very few under especially favorable circumstances. (I Rossi-Forel scale)
II.	Felt by only a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing. (I–II Rossi-Forel scale)
III.	Felt quite noticeably indoors, especially on upper floors of buildings, but many people do not recognize it as an earthquake. Standing motorcars may rock slightly. Vibration like passing truck. Duration estimated. (III Rossi-Forel scale)
IV.	During the day, felt indoors by many, outdoors by few. At night, some awakened. Dishes, windows, and doors disturbed; walls make creaking sound. Sensation like heavy truck striking building. Standing motorcars rocked noticeably. (IV–V Rossi-Forel scale)
V.	Felt by nearly everyone; many awakened. Some dishes, windows, etc. broken; a few instances of cracked plaster; unstable objects overturned. Disturbances of trees, poles, and other tall objects sometimes noticed. Pendulum clocks may stop. (V–VI Rossi-Forel scale)
VI.	Felt by all; many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight. (VI–VII Rossi-Forel scale)
VII.	Everybody runs outdoors. Damage "negligible" in buildings of good design and construction; "slight to moderate" in well-built ordinary structures; "considerable" in poorly built or badly designed structures. Some chimneys broken. Noticed by persons driving motorcars. (VIII− Rossi-Forel scale)
VIII.	Damage "slight" in specially designed structures; "considerable" in ordinary substantial buildings, with partial collapse; "great" in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Persons driving motorcars disturbed. (VIII+ –IX Rossi-Forel scale)
IX.	Damage "considerable" in specially designed structures; well-designed frame structures thrown out of plumb; "great" in substantial buildings, with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken. (IX+ Rossi-Forel scale)
X.	Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations; ground badly cracked. Rails bent. Landslides considerable from river banks and steep slopes. Shifted sand and mud. Water splashed (slopped) over banks. (X Rossi-Forel scale)
XI.	Few, if any (masonry), structures remain standing. Bridges destroyed. Broad fissures in ground. Underground pipelines completely out of service. Earth slumps and land slips in soft ground. Rails bent greatly.
XII.	Damage total. Waves seen on ground surfaces. Lines of sight and level distorted. Objects thrown upward into the air.

porting scale and the only available measure of severity for earthquakes occurring prior to the advent of seismographic monitoring.

Because magnitude is a measure of energy release and intensity is based on observed effects, correlation of the two parameters is difficult. However, a commonly used approximate relationship follows the general form

$$M = \tfrac{2}{3} I_0 + 1 \text{ (Gutenberg and Richter, 1956)}$$

where I_0 = maximum MM intensity
M = magnitude

While either magnitude or intensity can be used to characterize the size of an earthquake, neither provides direct input to seismic embankment analysis. The most common engineering parameter used in determining embankment behavior is peak acceleration, a, usually expressed as a percent of gravity. Correlations between intensity and peak acceleration are necessarily crude because of the previously discussed factors that influence intensity and its general empirical nature. An approximate empirical relationship between acceleration and intensity is of the form

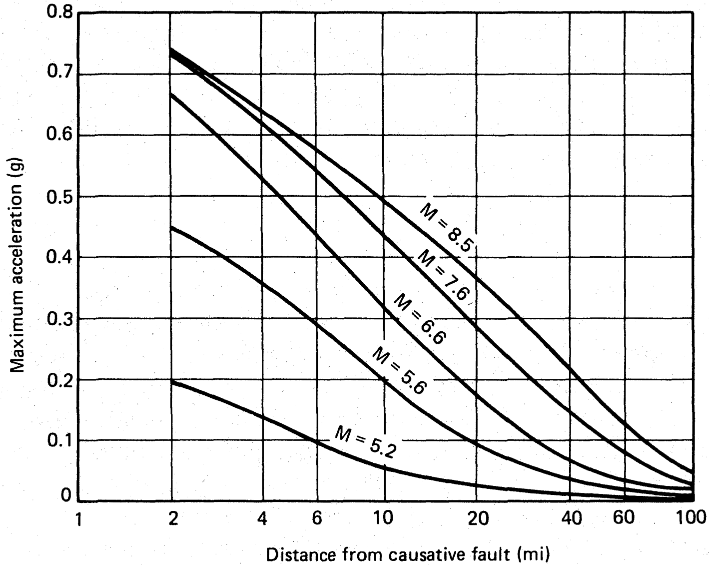
$$\log a = I/3 - \tfrac{1}{2} \text{ (Gutenberg and Richter, 1956)}$$

where I = local MM intensity
a = peak acceleration, cm/sec^2

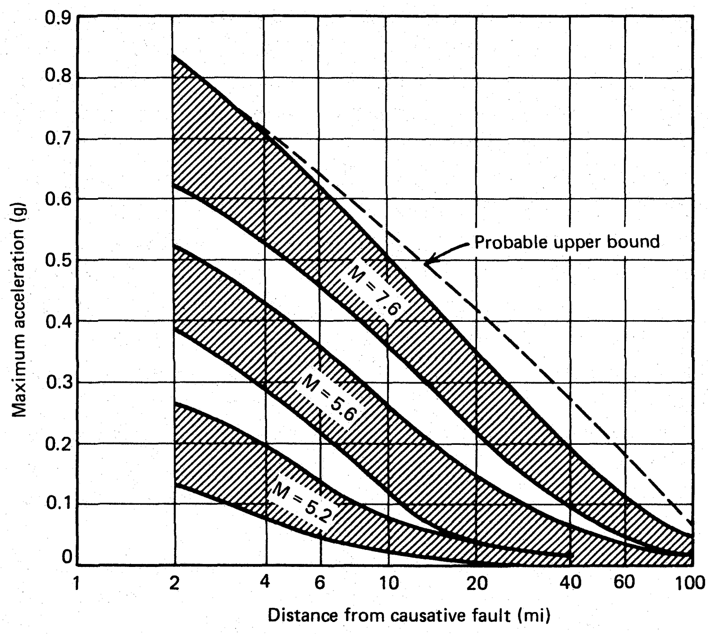
Relationships between magnitude and acceleration are available which are based on measurements of both parameters during significant earthquakes. It is also necessary to account for attenuation of acceleration with distance from the earthquake epicenter. Attenuation results from both geometric decay in energy as waves spread in a circular or elliptical pattern from the earthquake source, and also from damping due to the intrinsic properties of the rock through which the waves travel. A typical attenuation relationship for maximum acceleration in rock is shown in Figure 9.1 for the western United States (Schnabel and Seed, 1973). Relationships of this type apply strictly to the area in which they were derived and may be different in other geographic regions and geologic provinces (Nuttli, 1973). Also, attenuation functions may be dependent on the type of fault producing the earthquake (Bureau, 1978).

Estimation of Seismic Risk

Several procedures are available for estimating seismic risk in general and expected acceleration levels in particular for a given site. Taken as a group, these procedures form a hierarchy of increasing sophistication and increasing conservatism. At the crudest level are procedures for estimating design



(a)



(b)

Figure 9.1 Magnitude-acceleration and attenuation relationships for western United States. (From Schnabel and Seed, 1973.) (*a*) Average values for maximum acceleration in rock. (*b*) Ranges of maximum acceleration in rock.

acceleration on the basis of historic earthquake occurrences. More sophisticated are approaches that apply probabilistic methods to historic data in order to define the likelihood of occurrence of various acceleration levels. The most conservative technique applies deterministic methods based on postulated fault rupture to estimate the "maximum credible" earthquake and its associated site acceleration.

Historic Seismicity Approach

Evaluation of seismic risk for a particular site always begins by examining the record of historic earthquake occurrences in the vicinity, usually within a radius of about 100–200 mi. The presumption is that earthquakes which have occurred within the period of historic record provide a crude indication of the general level of seismicity in the area to be expected during the life of the embankment. Plotting of historic earthquake epicenters may also suggest the location of seismic source zones, such as active faults, that can be expected to generate significant activity in the future.

Catalogs of historic earthquakes can be obtained from various sources in the literature, including the "United States Earthquakes" series published annually by the U.S. Department of Commerce. Listings from several sources have been compiled by the U.S. Department of Commerce, National Oceanic and Atmospheric Administration (NOAA) in Boulder, Colorado. Computer printouts and plots of earthquake information are made available for any specified geographic location in the United States and portions of neighboring countries.

Examination of historic earthquake data is useful when performed in terms of the maximum acceleration experienced by the site during the period of historic record. Site accelerations can be estimated by applying previously described magnitude, intensity, and attenuation relationships to individual historic events. In many remote mining areas, the period of record is relatively short, usually not much more than 100 yr. If it is assumed for illustrative purposes that the maximum acceleration experienced by the site in a 100-yr period of record corresponds in a general way to the 100-yr return-period acceleration (which is statistically incorrect, but perhaps reasonable as a very crude first-order approximation), then the exceedance probability associated with the maximum historic site acceleration can be estimated.

For example, suppose that the maximum acceleration experienced at a particular site is 0.05 g over a 100-yr period of record, and assume that this represents the 100-yr return-period event. Further, suppose that the tailings impoundment under consideration will have an active (saturated) life of 20 yr. Using probability relationships presented in Chapter 4, it can be shown that the 0.05 g maximum historic acceleration would have an 18% chance of being equaled or exceeded during the active life of the facility. In many cases, this risk would be too high to justify 0.05 g as the design acceleration, and a higher design acceleration with a longer return period would be required to produce a more acceptable failure probability. Although historic data alone are not always sufficient to define design seismic acceleration, this example illustrates that historic earthquake data interpreted in terms of maximum site acceleration are useful to at least provide a lower bound to the design acceleration value.

Related to the historic earthquake approach are seismic zoning maps such as shown in Figure 9.2. A region is divided into zones that reflect in an approximate way relative numbers and sizes of historic earthquakes, and a seismic coefficient is assigned to each zone. The seismic coefficient in Figure 9.2 is a somewhat arbitrary number not necessarily intended to represent actual expected ground motions, as discussed by Seed (1973), but rather to be used as input to empirical, pseudo-static methods of embankment analysis. Seismic coefficients presented on zoning maps such as that shown in Figure 9.2 should not be confused with true earthquake acceleration.

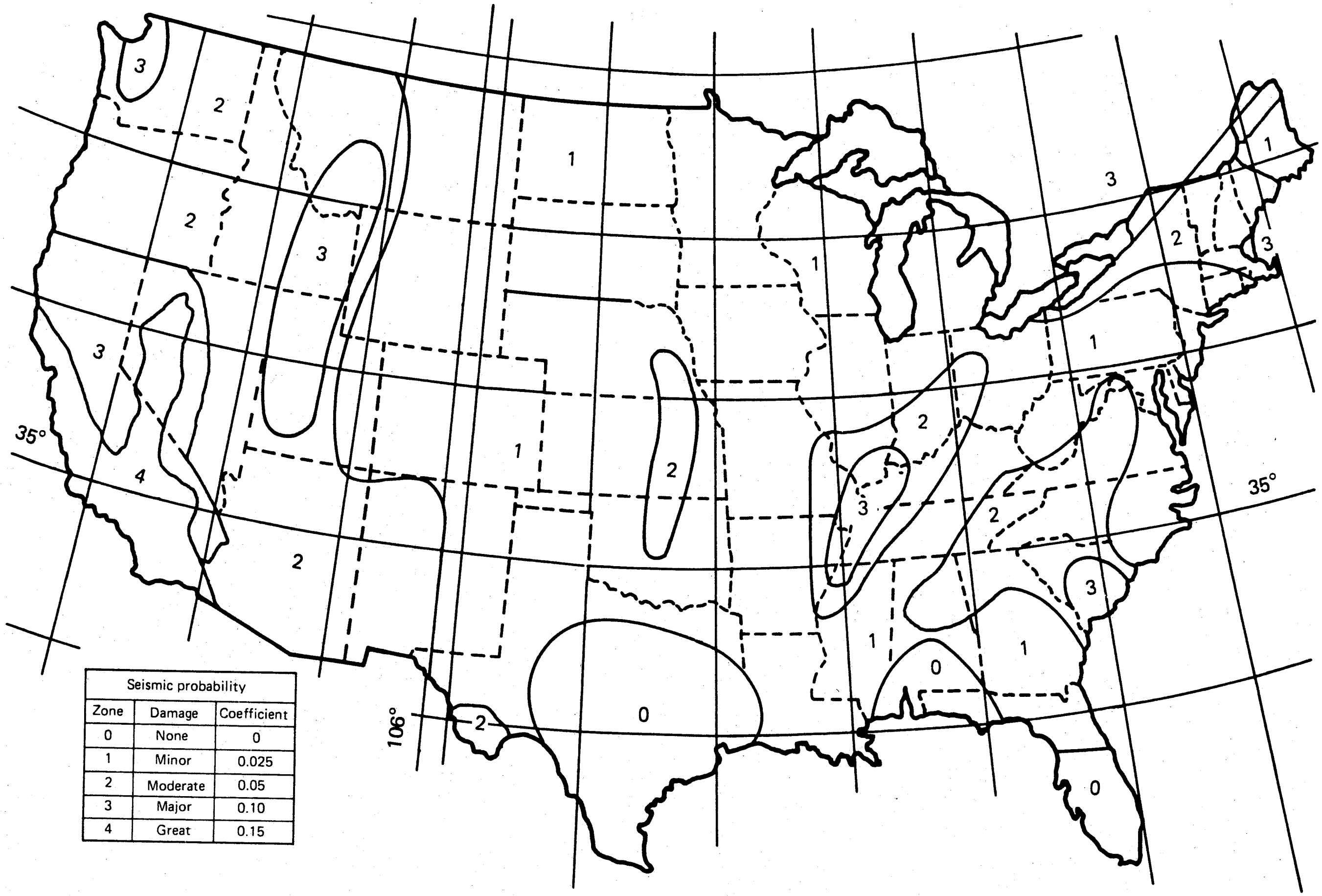
Other zoning maps, such as those presented by Nuttli (1973) for the central United States, do reflect actual accelerations. Zoning maps of this class are often based on detailed consideration of specific large historic earthquakes. A remaining problem with zoning maps of this class, however, is that the occurrence probability of the recommended design acceleration value is not rigorously defined. Therefore, failure probabilities to define the degree of risk cannot be ascertained.

Probabilistic Methods

Probabilistic seismic risk evaluations are a significant refinement to the historic earthquake approach. While still based in large part on the record of historic earthquake occurrences, the aim of probabilistic methods is to associate a unique probability of occurrence with each possible level of acceleration. This type of approach is based on the concept that earthquakes are possible in almost any area, but that their likelihood of occurrence is governed by established seismicity patterns. By defining a probability for a given level of acceleration, the designer is better able to quantify the risk associated with a particular design acceleration level.

The results of generalized seismic risk assessments are often portrayed as maps showing acceleration for various probability levels. Probability is often expressed as return period, which, as can be recalled from Chapter 4, is the inverse of the annual probability of occurrence or exceedance. Maps showing statistically derived acceleration levels for various return periods are shown in Figures 9.3–9.7 for the United States, Canada, and Mexico.

Algermissen and Perkins (1976), Milne and Davenport (1969), and Esteva (1976) describe the derivation of probabilistic seismic risk maps. Although details of the procedures vary, they share many common elements. Initially, a region is divided into a number of earthquake *source zones* by defining discrete subregions having significantly different levels of historic seismic



Seismic probability		
Zone	Damage	Coefficient
0	None	0
1	Minor	0.025
2	Moderate	0.05
3	Major	0.10
4	Great	0.15

Figure 9.2 Seismic zoning map of the United States, showing seismic coefficients for pseudo-static analyses. (From Corps of Engineers, 1977.)

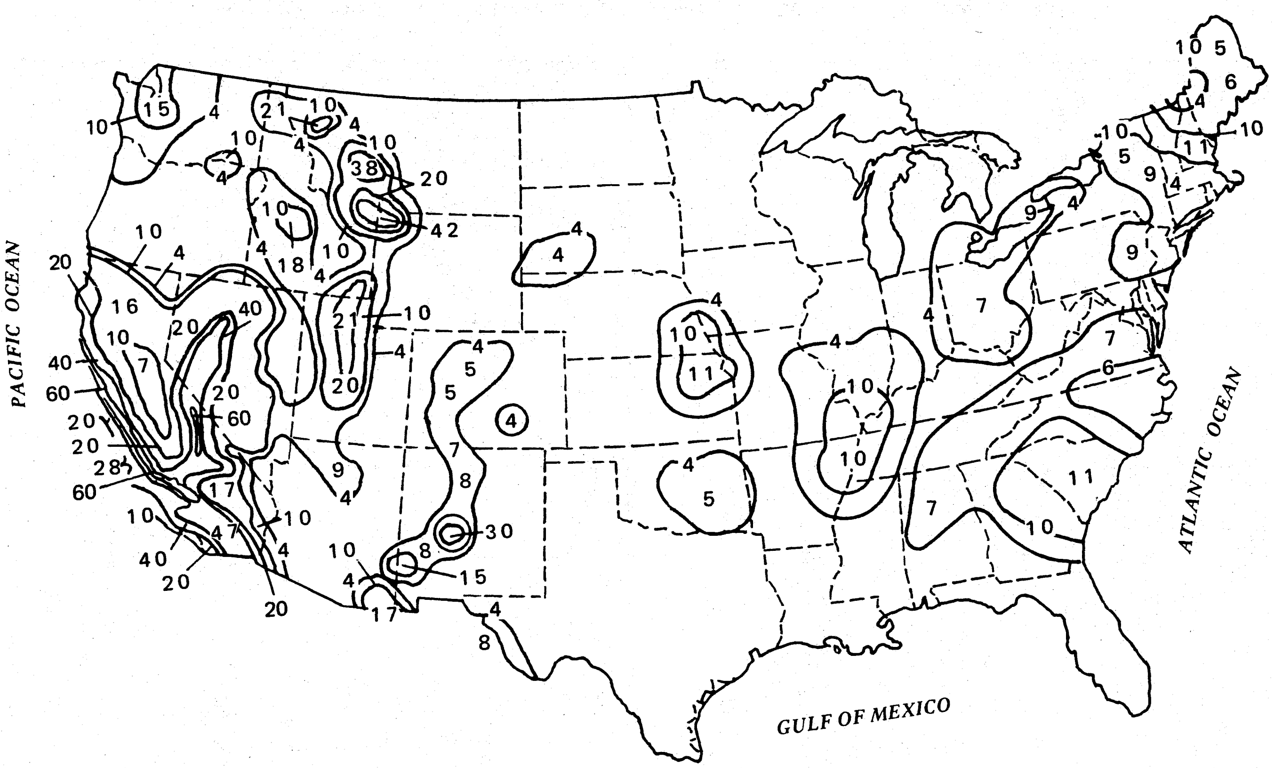
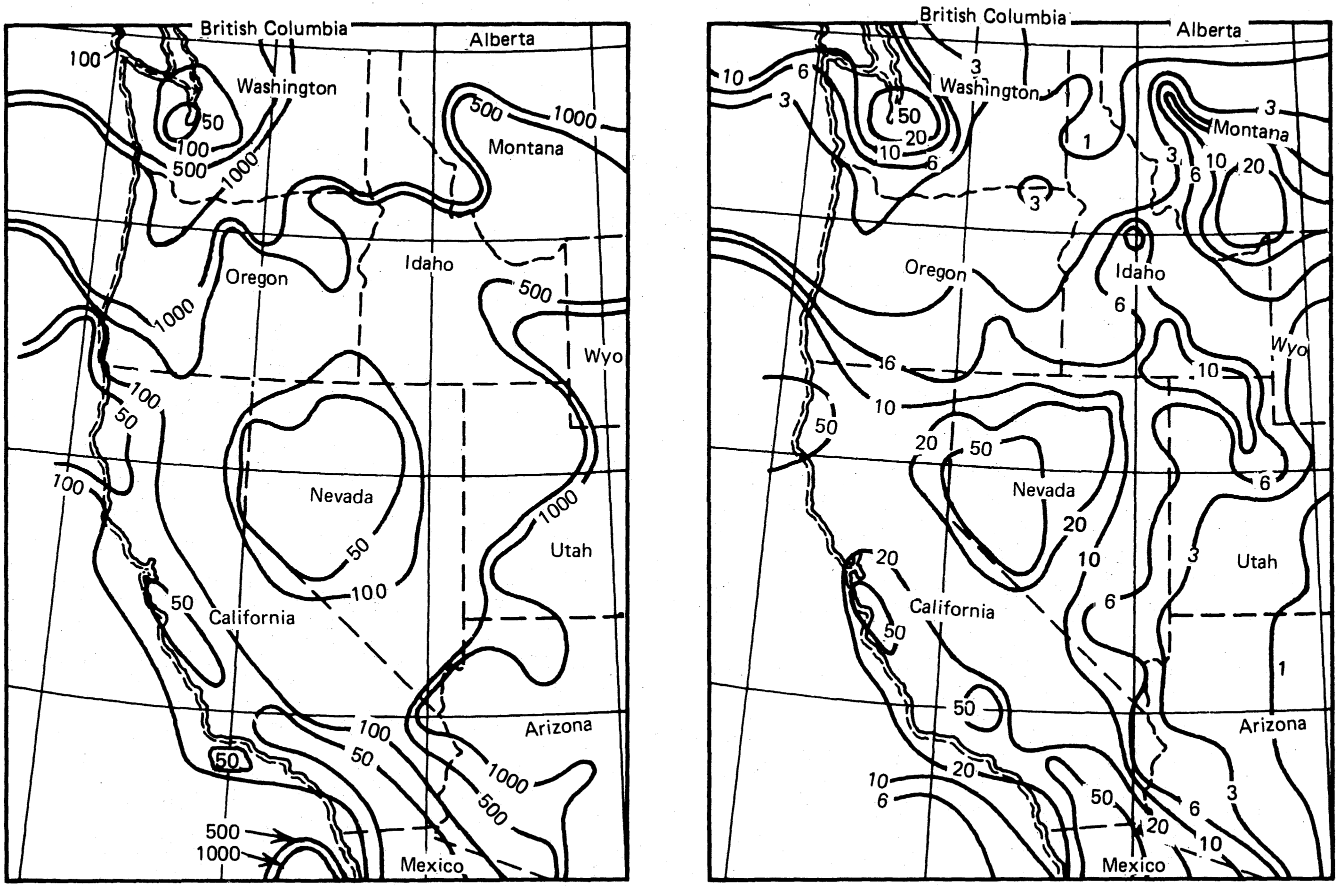


Figure 9.3 Bedrock accelerations in the United States. (percent *g*) for 475-yr return period. (From Algermissen and Perkins, 1976.)



(*a*) (*b*)

Figure 9.4 Earthquake return periods and accelerations for Western United States. (From Milne and Davenport, 1969.) (*a*) Return period (years) for $a = 0.10\ g$. (*b*) Acceleration (percent g) for 100-yr return period.

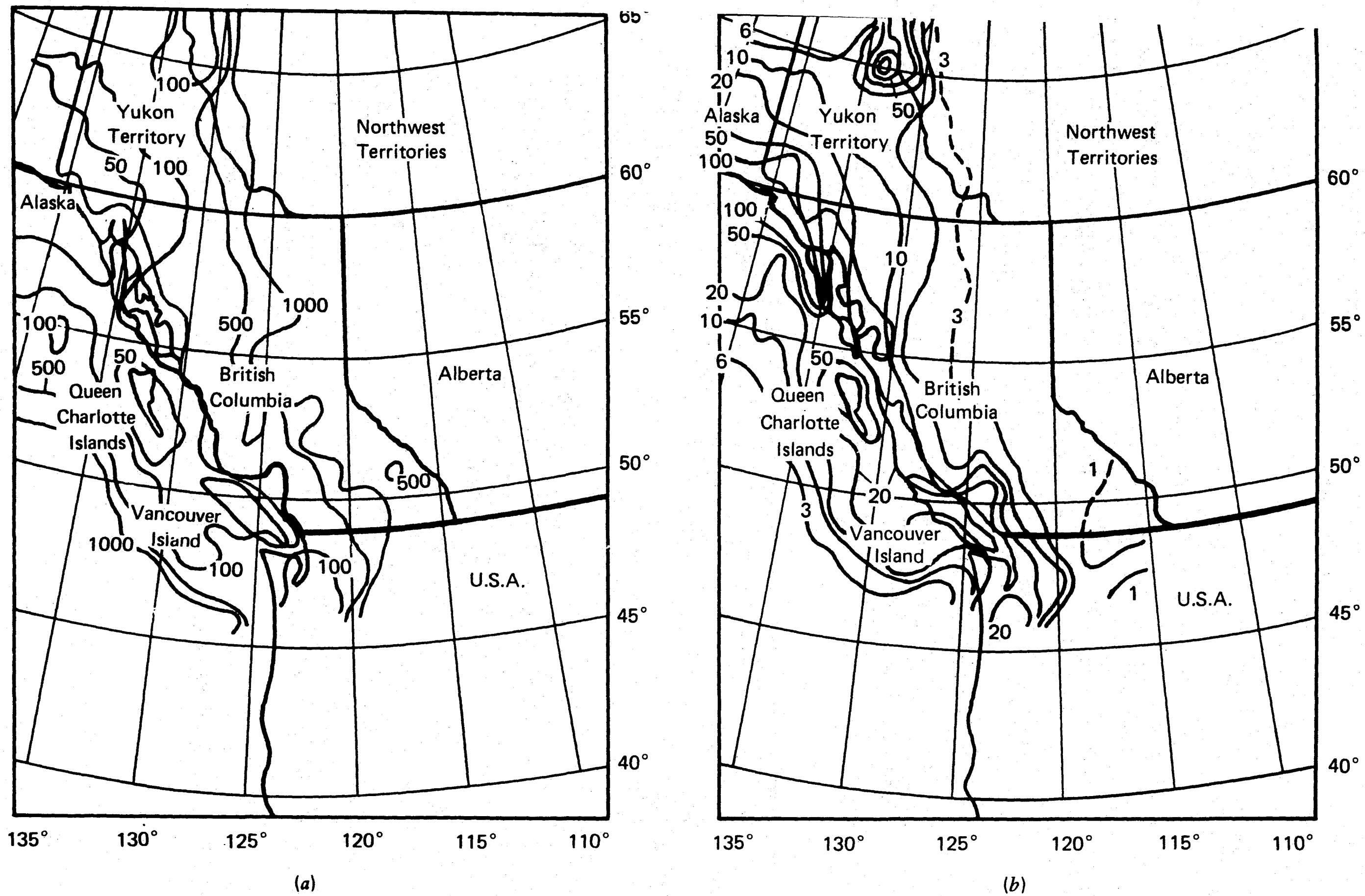
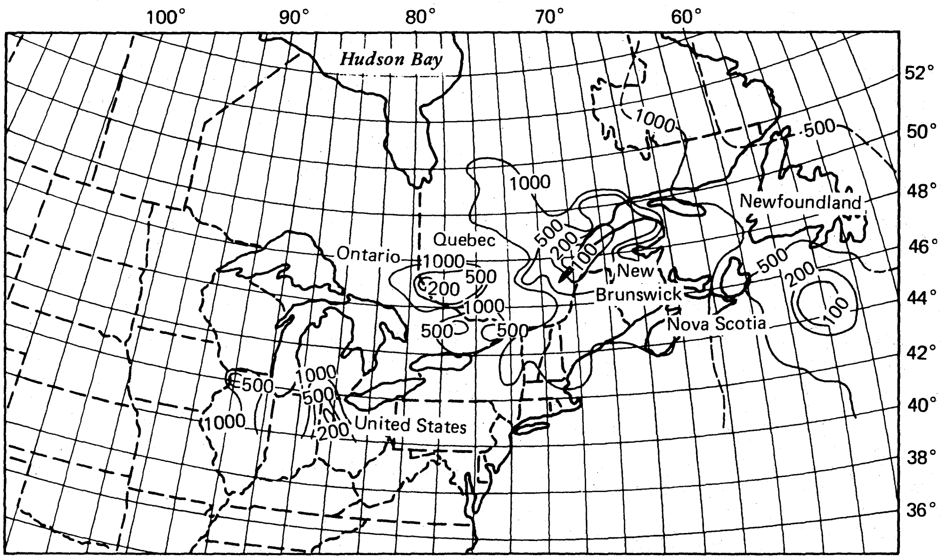


Figure 9.5 Earthquake return periods and accelerations for Western Canada. (From Milne and Davenport, 1969.) (*a*) Return period (years) for $a = 0.10\,g$. (*b*) Acceleration (percent g) for 100-yr return period.



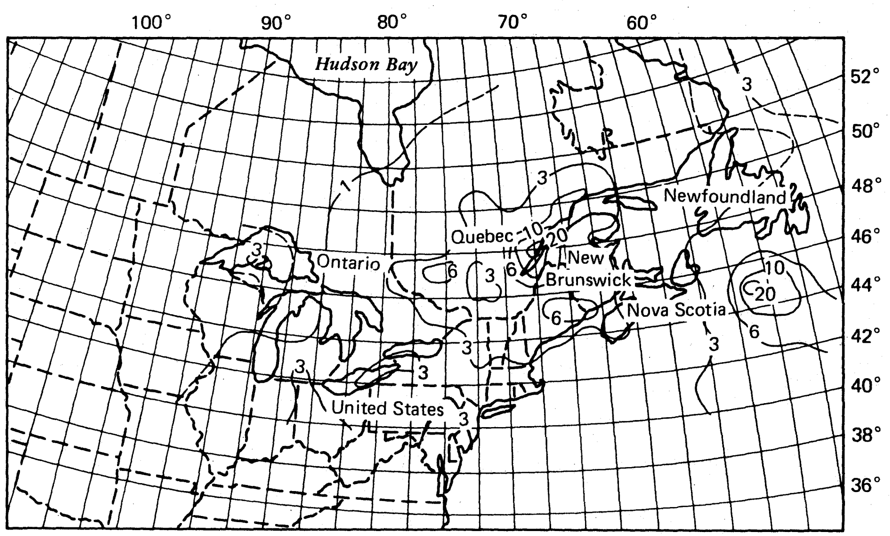


Figure 9.6 Return periods and accelerations for Eastern Canada. (From Milne and Davenport, 1969.) (*a*) Return period (years) for $a = 0.10\,g$. (*b*) Acceleration (percent g) for 100-yr return period.

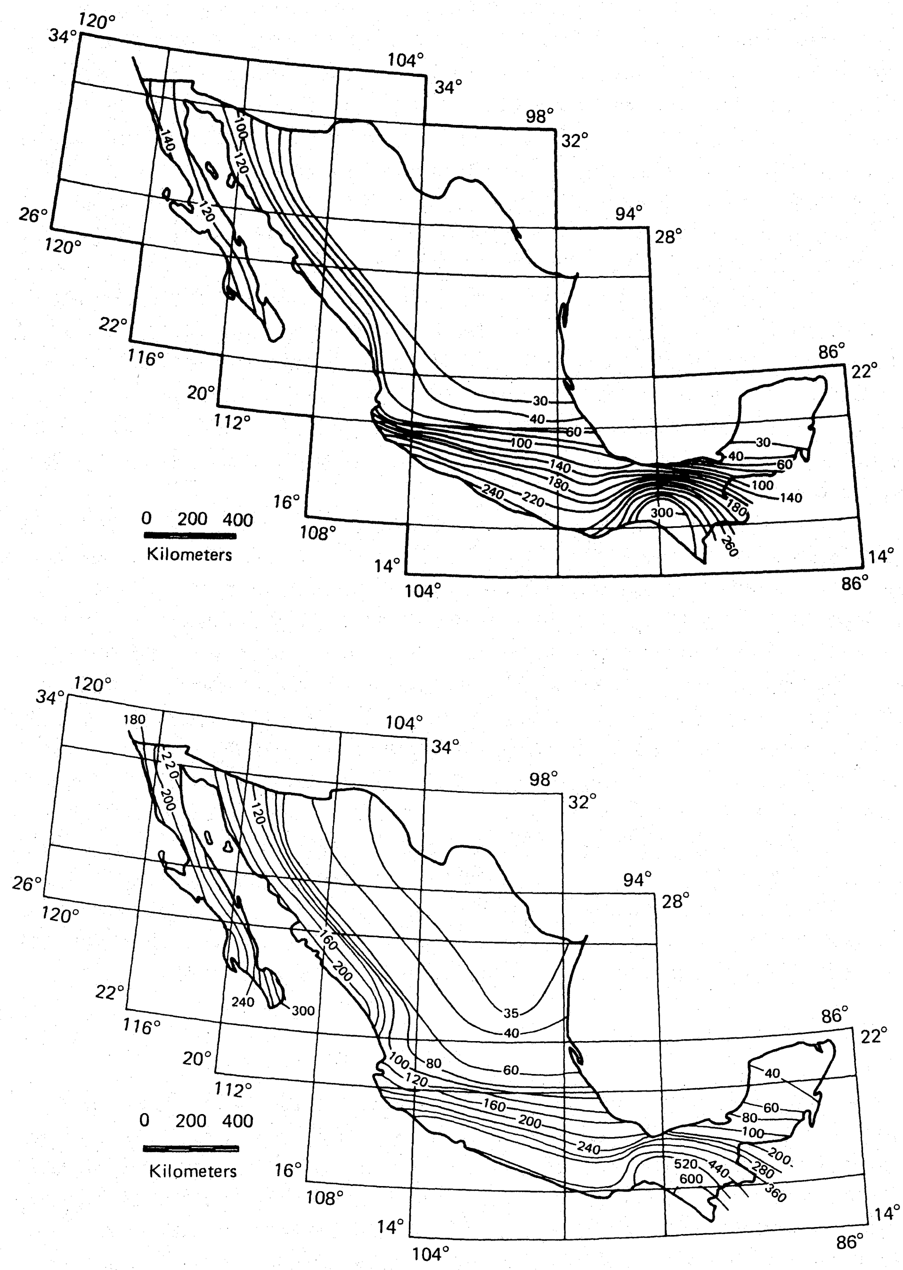


Figure 9.7 Accelerations in cm/sec² for Mexico. (From Esteva, 1970.) (*a*) 100-yr return period. (*b*) 500-yr return period.

activity. Also, an attenuation function (for example, as shown in Figure 9.1) is adopted. Within each source zone an earthquake recurrence rate is derived from the historic data, supplemented by geologic evidence for older events where available. Then a specific point is selected, and the exceedance probability of a given level of acceleration is determined for that point, assuming that the distribution of earthquakes with time in each source zone can be modeled by a probability function usually corresponding to a Poisson process. Since, for example, a given acceleration can be produced by either a small nearby earthquake or a large distant one, its probability of exceedance at the specified point is the sum of the probabilities of accelerations derived from large and small events from various source zones, accounting for attenuation with distance to the point under consideration. The process is repeated for a number of points sufficient to construct a map of acceleration contours. Similar procedures can be used to study a specific site under the influence of source zones that represent known active faults (Donovan and Bornstein, 1978).

The probabilistic approach represents a major refinement over less sophisticated methods of historic earthquake risk assessment. The derivation of earthquake recurrence rates for individual source zones accounts for the likelihood of earthquakes larger than those having occurred in the past by extrapolating from the period of historic record. However, criticism has sometimes been leveled against probabilistic methods on the basis that historic data are too limited to reflect geologic factors. For example, a fault could be present near a site showing evidence of geologically recent activity, such as fresh scarps. Although this fault could represent a major earthquake hazard, it might not be accounted for in a probabilistic risk analysis unless it had generated seismic activity during the brief period of historic record or unless recurrence rates could be inferred from geologic evidence. These objections can be addressed by use of a deterministic approach. Deterministic methods are a necessary adjunct to probabilistic approaches when active faults are known to be present in the site vicinity.

Deterministic Methods

In contrast to the probabilistic approach, deterministic methods for assessment of seismic risk are so named because they do not account for the likelihood of occurrence of a predicted acceleration. The resulting deterministic acceleration is considered to be the maximum that could occur on the basis of geologic data, without regard for past historic events. Thus, the acceleration produced by this procedure is often referred to as "maximum," "maximum probable," or "maximum credible." This approach is philosophically analogous to the Probable Maximum Flood concept discussed in Chapter 4.

Use of the deterministic approach is possible where potential earthquake sources can be identified on the basis of geologic or tectonic evidence. In

Table 9.2 Summary of Active Fault Mapping, Western United States

State	Reference
Montana	Witkind, 1975a
Wyoming	Witkind, 1975b
Idaho	Greensfelder, 1976
	Witkind, 1975c
Colorado	Kirkham and Rogers, 1978
	Witkind, 1976
Utah	Anderson and Miller, 1979
Nevada	Slemmons, 1967

practice, the deterministic method is most commonly applied in regions where active earthquake-generating faults exist and are exposed at the surface. In relation to seismicity assessment, *active faults* are most commonly defined as those showing evidence of past displacement during Holocene time, or within roughly the last 10,000–11,000 yr (Slemmons and McKinney, 1977). The presumption here is that, if a fault has undergone displacement during recent geologic time, there is some possibility, however small, that it could again rupture during the life of the facility under consideration, producing a major earthquake.

A number of methods are available for evaluating the presence of faults and determining their potential activity. Sources in the literature can provide useful compilations of known or suspected active faults, such as summarized in Table 9.2 for some of the seismically active mining regions in the western United States.

If the site under consideration is determined to be near one or more mapped active faults, remote sensing techniques are often applied for further evaluation of fault age and activity. As summarized by Russell et al. (1976), these techniques include ordinary black-and-white and color aerial photography; infrared or other satellite imagery, such as the ERTS series; side-looking airborne radar (SLAR); and low-sun aerial photography. ERTS and SLAR imagery is available for some areas through government sources. Low-sun angle photography is relatively inexpensive and very effective for identifying fault scarps showing signs of recent movement. This method, described by Slemmons (1977) and Sherard et al. (1974), is perhaps the single most useful remote sensing technique for active fault identification.

Still further evaluation may be necessary from field studies if unambiguous conclusions regarding fault age cannot be drawn from literature sources or remote sensing techniques. Field studies may include extensive trenching, very detailed mapping of offset soils on scarps, and carbon dating. Bucknam and Anderson (1978) and Wallace (1977) describe the use of scarp angle in unconsolidated sediments for determining fault scarp age.

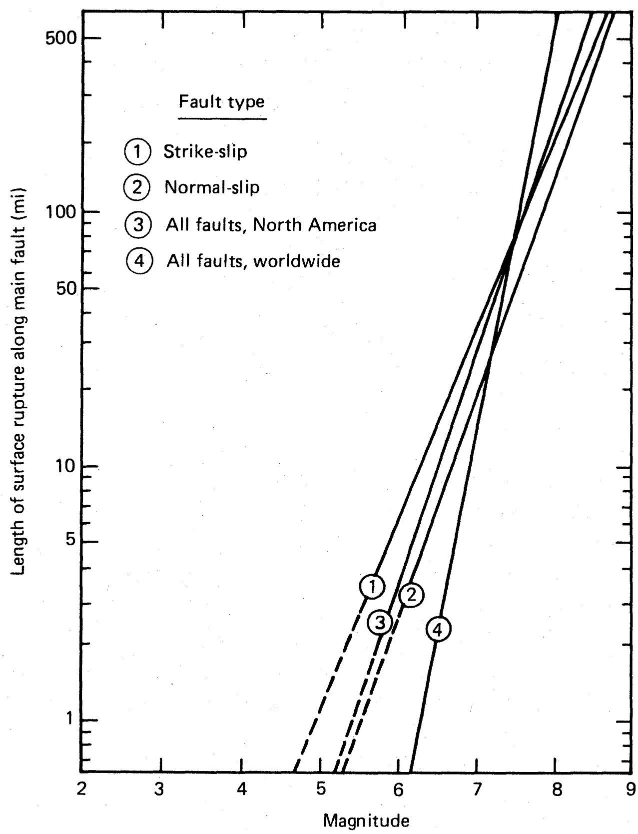


Figure 9.8 Fault rupture length versus magnitude. (From Slemmons, 1977.)

The result of fault assessments carried out by one or more means discussed above, in conjunction with background studies on regional geology and seismotectonics, will be to identify the location and length of active faults as well as the type or direction of movement they have experienced. From this information, methods are available for estimating the maximum earthquake magnitude that could result from rupture on a particular fault. Figure 9.8 shows an empirical relationship between earthquake magnitude and fault rupture length presented by Slemmons (1977). In using Figure 9.8, it is customary to assume that rupture along one-half of the total fault length could occur. The maximum earthquake magnitude so obtained can then be attenuated to the site using relationships such as shown in Figure 9.1 to provide a deterministic estimate of the maximum credible acceleration.

Selection of Design Earthquake

Evaluation of seismic risk by historic earthquake, probabilistic, and deterministic procedures will yield different estimates of potential site acceleration. Bell and Hoffman (1978) note that there is no single unique design earthquake and that any given level of seismicity can be specified only in conjunction with its associated risk. The problem remains to select a value appropriate for use as input to seismic stability analysis. This problem can be best addressed by considering the accelerations produced by the hierarchy of methods in conjunction with their occurrence or exceedance probabilities.

If an embankment is designed to withstand a given design acceleration, and if this acceleration is exceeded, it can be assumed that failure will result. To this extent, the probability of exceedance for a particular acceleration can be considered equivalent to the probability of embankment failure. By comparing accelerations of different return periods or failure probabilities, and by accounting for the consequences of embankment failure, exercise of judgment usually produces a reasonable design acceleration value.

Selection of a design acceleration can be illustrated by an example involving a tailings embankment in Montana with a planned active life of 20 yr. Site soils are sufficiently shallow and dense so that bedrock accelerations provide a reasonable approximation of accelerations at the ground surface. Initially, studies of regional geology and seismotectonics were performed to establish a framework for interpreting historic earthquake and fault information. Estimates of site acceleration were then made by all three methods discussed above.

In evaluating historic earthquakes, it was determined that the 1959 Hebgen Lake earthquake, the largest recorded in the Intermountain Seismic Belt, produced the most severe historic shaking at the site. Isoseismal mapping of reported intensities from this event was used to determine the intensity experienced at the site, which was in turn converted to acceleration. Statistical mapping was then referenced to determine return periods for various other levels of acceleration, and their annual probabilities of occurrence were used to calculate exceedance, or failure, probabilities according to methods presented in Chapter 4. Finally, fault mapping provided evidence of two active faults within 30–50 mi, and maximum site accelerations were determined for each using the deterministic approach. The resulting accelerations are shown in Table 9.3.

Inspection of Table 9.3 indicates that predicted accelerations span a considerable range, from 0.05 g to 0.13 g. In this case, the maximum historic acceleration of 0.05 g provides a reasonable lower bound to the acceleration appropriate for design. The value of 0.13 g that could result from half-length rupture on the nearest active fault is considered to provide an upper bound in this case.

Within these limits are three return-period accelerations. The 18% failure probability for the 100-yr event was judged to be an unacceptable risk, and the 475-yr acceleration of 0.08 g was considered to be too low in light of the presence of known active faults. Ultimately, the 1,000-yr return-period acceleration of 0.10 g was selected as the design value. The 2.0% failure probability was judged to be sufficiently low in light of the consequences of

Table 9.3 Example Comparison of Accelerations by Various Methods

Method	Source	Acceleration (g)	Return Period (yr)	Annual Probability of Occurrence	Exceedance or Failure Probability
(1) Maximum historic	Isoseismal mapping	0.05	—	—	Relatively high
(2) Probabilistic	Figure 9.4a	0.06	100	0.01	0.18
	Figure 9.3	0.08	475	0.002	0.04
	Figure 9.4b	0.10	1,000	0.001	0.02
(3) Deterministic	Active fault mapping	0.11–0.13	—	—	Practically zero

failure at this particular site. In addition, the selected design value, while not as large as that produced by the magnitude 7.0 maximum credible earthquake on the controlling active fault, would be consistent with the occurrence of an only somewhat smaller magnitude 6.8 event.

There are few rigid guidelines for establishing appropriate levels of risk for selecting design acceleration values, but in general exceedance probabilities should be at most a few percent over the active (saturated) life of the embankment in most cases. Although the general procedures followed in the above example are probably typical of common tailings embankment design practice, acceptable risk can be determined only in the context of the particular site conditions and consequences of failure in each individual case.

In summary, seismic risk should be assessed by pursuing several approaches. Initially, the regional geologic and seismotectonic setting should be established. In regions of low seismicity, extensive evaluation may not be required beyond an examination of historic earthquakes. Here the largest historic earthquake effects experienced at the site may provide a reasonable indication of seismic risk, particularly if the period of record is long. In more seismically active areas, probabilistic approaches are very valuable to the extent that they account for the likelihood of occurrence of earthquakes from several surrounding areas and also account for the probability of events larger than those that have occurred in the period of historic record. If the site lies within a region of known active faults, probabilistic approaches should be supplemented by detailed geologic studies and evaluation of seismic risk by deterministic procedures. The maximum credible earthquake may provide the only reasonable design value where the consequences of failure are intolerable in terms of loss of life, economic loss, or environmental damage.

The level of seismic risk assessment also depends on the type and detail of seismic stability analysis anticipated. In areas of low seismicity where only pseudo-static stability analyses will be performed, use of a seismic coefficient from zoning maps may be acceptable, recognizing that both the analysis and the coefficient are largely empirical. More sophisticated liquefaction analyses, however, require more detailed evaluation of the seismic input data. For detailed dynamic analyses, it is necessary to account for such additional factors as the response of the foundation to bedrock accelerations, and the expected duration of shaking.

SEISMIC STABILITY ANALYSIS

Methods for determining the ability of earth structures to resist earthquakes have been the subject of considerable recent discussion and investigation. Tailings embankments have played no small role in spurring the development of this technology, since it was in part the catastrophic failure of upstream-constructed tailings embankments in the 1965 Chilean La Ligua earthquake that brought the seismic liquefaction problem to light (Dobry and Alvarez, 1967). To the extent that tailings embankments are often constructed using relatively loose tailings in one form or another—either hydraulically deposited or cycloned materials—they may be particularly susceptible to the buildup of internal pore pressures during strong seismic shaking. In the extreme case, these pore pressures can induce liquefaction in all or part of the embankment, leading to catastrophic failure by flow-type sliding.

Response of Soils to Cyclic Loading

Chapter 8 discusses various classes of pore pressure produced by phreatic conditions, application of load, and statically applied shear stresses. To these sources of pore pressure must now be added pore pressures due to cyclic loading.

When an element of saturated soil is subjected to shaking, as simulated in the laboratory, the directions of the applied principle stresses change. In a laboratory triaxial or direct shear test sample subjected to undrained cyclic loading, the resulting internal shear stress reverses directions. This process is referred to as *stress reversal,* and the number of cycles of stress reversal applied to the test sample varies according to the duration of the expected earthquake. The level of stress applied during the test, or the *cyclic shear stress,* is related to the acceleration of the earthquake being modeled.

The response of the soil to cyclic stresses depends on a number of factors, but on the most elementary level, soil type can be considered the fundamental parameter. For saturated cohesive soils (clays) subject to undrained cyclic loading, pore pressures are generated but usually to only a limited degree. As a result, the static undrained shear strength of a clay may be reduced somewhat by cyclic loading, but the soil retains its basic integrity and liquefaction does not occur. Consequently, clayey soils are not usually of primary concern in determining embankment stability under earthquake loading conditions (Seed et al., 1978).

Another class of materials is represented by very coarse, permeable deposits of cohesionless material. In the context of tailings embankments, this class best applies to embankments constructed of coarse mine waste. High permeability of such materials prevents undrained conditions from developing during cyclic shear; in effect, excess pore pressures generated by cyclic loading dissipate as fast as they are generated (Sadigh et al., 1978). Here, too, liquefaction susceptibility is seldom of major concern.

Dense sands represent still another class. Most commonly such materials are encountered in centerline- or downstream-type tailings embankments constructed of cycloned tailings sand placed in lifts and compacted under controlled conditions. Pore pressures are generated in response to undrained cyclic loading and may momentarily reach high values. However, the tendency of the dense sands to dilate quickly reduces the pore pressure, causing the material to stabilize. Even after a large number of cyclic stress reversals, the cumulative pore pressure is limited, although the sample may experience some degree of permanent deformation. Severe loss of strength does not occur under these conditions, so again this class of material does not usually present dramatic problems for seismic stability. In applying criteria from several sources to tailings embankments, Mittal and Morgenstern (1977) conclude that compaction of tailings sands to relative densities in excess of about 50–60% is sufficient to preclude liquefaction under accelerations less than about 0.10 g, and that compaction to relative densities in excess of 75% should prevent liquefaction under higher accelerations. In questionable cases, the relative density required to prevent liquefaction can be determined from dynamic analyses of the embankment. The minimum relative density required to prevent liquefaction depends not only on tailings characteristics but also on dam height, saturation level, and ground motion characteristics, including acceleration and duration.

The most problematic class of materials consists of loose, saturated sands. At typical relative densities of 30–50% for uncompacted tailings, this category applies to most upstream embankments, as well as downstream and centerline types constructed of uncompacted tailings sands subjected to strong and/or extended seismic shaking. In undrained cyclic loading, pore pressures are generated during each application of cyclic stress reversal, and the effects become cumulative during the duration of shaking. This phenomenon is illustrated in Figure 9.9 by typical cyclic triaxial test results on tailings presented by Finn (1980). Under the application of uniform cycles of cyclic stress, pore pressures may reach the level of the applied confining stress. At this point the effective stress within the sample is momentarily zero, and *initial liquefaction* is said to have occurred. Following initial liquefaction, the sample may rapidly undergo excessive strain with only a few additional cycles of stress. Failure for materials of this class may be defined either at the point of initial liquefaction or at some arbitrary strain, commonly taken as 5% single-amplitude strain.

Pore pressure response in uncompacted tailings is influenced by variables related to stresses applied under both static and dynamic conditions, including the magnitude of initial effective consolidation stress, consolidation stress anisotropy, magnitude and direction of applied cyclic shear stress, and the number of cycles of stress reversal. These factors pertain to the way in which initial static stresses and earthquake-induced cyclic stresses are simulated in the laboratory. However, Seed (1976, 1979a) reviews other factors related to the nature of the material and the deposit that also govern liquefaction susceptibility. Many of these factors suggest that the liquefaction susceptibility of tailings may be greater than that for natural soil deposits at the same relative density. These factors include:

Grain Characteristics and Gradation. Grain shape and size characteristics can cause cyclic stress ratios required for initial liquefaction to vary by ±20% for the same relative density.

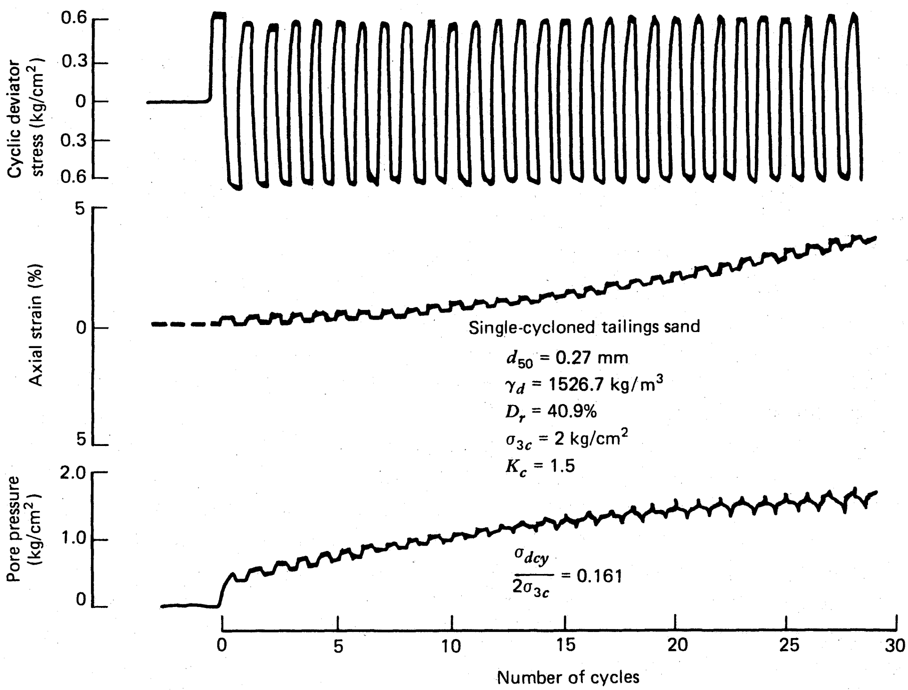


Figure 9.9 Typical cyclic triaxial test results for tailings sand. (From Finn, 1980.)

Method of Deposition. Laboratory studies on samples prepared to the same relative density but using different procedures suggest that cyclic stress ratios required for liquefaction can vary by as much as 200%. Sedimentation of material in water, similar to tailings deposition, produces the greatest liquefaction susceptibility.

Aging. The longer a deposit has experienced sustained load, the greater its liquefaction resistance is likely to be. Obviously much younger than natural deposits, tailings deposits are likely to have greater liquefaction susceptibility on this basis.

Previous Strain History. Natural deposits having experienced previous seismic shaking may exhibit increased liquefaction resistance during subsequent earthquakes, even if no densification occurred. This is unlikely to be a mitigating factor for most tailings deposits.

Overconsolidation. Some natural deposits have been overconsolidated because of geologic processes or groundwater fluctuations. This tends to increase liquefaction resistance. These beneficial effects are unlikely to accrue to the typically normally consolidated tailings deposit.

These factors suggest that comparisons between natural soil deposits and tailings deposits at equivalent relative densities must be made with caution, and that tailings samples for laboratory testing must be prepared in a way that reflects their depositional and environmental character in situ.

Methods of Seismic Analysis

Against this backdrop of soil behavior under cyclic loading, various procedures for analysis of embankments under seismic loading can be compared and evaluated. The various methods differ principally in the way they consider, or fail to consider, actual soil behavior under dynamic loading. It is useful to define three categories of analysis: those for foundation materials, those for compacted or clayey embankments, and those for embankments of uncompacted tailings.

Foundation Analysis

While the previous discussion has focused on the seismic behavior of embankment materials, it is also necessary to account for possible liquefaction of foundation materials upon which the embankment is constructed, particularly when the foundation consists of loose, cohesionless soils that are initially saturated or may become so under the influence of embankment seepage.

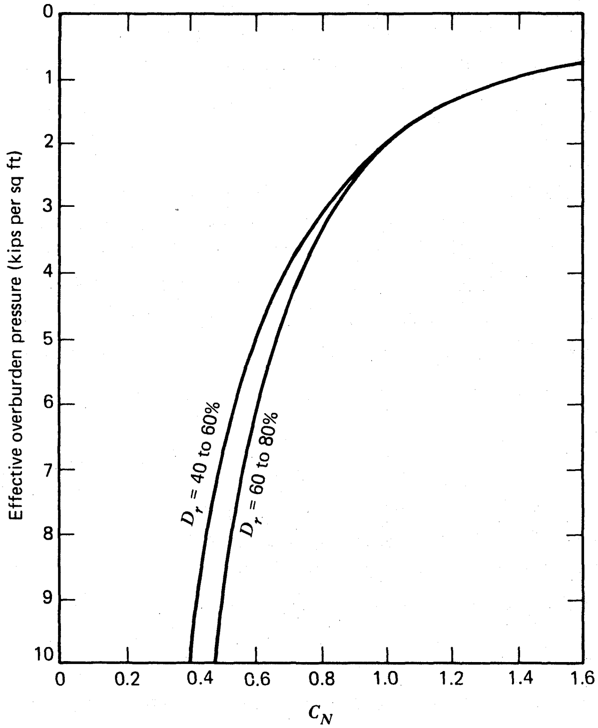
Simplified methods for evaluating liquefaction potential on level ground have been developed by Seed and Idriss (1971) and Seed et al. (1983) by comparing the characteristics of natural deposits that have been observed to either liquefy or remain stable under various levels of seismic shaking. Applying these procedures to an embankment foundation requires the assumption that if the deposit is shown to be stable in its prior state it will also remain stable under the influence of the embankment, provided that increased foundation saturation levels are accounted for.

Characterization of the deposit is on the basis of blowcount, N, from Standard Penetration Test (SPT) results corrected to account for depth according to the following relationship (Seed et al., 1983):

$$N_1 = C_N \times N$$

where N_1 = modified penetration resistance, blows per foot
N = SPT blowcount using a rope and drum system in borings drilled with mud. Measured blowcounts in the depth range 0–10 ft should be multiplied by 0.75.
C_N = correction factor given in Figure 9.10a

Characterization of the earthquake is made by relating the average cyclic shear stress ratio induced by the earthquake ($\tau_{av}/\bar{\sigma}_0$) to acceleration. This procedure is based on estimates which show that the equivalent uniform cyclic shear stress is about 65% of the maximum shear stress induced by the



(*a*)

Figure 9.10 Simplified liquefaction analysis data. (From Seed, et al., 1983.) (*a*) Recommended curves for determination of C_N. (*b*) Chart for evaluation of liquefaction potential for different magnitude earthquakes.

earthquake. A relationship between average cyclic shear stress ratio and acceleration of the following form results (Seed and Idriss, 1971):

$$\frac{\tau_{av}}{\bar{\sigma}_0} \cong 0.65\,(a_{max})\left(\frac{\sigma_0}{\bar{\sigma}_0}\right)(r_d)$$

where a_{max} = Maximum acceleration at the gound surface as a fraction of gravity acceleration

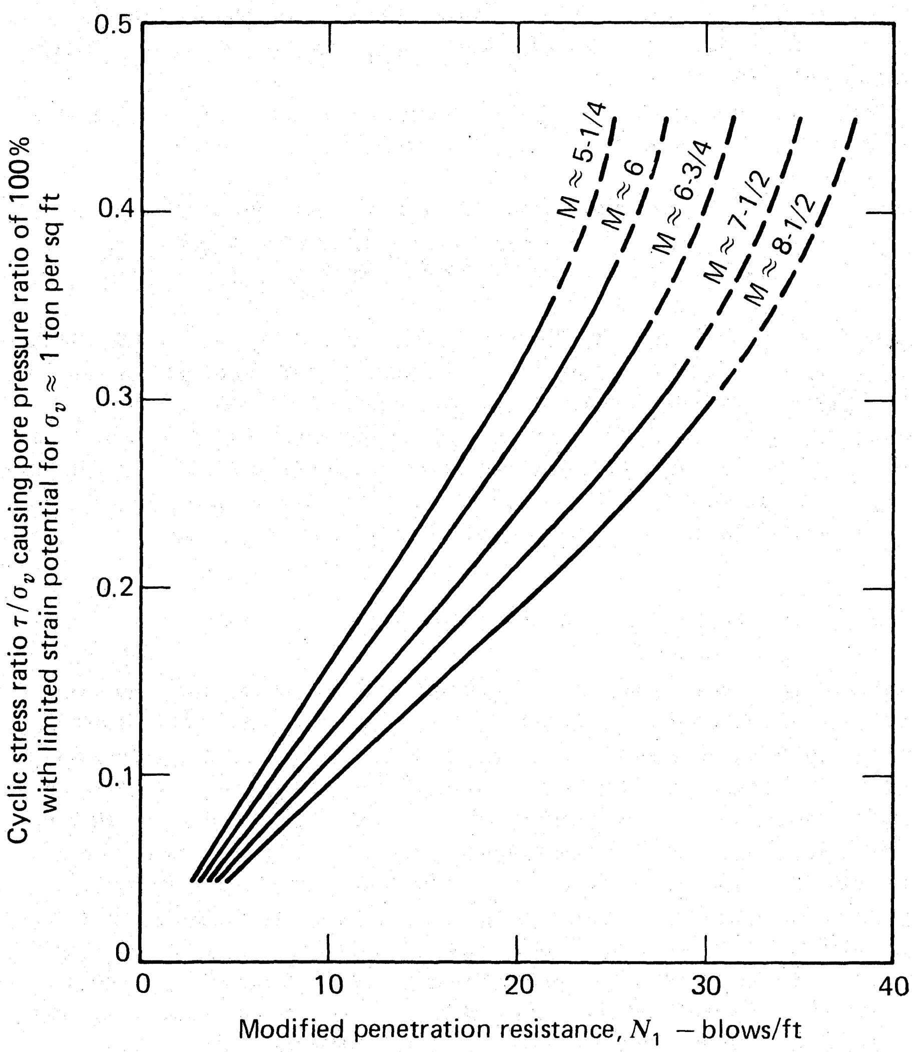
σ_0 = total overburden pressure on sand layer under consideration

$\bar{\sigma}_0$ = effective overburden pressure on sand layer under consideration

r_d = stress reduction factor varying from 1.0 at the ground surface to 0.9 at a depth of 30 ft

τ_{av} = average cyclic shear stress induced by the earthquake

For any given value of maximum ground surface acceleration and modified penetration resistance N_1, the possibility of cyclic liquefaction on



(b)

level ground can be addressed with reference to sites where liquefaction is known to have occurred. By evaluating liquefaction occurrences in a number of earthquakes worldwide and by applying scaling factors to account for earthquake magnitude and shaking duration, Seed et al. (1983) have proposed the relationship between N_1 and $\tau_{av}/\overline{\sigma}_0$ shown in Figure 9.10b. These data are intended to apply to relatively clean sands with $d_{50} > 0.25$ mm. However, field evidence indicates that silty sands are somewhat less vulnerable to liquefaction, and for silty materials ($d_{50} < 0.15$ mm), N_1 should be increased by 7.5 before entering Figure 9.10b. This correction can have a very significant effect on liquefaction evaluations for silty sand deposits (Seed et al., 1983).

These relationships can be used to develop a factor of safety against liquefaction, expressed as

$$\text{FS} = \frac{\text{uniform stress required to cause initial liquefaction or an acceptable limit of strain in } x \text{ cycles}}{\text{average shear stress induced by earthquake for } x \text{ cycles}}$$

Seed (1976, 1979a) notes that the acceptable factor of safety, besides being determined by level of confidence in the accuracy of input parameters, is also influenced by the density of the sand deposit under consideration. For dense materials unlikely to undergo large strains rapidly, relatively low factors of safety may be acceptable. For loose materials that may reach large strains very rapidly and abruptly, considerably more conservatism in the acceptable factor of safety is required.

Analysis of Compacted or Clay Embankments

Evaluations of dam behavior under actual earthquake loading have shown that well-constructed, compacted dams have experienced peak earthquake accelerations up to about 0.2 g with no detrimental effects and that dams constructed of clay on clay or rock foundations have withstood extremely strong shaking under accelerations of 0.35–0.8 g with no apparent damage (Seed et al., 1977, 1978). Rockfill dams have also performed well. Dams of this class are unlikely to experience failure due to seismic liquefaction. In the context of tailings embankments, they include downstream and water-retention type embankments constructed of clay, rockfill, mine waste, or of other material that has been well compacted under controlled conditions.

For embankments of this class, seismic analyses are commonly performed using either a pseudo-static or deformation approach.

Pseudo-static Method. The pseudo-static approach differs little from static analysis methods discussed in Chapter 8. In effect, the analysis is treated as a static problem, with the horizontal force applied by the earthquake expressed as the product of a seismic coefficient multiplied by the weight of the potential sliding mass.

Pseudo-static methods have long been customary, and they remain the workhorse for seismic stability analysis in cases where cyclic liquefaction or major pore pressure buildup is not anticipated. Nevertheless, the pseudo-static approach does not accurately model true embankment behavior under major earthquake shaking. As such, pseudo-static methods must be considered largely empirical in nature. Seed (1973, 1979b) summarizes the limitations of pseudo-static analyses. Objections to the method center around analytical details and the difficulty in selecting an appropriate seismic coefficient. Previous portions of this chapter have explained the use of empirical seismic coefficients that may or may not bear any relationship to actual accelerations determined on the basis of expected maximum acceleration. Even if maximum acceleration is used in the analysis, the nature of limiting-equilibrium assumptions requires that the earthquake force be applied uniformly within the sliding mass and as a sustained force. In actual fact, accelerations vary within the embankment because of its dynamic response to base motion, and there is little reason to necessarily believe that a peak inertial acceleration applied over a short time period is accurately represented by a sustained force.

These factors argue against placing great confidence in pseudo-static approaches except in regions of low seismic risk, or for compacted or clayey dams where significant generation of pore pressure is not anticipated. An alternative approach is to evaluate the deformation experienced by the embankment under the influence of seismic shaking.

Deformation Approaches. Deformation approaches may be applied to embankment materials that do not experience severe strength reduction during application of cyclic stresses. In effect, this method assumes that the embankment slope will not undergo complete failure by sliding during an earthquake but that the embankment will experience some degree of permanent deformation, an assumption verified by observed performance of compacted dams during earthquakes (Reséndiz et al., 1982). The basis for evaluating the suitability of embankment behavior under seismic shaking is whether or not the computed deformations lie within acceptable limits—for example, crest deformations small enough to prevent overtopping.

Procedures originally developed by Newmark (1965) have been used to estimate embankment deformations during earthquake shaking. These procedures use limiting-equilibrium methods to estimate permanent displacement experienced by a rigid-body failure mass. Other simplified techniques proposed by Makdisi and Seed (1977) include consideration of the dynamic response of the embankment. The embankment material is assumed to behave elastically at stress levels below failure and plastically above yield stresses. Either approach can be used in lieu of pseudo-static analysis to assess seismic embankment stability on the basis of allowable deformation. However, applicability of deformation-type approaches is limited to embankments of materials that experience little or no increase in pore pressure due to cyclic loading, including clayey materials, dry or partially saturated soils, or well-compacted saturated cohesionless materials which do not experience significant deformation under cyclic loading unless the static undrained strength is exceeded.

Analysis of Uncompacted Embankments

Embankments of uncompacted material, usually tailings, pose an entirely different analytical problem from the methods presented above. In uncompacted embankments, pore pressures can be expected to result from strong seismic shaking, and the material can no longer be assumed to retain all or most of its original static strength. Upstream embankments of hydraulically discharged tailings are the most problematic variety of uncompacted embankment, but this category also includes downstream- and centerline-type embankments that are saturated within significant portions and are constructed of uncompacted cycloned tailings.

Many types of techniques, ranging from very crude to extremely complex, are available for seismic stability analysis of uncompacted embankments, but most suffer from either theoretical limitations or practical drawbacks. In a hierarchy of complexity, these methods include empirical evaluations, simplified liquefaction analyses, pseudo-static methods in conventional or modified form, and full dynamic analyses.

Empirical Approach. On the most simplistic level, it is instructive to make a preliminary assessment of seismic stability on the basis of the observed performance of hydraulic fill embankments during earthquakes. Catastrophic seismic failures or near failures of uncompacted hydraulic fill embankments are well documented, but seldom is the performance of individual embankments evaluated in the context of the level of earthquake shaking they have experienced.

The most comprehensive catalog of upstream tailings embankment performance is provided by Dobry and Alvarez (1967) in their examination of the effects of 1965 magnitude 7–7.25 earthquake near La Ligua, Chile. The numerous active upstream tailings embankments examined, some of which failed and some of which did not, were at various distances from the earthquake epicenter. Figure 9.11a summarizes tailings embankments that did and did not fail, according to peak bedrock accelerations conservatively estimated using attenuation functions of Schnabel and Seed (1973). Supplementing these data are observations by Finn (1980) for upstream-constructed tailings embankments during the 1978 earthquake near Izo-Oshima, Japan (M = 7.0); observations by Robinson and Toland (1979) on the behavior of an upstream tailings embankment near Bishop, California; and data from the author's files.

The data in Figure 9.11a suggest that most upstream-constructed embankments have survived accelerations up to about 0.15 g. While the data is heavily biased toward the Chilean tailings embankments, most of these were constructed using very steep slopes (1.5 : 1 to 1.7 : 1) and at least some show evidence of having been operated with the decant pond very close to the embankment crest. It is therefore likely that many of these embankments were only marginally stable under static conditions even before the earth-

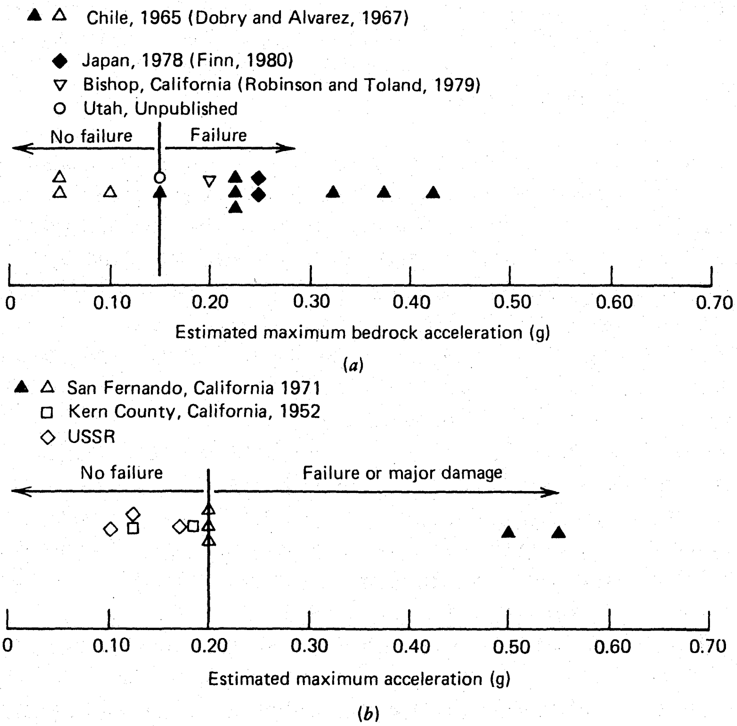


Figure 9.11 Performance of hydraulic fill embankments during earthquakes. Solid symbols indicate failure. Open symbols indicate no failure. (*a*) Upstream embankments. (*b*) Downstream embankments. (From Seed et al., 1977.)

quake and that they represent the least stable examples of upstream embankments. An additional factor to be considered is that relatively few data are available on the performance of embankments that have not failed under low accelerations. There are undoubtedly many such embankments that have survived small to moderate accelerations, but their behavior is seldom reported in the literature. Considering these factors, it appears that most upstream embankments with good control of pond water and phreatic surface, and with reasonable slopes, can survive accelerations up to about 0.15 *g*. It is also interesting to note that, of the considerable number of inactive upstream embankments inspected by Dobry and Alvarez, none failed, regardless of acceleration levels experienced.

Similar data presented by Seed et al. (1977, 1978) for conventional water dams constructed by hydraulic fill methods are presented in Figure 9.11b. The performance of these dams would be expected to be similar to that of downstream-type tailings embankments constructed of uncompacted cycloned sands. Figure 9.11b shows that none of the dams failed at accelerations less than 0.20 *g*, and Seed et al. (1977) conclude:

> Many hydraulic fill dams have performed well for many years and when they are built with reasonable slopes on good foundations, they can survive moderately strong shaking—say up to about 0.2g from magnitude 6.5 to 7 earthquakes with no harmful effects.

Simplified Liquefaction Analyses. Simplified liquefaction analyses of the form proposed by Seed and Idriss (1971) and Seed et al. (1983), as presented previously, have been applied to upstream embankments to compute a factor of safety against liquefaction. In applying the simplified methods of Seed et al. (1983) to tailings deposits, it is significant that an increased resistance to liquefaction is predicted for silty sands ($d_{50} < 0.15$ mm). This would apply generally to uncycloned tailings finer than about 40 mesh and having more than about 20% minus No. 200, which represent the majority of spigotted tailings deposits, as described in Chapters 1 and 2.

As previously discussed, however, a number of factors suggest that the liquefaction resistance of tailings may be different from that of natural deposits upon which the simplified approach is based. In an attempt to resolve this difficulty, some investigators have performed cyclic testing on the actual hydraulic fill material and used appropriate methods to adjust the laboratory strength to field conditions (Pyke et al., 1978).

Nevertheless, several problems with this approach remain and stem from the application of a one-dimensional level-ground method of analysis to the two-dimensional configuration of the embankment. First, the phreatic surface near the face of an upstream embankment is not horizontal but drops within the embankment section, making it difficult to know what groundwater level should be taken as representative for the simplified one-dimensional analysis. Second, acceleration is not constant over the surface of the embankment, but varies with the dynamic response of the embankment and foundation according to their soil properties and geometry, as illustrated in Figure 9.12. Thus, selection of an appropriate ground surface acceleration for input to the simplified analysis is also difficult. These factors suggest that the use of the simplified approach should be restricted to embankments of low height and should be performed using only cyclic test data on the actual tailings material.

Pseudo-static Method. Pseudo-static approaches are routinely used in practice for uncompacted tailings embankments, but they have even less theoretical validity than in their conventional application to compacted embankments because pore pressures due to cyclic loading are not accounted for. Finn (1980) describes the results of pseudo-static analyses performed on upstream tailings embankments that subsequently failed by liquefaction during the Iso-Oshima earthquake of 1978. By using a seismic coefficient corresponding to the actual maximum acceleration experienced by the embankments during the earthquake, failure might have been correctly predicted. However, considering the uncertainties in selection of appropriate strength

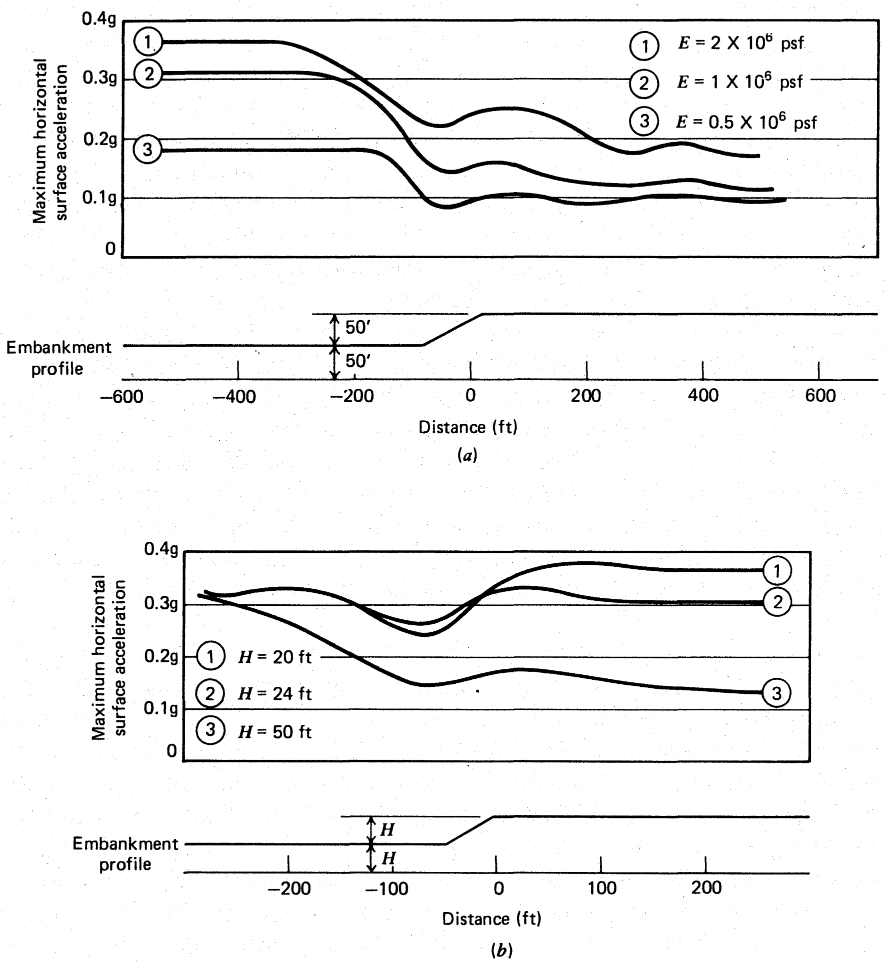


Figure 9.12 Effect of embankment response on surface acceleration for peak bedrock acceleration = 0.16 *g*. (From Idriss and Seed, 1967.) (*a*) Effect of material properties for 50-ft embankment. (*b*) Effect of embankment height and foundation soil thickness for $E = 1 \times 10^6$ psf.

values, such a successful prediction could have been largely accidental. In general, application of pseudo-static analysis to uncompacted embankments should be reserved for those cases where seismic risk is so low that it can be reasonably certain that significant pore pressures due to cyclic loading will not develop. As a precaution, total stress parameters should be used for saturated uncompacted materials of low to moderate permeability in order to account in a crude way for pore pressures due to shear generated by the rapidly applied earthquake loading.

Modified Pseudo-static Methods. Several investigators have attempted to address deficiencies in the conventional pseudo-static approach by accounting for pore pressures induced by cyclic shear in a more explicit way. One approach for doing so is to first consolidate a sample in the laboratory, to then subject it to a number of cycles of undrained shear corresponding to the magnitude and duration of the design earthquake, and finally to shear the sample in a static, undrained mode (assuming that liquefaction has not already occurred). A strength envelope results that is similar to a conventional envelope from static consolidated-undrained tests, except that the pore pressures induced by cyclic shear are incorporated. Lee and Roth (1977) describe the use of such a strength envelope in the analysis of a hydraulic fill dam. Similar procedures are described by Ramanujam et al. (1978).

Another modified pseudo-static approach is that developed by Klohn et al. (1978). Intended for application to centerline-type tailings embankments, this method assumes that the retained slimes liquefy and produce a rapidly applied load on the embankment itself, as described by Finn and Byrne (1976). The pore pressures due to undrained static loading during application of this inertial thrust, as well as those produced by cyclic loading, are incorporated into an effective-stress strength model. Seismic coefficients used in a series of pseudo-static analyses are based on peak ground acceleration corrected to account for dynamic response of the embankment.

Complete Dynamic Analysis. Dynamic analysis represents the state of the art for seismic analysis of uncompacted embankments subject to strong shaking. This type of analysis attempts to account explicitly and individually for all the factors needed to realistically model the behavior of embankments during earthquakes. However, in avoiding the assumptions used in other methods to make them more tractable, dynamic analyses are at a risk of becoming astoundingly complex. In effect, uncertainties that are dealt with on a gross level for other methods are separated into their various component parts for dynamic analyses. The result is not necessarily to reduce overall uncertainty, but to account for individual variables more explicitly.

Typical dynamic embankment analyses for conventional dams are described by Seed et al. (1975), Seed (1979a), Lee and Roth (1977), and Makdisi et al. (1978). Buckles et al. (1982) present the results of dynamic analyses performed on a hybrid upstream–downstream type tailings embankment, while Finn and Byrne (1976) and Dorey and Byrne (1982) describe dynamic analysis procedures applied to centerline-type tailings embankments. In a dynamic analysis of a centerline embankment, Lo et al. (1982) discuss the effects of assumed slimes liquefaction on stability of the embankment.

Seed (1979b) and Finn (1982) summarize procedures for performing dynamic analyses of conventional dams and tailings dams, respectively. In general, dynamic analyses incorporate the following steps:

1. *Determine the preearthquake static stresses in the embankment.* This is usually performed using a static finite-element anal-

ysis to determine the initial effective normal stresses and shear stresses along the potential failure plane. These stresses are necessary for proper interpretation of laboratory cyclic test data.

2. *Determine the dynamic response of the embankment and foundation to base rock motions.* This step requires that a bedrock acceleration time history be established, usually on the basis of a well-instrumented earthquake in a similar geologic environment and of similar size and duration. Based on this base motion time history, the response of the embankment is determined by dynamic finite-element modeling, using either equivalent-linear or nonlinear procedures. The end result is to determine the acceleration time history at every node in the finite-element mesh. These time histories must then be converted to an equivalent series of uniform cyclic stress applications using procedures such as those described by Lee and Chan (1972) in order to interpret laboratory test data.

3. *Evaluate the dynamic soil behavior.* This is ordinarily accomplished by performing cyclic triaxial tests under initial consolidation stress conditions corresponding to those indicated by the results of the static stress analysis, with the laboratory results corrected to account for several factors that influence in situ liquefaction behavior (Seed, 1976). Failure in laboratory samples may be defined in terms of initial liquefaction or with reference to a particular level of axial strain.

4. *Evaluate embankment stability.* Stability of the embankment may be addressed in terms of either stresses or strains occurring locally within the embankment. The stability of the embankment is evaluated by comparing stresses induced by the earthquake to those required to cause liquefaction or to exceed prescribed limits of local strain. The locations within the embankment for which induced stresses exceed those required to cause liquefaction or produce unacceptable strain determine the extent of liquefaction or unacceptable performance. Embankment deformations can be assessed qualitatively on the basis of *strain potential* for individual elements, which corresponds to the strain that would be experienced if the element were not constrained by the surrounding soil. Those elements with excessive strain potential may be considered to offer no effective resistance to slide movement within the embankment. Under these conditions, a stability analysis can be performed to indicate the overall factor of safety against sliding through zones of liquefaction or excessive deformation (Seed et al., 1975). Alternatively, cyclic-induced pore pressures for individual elements, together with effective-stress strength parameters, can be incorporated into a limit-equilibrium analysis to compute a

post-earthquake factor of safety. In addition, total embankment deformations can be calculated on the basis of gravity loads and softened material properties to determine whether they lie within acceptable limits (Lee and Roth, 1977).

Figure 9.13 shows the results of a dynamic analysis performed on a centerline-type tailings dam constructed of uncompacted cycloned tailings sand at a relative density of 50%. The design earthquake is taken as a magnitude 6.5 event producing a peak ground acceleration of 0.15 *g*. At this particular site the design acceleration has been determined by probabilistic analysis to have a recurrence interval of 1,000 years and an exceedance probability of 2% over the saturated life of the embankment.

In this example the starter dike is relatively impervious with respect to the cycloned tailings sand, resulting in zones of saturation at the base of the dam and also along the interface between the starter dike and the cycloned sand fill. Results of the dynamic finite-element analysis on Figure 9.13 show that by the end of earthquake shaking the upper portion of the impounded slimes have undergone initial liquefaction. Cyclic pore pressure ratios for saturated elements not experiencing initial liquefaction are shown on Figure 9.13. Further evaluation of analysis results could, for example, include limit-equilibrium stability analyses along the potential failure surface shown using effective-stress strength parameters in conjunction with the computed cyclic pore pressures.

SUMMARY

The analysis of tailings embankment stability under seismic conditions is a task of considerable complexity involving no small degree of uncertainty. Even prior to the analysis itself, a major issue is the assessment of seismic risk and selection of design earthquake parameters. This can be done on either a probabilistic or a deterministic basis, depending on the degree of conservatism required and the seismotectonic setting of the site. Regardless of the basis for the design earthquake, a systematic procedure is to evaluate ground motion parameters (principally acceleration) using several methods ranging from evaluation of historic earthquakes, to probabilistic analysis, to deterministic analysis based on postulated fault rupture. In the end, there can be no one unique design value, but rather an array of values with a certain level of risk associated with each. Ultimate selection of parameters for use in seismic analysis depends on the nature of the structure and the consequences of failure.

Methods for seismic analysis are varied and depend on the type of structure under consideration. For embankments that because of the dense, clayey, or pervious nature of their fill would not be expected to undergo development of significant pore pressure during cyclic loading, appropriate

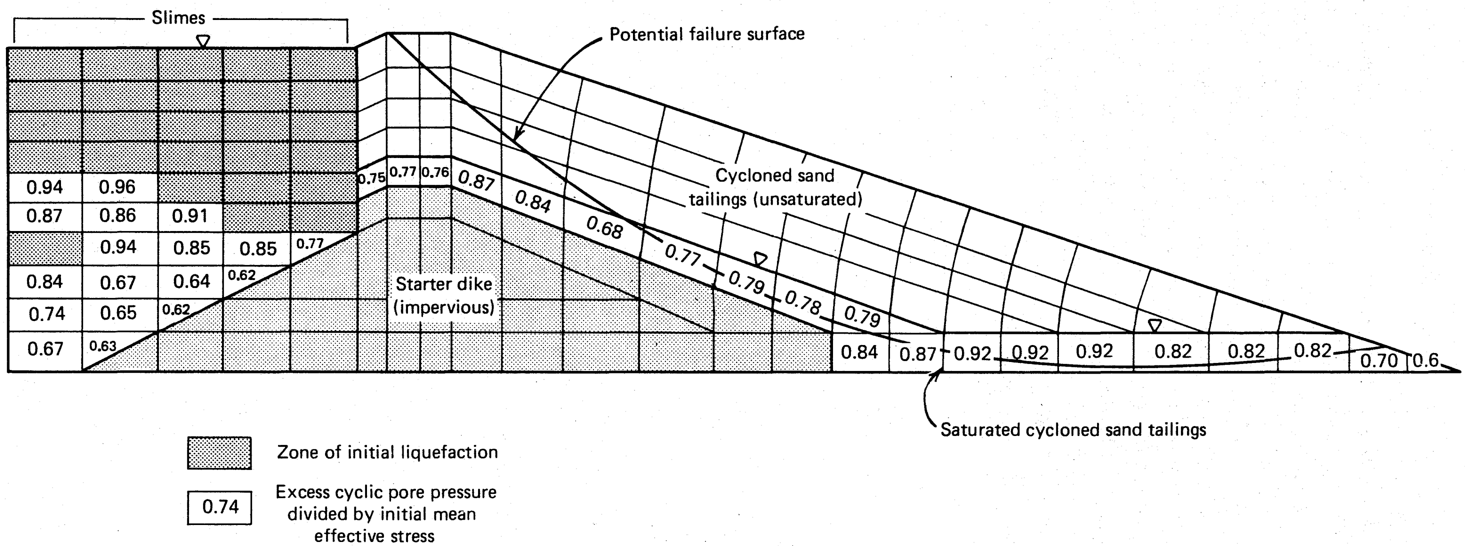


Figure 9.13 Results of dynamic analysis for centerline cycloned sand tailings dam.

analytical techniques may include pseudo-static analyses or methods based on embankment deformation.

For embankments of uncompacted material, an initial guide to embankment performance is the behavior of similar structures under earthquake shaking. Analytical approaches may include simplified liquefaction analysis, pseudo-static analysis, and modified pseudo-static procedures. Each method involves unique limitations, and none is theoretically rigorous. Best used in combination where possible, these methods are likely to give a reasonable indication of seismic stability, provided that their approximate nature is kept in mind. Ordinarily reserved for uncompacted embankments involving high hazard and strong seismic shaking are dynamic analyses. These methods involve very complex and sophisticated procedures both in modeling and in laboratory testing. Interpretation of input data as well as results requires an extraordinary degree of judgment and insight.

10

Analysis of Seepage and Contaminant Transport

WILLIAM HIGHLAND

The strongest argument of the detractors of mining is that the fields are devastated by mining operations. . . . Further, when the ores are washed, the water which has been used poisons the brooks and streams, and either destroys the fish or drives them away. . . . Thus, it is said, it is clear to all that there is greater detriment from mining than the values of the metals which the mining produces.

Agricola, *De Re Metallica,* published in 1556

Of all the issues related to tailings disposal, there is perhaps none so environmentally sensitive or analytically complex as impoundment seepage. Geotechnical engineers have traditionally considered seepage through dams as primarily an issue of structural significance. More than a few dams, tailings embankments among them, have failed because of structural problems related to seepage, such as piping and poor phreatic surface control. As discussed in Chapter 8, proper design and analysis methods must recognize the structural importance of seepage. On the other hand, traditional geotechnical principles do not adequately speak to the issue of accurately determining either the quantity of seepage that escapes from a tailings impoundment or the movement through groundwater of contaminants it may contain. Invoking the principles and techniques of hydrogeology aids but still does not solve the seepage analysis problem, since practitioners of this field have traditionally been concerned more with extracting water from an aquifer in one way or another than analyzing the details of seepage discharged into it.

Major steps in understanding the impoundment seepage problem have come about very recently as the result of input from the field of soil science and related areas where movement of water through unsaturated soil has long been of concern. Further progress has been made by applying basic physical chemistry to the previously neglected areas of tailings impoundment effluent and effluent–soil interactions. Still, for all the recent advances, analysis of seepage and contaminant transport from tailings impoundments is not yet a mature science.

Regulatory developments concerning protection of groundwater from contamination by hazardous wastes will continue to prompt further development of analytical techniques. Tailings impoundment seepage considerations have increased in importance to the point that they occasionally become the tail that wags the design dog for impoundments that retain toxic effluents. Seepage protection measures can in some cases easily outweigh the cost of all other physical impoundment facilities combined, and licensing or permitting of new mines not infrequently hinges on the effects of tailings impoundment seepage on groundwater. In addition to regulatory and environmental importance, it is becoming more commonly recognized by mine operators that accurate prediction and mitigation of seepage losses can minimize costs associated with obtaining mill makeup water and adding costly reagents to the mill process water stream.

Chapter 10 discusses factors pertaining to the analysis of seepage from tailings impoundments and the transport of effluent-borne contaminants through soil and groundwater. Initial portions of the chapter provide an overview of seepage impacts and discuss investigation tools for both seepage and geochemical analyses. Subsequent portions discuss analytical methods and present an array of techniques for evaluating both seepage quantities and contaminant transport, ranging from relatively simple to highly complex.

OBJECTIVES OF SEEPAGE EVALUATIONS

Major factors affecting the siting of tailings disposal impoundments often overshadow the costs of providing seepage control systems, as discussed in Chapter 5. Therefore, the designer is generally confronted with a hydrogeologic situation by default rather than choice. In those cases where seepage control constitutes a major proportion of tailings disposal costs, hydrogeology should certainly be a major factor in the siting decisions.

The major objective of seepage evaluation is to provide a tailings impoundment design that will mitigate environmental damage and minimize seepage loss of mill effluent that might otherwise be economically recycled to the mill. Environmental damage may result when mill effluent seepage containing toxic substances enters groundwater resources that flow toward points of water supply or natural discharge areas, as shown in Figure 10.1. Seepage impacts are mitigated by chemical reactions between seepage and subsurface materials, which can be utilized to provide a buffer zone between the impoundment and points of discharge.

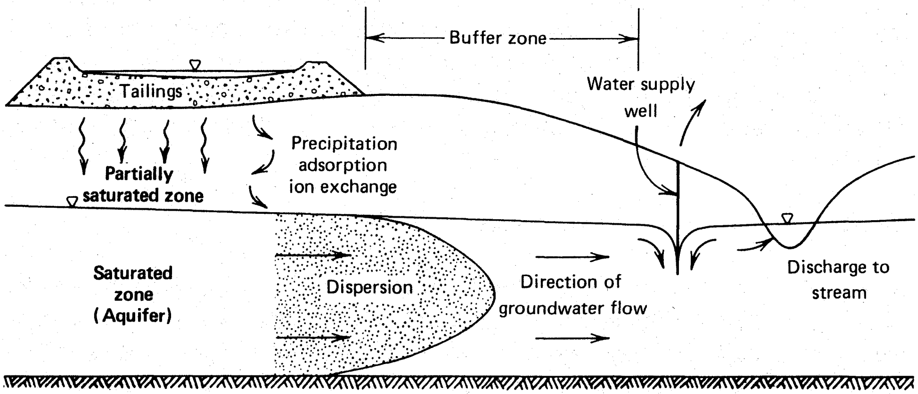


Figure 10.1 Groundwater flow and contaminant transport processes.

To achieve the objectives of a seepage evaluation, it is necessary to:

1. Establish the geologic, hydrologic, and geochemical conditions in the vicinity of a proposed impoundment prior to construction.
2. Physically and chemically characterize the tailings, effluent, and any construction materials to be used, such as earthen liners, drain materials, and embankment materials.
3. Conduct sufficient field and laboratory tests to satisfy 1 and 2 within the impoundment area and an area 1,000–2,000 ft outside the impoundment.
4. Investigate the need and alternate methods for seepage control through predictive models.
5. Select a control method, if necessary, that minimizes the change or impact to the natural system at reasonable economic and environmental cost.

OVERVIEW OF SEEPAGE IMPACTS

Physical Factors Affecting Seepage Impacts

Permeability is the physical factor that most influences groundwater contamination potential. For example, high permeability of the tailings, the foundation materials, and the uppermost aquifer will almost certainly result in effects of tailings seepage on groundwater. Moreover, differences between the permeabilities of the tailings, the foundation, and the aquifer, as well as the variability of permeability within each material, produce the wide variety of responses in groundwater systems that have been observed at existing tailings disposal sites. In order to introduce concepts, the discussion that follows assumes a tailings impoundment located on homogeneous materials that are initially partially saturated between the base of the impoundment and the groundwater table. In this simple context, the range of typical seepage situations can be examined and then extrapolated to more complex situations.

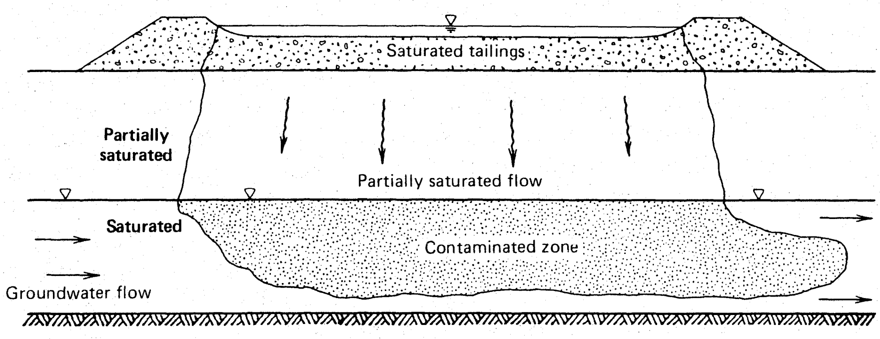
Response of Flow System to Seepage

In some fortunate but rare circumstances, the tailings impoundment has little or no effect on the underlying groundwater regime. The obvious examples include tailings impoundments on naturally impervious foundations or those that incorporate essentially impervious liners. In such cases, the low quantity of seepage escaping the impoundment has a negligible effect on the flow system.

Even for impoundments with high seepage losses, little or no water level response in an aquifer can also occur in specific geologic environments, as shown in Figure 10.2a. Small water level response is possible only if both the partially saturated zone below the impoundment and the aquifer are highly permeable in comparison to the tailings. Geologically, high permeability foundations are relatively common. In large areas of Florida, the southeastern United States, and the west coast of Australia, for example, coastal or alluvial sands overlie slightly to highly karstic limestone. High foundation permeability is also commonly found in mountainous regions, where steep topography frequently limits impoundment sites to permeable gravels comprising alluvial valley bottoms or mountain-front pediments. These and similar hydrogeologic environments would probably show little or no change in water levels in response to disposal of tailings having moderate to low permeability. In general, little response is observed when seepage quantity is approximately 10% or less of the through-flow in the underlying aquifer.

Three major features distinguish the limiting condition where little or no change in the original flow system takes place:

1. Flow in the tailings is fully or nearly saturated. Seepage movement is vertically downward, and the quantity of seepage leaving the impoundment depends entirely on the permeability of the tailings or any impoundment liner rather than on the permeability of the foundation materials, which freely transmit as much fluid as can be supplied by the impoundment.
2. Flow between the base of the impoundment and the groundwater table remains in a partially saturated state, but the moisture content of the foundation materials increases. Flow in the partially saturated zone is also primarily vertically downward.
3. Saturated flow in the uppermost aquifer remains primarily unidirectional and horizontal. Stated in other words, mounding of water



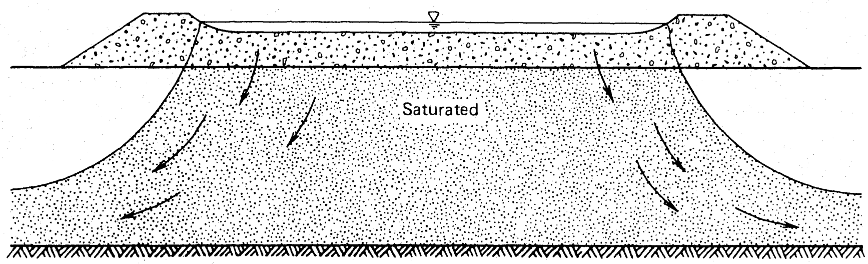


Figure 10.2 Aquifer responses to tailings impoundment seepage. (*a*) Flow directions unchanged by seepage. (*b*) Flow system dominated by seepage.

levels below the impoundment is insignificant. Although chemical contaminants may significantly alter the chemical composition of the groundwater, flow directions and water levels in the aquifer remain relatively unchanged.

Flow System Dominated by Seepage

In most geologic situations likely to be encountered in tailings impoundment design, seepage will eventually result in measurable amounts of groundwater *mounding*. Mounding results in increased saturated aquifer thickness near the impoundment and aquifer flow that moves more or less radially away from the impoundment. The extreme case occurs when seepage completely saturates subsurface materials and results in the progressive growth of a groundwater mound that eventually moves upward to form a direct saturated connection between the tailings and aquifer, as shown in Figure 10.2b.

The transition from partially saturated to saturated foundation conditions may span a period of many years or even tens of years depending on foundation permeability. The time span required for saturation is particularly important in assessing the possibility of groundwater contamination since tailings disposal may cease prior to complete saturation of the foundation.

Measurable changes in either aquifer water levels or saturation below the tailings can be expected when permeability of the foundation materials is within two orders of magnitude of the tailings permeability (either higher or lower) or when the quantity of seepage is greater than 10% of the throughflow in the uppermost aquifer. McWhorter and Nelson (1979) describe three stages in the development of mounding as follows:

Stage 1. Seepage from the disposal site produces a *wetting front* that moves vertically downward toward the water table. The wetting front will at least result in an increase in the moisture content of the foundation materials and may result in complete saturation as the front moves downward.

Stage 2. This stage is characterized by a rising groundwater mound in the underlying aquifer where the rate of rise depends on the seepage rate, the fraction of the pore space in the foundation material that remains available for water storage, and the tendency for groundwater to spread laterally as the mound rises.

Stage 3. The groundwater mound rises to the base of the impoundment, resulting in a saturated hydraulic connection between the tailings deposit and the groundwater.

The stages in seepage development are shown diagrammatically in Figure 10.3. For fully developed Stage 3 conditions, the flow system is characterized by the following features:

1. Flow within the tailings deposit itself is saturated and predominantly in the vertical direction, with a small horizontal component of flow.
2. Flow in the foundation materials is completely saturated below the tailings impoundment and has both vertical and horizontal components of flow, resulting in seepage movement downward and radially outward from the impoundment.
3. Flow directions in the underlying aquifer have been completely altered by seepage from the tailings impoundment. Vertical gradients are present below the impoundment, and flow is radially outward from the impoundment in all directions. Mounding results in seepage that eventually displaces groundwater below and in the near vicinity of the impoundment.

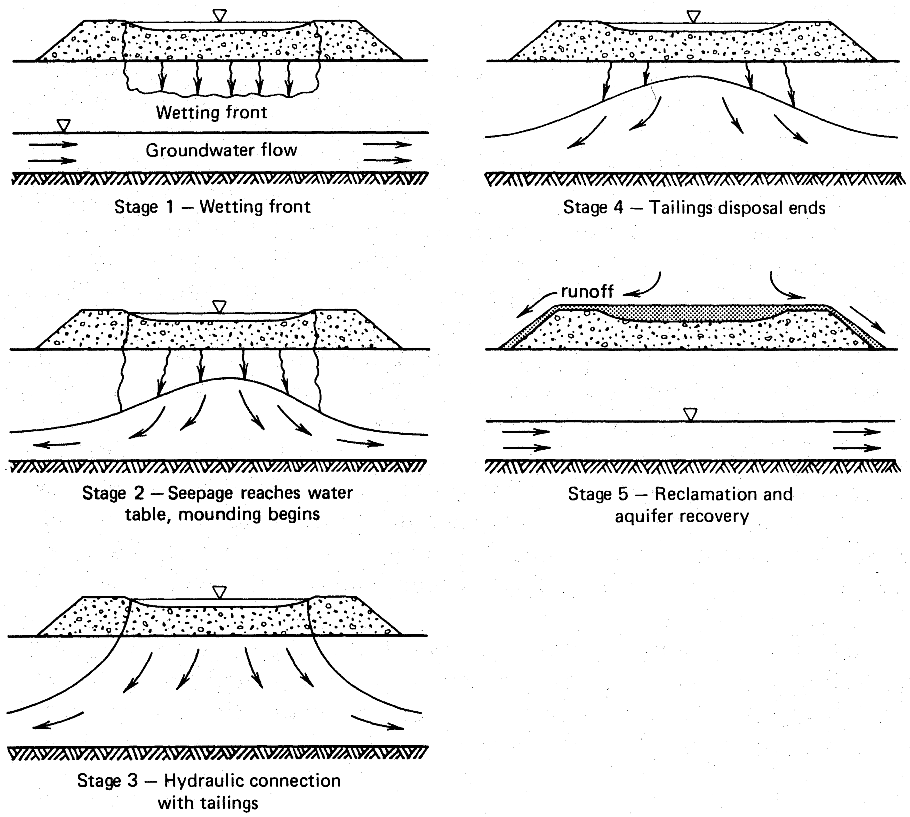


Figure 10.3 Stages in seepage development. (From McWhorter and Nelson, 1979.)

Taking a longer term view of seepage from tailings impoundments, two additional stages in seepage conditions can be added to those suggested by McWhorter and Nelson, as shown in Figure 10.3. At the completion of disposal operations, supply of effluent to the pond ceases. Gradually fluid stored in the tailings voids will drain by gravity and to a lesser extent be released as a result of tailings consolidation. As the supply of seepage to the underlying aquifer dwindles, the groundwater mound will begin to decline. After a long period of time, typically on the order of 10–50 yr or longer, the tailings will be drained to their field capacity. Infiltration of precipitation will continue over the long term, the amount depending on climate, the permeability of the tailings, and the type of reclamation measures. Long-term infiltration, however, will not likely produce more recharge to the groundwater than occurred over the same area prior to impoundment construction. Ultimately then, the groundwater regime will eventually reestablish its original configuration in most cases.

More Complicated Responses

Geologic discontinuities and heterogeneity result in physical aquifer responses that can be considerably more complicated than those described above. An evaluation of the degree of fracturing, jointing, and faulting is necessary in all foundation seepage investigations to assess the potential for concentrated seepage through discontinuities. For example, shales containing a high percentage of montmorillonitic clays are particularly susceptible to fracturing as a result of shrinking or swelling after changes in moisture content and therefore may not necessarily provide an ideal foundation for a disposal site. Other complexities include clay layers or lenses in the partially saturated zone producing "perched" seepage; igneous intrusions producing partial barriers to flow; and extremely heterogeneous conditions resulting from the original depositional environment, such as fluviatile sediments. Such complexities generally result in preferential flow paths rather than uniform aquifer response. Detection of seepage in complex geologic environments requires a detailed knowledge of the geologic conditions and a reasonable amount of experience and judgment. Examples of preferential flow paths resulting from geologic discontinuities and heterogeneity are shown in Figure 10.4.

Methods for conducting hydrogeologic investigations are briefly summarized in a following section. An appreciation of the geologic controls on groundwater flow is essential in any seepage evaluation, and the importance of obtaining reliable geologic data cannot be overemphasized.

Other Physical Factors

Although permeability is the primary factor affecting groundwater contamination potential, the typical responses described above have highlighted other physical characteristics that influence seepage quantity and hence groundwater contamination potential. Physical factors other than permeability that limit the amount of seepage reaching groundwater include the initial moisture content and total thickness of the partially saturated zone; the distribution of tailings within the disposal area; and engineered seepage barriers such as earthen or synthetic liners.

Depth to groundwater is an important factor in assessing groundwater contamination potential. In many arid regions, such as portions of the western United States, the partially saturated zone extends many hundreds of feet below the surface. These large zones of partially saturated material in low-rainfall environments can serve effectively in storing seepage and thereby greatly reduce the rate at which seepage enters groundwater aquifers. Conversely, in humid regions where groundwater is near the surface, Stage 3 conditions can be reached quickly.

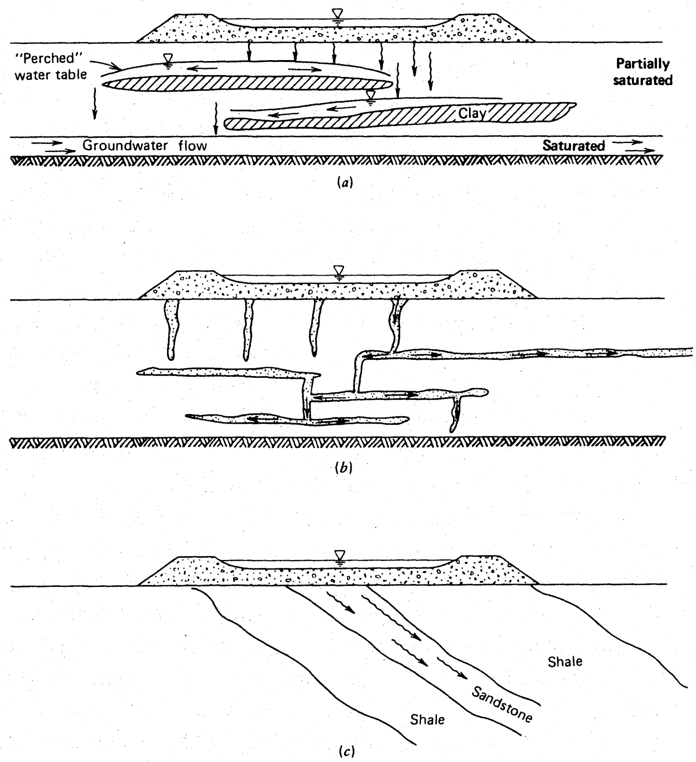


Figure 10.4 Geologic factors affecting seepage movement. (*a*) Clay layers in partially saturated zone. (*b*) Flow in fractures. (*c*) Flow through permeable inclined strata.

Another physical factor influencing seepage quantity is impoundment size. For unlined impoundments on foundations more pervious than the tailings, seepage loss can be considered directly proportional to the size of the impoundment area covered by water. A simplistic but important observation in such cases is that seepage can be reduced by minimizing the size of the decant pond during impoundment operation, for example, by increasing mill water reclaim rates. In a layout context, seepage can be minimized by reducing the area covered by the impoundment and increasing its height, again assuming relatively pervious foundation conditions. While this criterion may be at odds with optimum fill efficiency considerations discussed in Chapter 5, a compromise solution can be to construct a segmented impoundment to minimize the area covered by water at any given time.

Engineered seepage controls constitute a final physical influence on seepage quantities. Three main categories of controls are available: barriers, collectors, and liners. These control methods and other factors pertaining to seepage mitigation are discussed at length in Chapter 11.

Geochemical Factors Affecting Seepage Impacts

The physical movement of seepage is a necessary prerequisite for contamination problems to result from the disposal of tailings. This rather trivial observation is intended only to highlight the order in which a rational investigation should proceed. If examination of the physical flow processes for a particular impoundment design indicates little seepage movement, then the need for detailed contaminant attenuation studies is questionable. On the other hand, to ignore geochemical factors in cases of high seepage loss of effluent containing toxic, particularly metallic, contaminants may be to derive an entirely misleading picture of groundwater contamination potential.

Chemical responses of groundwater to tailings disposal are an environmentally sensitive issue. Although the physical impacts associated with tailings impoundment seepage may occasionally result in such undesirable effects as artificially introduced marsh areas or damage to nearby structures, the chemical effects can often far overshadow these.

The first and foremost chemical factors affecting groundwater contamination potential are the pH, salinity, and toxicity of the tailings effluent. A summary of the chemical composition of several mill effluents in comparison with water quality standards has been presented previously in Chapter 1. Mill effluents that are products of leaching processes are most likely to cause groundwater contamination, all other factors being equal. Therefore, the acid-leach effluents commonly associated with uranium milling and the alkaline-leach process associated with alumina refining are examples of mill effluents that have a high potential for groundwater contamination. A related problem is tailings that may develop low pH effluent due to pyrite oxidation. Leaching processes that rely heavily on oxidation and toxic reagents (for example, cyanide used in gold extraction) also have a high potential for groundwater contamination.

At the other end of the spectrum are effluents associated with physical crushing and wash separation techniques, such as taconite processing and phosphate beneficiation, which result in only small additions of dissolved solids to the original water and therefore have low potential for adversely affecting groundwater quality, regardless of seepage quantity. Similarly, many flotation processes that do not require extensive pH modification produce effluents that are relatively innocuous from a chemical standpoint.

As with the physical factors affecting groundwater contamination, reduction of groundwater contamination potential can be achieved through a combination of natural and designed geochemical factors. It is only recently that many investigators have come to realize that chemical reactions of seepage with native soils and groundwater can be a very effective means of reducing levels of contaminants in mill effluent seepage (Shepherd and Cherry, 1980; Guarnaschelli and Shields, 1978; van Zyl and Caldwell, 1979). Much of the recent work in this area has resulted from the intensive investigation of uranium tailings disposal. It was long known that many older uranium tailings impoundments were losing a considerable amount of effluent through seepage and that the effluent itself was high in dissolved radioactive contaminants and heavy metals. Therefore, it was initially assumed that large-scale contamination of aquifers with highly toxic substances was widespread near these impoundments. After years of study, few cases of widespread contamination were shown to exist, and of those that do exist, the contamination observed is by primarily nontoxic salts, such as sulfates and chlorides (Rahn and Mabes, 1978). The puzzling absence of radiological and metallic contaminants led to more intensive investigation of the geochemical processes which retard the movement of certain contaminants in groundwater flow.

Chemical reactions resulting from tailings pond seepage are most easily investigated in stages that correspond to stages in the physical flow processes shown in Figure 10.3. Seepage first passes through clay liners if they are used in a design. Clay liners can be used to reduce the concentration of contaminants in seepage as well as to restrict physically the quantity of seepage itself. Therefore, from a geochemical standpoint, one is interested in the amount of contaminant that will be removed by a clay liner, the rate at which removal efficiency may decrease with time, and the total capacity of the liner material to react with seepage.

The next stage in the seepage flow process is movement of seepage through the partially saturated zone. If a liner is not used, the partially saturated foundation soils are the first zone of soil contact. Here a knowledge of soil reactivity in the partially saturated zone as well as any potential for contaminant attenuation is desirable. Mixing of seepage with moisture stored in the partially saturated zone will also result in dilution of contaminant concentration. Seepage may eventually reach the underlying aquifer, where reaction of contaminants with aquifer solids, reactions with native groundwater, and dilution by mixing of seepage and groundwater are all potential mechanisms that reduce concentrations of dissolved contaminants.

In some cases, total reliance on the above-mentioned natural mechanisms will provide adequate seepage control from a groundwater quality standpoint, but designed seepage-control measures may be necessary to supplement natural attenuation mechanisms. In extreme cases, seepage loss through the impoundment must be almost totally eliminated through design measures. These situations arise when costs for locating disposal sites at hydrogeologically favorable sites are far greater than designing and constructing "zero-seepage" facilities.

Summary of Parameters Influencing Seepage Impacts

The overview of stages in the development of seepage resulting from tailings disposal has highlighted the physical and chemical factors that affect groundwater contamination potential. Similarly, the parameters that must be measured to evaluate the design and siting requirements for prevention of

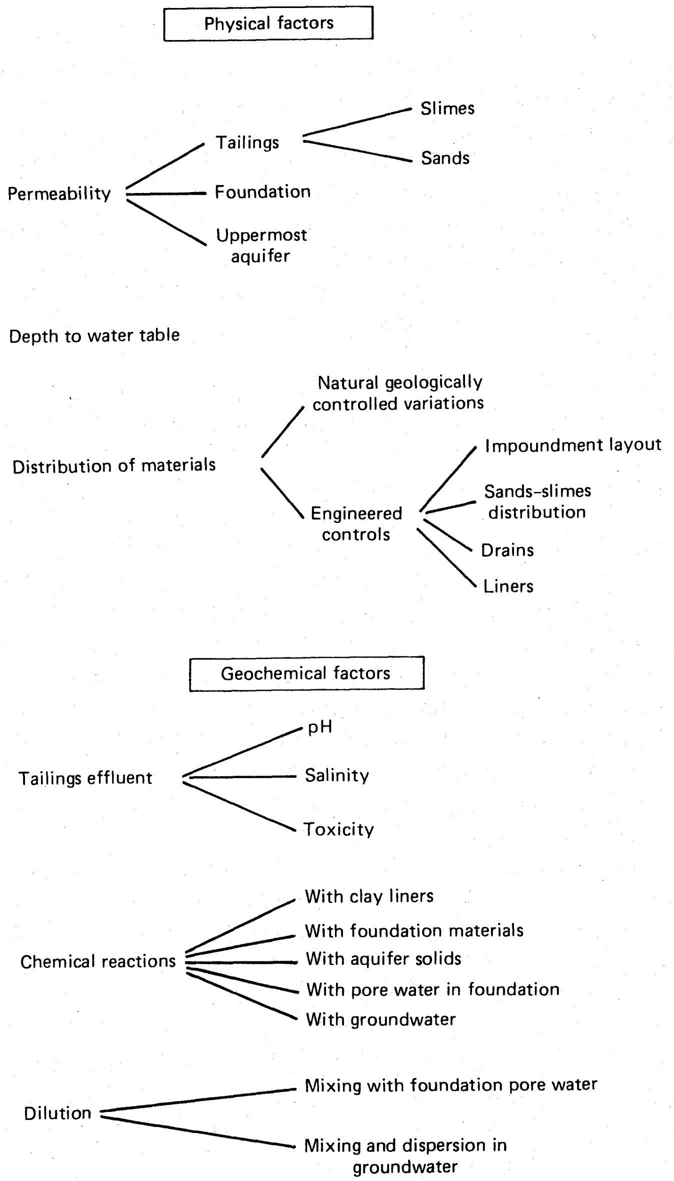


Figure 10.5 Summary of factors affecting groundwater contamination potential.

groundwater contamination have also been addressed. The physical and chemical factors affecting groundwater contamination potential are summarized in Figure 10.5.

The primary factors affecting groundwater contamination potential are the toxicity and concentration of individual contaminants in the tailings effluent as well as the permeability of the tailings, foundation, and uppermost aquifer. Reliance on natural physical and geochemical control of seepage is preferable to designed controls, when possible. However, tailings impoundments located on highly permeable materials and which contain concentrated or toxic effluents have an inherently high groundwater contamination potential regardless of stringent design measures to control seepage since even small localized liner failures can cause widespread groundwater contamination.

The magnitude of seepage impacts resulting from tailings disposal is dependent first on the amount and rate of seepage movement, and second on the chemical reactions and physical dilution processes that take place as the seepage moves. Therefore, quantification of the physical characteristics of the tailings and foundation materials is logically followed by chemical characterization of the tailings effluent, groundwater, and foundation materials.

INVESTIGATION METHODS

Hydrogeologic Investigations

Establishing a coherent picture of subsurface stratigraphic and geologic structural relationships beneath a tailings impoundment is a critical component of any seepage evaluation. The methods of conducting a field investigation to obtain hydrogeologic data depend on many site-specific variables, such as geology, depth to water table, and terrain. Selection of drilling equipment, use of geophysical logging, and other types of tests conducted in the field are difficult to generalize. Similarly, many methods are available for the analysis of data from borehole tests, pumping tests, and measurement of baseline water quality.

The methods available for these types of investigations are widely published and discussed in many widely available sources. Table 10.1 summarizes some of the available references applicable to hydrogeologic investigations for tailings impoundments.

Objectives

It should not be surprising that the basis of any seepage evaluation depends on knowing the nature of the materials through which the seepage will pass and the characteristics of the groundwater regime that it may affect. The purpose of hydrogeologic investigations is to address these issues by providing background data on the geologic and groundwater setting beneath the impoundment site, as well as direct field measurement of physical parameters where possible.

Characterization of broad geologic conditions and minor geologic variables provides the framework for further development of seepage models. The investigation must determine, for example, whether the principal flow

Table 10.1 Reference Sources for Hydrogeologic Investigations

Reference	Methods Described
Bouwer (1978)	General text on groundwater hydrology. Excellent explanations of partially saturated flow behavior.
Davis and DeWiest (1966)	General text on groundwater hydrology. Emphasis on geologic factors.
Domenico (1972)	General text on groundwater hydrology. Concise theoretical treatments.
Freeze and Cherry (1979)	General text on groundwater hydrology. Perhaps the best textbook available. Covers all facets of groundwater hydrology, including geochemical principles.
Walton (1970)	General text on groundwater hydrology. Many case histories of groundwater supply.
U.S. Environmental Protection Agency (1977)	Guidelines for monitoring procedures, sampling and analysis techniques for solid waste disposal facilities.
Keys and MacCarey (1971)	Geophysical logging methods applied to groundwater investigations.
U.S. Bureau of Reclamation (1977)	Practical guide to conducting field investigations.
Kruseman and DeRidder (1976)	Summary of methods of conducting and analyzing data from pumping tests.
Wilson (n.d.)	Measurement and monitoring of the partially saturated zone.

medium consists of porous soil or fractured rock, and the extent to which permeable materials are present that could provide preferential flow paths. The degree of uniformity and isotropy of the materials is also important, as well as determining the presence of continuous, low-permeability layers that may define flow boundaries.

Fundamental aquifer characteristics must also be assessed. These include the number of individual aquifers, their degree of isolation, and whether aquifer flow is confined or unconfined. Within each aquifer, water levels should be measured in a sufficient number of boreholes to accurately define groundwater movement between aquifers, flow directions, gradients, and velocities. On a more regional scale, it is important that areas of groundwater recharge and discharge be defined.

The location of all wells in the impoundment site vicinity must be cataloged, since the location of groundwater withdrawal points may be a major factor in determining the extent of any allowable contaminant migration and the width of required buffer zones between the wells and the impoundment.

In addition, baseline water quality data should be obtained both upgradient and downgradient from the impoundment site. These data will aid in determining the amount of contaminants that may be introduced into the groundwater without exceeding specified limits, and they may also provide an invaluable record of initial groundwater quality conditions in the event that future contamination questions arise during impoundment operation. References such as those listed in Table 10.1 provide discussions on groundwater sampling and analysis programs.

Aquifer Flow

For a variety of historical reasons, a special terminology for describing flow in aquifers has evolved. *Hydraulic conductivity* is a measure of the amount of water transmitted through a material of unit cross-sectional area. *Transmissivity* is a logical extension of this concept and is merely the product of hydraulic conductivity and saturated aquifer thickness. Therefore, the quantity of water flowing through a unit width of aquifer is simply:

$$Q = Ti$$

In the above equation, T represents transmissivity. Gradient, i, is the difference in total head measured at two points divided by the flow distance between those points, or in simplistic terms, the slope of the water table.

Another parameter of importance in saturated flow measurements describes the water-storage mechanisms active when an aquifer receives or releases water. Various terms have been used in this connection including *specific yield, specific storativity*, and *storage coefficient*. Strictly speaking, changes in the fluid mass stored per unit volume in a porous medium may be the result of three processes:

1. Filling or draining of pore space.
2. Compression of the fluid (change in fluid volume per unit change in pressure).
3. Changes in porosity (for example, consolidation of slimes).

The storage coefficient is defined as follows (Domenico, 1972):

$$S_c = m\gamma(\alpha + n\beta)$$

where S_c = storage coefficient
m = aquifer thickness
α = factor related to compressibility of aquifer matrix
γ = unit weight of fluid
β = factor related to compressibility of fluid
n = porosity

Therefore, the storage coefficient is a dimensionless number that relates the volume of water released from aquifer storage to a unit decline in pressure head per unit area of aquifer.

In most tailings impoundment seepage evaluations, the foundation and tailings can be assumed incompressible with little error ($\alpha = 0$). In certain situations, such as long-term evaluation of seepage from tailings slimes or impoundments constructed on soft, compressible soils, it may be necessary to account for consolidation-induced changes in porosity and hydraulic conductivity in order to obtain reasonable estimates of seepage.

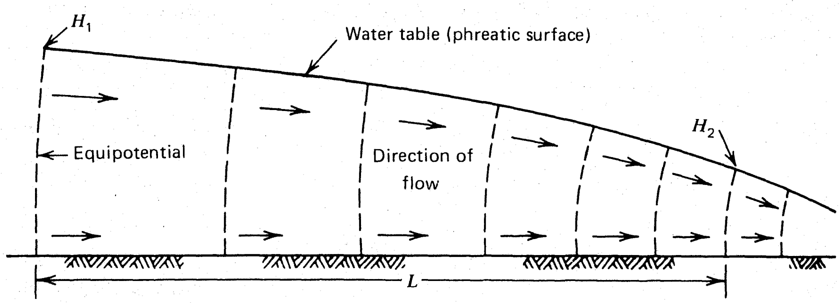
Groundwater hydrologists distinguish two major types of aquifers. *Unconfined* aquifers release or store water through changes in water level, and therefore storage is primarily a function of porosity ($\alpha = 0$, $\beta = 1.0$). *Confined* aquifers release or store fluids on the basis of changes in fluid and aquifer volume in response to changes in pressure. A schematic representation of confined and unconfined aquifers, highlighting the difference between the two, is presented in Figure 10.6.

Most seepage evaluations related to the design of tailings impoundments are concerned primarily with the uppermost aquifer, which is normally unconfined. Confined aquifers require relatively impervious layers above and below to maintain the fluid under pressure, which generally precludes contamination from surface impoundments. Occasionally, tailings disposal can affect confined aquifers—for example, disposal methods involving open-pit or underground mine backfilling. In most cases, however, unconfined aquifers are of interest and storage coefficient is adequately approximated by specific yield (fraction of pore space that actually releases water) or total porosity.

Field Measurements

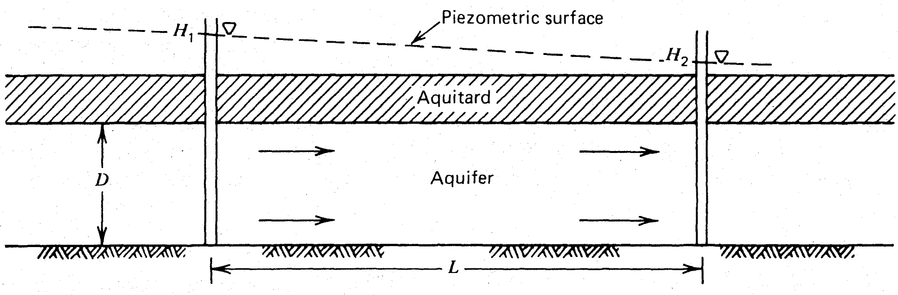
An important purpose of hydrogeologic investigations is to perform field permeability measurements, usually by either borehole tests, pumping tests, or both. Borehole tests to estimate hydraulic conductivity can generally be conducted within 1 hr to 1 day. Three tests are most common: packer tests, open borehole falling head tests, and "slug" tests conducted in piezometers. The accuracy of these tests can be considered order of magnitude but they are still valuable in assessing quickly the degree of hydraulic conductivity variation at a site and the requirements for pumping tests. Methods for analyzing borehole test data can be found in Cedergren (1967), Lambe and Whitman (1969), U.S. Bureau of Reclamation (1977), and Freeze and Cherry (1979). A summary of equations used in the analysis of borehole test data is presented in Table 10.2.

Pumping tests are the preferred means to estimate saturated flow variables for an aquifer. The first theoretical derivation relating the transient drawdown in an aquifer resulting from pumping was presented by Theis (1935) and is most applicable to confined aquifers that instantaneously re-



Features: Nonlinear gradient
$T = KH$
$$Q = Ti = \frac{K(H_1^2 - H_2^2)}{2L}$$
Storage change by draining or filling of pore space
$S_c = 0.01$ to 0.30

(*a*)



Features: Linear gradient
$$Q = KD\,\frac{(H_1 - H_2)}{L}$$
Storage change by compression or expansion of aquifer and fluid
$S_c = 10^{-6}$ to 10^{-3}

(*b*)

Figure 10.6 Aquifer terminology. (*a*) Unconfined aquifer. (*b*) Confined aquifer.

lease water from storage. Analysis of pumping test data from unconfined aquifers is most commonly performed with methods described by Boulton (1963) that account for the delayed release of water from storage commonly observed in unconfined aquifers. A correction factor that is applied to the Theis method allowing for decreases in transmissivity was developed by Jacob (1944) and is listed in Table 10.2.

The subject of pumping test data interpretation is extensively discussed in the literature. Two excellent references summarizing methods of conducting pumping tests and interpretation of results are presented in Kruseman and

Table 10.2 Summary of Equations for Aquifer Test Data Analysis

Test Type	Equation	Comments
Packer	$K = \frac{Q}{2\pi L(\Delta H)} \ln (L/R_1)$	used only in rock or soils not susceptible to cave-in
Slug, open borehole	$K = \frac{r^2}{2L} \ln \frac{L}{R_1} \left[\frac{\ln (h_2/h_1)}{(t_2 - t_1)}\right]$	can be performed in open hole or completed well
Pumping, transient analysis	$T = \frac{Q}{4\pi s} W(u_y, R_2/B)$	tabulated values of $W(u_y, R_2/B)$ in Walton (1970) and Kruseman and DeRidder (1976)
Pumping, steady state	$K = \frac{Q \ln (R_2/R_1)}{H_2^2 - H_1^2}$	used to estimate K from wells that have been pumped continuously for long periods of time
	$s_{cor} = s - (s^2/2D)$	drawdown correction factor for unconfined aquifers

Notes: L = length of aquifer interval tested; r = casing radius; R_1 = borehole radius; R_2 = distance between pumped well and observation well; D = aquifer saturated thickness prior to pumping; s = drawdown of water level resulting from pumping; t = time; h = head differential.

DeRidder (1976) and Lohman (1972). A summary of unconfined pumping test equations is presented in Table 10.2, and test methods are shown diagrammatically in Figure 10.7.

In summary, a hydrogeologic investigation normally consists of a drilling program to delineate geology and aquifers. These investigations can, and should, be integrated with geotechnical investigations. During drilling, observation wells are installed to measure water levels from which horizontal and vertical groundwater flow rates and directions can be estimated. Quick estimates of hydraulic conductivity are obtained through a combination of open borehole tests and slug tests in completed observation wells. The results of borehole tests provide the basis for planning and design of pumping tests, if necessary. The results of all field and laboratory testing ultimately provide the physical parameters necessary for seepage analysis.

Geochemical Investigations

Tailings effluents, particularly those produced by acid or alkaline mill processes, are a complex mixture of potentially toxic chemical compounds. The

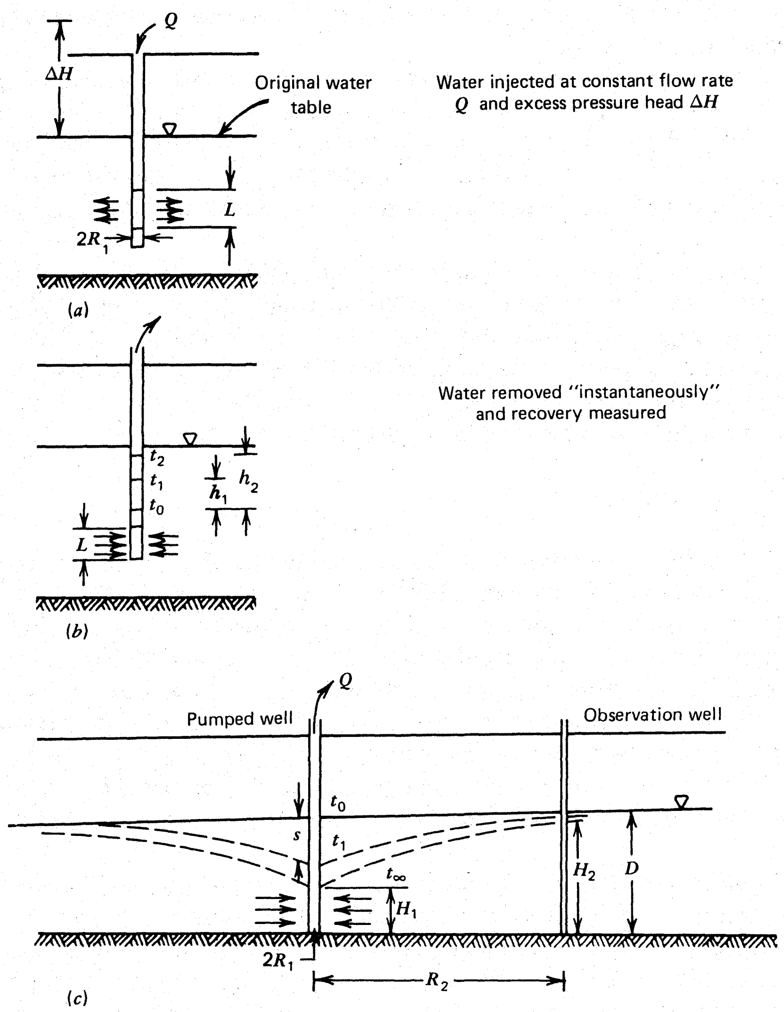


Figure 10.7 Aquifer test methods. (*a*) Packer test. (*b*) Slug test. (c) Pumping test.

amount of contaminant removal resulting from chemical reactions as seepage moves through the subsurface is an important consideration for groundwater quality and protection of the environment. Contaminant removal is a result of dilution processes and chemical reactions between seepage and (a) foundation soils; (b) aquifer solids; (c) pore water stored in the partially saturated zone; and (d) groundwater.

Evaluation of possible chemical reactions relies on available theoretical concepts in geochemistry. However, the complex composition of some tailings effluents can be simplified by measurement of controlling parameters, such as pH, in laboratory and field investigations. Empirical relationships are then used to relate the controlling parameters to concentrations of metals and other ions in solution.

There is a lack of standardized test methods for measurement of geochemical parameters, making it difficult to compare results from similar investigations directly or to apply previous test results to information gathered at new sites. Nonetheless, geochemical investigations provide a powerful method for assessing the impact of seepage from tailings impoundments.

The range of geochemical parameters that need to be assessed in an investigation depends on the nature of the impounded waste and its interaction with an impoundment liner and the foundation materials. A discussion of the geochemical parameters required for evaluating tailings impoundment seepage is given by Shepherd and Cherry (1980), who provide some suggestions for standard test procedures for uranium mill wastes.

Geochemical Parameters

Chemical reactions are often far more significant than dilution by volumetric mixing in reducing contaminant concentrations, particularly for acid mill effluents. Chemical reactions depend on the mineral compositions of the geologic materials contacted by the mill effluent, the chemical composition of the effluent, the chemical composition of the subsurface waters, temperature, and pressure. Adding to the complexity of geochemical interactions is the fact that individual chemical reactions can proceed at vastly different rates. However, the empirical tests described herein provide simplified methods for obtaining useful information from these complex geochemical systems.

The *equilibrium state* is a useful concept in geochemistry that is applicable to evaluations of seepage from tailings impoundments. The primary factors controlling the final equilibrium state are pH, oxidation potential (Eh), temperature, and pressure, with pH normally being the most important parameter. Tailings seepage that enters the subsurface is chemically out of equilibrium with soils and groundwater, resulting in chemical reactions that proceed toward a new chemical equilibrium state. On a crude basis, effluents can be characterized by their level of total dissolved solids (TDS) and pH. Figure 10.8 illustrates the difference between typical TDS and pH levels for several types of effluents and initial groundwater quality as approximated by drinking and livestock water quality standards. The conditions shown in Figure 10.8 represent those prior to establishment of a new equilibrium state.

An extensive literature containing thermodynamic data for many simple aqueous chemical systems is available. These data are useful in examining possible reactions of tailings effluents with subsurface soils and waters but give no information on the rate at which the reactions proceed or the effects of complex chemical systems on particular chemical species. The interested reader can find general discussions on thermodynamic theory and available

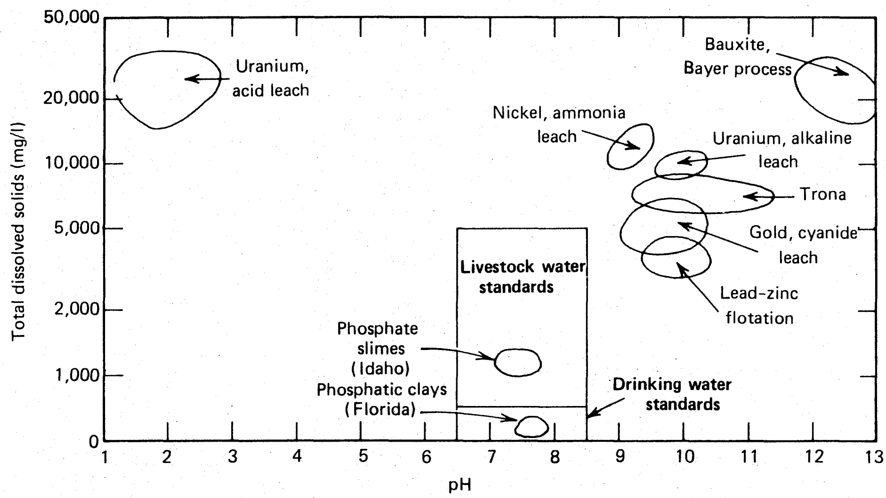


Figure 10.8 Chemistry of mill effluents.

data for simple chemical systems in Freeze and Cherry (1979), Garrels and Christ (1965), and Stumm and Morgan (1981). A comprehensive description of colloid geochemistry, an important consideration for waste effluents in contact with soils, is outlined in Yariv and Cross (1979).

Six types of chemical reactions are important in the context of tailings seepage evaluations:

1. *Neutralization Reactions.* These are reactions that occur as a result of highly alkaline or highly acidic solutions contacting groundwater and/or reactive subsurface materials. A commonly encountered example is the reaction that occurs between acidic tailings effluents and carbonate minerals in limestone, carbonate or bicarbonate ion in groundwater, or carbonate minerals in alkaline soils in regions of dry climate. Neutralization reactions would also occur between alkaline effluents and acidic soils in humid regions.
2. *Oxidation-Reduction Reactions.* These reactions are particularly important for solubility and complexing of elements that display more than one valence state depending on the oxidation potential of the fluid—for example, iron (II)–iron (III) and uranium (IV)–uranium (VI). Some mill process fluids, including uranium and gold, are maintained at high oxidation states to enhance mineral solubility. Seepage of highly oxidized effluent that contacts reduced groundwater or soils may result in precipitation reactions.
3. *Precipitation Reactions.* Both neutralization reactions, which change seepage pH, and oxidation-reduction reactions, which change the oxidizing potential of seepage, affect the solubility of dissolved species in solution. For example, as acidic seepage neu-

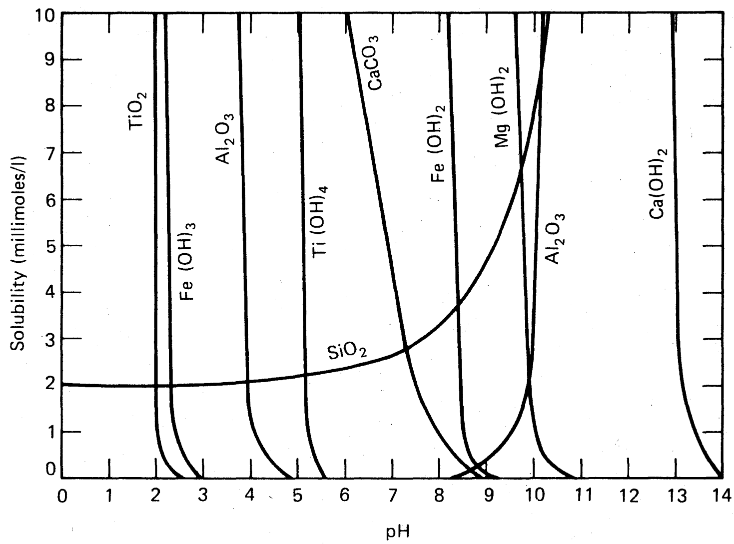


Figure 10.9 Solubility as a function of pH. (Reprinted from Loughnan, 1969.)

tralizes, hydroxyl ions become available to form insoluble metal hydroxides. If a dissolved ionic contaminant forms a solid precipitate of low solubility, its mobility and potential for groundwater contamination will be sharply reduced or eliminated. Precipitation reactions may result from changes in any or all of the four thermodynamic variables: pH, Eh, temperature, or pressure. Precipitation of carbonates from groundwater discharging from springs is a well-known example of pressure dependent solubility. However, precipitation reactions resulting from temperature or pressure changes in tailings seepage can normally be considered minor in relation to precipitation reactions resulting from neutralization. The solubility of many chemical species in mill effluent solutions is highly dependent on solution pH. Therefore, neutralization of seepage is an important factor in removal of contaminants.

The solubility of simple elemental compounds as a function of pH is displayed in Figure 10.9. As can be observed, some compounds are soluble only in acidic conditions, some are soluble only in alkaline environments, and some are soluble in both. For example, silica is relatively insoluble at neutral and highly acidic pH but very soluble in alkaline environments, whereas alumina is insoluble in the neutral pH range (5–8.5) but very soluble outside this range. From the information presented in Figure 10.9, it is apparent that neutralization of acid seepage will lead to precipitation of such metals as aluminum, iron, and titanium, with similar effects on more toxic metals such as cadmium, lead, and radium.

Elements having the highest potential for groundwater contami-

Table 10.3 Solubility of Trace Element Compounds

Toxic Elements at Concentrations < 1 mg/l	Compound	Interim Primary Drinking Water Standards (mg/l)	Solubility in Distilled Water at 1 atm & 25°C (mg/l)
As	As_2S_3	0.05	0.5
	As_2S_5		1.4
	$H_3AsO_4 \cdot \frac{1}{2}H_2O$		>1,000
	As_2O_3		>1,000
Cd	$CdCO_3$	0.01	Insoluble
	$Cd(OH)_2$		Insoluble
	CdS		Insoluble
Cr	CrO_3	0.05	>1,000
	Cr_2O_3		Insoluble
CN	$AgSCN$	0.02	0.2
	$CuCN$		2.6
	$CuSCN$		Insoluble
Pb	$Pb(AsO_4)_2$	0.05	Insoluble
	$Pb(CO_3)$		1.7
	$Pb(OH)_2$		160
	$PbSO_4$		45
Hg	HgO	0.002	50
	HgS		Insoluble
Se	H_2SeO_3	0.01	>1,000
Cu	$Cu(OH)_2$	(Toxic to fish)	Insoluble
	Cu_2S		Insoluble

Source: Aylward and Findlay (1974)

nation at wells or discharge areas remote from the impoundment area are those that are toxic at concentrations less than 1 mg/l. These elements are listed in Table 10.3 in comparison with the measured solubility of typical compounds that could be expected from precipitation reactions. As indicated, selenium and arsenic, if present in tailings effluent seepage, may not be removed by result of neutralization reactions.

Under conditions of uniform temperature and pressure, the *stability* of a solid phase mineral is mainly controlled by the acidity or alkalinity and the oxidizing potential of the solution it is in contact with. In simple terms, the oxidizing potential is related to the amount of dissolved oxygen in solution, although other compounds may also act as oxidants.

Reactions in which oxygen is consumed, for example, pyrite oxidation, lower the oxidation potential of the solution. The shaded areas in stability diagrams such as those in Figure 10.10 indicate regions in which particular elements are thermodynamically stable as solid mineral phases. It should be noted, however,

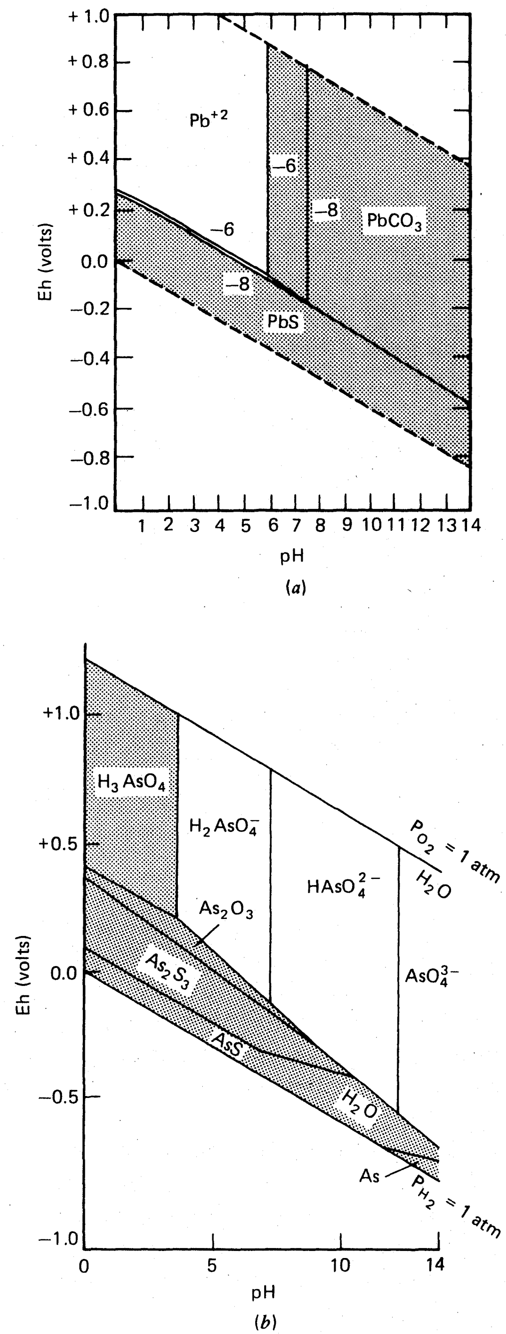
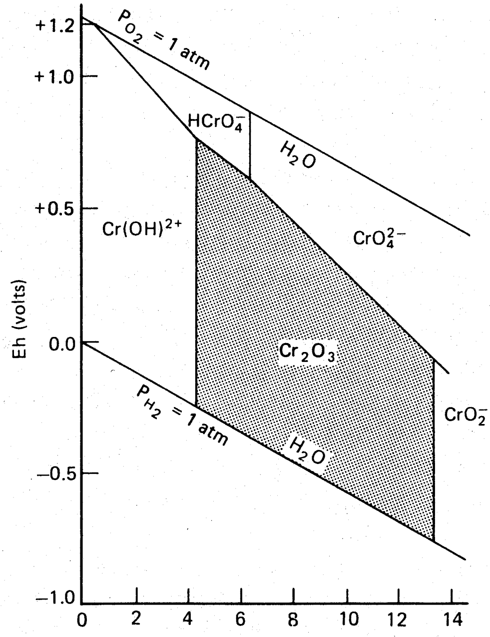
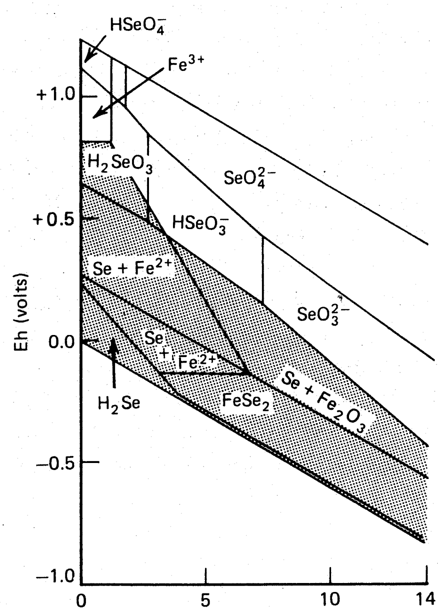


Figure 10.10 Stability diagrams for selected trace elements. Shaded areas represent solid phase. (From Longmire et al., 1981.) (*a*) Lead. (*b*) Arsenic. (*c*) Chromium. (*d*) Selenium.



P_{O_2} = 1 atm
$HCrO_4^-$
H_2O
$Cr(OH)^{2+}$
CrO_4^{2-}
Cr_2O_3
CrO_2^-
P_{H_2} = 1 atm
H_2O
Eh (volts)
pH
+1.2
+1.0
+0.5
0.0
−0.5
−1.0
0
2
4
6
8
10
12
14

(*c*)



$HSeO_4^-$
Fe^{3+}
H_2SeO_3
SeO_4^{2-}
$HSeO_3^-$
$Se + Fe^{2+}$
SeO_3^{2-}
Se + Fe^{2+}
$Se + Fe_2O_3$
$FeSe_2$
H_2Se
Eh (volts)
pH
+1.0
+0.5
0.0
−0.5
−1.0
0
5
10
14

(*d*)

that stability diagrams in Figure 10.10 give no indication of the rate at which the end products are produced, and that the systems shown are chemically very simple. Longmire et al. (1981) summarize the stability diagrams for most elements which are toxic at concentrations less than 1 mg/l, as well as several other major elements, such as iron.

On an empirical level, the equilibrium approach provides useful guidelines for estimating groundwater contamination potential. If pH and Eh of the seepage as a function of travel distance can be estimated or measured, upper bounds on concentrations and mobility of trace metals and toxic elements can also be estimated (Longmire et al. 1981). The use of pH and Eh, estimated from thermodynamic data or by direct measurement, can be used to make qualitative assessments of contaminant migration potential. Geochemical control measures can then be designed to prevent migration. An example of this approach is the barium chloride treatment used to coprecipitate radium from uranium tailings effluent mentioned in Chapter 1.

4. *Adsorption.* Adsorption is not a chemical reaction in the strict sense, but represents an important method by which trace elements are bonded or attached on soil particles. The exact mechanism of removal by adsorption is poorly understood, but cations are usually adsorbed to a much greater degree than anions. The anions sulfate, chloride, and nitrate generally show little or no attenuation by adsorption after contact with soils.
5. *Ion Exchange.* This mechanism is normally attributed to the special chemical properties associated with clay minerals. Clay minerals contain interlayers of cations that are readily exchangeable with cations in solution. Thus, it is possible to exchange toxic cations in the effluent, such as lead, for ions in the clay minerals with low toxicity, such as calcium.
6. *Biological Reactions.* Ordinarily of importance mainly in surface flow, this class of reactions involves consumption and immobilization of pollutants by microorganisms. As noted in Chapter 1, biological reactions are used to mitigate sulfate contamination in surface ponds. Also, Brierly et al. (1980) describe the use of algae in surface ponds to reduce heavy metal concentrations from effluent mine water. Bacteria also play an important role in the rate at which pyrite oxidation proceeds.

Laboratory Investigations

Consideration of the above chemical mechanisms enables the priorities for conducting a geochemical investigation to be established. A logical sequence of studies is as follows:

1. Chemical characterization of the tailings effluent, tailings solids, groundwater, foundation materials, and aquifer materials. This should include simple index tests and then, if warranted by the toxicity of the tailings effluent, detailed chemical and mineralogical analyses.
2. Identification of key geochemical buffering zones—for example, clay liners for impoundments over high-permeability aquifers, and materials in the partially saturated zone for unlined impoundments.
3. Laboratory testing of seepage–groundwater interactions.

Index tests that provide simple characterization of materials are summarized in Table 10.4. These tests provide preliminary information that aids in the planning of detailed geochemical laboratory investigations. For example, the calcium carbonate content of a soil provides a very good indication of the amount of acidic effluent that can be neutralized by the soil during seepage flow. Clay content and cation exchange capacity are also indicative of soil reactivity. A comprehensive summary of soil properties, particularly the physical and chemical properties of clay minerals, can be found in Mitchell (1976). Procedures for conducting the index tests listed in Table 10.4 are discussed by Black (1965).

Three types of detailed laboratory experiments to determine contaminant uptake of soils are available: column tests, batch tests, and sequential batch tests, each with unique advantages and disadvantages. In any geochemical investigation it must be realized that tests represent an accelerated version of the events that may occur at the actual site. When the reactions proceed rapidly in the laboratory, concentrations of reactants and products are controlled by their equilibrium states. In most seepage evaluations for tailings impoundments, precipitation reactions controlled by pH can usually be assumed to be rapid with little error. If, however, reactions proceed slowly,

Table 10.4 Index Tests for Geochemical Properties of Soils

Test	Comment
Cation exchange capacity	High cation exchange capacity is associated with montmorillonitic clays, effective in adsorbing contaminants.
X-ray diffraction	Mineral identification allows correlation of geochemical properties with similar materials.
Sediment paste pH	Indicative of soil buffering capacity.
Calcium carbonate equivalent	Usually directly related to neutralization capacity.
Particle size distribution	Proportion of clay particles, usually those less than 2 microns in size, aids in estimating cation exchange potential.

they are said to be *kinetically* or *rate controlled,* and such reactions may not be accurately reflected in laboratory tests. Even though pH-dependent reactions may proceed rapidly, the reactions that alter pH may proceed slowly and therefore also control their rates. An example of a rate-controlled reaction is oxidation of pyrites in tailings or solutioning of limestone that produces karst.

Measurements of pH, total dissolved solids, and major ions identified in the tailings effluent analyses, as well as any known toxic compounds, should be included for analysis in a laboratory geochemical investigation. Measurement of oxidation potential is also important when highly oxidized process effluents will contact organic or other soils with low oxidation potential. However, the measurement of oxidation potential is difficult.

Column tests have traditionally been used to investigate attenuation of contaminants and are discussed by Freeze and Cherry (1979). Tailings effluent is passed through a column of soil, and contaminant discharge concentrations are measured at regular time intervals. A plot of discharge concentration versus the ratio of fluid volume passed through the column to the mass of soil in the column is prepared as shown in Figure 10.11a. Column tests provide the advantage of being able to vary fluid velocity and observe the resulting effect on attenuation. Unrealistically high gradients often required in column tests on low-permeability materials result in fluid velocity orders of magnitude higher than expected field velocities, possibly affecting chemical reactions that depend on partial pressure of gases such as carbon dioxide. Column tests require relatively long periods of time to complete for low-permeability materials, such as clay liners.

Batch tests are a convenient method of avoiding the long delays associated with column tests. Tailings effluent is added in small increments to loose soil and mixed for a short period of time, usually less than 1 day. Batch tests provide quick results but may overestimate the amount of attenuation capacity because of the increased surface area available for reaction during mixing. Both batch and column tests simulate fluid movement through a fixed volume of soil and therefore are applicable to investigations of relatively thin clay liners or soil horizons for which it is anticipated that the entire reactive capacity may be exhausted during seepage flow.

Griffin et al. (1980) describe a sequential batch test method that simulates contaminant removal as a function of travel distance through the soil. Sequential batch tests are best suited to evaluating attenuation in thick sequences of material, such as partially saturated foundations, or movement through aquifers. The tests are performed by mixing a fixed volume of tailings effluent with soil for approximately 1–1.5 hr. The effluent is decanted from the slurry and then added to a fresh batch of soil. This process is repeated several times and results are plotted as shown in Figure 10.11b. The soil–effluent ratio can be directly related to the seepage travel distance required to remove contaminants. Sequential batch tests have the same limitations as batch tests but provide inexpensive and fast results.

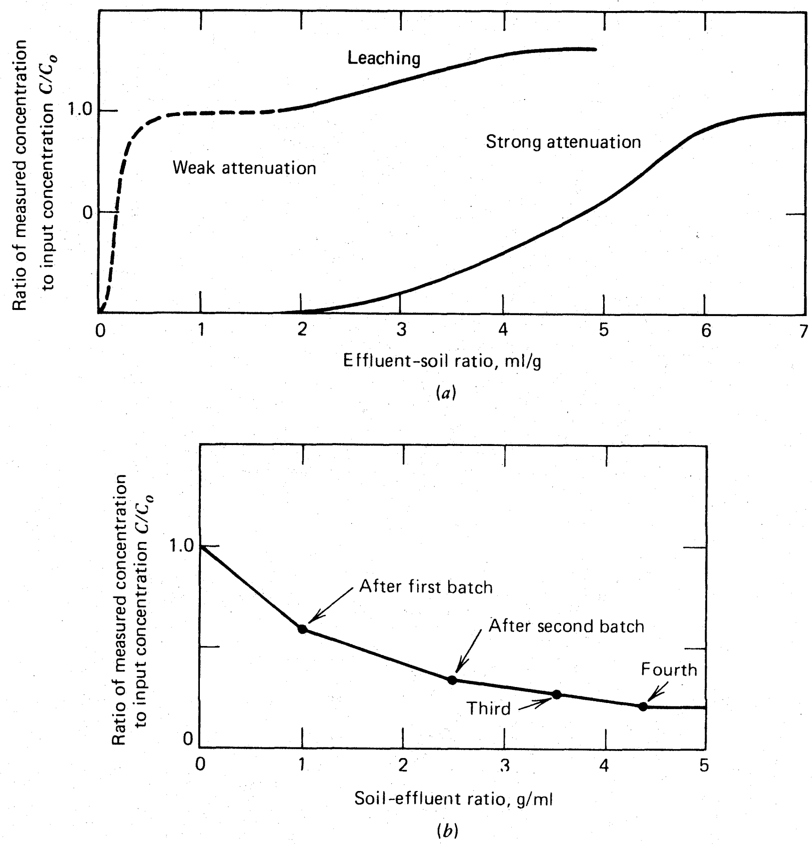
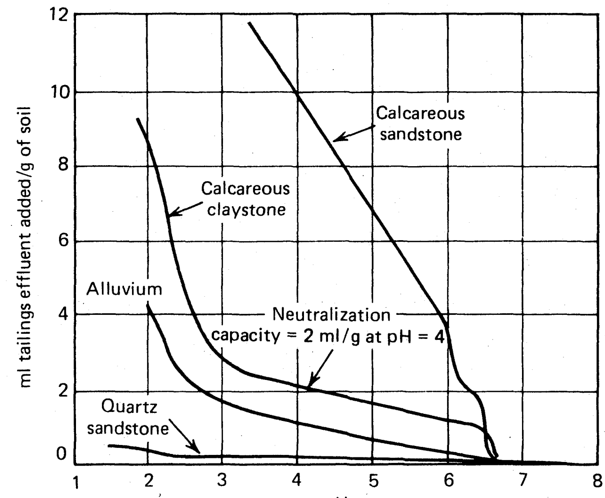


Figure 10.11 Typical geochemical laboratory test results. (*a*) Column or batch tests. (*b*) Sequential batch tests.

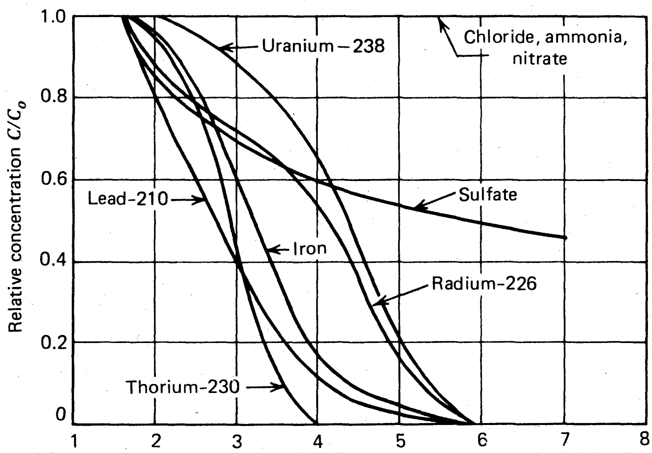
Since pH controls the solubility of many contaminants, particularly trace metals, the *neutralization capacity* of a material is a fundamental parameter for measurement. Neutralization capacity is defined as the volume of tailings effluent that must be added per unit mass of soil to reach a specified pH level. Figure 10.12 displays batch test results as a function of pH for acid-leach uranium effluent showing the neutralization capacity to pH 4 for several material types. For these tests, it was observed that control samples neutralized with calcium hydroxide showed similar concentrations of metals and radionuclides, demonstrating that pH is the parameter controlling contaminant transport.

Distribution Coefficient

The combined effects of contaminant removal by adsorption and ion exchange can be quantified with the *distribution coefficient*. The distribution



(*a*)



(*b*)

Parameter	Approximate initial concentration (C_0)
Sulfate	24,000 mg/l
Chloride	1,600 mg/l
Nitrate	30 mg/l
Ammonia	35 mg/l
Iron	2,400 mg/l
Radium-226	3,300 pCi/l
Uranium-238	6,500 pCi/l
Lead-210	24,000 pCi/l
Thorium-230	149,000 pCi/l

Figure 10.12 Example batch test results. (*a*) Neutralization capacity. (*b*) pH-dependent solubility.

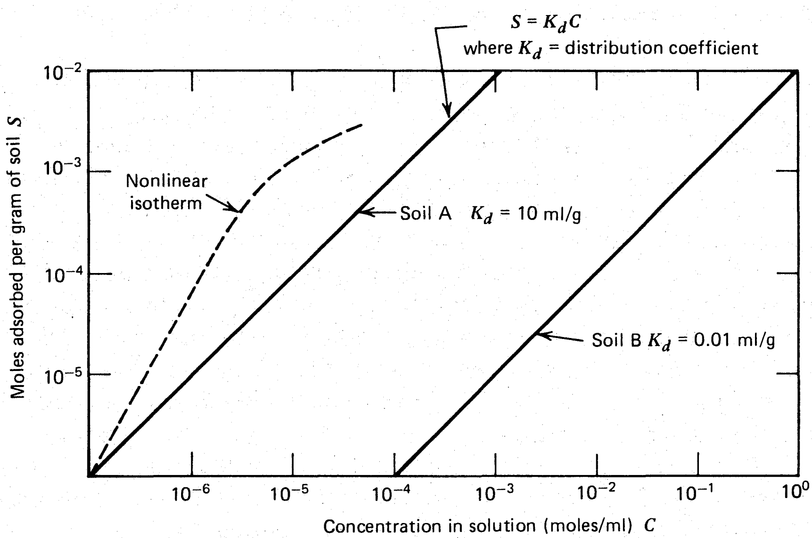


Figure 10.13 Calculation of distribution coefficient.

coefficient is a "partitioning" coefficient: the amount of contaminant removed is assumed to be directly proportional to the mass of solids contacted by the fluid. Therefore, data obtained from column tests or batch tests are suitable for making estimates of the distribution coefficient. A plot of solution concentration versus concentration retained in the solids is prepared as shown in Figure 10.13. A linear relationship between the moles adsorbed per unit mass of solids and the solution concentration meets the definition of the distribution coefficient K_d, most commonly expressed in ml/g. A detailed mathematical treatment of the distribution coefficient as well as nonlinear relationships is presented in Bear (1979). Taylor (1980) presents an empirical extension of the distribution coefficient by relating it to pH.

The distribution coefficient has been used in many contaminant transport investigations, as discussed in Freeze and Cherry (1979), but it is subject to the following limitations:

1. The concentrations of the contaminant must be relatively low.
2. Flow must not be concentrated within geologic discontinuities.

Since adsorption and ion exchange mechanisms have limited capacity to remove contaminants, high effluent contaminant concentrations will quickly exhaust the removal capacity of the host material. Similarly, fractured media, causing channelized flow of effluent seepage, would provide much less surface area for reaction than the material tested in the laboratory, resulting in overestimates of contaminant removal. Other concerns with the concept of the distribution coefficient are discussed by Reardon (1981).

METHODS FOR EVALUATING SEEPAGE QUANTITIES AND IMPACTS

There are many methods for estimating seepage loss and contaminant transport, ranging from the simple water balance method to complex numerical models. Simple methods of analysis are used during the preliminary design stage to estimate the magnitude of groundwater contamination potential and also to check the results of more sophisticated analyses. The preliminary design estimates may be sufficient if, in conjunction with worst-case assumptions, groundwater contamination potential is shown to be low. More sophisticated analyses are required if the preliminary estimates indicate potential for severe groundwater contamination, if the tailings effluent contains high concentrations of toxic compounds that do not readily react or decompose, or if regulatory agency guidelines require a specific type of analysis. Highly sophisticated methods are reserved for cases where they may provide significant savings in cost of seepage control systems, where the geometry of the tailings impoundment and geology are complex, or where detailed analyses of partially saturated flow behavior are required.

In the following sections, three types of seepage evaluation methods are described that represent increasing levels of sophistication—lumped parameter methods, analytical methods, and numerical methods, as summarized in Table 10.5. All contaminant transport analyses require that seepage velocity be known. Therefore, groundwater flow directions, gradients, and velocities should be well established prior to evaluating contaminant transport.

Lumped Parameter Methods

The lumped parameter approach could also be called the "black box" method. Overall effects are estimated without accounting for specific mechanisms that produce these effects. Seepage estimates are based on the net difference between all inflows and outflows to the impoundment. This is in direct contrast to analytical and numerical methods, which rely on detailed estimates of permeability, impoundment depth, location of geologic boundaries, and other factors. Although the estimates obtained with lumped parameter methods are often very crude, the methods are valuable in establishing realistic upper bounds on the magnitude of seepage to be expected. Use of these simple methods as a check can avoid the embarrassing but not uncommon prediction of seepage quantity that exceeds inflow to the impoundment.

Impoundment Water Balance

Water balance calculations for the purpose of estimating long-term water excess or deficit in a tailings impoundment have been discussed in Chapter 4. The water balance can also be used to estimate the maximum possible quantity of seepage. Using the example presented previously in Table 4.2, the maximum possible seepage quantity is the residual of the balance be-

Table 10.5 Summary of Methods for Evaluating Seepage Loss and Impacts

Method	Application
Lumped parameter Impoundment water balance Partially saturated zone storage Salt balance Neutralization capacity Attenuation distance	Used to establish bounds and preliminary comparative evaluations, and as a check on numerical analyses
Analytical Darcy's Law Partially saturated flow Mounding equations Contaminant transport	Used to provide detailed estimates of seepage movement with time for uniform geologic conditions
Numerical methods	Used to analyze complex geometry, partially saturated flow, and chemical reactions

tween inputs and outputs of 183 gpm. Therefore, without any knowledge of tailings or foundation permeability, an upper bound is established.

Although the example in Chapter 4 provides a long-term average estimate for a fully developed tailings impoundment, the water balance method is easily extended to predicting maximum possible seepage quantity at any stage in the development of sidehill or cross-valley impoundments. Filling of sidehill and cross-valley impoundments progressively increases the decant pond area, which in turn alters quantities of evaporation, direct precipitation, and tributary runoff. Similarly, the effects of changes in mill production rate during the life of the operation can be estimated rapidly.

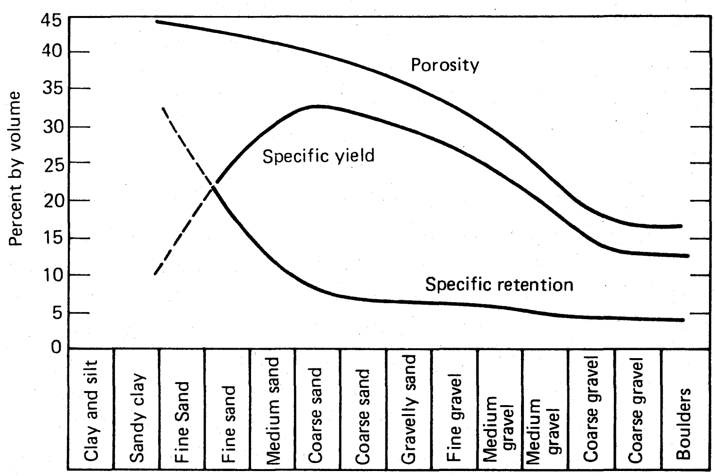
Storage in the Partially Saturated Zone

Capillary forces that develop in partially saturated soils result in retention of water in the soil matrix. *Specific retention* (SR) of a soil is defined as the fractional volume of water retained by a sample initially saturated and then allowed to drain freely under the influence of gravity. *Specific yield* (SY) is the fractional water volume that drains out. Therefore, porosity is equal to the sum of specific yield and specific retention. Typical values of specific retention, specific yield, and porosity are displayed in Figure 10.14.

The initial moisture content of soils in arid environments can be well below the specific-retention value. Initial seepage from impoundments entering soils in arid environments will be consumed in filling this available pore space and will be prevented from entering the groundwater in underlying aquifers. The volume of fluid that can be permanently retained in the partially saturated zone is given by

	Porosity (percent)
Unconsolidated deposits	25–40
Gravel	25–40
Sand	25–50
Silt	35–50
Clay	40–70
Rocks	
Fractured basalt	5–50
Karst limestone	5–50
Sandstone	5–30
Limestone, dolomite	0–20
Shale	0–10
Fractured crystalline rock	0–10
Dense crystalline rock	0–5

(*a*)



(*b*)

Figure 10.14 Relationship between porosity, specific yield, and specific retention. (From Wilson, n.d.) (*a*) Range of porosity values in unconsolidated deposits and rocks. (*b*) Variation of porosity, specific yield, and specific retention with grain size.

$$V_r = (\mathrm{SR} - \theta_i)V_t$$

where V_r = volume of storage available
SR = specific retention
θ_i = initial volumetric moisture content (volume of water divided by total volume) where $\theta = wG\left[\frac{1}{wG + 1/G}\right]$ from Chapter 1
V_t = total pore volume beneath the impoundment and above the water table

Table 10.6 Example Calculation of Storage Available in Partially Saturated Zone

Porosity (n)	= 0.40
Water content (w)	= 0.04
Specific retention (SR)	= 0.20
Specific gravity of solids (G)	= 2.65
Impoundment area	= 120 acres
Depth to water table	= 100 feet
Maximum impoundment seepage rate	= 183 gpm (295 acre feet/yr)

Item	Volume (acre feet)
Total pore volume	4800
Less initial water content	−763
Equals maximum storage (at full saturation)	4037
Storage volume at specific retention	2400
Less initial water content	−763
Equals permanent storage	1637

The volume of fluid required to completely saturate the partially saturated zone is

$$V_s = (n - \theta_i) V_t$$

A crude estimate of the time required to reach the specific retention moisture content below the entire impoundment is obtained with

$$t = V_r / q$$

where t = time
q = average seepage rate

Continuing with the same example used for the water balance, estimation of available permanent storage in the partially saturated zone in relation to the maximum seepage quantity can be used to provide preliminary estimates of groundwater contamination potential. For the same hypothetical impoundment, it is assumed that the depth to the water table is 100 ft and the foundation is silty sand with initial average water content of 4% (equivalent to 6.4% by volume). Referring to Figure 10.14, the specific retention for the foundation material is estimated as 20% and porosity is approximately 40%. Pore volume, volume of seepage, and volume available for storage are listed in Table 10.6. In this example, the partially saturated zone can permanently retain 1637 acre feet, the equivalent of 5.5 yr of seepage at a maximum rate escaping the impoundment of 183 gpm. Temporarily the partially saturated zone can retain up to 4037 acre feet but this storage will be reduced after impoundment operation ceases and the soil drains to specific retention.

Salt Balance

The salt balance is a lumped parameter method for estimating the impacts of seepage on water quality. The salt balance follows from the observation that the amount of contaminant introduced into an aquifer cannot exceed the total amount contained in seepage that has left the impoundment. A salt such as chloride that does not participate in chemical reactions and would not be leached from most geologic materials is best for illustration purposes.

The contaminant concentration in seepage completely mixed with groundwater flowing beneath a tailings impoundment is

$$C_m = (C_s V_s + C_g V_g)/(V_g + V_s)$$

where C_m = contaminant concentration in completely mixed seepage and groundwater
C_s = contaminant concentration in seepage prior to mixing
C_g = contaminant concentration in groundwater prior to mixing
V_s = volumetric flow rate of seepage
V_g = volumetric flow rate of groundwater

The relationship between mixed concentration and seepage to groundwater flow ratio is shown in Figure 10.15, assuming zero initial contaminant concentration in the groundwater. Complete mixing may not occur until flow has progressed 1 mi or more away from the impoundment. The salt balance calculations are therefore appropriate for making crude estimates of water quality impacts at more distant points of groundwater use outside mining property boundaries. Use of the salt balance method in the near vicinity of the impoundment will underestimate contaminant concentrations.

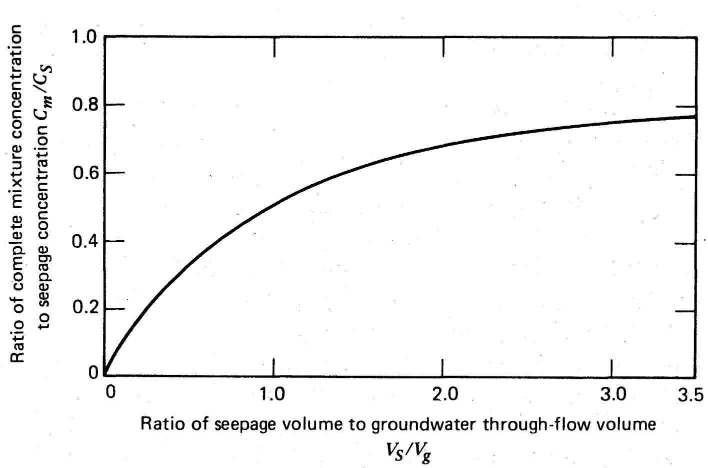


Figure 10.15 Completely mixed concentrations versus ratio of seepage to groundwater through flow volume.

As an example, uranium mill effluent typically contains chloride at a concentration of about 300 ppm. If seepage were estimated as 20% of the volume of groundwater flowing beneath an impoundment ($V_s/V_g = 0.20$ on Figure 10.15), chloride concentration could be expected to be 50 ppm above the ambient concentration in the groundwater on the basis of complete mixing. This example demonstrates that mill effluents that do not themselves meet water quality standards (250 ppm for chloride) can meet water quality criteria at a point of use sufficiently far from the impoundment by sensible use of buffer zones and dilution. Current U.S. Environmental Protection Agency regulations for surface water quality are based on similar concepts.

Neutralization Capacity

Mill effluents with pH far from neutral are of particular concern because of the high solubility of potentially toxic elements, including heavy metals, outside the neutral range. Neutralization capacity of soils can be used to estimate the amount of pH buffering that will take place in clay liners and/or the partially saturated zone prior to seepage reaching groundwater aquifers. A bulk neutralization estimate is based on data obtained from batch or column tests and is applicable to clay liners and foundation materials of relatively low permeability; high permeability foundations that transmit seepage at velocities in excess of 1–2 ft/day may not provide sufficient contact time for neutralization reactions to proceed to completion.

Using the same impoundment assumptions utilized in the partially saturated zone storage calculations, a comparison of a clay-lined with an unlined impoundment is considered, as shown in Table 10.7. This example assumes a tenfold decrease in seepage rate by installation of a liner com-

Table 10.7 Example Comparison of Clay Liner and Partially Saturated Zone Neutralization Capacity

	Clay Liner	Partially Saturated Zone
Thickness	2 ft	100 ft
Effluent pH	2	2
Neutralization capacity to pH = 4	2 ml/g	0.3 ml/g
Porosity	45%	40%
Unit weight	90 pcf	100 pcf
Impoundment seepage rate	20 gpm	183 gpm
Impoundment area	120 acres	120 acres
Seepage volume neutralized to pH = 4	2.25×10^8 gal	1.90×10^9 gal
Equivalent years of seepage	21.4	19.5

posed of clay having a neutralization capacity approximately 10 times greater than that of soils in the partially saturated zone. For this particular comparison, it is evident from Table 10.7 that installation of the clay liner would provide no more neutralization capacity and consequent attenuation of toxic elements than would already be present naturally in the partially saturated zone.

Attenuation Distance

During preliminary evaluations of groundwater contamination potential, it is often of interest to quickly estimate the maximum possible migration distance required to attenuate toxic contaminants to acceptable concentrations. Data obtained from sequential batch tests proposed by Griffin et al. (1980) is most useful for this purpose.

Using the same parameters assumed for the partially saturated zone shown in Table 10.6, it is further assumed that chemical analyses of the mill effluent under consideration have identified arsenic, with an average concentration of 1 mg/l, to be the toxic ion with the highest potential for exceeding groundwater drinking standards (0.05 mg/l). It is also observed from sequential batch test data for the silty sand foundation materials that the soil–effluent ratio required to reduce the arsenic concentration of seepage to 5% of the effluent concentration is 2 g/ml. Using the above assumptions together with those listed in Table 10.6, the distance that arsenic will migrate vertically through the foundation in 20 yr is calculated as follows:

$$20 \text{ yr} \times \frac{183 \text{ gpm}}{120 \text{ acres}} \times 2 \text{ g/ml} \div 100 \text{ pcf} \times \begin{matrix}\text{unit}\\ \text{conversion}\\ \text{factors}\end{matrix} \cong 60 \text{ ft}$$

Griffin notes that calculations based on sequential batch tests do not account for mixing with pore water stored in the partially saturated zone or waters introduced by rainfall infiltration and therefore should provide reasonably conservative estimates of contaminant transport distance.

Analytical Methods

Analytical methods are applicable to simple systems characterized by uniform geometry and material properties such that description using exact mathematical formulas is reasonable. The predictions possible with analytical equations are more detailed than those that can be made with lumped parameter methods but require more input data. With basic information on permeability, dispersion coefficient, and distribution coefficient, order of magnitude estimates of such items of interest as mounding height, time required for seepage to reach groundwater, and amount of dilution can be made.

Darcy's Law

All methods of calculating flow in saturated and partially saturated materials are based ultimately on Darcy's Law:

$$Q = KiA$$

where Q = volumetric flow rate
K = hydraulic conductivity
i = hydraulic gradient (headloss per unit distance of flow)
A = area of flow

Darcy's Law is directly analogous to Fourier's Law, used in heat transfer calculations, and Ohm's Law, used in calculations of electrical current. Fourier's Law, for example, states that heat loss is linearly proportional to the thermal conductivity of the material, the temperature gradient, and the surface area available for heat transfer. These conditions are physically analogous to Darcy's Law, which states that volumetric fluid flow rate is linearly proportional to the pressure gradient and the area available for flow. An important but frequently overlooked extension of this analogy concerns flow through materials in series. When seepage passes through several soils of differing permeability, the resistance to flow is governed not only by permeability but also by the distance of flow in each. Thus, a higher-permeability stratum may be more effective than a lower-permeability layer in retarding seepage if its thickness is much greater. This concept is discussed further in Chapter 11 in conjunction with design of clay liners.

Geotechnical engineers use the term *permeability* for the quantity designated *hydraulic conductivity* above. Although the terminology used is not overly significant, it is important to realize that true permeability depends not only on the properties of the tailings solids or foundation materials but also on the properties of the fluid flowing through the soil, in the following manner (Hubbert, 1940):

$$K = \rho g k/\mu$$

where ρ = fluid density
g = gravitational acceleration
μ = fluid viscosity
k = intrinsic permeability

Although density and viscosity differences between most tailings effluents and water are small and can normally be neglected, heated effluents, effluents with high concentrations of dissolved solids, or those with added surfactants may flow at significantly different rates than water. Total dissolved solids levels of several tailings effluents listed previously in Table 1.7

approach that of seawater, and some extraction processes, such as those used for oil sands and trona, produce heated effluent. Density differences are particularly important since they are reflected in both the permeability and the fluid pressure terms of Darcy's Law. The large movements of salt-water–freshwater interfaces in coastal areas resulting from small declines in water table elevation highlight the importance to be attributed to small differences in density.

Other factors affecting permeability that are significant for evaluation of seepage from tailings impoundments are changes resulting from consolidation and chemical reactions. Compressible soils, such as normally consolidated clays and tailings slimes, require measurements of hydraulic conductivity at several levels of stress to estimate reductions in hydraulic conductivity as a function of void ratio, for example, as shown in Figure 2.6. Chemical reactions may take place that can either clog pore space or dissolve the solid particles in the soil matrix. Recent investigations of interactions between acid uranium tailings effluents and clay liners show that chemical precipitation products reduce permeability by approximately one order of magnitude initially. Gee et al. (1980) found no significant change in permeability after the initial decrease, whereas Crim et al. (1979) found permeability to decrease and then gradually increase over a period of 12 mo to levels comparable to those observed with water. Conversely, although not reported in the literature, highly calcareous soils could be expected to increase in permeability after prolonged contact with acid effluents. Therefore, laboratory measurements of permeability should use tailings effluent rather than water when chemical reactions are expected.

In addition to estimates of seepage quantity, seepage velocity is a parameter of interest and is easily obtained by the following relationship:

$$v = Ki/n$$

where n is porosity, defined in Chapter 1. The difference between volumetric flow rate and velocity is particularly important in fractured media, which are normally characterized by very low porosity resulting in relatively high velocity and large transport distances for small volumetric flow.

The one-dimensional version of Darcy's Law is the simplest analytical formula available and is commonly used to estimate seepage from impoundments on highly pervious foundations where the tailings alone control seepage and to estimate seepage through clay liners. As an example, assume that a clay liner 3 ft thick with permeability of 1×10^{-7} cm/sec is to be installed over a highly pervious impoundment base. Assume also that the final height of the impoundment will be 50 ft. Provided the tailings above the liner and the underlying foundation materials have negligible flow resistance in comparison to the liner, seepage from the hypothetical impoundment at full height is

$$Q = 1 \times 10^{-7} \text{ cm/sec} \times 50/3 \times 120 \text{ acres} = 130 \text{ gpm}$$

When applied to unlined tailings impoundments on pervious foundations, Darcy's Law is often found to overestimate seepage and should be checked against upper-bound seepage values determined by water balance methods.

One-Dimensional Partially Saturated Flow

During partially saturated flow, gravity gradients are no longer the sole factor governing seepage. In addition are gradients derived from capillary suction of water held in tension in the partially saturated voids of the soil. The lower the degree of saturation of the soil, the higher the suction pressure. In addition, hydraulic conductivity is no longer constant for a given material type but varies with moisture content or degree of saturation, becoming much lower at reduced saturation. Figure 10.16 shows that suction pressure can increase by two to three orders of magnitude for a 50% reduction in volumetric moisture content. Similarly, the same reduction in volumetric moisture content will reduce hydraulic conductivity compared to fully saturated values by three orders of magnitude. The quantity of seepage during unsaturated flow is therefore governed by interactive, moisture-content determined variations in suction pressure and hydraulic conductivity.

Refinement of Darcy's Law, as applied to multilayered soils and accounting for the additional driving force provided by capillarity, is presented by McWhorter and Nelson (1979) for estimating seepage losses from tailings impoundments overlying partially saturated foundation soils. The analysis presented herein is a brief summary of the methods presented in more detail in their publication.

Based on 50 sets of data, McWhorter and Nelson present an empirical equation for estimating the displacement pressure head in partially saturated soils:

$$h_d = -9.66\left(\frac{K}{n - \mathrm{SR}}\right)^{-0.401}$$

where h_d = displacement pressure head in centimeters of water, a negative pressure
K = saturated hydraulic conductivity in centimeters per second

and n and SR are porosity and specific retention, as defined previously. Alternatively, displacement pressure may be measured in the laboratory using methods described in Bouwer (1978).

A typical profile view of a clay-lined impoundment is shown in Figure 10.17. If the permeability and thickness of the liner and/or the tailings slimes control the quantity of seepage, then flow behind the wetting front in the foundation material will be in a partially saturated state. Alternatively, if the resistance of the foundation materials controls the rate of seepage, then the flow behind the wetting front will be saturated. The criterion for deter-

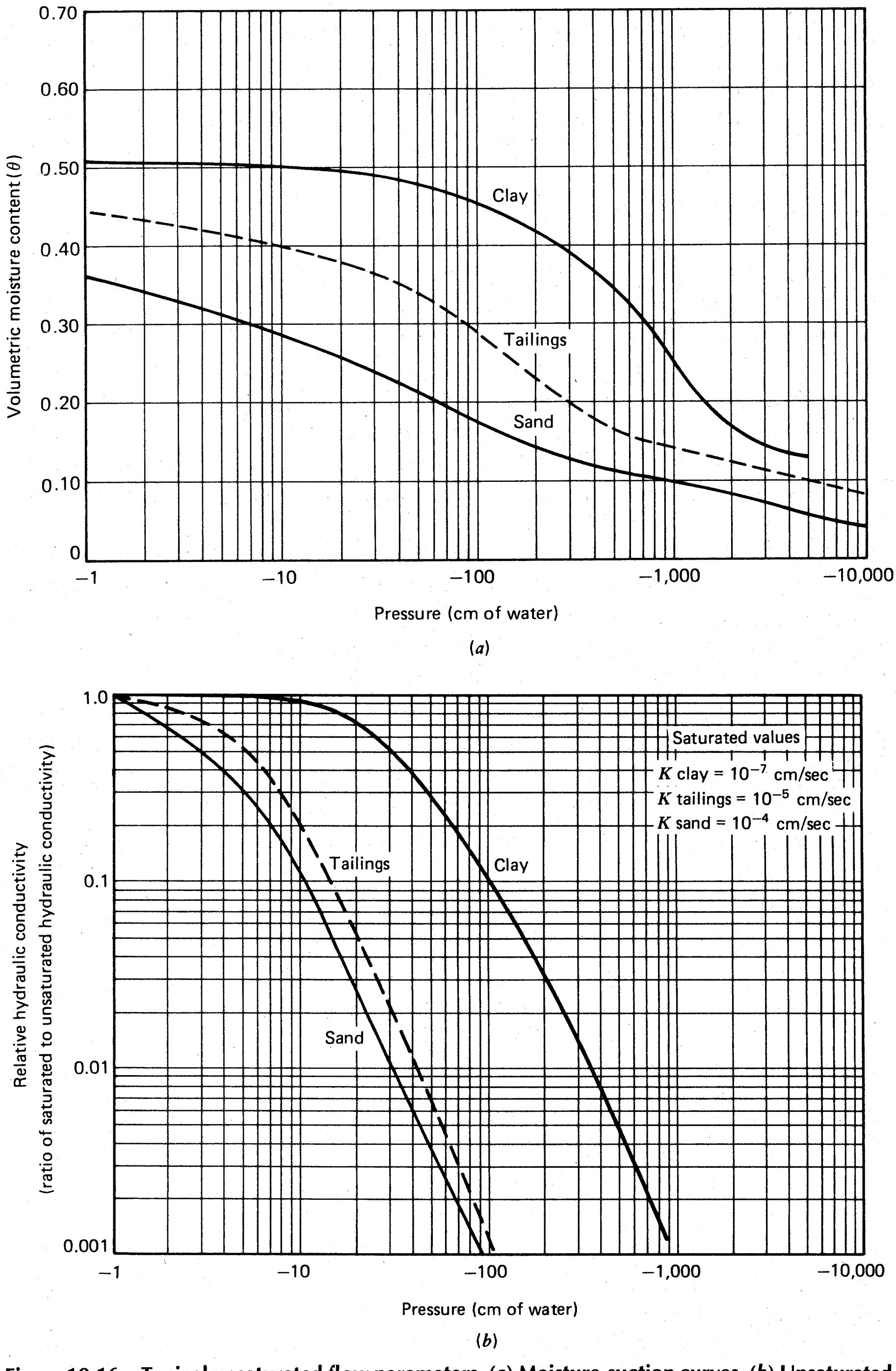


Figure 10.16 Typical unsaturated flow parameters. (*a*) Moisture-suction curves. (*b*) Unsaturated hydraulic conductivity.

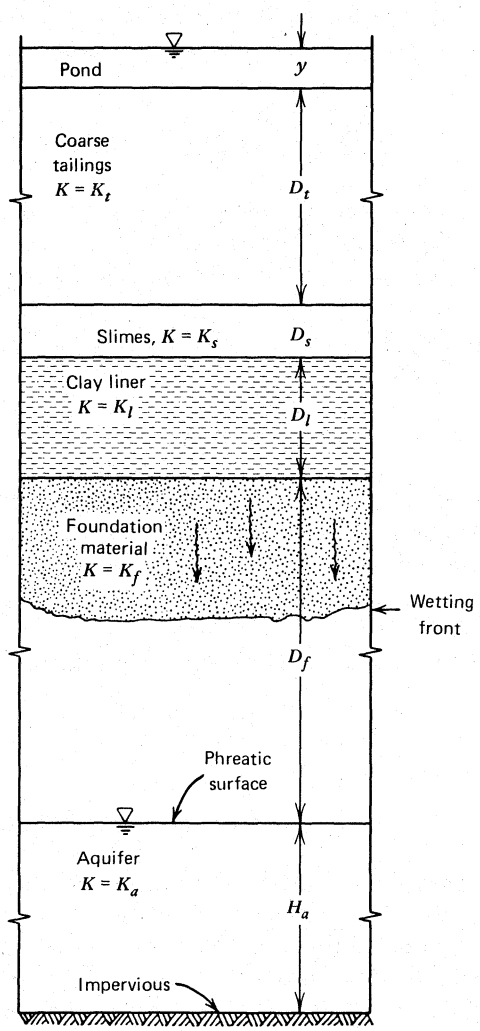


Figure 10.17 Flow regime in McWhorter–Nelson model. (From McWhorter and Nelson, 1979.)

mining whether seepage rates are controlled by the foundation materials or the impoundment and liner is

$$y + D_t + D_s + D_l - K_f\left(\frac{D_t}{K_t} + \frac{D_s}{K_s} + \frac{D_l}{K_l}\right)\begin{cases} < h_d \text{ unsaturated} \\ \geq h_d \text{ saturated} \end{cases}$$

with parameters as defined in Figure 10.17. When seepage rates are controlled by the foundation materials, there is normally a low potential for groundwater contamination. A discussion of this case is presented in McWhorter and Nelson (1979) and is not considered further in this treatment.

When seepage rates are controlled by the resistance of the tailings and/or liner, the seepage rate per unit area may be approximated as follows:

$$q = \frac{y + D_t + D_s + D_l - h_d}{(D_t/K_t) + (D_s/K_s) + (D_l/K_l)}$$

where q is the volumetric seepage rate per unit area and the other parameters are as defined in Figure 10.17. It can be shown that this equation reduces to Darcy's Law when capillarity is ignored and flow through a single layer of material is considered.

The volumetric moisture content behind the wetting front, which is below saturation, is calculated as

$$\theta_I = (n - \mathrm{SR})(q_m/K_f)^{\lambda/(2+3\lambda)} + \mathrm{SR}$$

where q_m = the average seepage rate during Stage 1 (time from start of disposal until seepage reaches the water table)
λ = an empirical pore size index that is normally within the range 1–3 and can be assumed equal to 2

The time required for seepage to reach the water table is simply

$$t = D_f \frac{(\theta_I - \theta_i)}{q_m}$$

where θ_i is the initial volumetric moisture content in the foundation.

Example calculations comparing a clay liner overlain by coarse tailings (Case 1) and slimes used to control seepage without a clay liner (Case 2) are shown in Table 10.8. For simplicity, the depth of tailings within the impoundment has been assumed constant. Example calculations provided in McWhorter and Nelson (1979) detail the calculation procedures for tailings, slimes, and fluid depths that vary with time. The assumptions used in this example are similar to those used in the previous examples and therefore provide a means of comparing the results of several different types of analyses. This example demonstrates the effectiveness of using slimes to control seepage rates. It can be concluded that the main benefit of placing a clay liner in situations similar to those assumed in this example is the uptake of contaminants through adsorption and precipitation reactions, rather than the reduction in seepage rate.

Mounding Equations

Seepage that reaches the water table causes fluid levels in the aquifer to rise. The rate of mound rise is a function of the horizontal transmissive capacity

Table 10.8 Comparison of Seepage from Lined and Unlined Impoundment during Stage 1

	Assumed Parameters				
	Fluid (y)	Tailings (t)	Slimes (s)	Liner (l)	Foundation (f)
Case 1					
Thickness (ft)	5	20	25	0	100
Saturated hydraulic conductivity (cm/sec)	—	1×10^{-4}	1×10^{-6}	—	1×10^{-4}
Porosity (%)	—	—	—	—	40
Specific retention (%)					20
Case 2					
Thickness (ft)	5	45	0	3	100
Saturated hydraulic conductivity (cm/sec)	—	1×10^{-4}	—	1×10^{-7}	1×10^{-4}
Porosity (%)	—	—	—	—	40
Specific retention (%)					20

Summary of Calculations:	Case 1	Case 2
h_d (cm)	−203	−203
q (ft^3/yr per ft^2)	2.25	1.96
θ_l (%)	27.7	27.5
Stage 1 duration (yr)	9.5	10.8

of the aquifer and the storage remaining in the partially saturated zone at the end of Stage 1 seepage. The transmissive capacity of the aquifer depends in turn on the horizontal hydraulic conductivity (assumed constant) and the average saturated thickness of the aquifer (which increases with time). Storage available in the partially saturated zone depends on the difference between foundation porosity and the moisture content of the foundation at the end of Stage 1, as well as on the distance between the base of the impoundment and the top of the mound, which decreases with time. As a result of these interactions, aquifers of high hydraulic conductivity display little or no increase in water level, as illustrated previously in Figure 10.2a.

Analytical equations that estimate the amount of mounding below an impoundment assume a constant seepage rate, a valid approximation until the mound approaches the base of the impoundment. An estimate of the average seepage rate is necessary for impoundments that change in height with time.

An analytical solution for mounding below square or rectangular impoundments was originally developed by Hantush (1967) and is discussed in the context of artificial recharge to aquifers in Walton (1970) and Bouwer

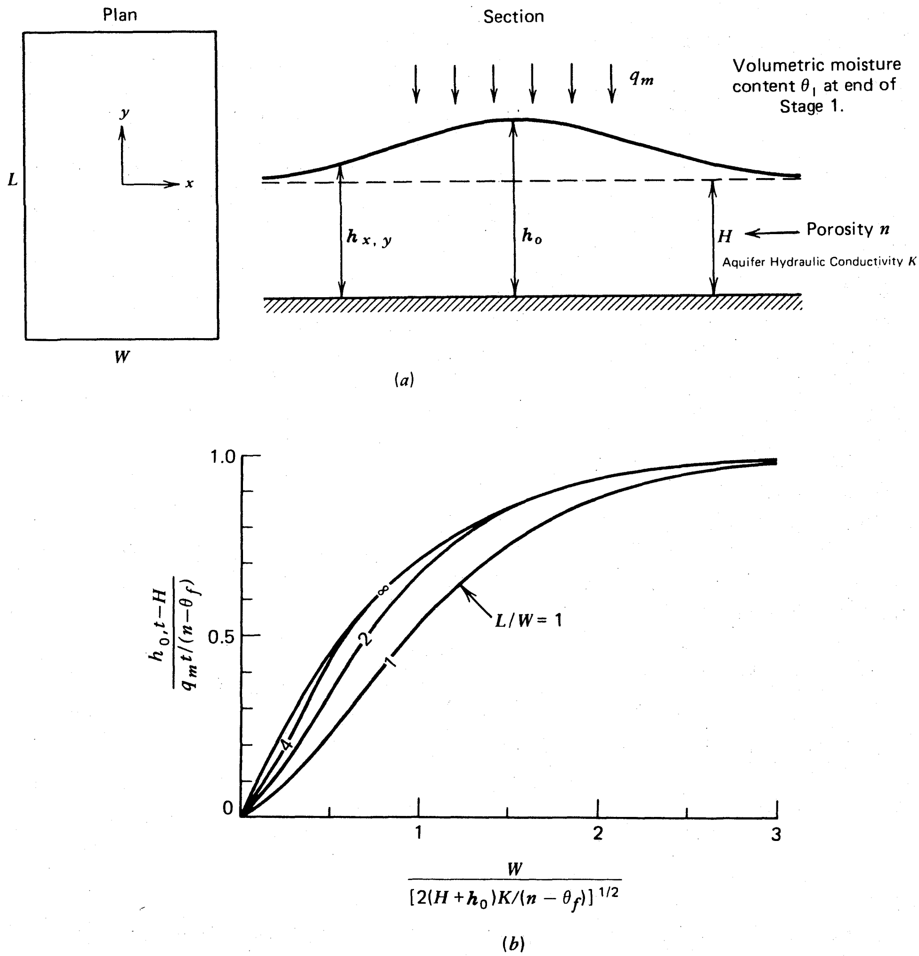


Figure 10.18 Relationships for determining rise of groundwater mound. (*a*) Geometry. (From *Groundwater Hydrology* by H. Bouwer. Copyright © McGraw-Hill. Used with permission of McGraw-Hill Book Company.) (*b*) Graphical solution. (Reprinted from Bianchi and Muckel, 1970.)

(1978). A similar solution for circular impoundments is presented in McWhorter and Nelson (1979), although Bouwer notes that circular impoundments may be approximated as squares of equivalent area. A graphical solution for mounding under the center of an impoundment as a function of time is presented in Figure 10.18. Calculations are performed iteratively by making an initial estimate for the mound height, calculating the dimensionless parameters shown in Figure 10.18, comparing with the graphical solution, and refining the estimate for mound height until the calculated values for the dimensionless parameters match the graphical solution.

Continuing with the example used to estimate Stage 1 seepage quantity and duration in Table 10.8, the duration of Stage 2 can be estimated. It can be recalled that the mean seepage rate was estimated as approximately 2 ft^3/yr per ft^2 for foundation porosity of 40%, volumetric moisture content of 27% at the end of Stage 1, and foundation hydraulic conductivity of 10^{-4} cm/sec. It is also assumed that the initial saturated thickness of the aquifer is 50 ft. Under these assumptions, the duration of Stage 2 (that is, the time required for the mound to reach the base of the impoundment) is approximately 10 yr.

The total duration of Stage 1 and Stage 2 for the example calculations is approximately 20 yr, which in many cases would approximate the duration of tailings impoundment operation. It can be observed that if operations ceased at the end of Stage 2, dissipation of the mound would require approximately 10 yr if all seepage stopped abruptly. Since drainage of the tailings will continue after termination of disposal operations, seepage will continue to enter the aquifer for a significant period of time after tailings discharge ends.

Contaminant Transport

Analytical solutions describing contaminant transport are severely limited in their application to tailings impoundment seepage as a result of the following assumptions necessary to obtain exact mathematical formulations:

1. Saturated flow.
2. Uniform groundwater velocity.

Therefore, analytical contaminant transport equations are rigorously applicable only after seepage has entered an aquifer and provided that the seepage causes little or no mounding. It is apparent from the examples described previously that mounding is insignificant only for the limiting cases of low seepage rate and/or high aquifer permeability. Nonetheless, the analytical solutions are applicable to these limiting situations and may be used for approximate estimates or as checks on more sophisticated numerical methods of analysis.

Two parameters used in contaminant transport that have not been previously described are the *dispersion coefficient* and the *retardation coefficient*. A brief description of the parameters is presented; an in-depth discussion including an extensive bibliography can be found in Freeze and Cherry (1979).

Dispersion is a mechanical mixing process resulting from the irregular geometry and distribution of pores in an aquifer. The average groundwater velocity is the mean of the variable velocities of individual fluid elements, some of which travel faster and some slower than the average. With the exception of very slow moving fluids, in which diffusion may be significant, the dispersion coefficient parallel to the direction of fluid motion is

$$D_l = \alpha_l \times v$$

where D_l = longitudinal dispersion coefficient
α_l = longitudinal dispersivity
v = average groundwater velocity

Dispersion transverse to the direction of flow is also assumed proportional to the average flow velocity but is less than the longitudinal dispersion. The ratio of longitudinal to transverse dispersivity is generally in the range 3–10. Several investigators have assumed on the basis of empirical observation:

$$\alpha_l \cong 3.3\ \alpha_T$$

where α_T is transverse dispersivity (Bredehoeft and Pinder, 1973; Konikow and Bredehoeft, 1974). Freeze and Cherry (1979) note that numerous laboratory studies have shown longitudinal dispersivity to range from 10^{-4} to 2×10^{-2}m, yet field values from tracer tests or back calculation of actual contaminant plumes consistently show α_l to be on the order of meters to tens of meters. As a result, realistic values of dispersivity must be obtained by field measurements or from the few reported investigations in similar geologic environments (Freeze and Cherry, 1979; Pinder, 1973).

Complete dispersion results in complete mixing of the contaminants with groundwater, which was described previously in lumped parameter terms. For the same amount of contaminant introduced to an aquifer, high-velocity aquifers, such as karstic limestone and alluvial gravels, will produce larger contaminant plumes at lower concentration than aquifers of low velocity, which will tend to retain the plume in a smaller and more concentrated form.

For constant groundwater velocity, the retardation coefficient is related to the distribution coefficient obtained from geochemical laboratory investigations as follows:

$$R_d = \frac{v}{v_c} = 1 + \frac{\rho}{n} K_d$$

where v = average groundwater velocity
v_c = the velocity of fluid that has a concentration 50% of that introduced into the aquifer
ρ = bulk density of the aquifer solids
n = porosity
K_d = distribution coefficient

Freeze and Cherry note that when the distribution coefficient is 1 mg/l, R_d will range from 5 to 11 for most materials. Therefore, for distribution coefficients that are 10 mg/l or higher, the contaminant is essentially immobile in the aquifer. As noted previously, the distribution coefficient must be used with extreme caution for pH-dependent contaminants.

Wilson and Miller (1978) provide a three-dimensional solution for contaminant transport of an instantaneously injected waste that accounts for dispersion and retardation subject to the limitations described above:

$$C_{(x,y,z,t)} = \frac{M}{8(\pi t)^{3/2}(D_x\, D_y\, D_z)^{1/2}} \exp\left[\frac{R_d}{4t}\left(\frac{-\,(x - vt)^2}{D_x} \frac{-y^2}{D_y} \frac{-z^2}{D_z}\right)\right]$$

where $C_{(x,y,z,t)}$ = contaminant concentration at location x, y, and z and at time t relative to point of injection

M = mass of contaminant injected at point source

D_x, D_y, D_z = dispersion coefficients in x,y, and z directions

R_d = retardation coefficient

v = average groundwater velocity

Based on the restrictive assumptions that apply to analytical contaminant transport equations, the need for numerical analyses is evident.

Numerical Methods

Numerical methods are required for calculations of several simultaneous or interactive processes, such as changes in effluent contaminant concentration resulting from dilution with aquifer pore water, changes in contaminant solubility as a function of pH, and contaminant adsorption. Geometric complexities also call for the use of numerical models, for example: complex distribution of geologic units, impoundments that change significantly in size and height over the disposal period, interactions with water supply wells, or irregularly shaped flow boundaries, such as sinuous stream channels. Numerical techniques are also required to model partially saturated flow in two dimensions. These models are used in cases where mill effluent seepage contains highly toxic compounds that may persist in the subsurface and contaminate existing or potential groundwater supply wells, or that may emerge at surface discharge localities in springs or streams.

The literature contains several examples of numerical analyses of contaminant transport (see, for example, Pickens and Lennox, 1976; Konikow, 1977). However, there are few published examples where tailings impoundment design has been the purpose of conducting a numerical contaminant transport analysis. Those that do exist have thus far been limited to uranium mill waste impoundments. These examples are discussed after a brief description of numerical analysis techniques.

Available Numerical Methods and Models

The theoretical basis for numerical modeling of groundwater flow has gradually developed over the past 20 yr. The parallel development and availability of computers has resulted in an exponential increase in the use of numerical models for groundwater analysis. This trend is expected to continue as the cost of microcomputers decreases and software becomes readily available.

Discussions of the basic groundwater flow and contaminant transport equations and their corresponding finite-difference and finite-element approximations may be found in most textbooks on groundwater hydrology. A readable, introductory treatment of the subject is presented in Freeze and Cherry (1979), while more detailed mathematical treatments may be found in Bear (1972), Bear (1979), Rushton and Redshaw (1979), and Zienkiewicz (1971).

Numerical models may be classified in four groups:

1. Flow models.
2. Coupled flow and contaminant transport models.
3. Partially saturated flow models.
4. Coupled, partially saturated, flow/contaminant-transport models.

A partial list of generally available models is presented in Table 10.9.

Application

Although application of numerical models to specific problems is not easily generalized, typical stages in model development are as follow:

1. *Construction of a Physical Model.* This requires detailed evaluation of the hydrogeologic regime, including definition of permeability, porosity, moisture content, neutralization capacity, and distribution coefficients. These parameters must be developed for the full range of materials present, including natural soil and rock; materials to be used in construction of the impoundment, such as clay liners; and the tailings. Identification of aquifer characteristics together with regional recharge and discharge areas is also an important component of physical model construction.
2. *Selection of Model Boundaries.* The boundaries for a numerical model can correspond to hydrogeologic boundaries, such as rivers, obstructions to flow, or planes of symmetry. Alternatively, boundaries can be chosen sufficiently far from the impoundment so that the effects of tailings seepage at the boundary are negligible. The selection of boundaries heavily influences the results of a numerical analysis, since the concentrations and pressures at the boundaries must be specified as known and constant quantities.
3. *Superposition of Discretized Mesh on Physical Model.* The numerical approximations of the differential equations assume either linear changes between the points at which calculations are per-

Table 10.9 Representative Numerical Models

Type of Model	Numerical Method	Comment	Reference
2-D Flow	Finite element	Developed specifically for analysis of flow through tailings dams	Kealy and Busch (1971)
2-D Flow	Finite difference	Well documented, easy to use	Trescott et al. (1976)
2-D Coupled flow and contaminant transport	Hybrid finite difference–method of characteristics	Well documented	Konikow and Bredehoeft (1977)
Partially saturated flow	Finite element	TRUST, several adaptations by various investigators	Narasimhan and Witherspoon (1977, 1978)
Partially saturated flow with chemical transport	Finite element	Recently developed	Gureghian (1981)

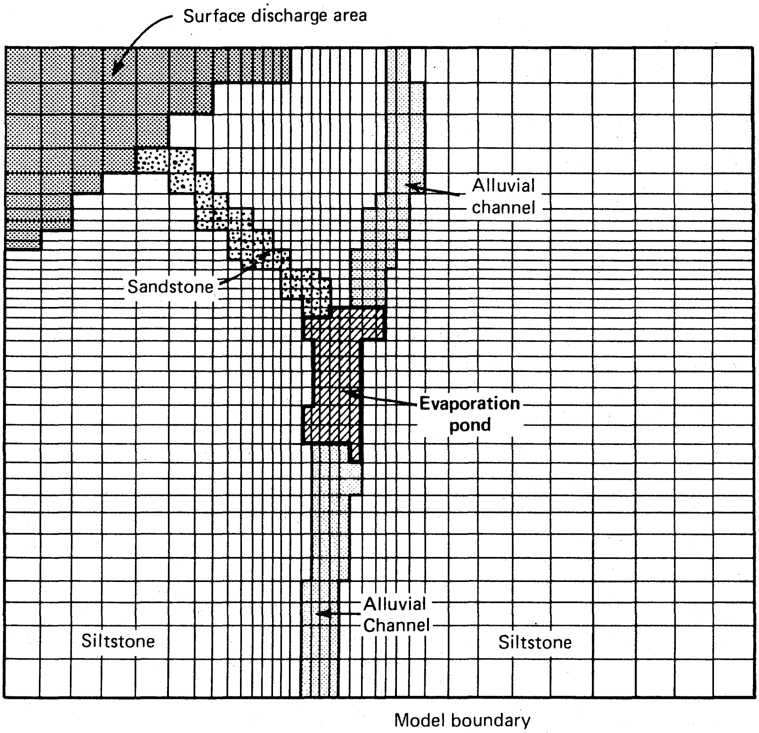


Figure 10.19 Example of finite-difference mesh, plan view.

formed or, in the case of finite-element models, simple polynomial variations. In either case, mesh size should be small in regions where large changes in water levels and/or concentrations are expected over relatively short distances. Obviously, fine mesh size is required at the interface between the tailings impoundment and the surrounding liner, foundation, or aquifer materials. An example of a physical model and a superimposed finite-difference mesh is shown in Figure 10.19.

4. *Specification of Initial Conditions.* Flow models require the initial or starting values of water level at each point of the mesh. Coupled flow/contaminant-transport models also require initial concentration of any chemical species. Partially saturated flow models require specification of initial moisture content or corresponding suction pressure values.
5. *Model Calibration.* Calibration of a numerical model provides a check on the accuracy of the specified initial and boundary conditions. Many impoundments are constructed over hydrogeologic regimes that display spatially constant water levels, flow directions, and background contaminant concentrations prior to im-

poundment construction. Short-duration model calculations performed for calibration should show little change from the initial values when boundary conditions have been correctly specified.

Models that use numerical methods of analysis can be used for a variety of purposes ranging from detailed analysis of partially saturated flow and chemical reactions in a clay liner, to evaluation of tailings seepage impacts on regional groundwater discharge areas. The range of applications is best illustrated with examples and selected case histories from the literature.

Examples

The use of numerical models to evaluate the impacts of tailings impoundment seepage have to date been limited to uranium mill waste effluents because of the high levels of toxic and radioactive compounds in the mill effluent. As discussed previously, transport of many of these compounds is highly dependent on pH. Therefore, the range of impacts associated with the disposal of these effluents is represented by migration of the wetting front, which represents the maximum seepage migration distance, and by modeling of pH changes, which will control the solubility of most metals and radionuclides such as radium-226 and thorium-230.

Bureau (1981) examined the penetration of the wetting front through partially saturated clay liners and foundation materials by comparing the results of the McWhorter-Nelson method of analysis with the numerical model TRUST described by Narasimhan and Witherspoon (1977, 1978), as shown in Figure 10.20. As observed by Bureau, the results of the detailed numerical simulations agree reasonably well with the results predicted by the McWhorter-Nelson method, which provides conservative estimates of wetting front migration.

An example of a coupled, partially saturated, flow/contaminant-transport analysis of seepage for a proposed trench disposal system described by Sharma (1981) is shown in Figure 10.21. The colluvial–alluvial soils surrounding the trench have a high calcium carbonate content that neutralizes most of the effluent leaving the trench. This case also demonstrates effective use of the partially saturated zone for disposal and treatment of mill effluent.

Taylor and Antommaria (1979) describe the first application of numerical contaminant transport modeling techniques to evaluation of uranium effluent seepage impacts on groundwater at an existing tailings disposal site in Wyoming. The impoundment considered is a conventional cross-valley impoundment operated for over 20 yr. The results of this investigation provide valuable insight to the long-term effects of contaminant uptake by geochemical reactions in a real rather than a laboratory environment. Taylor and Antommaria conclude that pH is the controlling factor on contaminant transport, as shown in Figure 10.22. A similar case history of seepage from an

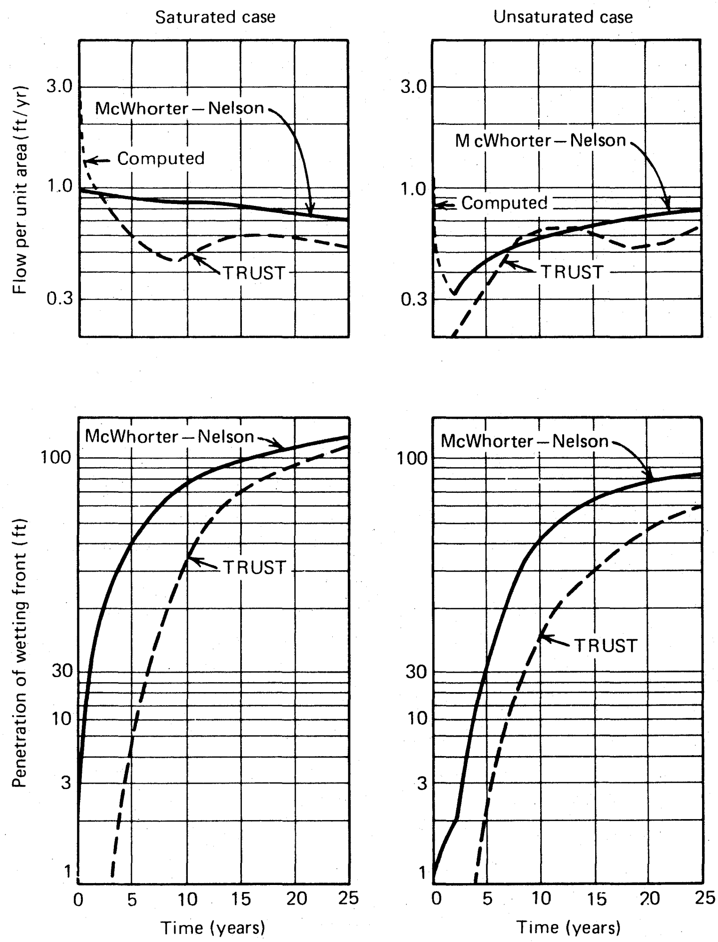
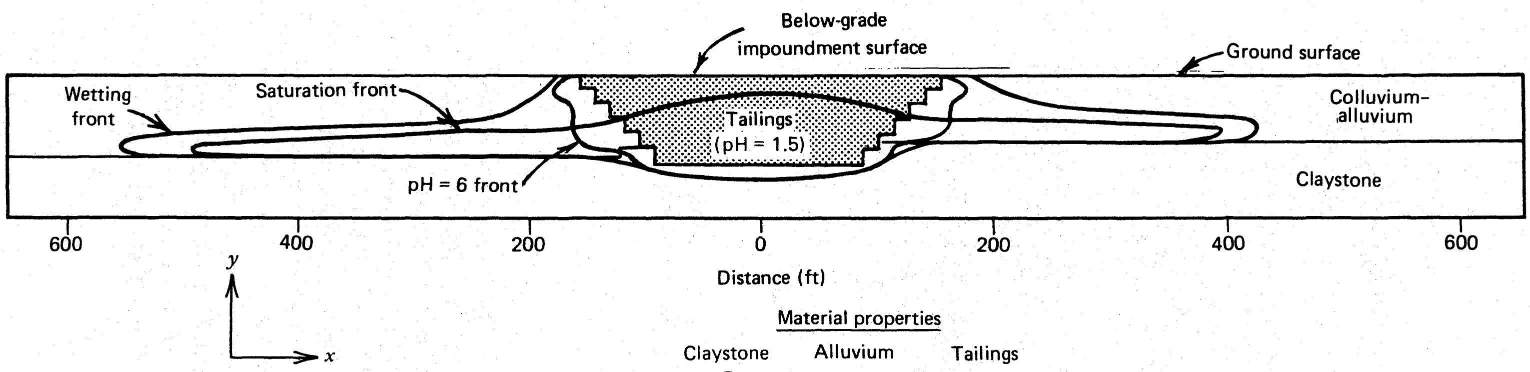


Figure 10.20 Comparison of model results. (From Bureau, 1981.)

evaporation pond in Wyoming, also operated for over 20 yr, is described by Highland et al. (1981).

Description of seepage impacts resulting from disposal of uranium mill tailings in former open-pit mines can be found in Nelson, Reisenhauer, and Gee (1980). This reference provides a good example of the most sophisticated level of seepage impact evaluations combining the measurement of partially saturated flow parameters, laboratory measurement of geochemical parameters, and subsequent use of numerical methods of analysis.

Figure 10.21 Example of coupled, partially saturated flow–contaminant transport model to evaluate seepage of acid mill effluent. Data shown are for 1 yr after end of disposal operations. (From Sharma, 1981.)



Material properties

	Claystone (1)	Alluvium (2)	Tailings (3)
Bulk density (g/cm^3)	2.34	1.68	1.36
Porosity (%)	11.4	36.6	51.0
Saturated hydraulic conductivity x direction (cm/sec)	5×10^{-5}	5×10^{-4}	1×10^{-5}
Saturated hydraulic conductivity y direction (cm/sec)	5×10^{-8}	1×10^{-5}	1×10^{-5}
Hydrogen ion distribution coefficient (ml/g)	11.3	8.2	
Initial degree of saturation (%)	54.0	28.0–46.0	100

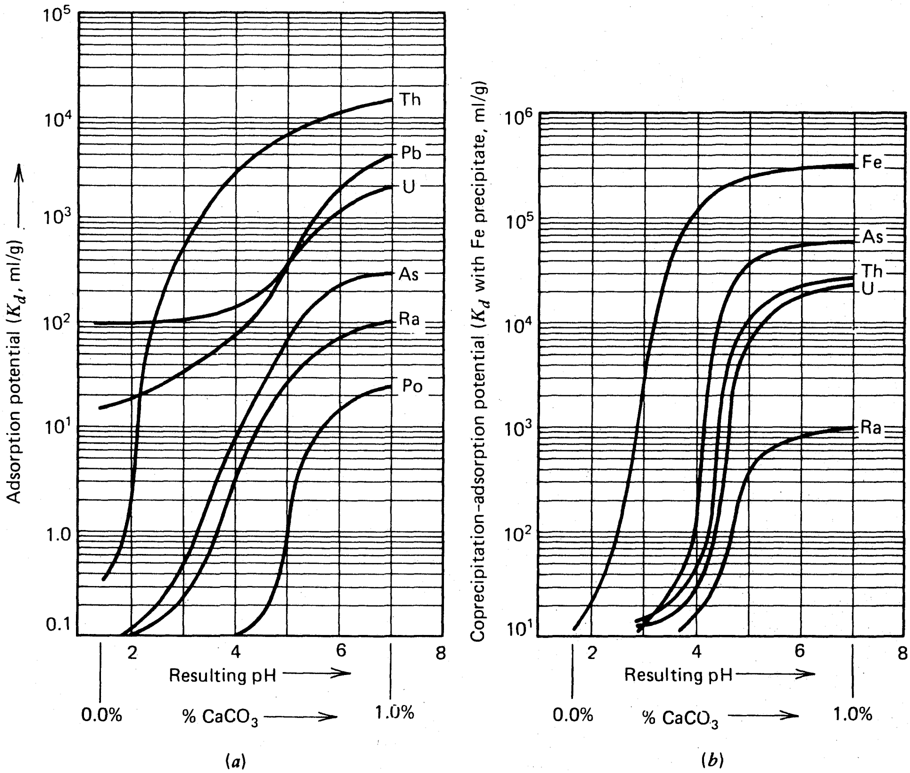


Figure 10.22 Contaminant uptake as a function of pH. (From Taylor, 1980.) (*a*) Adsorption geochemical reaction effects. (*b*) Coprecipitation and adsorption geochemical reaction effects.

SUMMARY OF SEEPAGE EVALUATION

Evaluation of tailings impoundment seepage impacts requires assessment of both the physical and chemical properties of the tailings, the mill effluent, and the surrounding hydrogeologic environment. The level of sophistication required for the analysis of impacts is dependent on the toxicity and concentrations of the mill effluent contaminants and the proximity of the impoundment to high-quality groundwater resources, points of water supply, or groundwater discharge areas. For mill effluents of low toxicity and/or impoundments located in favorable hydrogeologic environments, the simple lumped parameter methods of analysis are sufficient to evaluate the impacts of seepage. For highly toxic mill effluents or impoundments located near points of water supply, analytical methods can be applied to cases of uniform geometry, while numerical methods are necessary for more complex situations.

The preferred method of mitigating seepage impacts is to locate impoundments where geochemical attenuation of contaminants through reactions with in situ geologic materials can occur before seepage reaches groundwater resources. When consideration of other siting constraints does not allow location at a hydrogeologically favorable site, design methods of controlling seepage may be required, which is the subject of Chapter 11.

11

Seepage Control Methods

The sand is put back in the mine, where concrete is poured on it to make platforms for upward mining. Thus, the mine consumes its own tailings, sparing in large measure the beauty of its environment—but not sparing it entirely. Because crushed rock expands in volume, all cannot be put back into the mine, and one of the Black Hills is a flat-topped mountain of black sand. And, as it happens, Whitewood Creek flows black after it passes the mine.

John McPhee, *Encounters with the Archdruid*

Seepage from tailings impoundments and the effects of seepage-borne contaminants on groundwater have been among the most poorly understood aspects of tailings disposal technology. Yet the mitigation and control of tailings impoundment seepage is rapidly becoming one of the key issues in environmental and regulatory evaluation of mining projects. Chapter 10 has explained the basic mechanisms and evaluation methods for seepage movement and contaminant transport. However, in cases where mill effluent contains toxic constituents that could move freely in the groundwater regime, the problem remains to mitigate or eliminate these adverse effects. The purpose of Chapter 11 is to identify and evaluate the effectiveness of various seepage-control measures.

Sweeping changes have occurred recently in the state of the art of tailings impoundment seepage control along with increasing regulatory and environmental attention to seepage. In many cases, heavy emphasis is placed on the use of impoundment liners, but the implications on the economics of tailings disposal are apparent when it is considered that the cost of impoundment liners may approach or exceed several million dollars. Consequently, it is necessary to have an understanding of the full range of available seepage-control measures and their effectiveness in specific situations before making a decision on the appropriate option. To this end, Chapter 11 discusses some of the objectives and purposes of seepage control. Once the purpose of seepage control is established, control methods can be evaluated according to three general categories: seepage barriers, seepage return systems, and liners. Each of these three categories is discussed in detail in subsequent portions of the chapter, along with guidelines for evaluating effectiveness and suitability to specific site conditions.

OBJECTIVES OF SEEPAGE CONTROL

In Chapter 3, the historical evolution of tailings disposal methods is discussed, progressing from the early days, when tailings—without knowledge of or concern for their ultimate fate—were allowed to flow into streams, up to present-day disposal methods, which ideally match the tailings characteristics, site characteristics, and disposal methods in a somewhat rational manner. Seepage-control methods have developed according to a similar progression, beginning with a historical ignorance of seepage effects, but at present are not quite so far down the evolutionary path. By analogy to disposal methods, seepage-control methods must be matched to the chemical characteristics of the seepage and the specific site conditions. But the recent flurry of concern for controlling seepage has come about so rapidly that the rush to implement elaborate and costly control measures may have outstripped an understanding of exactly what these measures are intended to achieve.

Because a sound understanding of tailings-related geochemical processes as explained in Chapter 10 has only very recently been developed, it has not always been possible to fully determine the specific effects of seepage and to select a control method best suited to minimizing these effects. Now, however, with the principles of Chapter 10 in mind, it may be of value to refocus attention on some of the more fundamental issues related to the purpose and intent of seepage control.

Chapter 1 explained the sources of chemical constituents in tailings effluents based on ore type and mill processes, and also provided some guidance on assessing the relative toxicities of these constituents. Chapter 10 outlined some of the geochemical and hydrogeologic processes that influence the movement of contaminants through soil and groundwater. Combining these topics, some general principles related to seepage control emerge:

1. Not all mill effluents contain toxic constituents. Depending on ore type, mill process, and pH, contaminants may range from toxic heavy metals (that is, cadmium, selenium, arsenic) to such relatively innocuous materials as sulfates or suspended solids. Moreover, concentrations of these constituents, which determine their hazard, vary widely in various effluents.
2. For mill effluent that does contain toxic constituents, it is not necessarily the case that seepage of this effluent will result in pervasive groundwater contamination. Geochemical processes may retard or inhibit movement of some constituents, and these processes are often most effective in reducing mobility of the most troublesome metallic ions associated with low-pH effluents.
3. If toxic constituents do enter the groundwater regime, the ultimate effects on the groundwater environment must be determined be-

fore deciding on a seepage-control strategy intended to minimize these effects. Essential to this determination are hydrogeologic factors, baseline water quality, and intended use of the groundwater resource both present and future.

From these principles, it seems reasonable that the type of seepage-control strategy should match the chemical conditions of the effluent and the geochemical and hydrogeologic conditions of the site. Taylor (1980) defines three types of systems used with regard to uranium tailings seepage control, which can be extended to tailings in general:

Type I System. Seepage from the impoundment is essentially uncontrolled. Due either to a lack of troublesome contaminants in the mill effluent or to uptake of these contaminants by geochemical processes, groundwater contamination potential is not serious, irrespective of seepage quantity.

Type II System. In this case, impoundment effluent is partially retained, but some seepage loss is anticipated. Contamination potential is greater than for Type I effluents, and a higher level of seepage–contamination analysis is required, along with provisions to monitor groundwater quality.

Type III System. Here, seepage is totally restricted by structural measures. Ordinarily very costly, these measures, such as impoundment liners, attempt to achieve "zero discharge" of seepage from the impoundment.

Along with matching the seepage-control strategy to effluent and site conditions, it is necessary to establish the objectives of groundwater protection. In Chapter 6 it is emphasized that, when selection from a range of alternatives is made, it is essential that the objectives of the selection process be defined, and there is no better example than for determining seepage-control measures. The objectives of groundwater protection may be to prevent adverse health effects on groundwater users, or to keep contaminants at a given concentration from migrating beyond the project site boundary, or to prevent any escape of contaminants whatsoever from the impoundment. The influence of these different objectives on required seepage-control measures can be substantial, as illustrated in Table 11.1. The objectives in Table 11.1 are generally arranged in an increasing hierarchy of restrictiveness on the impoundment types required to meet them. For all objectives but the last, however, the geochemical and hydrogeologic nature of the site together with the mill effluent are important factors in determining the type of seepage-control strategy, and either a Type I, Type II, or Type III impoundment may be suitable depending on the specific circumstances.

It is apparent, then, that considerable thought must be given to the objectives of seepage control as well as to site and effluent-related factors before

Table 11.1 Groundwater Protection Objectives

Objective	Impoundment Type Required	Pertinent Site and Effluent-Related Conditions
(1) Prevent contaminant migration beyond site boundary	I, II, or III	Geochemical factors; groundwater gradient; buffer zones
(2) Restrict concentrations of specific contaminants in groundwater to defined levels	I, II, or III	Baseline water quality; geochemical and hydrogeologic factors; effluent composition
(3) Prevent adverse health effects on groundwater users	I, II, or III	Locations of wells; geochemical and hydrogeological factors; effluent composition
(4) Prevent introduction of toxic contaminants into groundwater	II or III	Geochemical and hydrogeologic factors; effluent composition
(5) Prevent any seepage release from impoundment ("zero discharge")	III only	Independent of all effluent, geochemical, and hydrogeologic factors

determining what level of seepage control is required. After making this decision, it remains to evaluate the various options available to achieve the intended result, be they in the category of seepage barriers, return systems, or liners. In the remaining portions of Chapter 11, these options are discussed individually in detail. For the most part, they apply to cases where it has been determined that a Type II or Type III system is required.

SEEPAGE BARRIERS

As illustrated in Figure 11.1, seepage barriers include cutoff trenches, slurry walls, and grout curtains. The use of barriers requires that the embankment design incorporate an internal zone of impervious fill to which the barrier can connect, providing an obvious limitation to the use of barriers for pervious cycloned sand or mine waste structures without a core. For raised embankments, seepage barriers must be installed in conjunction with starter dike construction. Consequently, they will underlie the upstream portion of downstream-type embankments and be located beneath the central portion of centerline embankments. Seepage barriers are not generally compatible with upstream-type embankments because this embankment type does not and cannot contain an impervious fill zone in the region of the embankment face. In fact, as explained in Chapter 8, a pervious foundation has a benefi-

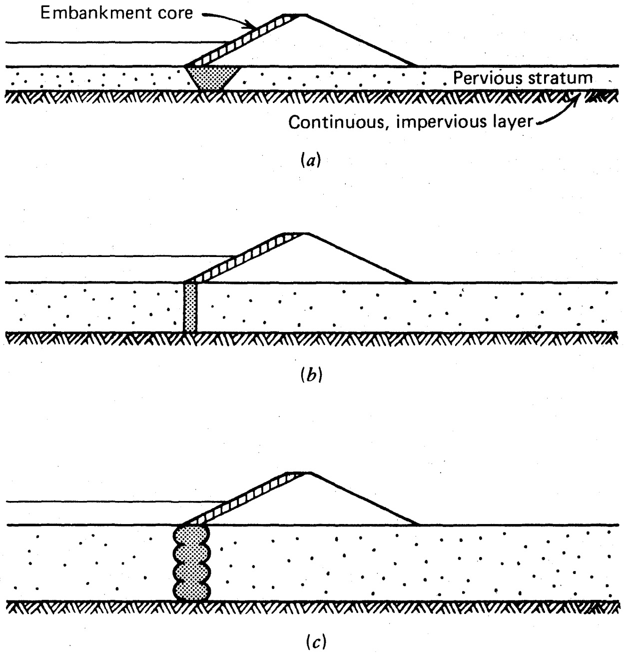


Figure 11.1 Seepage barriers. (*a*) Cutoff trench. (*b*) Slurry wall. (*c*) Grout curtain.

cial effect on stability of upstream embankments, and to block foundation seepage could cause a dangerous rise in the embankment phreatic surface.

Seepage barriers function by restricting lateral migration of seepage. As a consequence, they are fully effective only when pervious foundation materials are underlain by a continuous impervious stratum of natural material that prevents vertical flow. As shown by Cedergren (1967), a seepage barrier must completely penetrate the pervious foundation layer with a tight seal to the impervious stratum in order to reduce seepage significantly. For example, a barrier that penetrates 90% of the pervious stratum reduces seepage by less than two-thirds.

Cutoff Trenches

Usually relatively economical to construct when natural clays are present for use as compacted trench backfill, cutoff trenches are perhaps the most widely used seepage-control method for tailings embankments. Cutoff trenches are commonly installed to depths ranging from 5 to 20 ft, depending on the depth to the impervious layer, but they have been extended to as much as 60 ft beneath some tailings embankments. A major limitation of cutoff trenches is that their excavation more than about 10–15 ft below the water table quickly becomes impractical unless expensive and time-consuming construction dewatering methods are used. However, cutoff trenches, unlike other types of barriers, are not limited by the type of material present in the foundation; bouldery soils such as glacial tills and weathered or fractured bedrock can be excavated relatively easily. Also, the bottom of the trench can easily be inspected to ensure that all pervious materials have been penetrated and that the backfill achieves a good bond with the underlying impervious stratum.

Slurry Walls

Slurry walls can also be used to penetrate a pervious foundation stratum. The technique involves excavating a narrow trench with a backhoe whose sides are supported by a bentonite slurry. The trench is backfilled either with a slurry of soil and bentonite or with bentonite containing cement additives. Permeabilities of 10^{-7} cm/sec can be achieved with soil–bentonite walls (D'Appolonia, 1980), with permeabilities in the range of about 10^{-5} cm/sec for cement–bentonite cutoffs. Slurry wall cutoffs have been used for both tailings embankments and conventional water-retention dams (*Engineering News Record,* 1976, 1978). While slurry wall cutoffs have been constructed to depths as great as 100 ft, time and economic considerations usually limit practical depths to about 40 ft for tailings embankments.

A major advantage of slurry walls is that they can be constructed in saturated foundations where cutoff trenches would be impractical to excavate. Low permeabilities can be achieved with soil–bentonite, but the effects of the effluent on permeability must be accounted for. In addition, the embankment must be able to accommodate any deformation that might result from the presence of a soft soil–bentonite slurry trench beneath it.

On the other hand, slurry walls are relatively costly and cannot easily penetrate fractured bedrock or more than occasional boulders, limiting their usefulness in many types of coarse or highly fractured materials where barriers are most needed. The slurry trench must be excavated on a nearly flat surface. Consequently, for valley-bottom or sidehill sites with steep abutments, major excavations may be required prior to slurry wall installation. Considering these factors, slurry wall barriers are best suited to reasonably flat sites where the pervious material to be cut off is saturated, relatively shallow, and relatively fine grained.

Grout Curtains

Grout curtains have long been routinely constructed to reduce foundation permeability for major water-retention dams and can be installed to depths well in excess of 100 ft. Grouting techniques, while largely an art, are well developed (Albritton, 1982). The purpose of conventional grouting of dam foundations is structural rather than environmental. That is, grouting is intended to reduce foundation permeability to the extent that foundation pore pressures are not excessive; the quantity of foundation underseepage is of secondary importance in conventional water dam applications.

Several types of grout are available, ranging from portland cement to such chemical grouts as sodium silicate and acrylic resins. The type of grout required depends on the size of the voids that must be penetrated. However, economic considerations seldom allow the use of other than cement grout for dam foundation cutoffs. In order for grout particles to penetrate voids in soils, the ratio d_{15} of the soil to d_{85} of the grout particles must be greater than 25, in practice limiting cement grout applications to soils in the medium sand to gravel range with initial permeabilities greater than about 5×10^{-3} cm/sec (Einstein and Barvenik, 1975). Similarly, groutable fissures in rock must be wider than about 0.75 mm for cement grouting to have appreciable effect. Thus, cement grouting is usually effective for only relatively coarse soils and fractured rock with continuous, open joints.

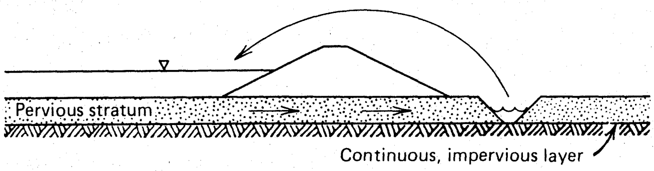
Grouting seldom reduces the permeability of the grouted material to less than about 10^{-5} cm/sec (Einstein and Barvenik, 1975), a value that, although adequate for water dam purposes, is not often sufficient to restrict seepage from tailings impoundments to an acceptable degree. For this reason and because of the high cost of grouting, its limitation to only coarser materials, and potential problems related to acid and sulfate attack of the grout by many tailings effluents, grouting does not ordinarily find wide application to tailings impoundment seepage control. It has been used, however, as a remedial measure to control seepage from existing impoundments. In one such case (Dodds, 1979), a remedial grout curtain of clay, cement, bentonite, and calcium chloride was constructed beneath an existing tailings embankment. Pumping tests conducted after completion of the grouting showed that failure of the grout to penetrate a thin zone immediately above the impervious layer rendered the curtain largely ineffective in reducing seepage quantities. Yet contaminant concentrations in the seepage effluent were reduced by the longer seepage flow path and increased geochemical retardation.

SEEPAGE RETURN SYSTEMS

Unlike seepage barriers, return systems do not attempt to restrict seepage flow but rather to collect it, thereby eliminating or minimizing migration of contaminants in the groundwater. Two basic forms of return systems operate on this principle: collector ditches–sumps and collector wells, as shown in Figure 11.2.

Collector Ditches

Collector ditches are often effective and inexpensive, making them a first line of defense for seepage control, whether used alone or as a backup for other seepage-control measures. Collector ditches are usually excavated along the downstream toe of an embankment, draining to one or more



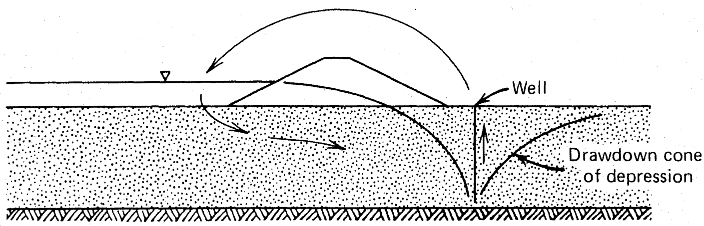


Figure 11.2 Seepage return systems. (*a*) Collector ditch/sump. (*b*) Collector wells.

sumps, where collected seepage is pumped back to the impoundment. For narrow cross-valley impoundments, sometimes only a sump or collector pond is constructed. Klohn (1979b) describes several examples of collector ditches and sumps for seepage control.

Collector ditches are similar to cutoff trenches insofar as they are most effective for a relatively shallow pervious zone that is underlain by a continuous low-permeability stratum. To be fully effective, the ditch must completely penetrate the pervious stratum, but even if this is not possible, the ditch will still collect and return that portion of the seepage passing through the embankment itself. Collector systems do not require that the embankment contain an impervious zone, and they are useful for pervious embankments of upstream, downstream, or centerline type.

The depth of collector ditches is ordinarily determined by the thickness of the pervious stratum, and they have been installed to depths approaching 50 ft in unusual cases. Because flow is concentrated within the pervious layer, exit gradients into the ditch are often high. Problems occasionally arise with piping that may start on the ditch face and progress back under the dam foundation unless the ditch is backfilled with filter sand, or gravel encapsulated in filter fabric.

Collector Wells

Collector wells incorporate the same principles as collector ditches. Contaminated seepage is intercepted by a line of wells at or near the embankment toe, pumped out, and returned to the impoundment. An impervious lower layer is desirable but not essential if the wells penetrate deeply enough to intercept the contaminant plume. Clean-water injection wells are sometimes advocated downgradient from the collector wells to reverse the flow gradient and provide further security to the collector well system (Klohn, 1979b; Soderberg and Busch, 1977).

Collector wells are expensive and not a method of choice for seepage control. Their effectiveness may be limited in foundations of low or variable permeability. However, they may be a remedial measure of last resort to prevent further damage to an already contaminated aquifer, or as an aquifer cleanup measure.

LINERS

Liners constitute the final category of seepage-control measure and are usually reserved for conditions where a Type III system is called for because of stringent groundwater protection requirements and relatively high concentrations of toxic constituents in the mill effluent. Liners of any type are inherently high in cost, but their effectiveness is at least somewhat less in doubt than high-cost barrier systems, principally because a liner is a surface installation that can be constructed under controlled conditions and inspected. This does not guarantee, however, that even a properly constructed liner will function as intended during actual operation or that any lined impoundment will be a "zero discharge" facility. Liners do have a major advantage over seepage barriers or seepage return systems insofar as they are completely independent of subsurface conditions. Whereas the effectiveness and construction feasibility of grout curtains, slurry trenches, and collector ditches or wells depend on the presence of a lower impervious layer as well as the nature of the material to be penetrated, liners suffer no such limitations and can be constructed on any surface sufficiently dry and competent to allow for normal earthwork operations, without regard for the nature of subsurface soil, rock, or groundwater conditions. Liners, however, must be resistant to both chemical attack by the retained effluent and a variety of types of physical disruption.

Because of the critical nature of seepage problems in those cases where liners are required, there is considerable debate over the relative merits and effectiveness of the various kinds of liners and liner materials. The major categories of liners include: tailings slimes, clay liners, and synthetic liners. Each of these categories contains a large number of different material types. The purpose of the ensuing discussion is to identify those types of liners that are most commonly considered for tailings impoundment applications and to consider their relative advantages and disadvantages.

Tailings Slimes

First formally proposed by Soderberg and Busch (1977), the concept of slimes liners is simple. Tailings slimes are spigotted in the impoundment, and seepage reduction is achieved by virtue of the low permeability of the tailings themselves. It is due at least in part to this apparent simplicity that slimes liners have not found wide favor with environmental regulatory authorities; on the surface, the method appears to be too similar to a normal spigotting procedure to lend credibility to claims of seepage reduction. Nevertheless, slimes liners constitute a legitimate and comparatively inexpensive seepage-control method for certain types of tailings, with effectiveness that can be comparable to or better than clay or synthetic liners. Slimes liners also have certain unique advantages not offered by other types.

It goes without saying that, for a slimes liner to be constructed, slimes must constitute a considerable fraction of the whole mill tailings. Usually, more than about 40% passing the No. 200 sieve would be sufficient. Review of data provided in Chapter 1 shows that this criterion is met for the majority of mill tailings. Slimes liners generally require that the fine fraction be separated from the whole tailings by cycloning. A second criterion is that to have an effectiveness comparable to other types of liners on pervious foundations, the slimes must have a consolidated permeability approaching 10^{-6} cm/sec. Data presented in Chapter 2 show that this criterion is not unreasonable for many types of tailings slimes.

To illustrate the comparative effectiveness of a hypothetical slimes liner, consider the example shown in Figure 11.3. Figure 11.3a shows a 50-ft deep impoundment, the lower half of which consists of cycloned slimes overlain by discharged whole tailings. For comparison, Figure 11.3b shows an impoundment of equal depth underlain by a 2-ft thick clay liner. In this example, both impoundments are underlain by pervious sand with the groundwater at depth. Simple one-dimensional saturated (Darcy) flow is assumed.

Comparison of the seepage quantities in Figures 11.3a and 11.3b shows that the slimes liner allows slightly less seepage than a 2-ft thick clay liner with a permeability of 10^{-7} cm/sec. In effect, the higher permeability of the slimes liner in comparison to the clay liner is compensated by its greater thickness. To be rigorous, the seepage from the slimes liner would also have to include consolidation-induced seepage, but the effectiveness of the two types of liners would still be essentially comparable.

Slimes liners have several disadvantages. First, normal spigotting procedures must be modified to distribute the slimes around the entire perimeter of the impoundment, and ponded water must be carefully controlled and minimized. As shown in Figure 11.4, normal spigotting of tailings from only the embankment crest produces an area of direct contact between the decant pond and natural soil at the rear of the impoundment. Kealy et al. (1974) show that direct contact of ponded water and pervious natural soils provides a major conduit for seepage unless the slimes everywhere underlie the decant pond. Second, the sands separated from the whole tailings must also be disposed of, requiring a smaller conventionally lined impoundment. However, if the cycloned sands can be used in embankment construction, this disadvantage is easily overcome. Finally, a slimes liner offers no opportunity for attenuating contaminant movement by geochemical processes within the liner itself, and protection of groundwater quality must come

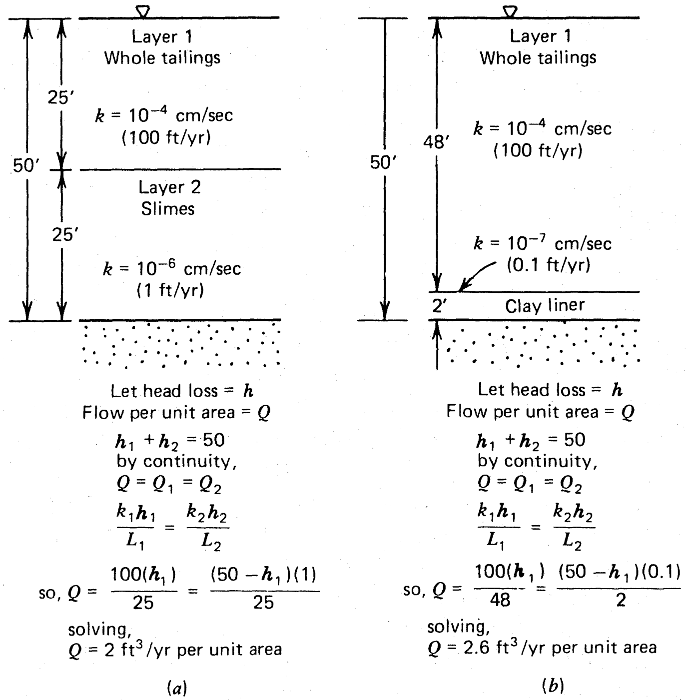


Figure 11.3 Example liner seepage comparison. (*a*) Slimes liner. (*b*) Clay liner.

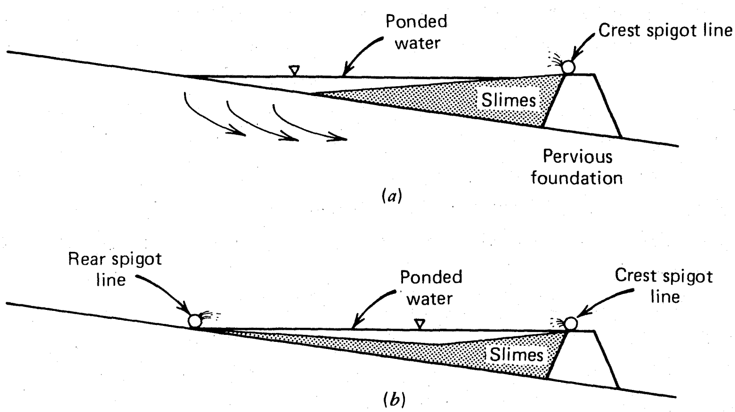


Figure 11.4 Control of seepage by slimes spigotting procedures. (*a*) Major seepage at water–foundation contact. (*b*) Foundation sealing by rear spigotting of slimes.

about as a result of reduction in seepage quantity alone, together with any contaminant uptake that may occur within the underlying natural materials.

On the other hand, other types of liners, both clay and synthetic, may be susceptible to cracking or rupture from foundation settlement, which may be considerable under loads imposed by typical tailings depths even for moderately dense foundation soils. This is not the case for slimes liners, which can easily withstand major foundation settlement or even seismic liquefaction without impaired effectiveness. Similarly, because the slimes are spigotted continuously, damage due to desiccation cracking or physical exposure is not of concern.

In summary, slimes liners are a promising concept that offer great potential for cost savings compared to other forms of liners. However, because of the careful spigotting and water-control procedures required, slimes liners are best suited to impoundments with low tributary inflow and mills with high mill water recirculation potential. Slimes liners are a logical adjunct to cycloned-sand embankments.

Clay Liners

Clay liners have long been used to reduce seepage from water storage reservoirs and toxic waste impoundments, and their application to tailings impoundments is straightforward. The term "clay" is not exclusive; satisfactory liners have been constructed using a variety of soil types, including CH, CL, ML, and SC in Unified Soil Classification terminology. Also, clay liners include compacted soils that incorporate additives.

The most common such additive is commercial bentonite. Blended with natural sandy soils in a traveling pugmill or central plant in proportions ranging from about 2% to 6% by weight, bentonite additives, where economically available, can provide a satisfactory liner material with permeabilities on the order of 10^{-6} cm/sec. However, the practices of spreading dry bentonite on the ground surface or mixing in place with a disc harrow are seldom suitable.

Another form of additive consists of chemical dispersants, including tetrasodium phosphate, sodium tripolyphosphate, soda ash (calcium carbonate), sodium chloride, sodium hexametaphosphate, and sodium silicate. Laboratory evidence and field experience indicate that clay permeability can sometimes be reduced about one order of magnitude if thorough and complete mixing is achieved (Dane, 1976; Lambe, 1974). Chemical dispersants modify the structure of a clay by producing a dispersed (face-to-face) structure of individual particles. An important question is the degree to which the effects of dispersants may be reversed by low-pH effluents. Laboratory work by Dane (1976) suggests that, once a dispersed structure is achieved in the compacted material, it is difficult for any large conductive pores to develop even if the clay is later subjected to a chemical environment favoring reaggregation.

Properties

The properties of compacted clays, either in a natural state or with any of the above-described additives, are largely a function of placement and compaction procedures. Properties of primary concern in liner applications are permeability, volumetric stability, flexibility, and piping resistance. These properties are strongly influenced by the placement moisture content. The degree and also the method of compaction in the field determine the density and the structure of the clay fill. In general, the kneading-type action produced by sheepsfoot compactors is most effective in breaking down agglomerations of clay particles and producing a *dispersed* and therefore lower-permeability structure.

The saturated permeability of compacted clays spans a wide range, with laboratory values most commonly ranging from about 10^{-5} cm/sec to 10^{-8} cm/sec (Lambe, 1955; Mitchell et al., 1965). The most important influence on permeability is clay mineralogy, which is specific to each deposit. However, moisture content and compaction conditions are important variables in determining the structure of the compacted clay. Permeability may vary over several orders of magnitude depending on the method and degree of compaction. Also, for a compacted clay at a given dry density, the placement moisture content will have a major effect on permeability. For example, Mitchell et al. (1965) show that for a particular type of clay, the laboratory permeability at 15% placement moisture content is 10^{-7} cm/sec. Reducing the placement moisture by only 2% at the same density increases permeability to 10^{-5} cm/sec, a change of two orders of magnitude. This change comes about because compaction at moisture contents above the optimum moisture content produces a low-permeability dispersed structure, while only slightly drier material will have a more permeable *flocculated* (edge-to-face) particle structure.

Laboratory tests do not account for a number of factors that influence permeability in the field, such as desiccation, the size of clay aggregates, and the effects of foreign material. In an excellent summary of the factors affecting clay liner permeability, Daniel (1981) concludes that it is fortunate to be able to predict in situ compacted clay permeability to within one order of magnitude from laboratory tests. The effects of such uncertainty (which may produce a tenfold variation in predicted seepage quantities) are clear, and these uncertainties are magnified when saturated permeabilities are used to derive unsaturated values for use in seepage analysis.

Other relevant properties of clay for liner applications include volumetric stability (shrink–swell potential) and flexibility or brittleness. Volumetric stability influences the degree to which a clay may crack on drying, and flexibility influences the ability of the compacted material to withstand settlement without cracking. Clays of relatively high plasticity and high placement moisture content not only have generally lower permeability but are usually relatively flexible. Unfortunately, these same characteristics usually result in an increased tendency to shrink and crack upon drying.

A final property of interest is dispersive piping potential. As discussed in Chapter 7, it is necessary to perform laboratory tests using the actual mill effluent to accurately gauge the susceptibility of a clay liner material to dispersive piping.

Effluent Reactions

Possible reactions between clay liner material and mill effluent, particularly low-pH uranium mill effluent, have been the subject of much concern. Possible mechanisms of permeability increase could come about by effluent-induced changes in the clay mineralogy or by breakdown of particles by acid attack. The results of considerable recent research (Gee et al., 1980; Nasiatka et al., 1981; Uziemblo et al., 1981) indicate that for low-pH effluents and for most clays, any damaging effect of acid is more or less counterbalanced by the plugging effect of precipitates that form from solution in the voids of the clay as a result of neutralization. Research to date has not shown acid attack of clay liners to be a pervasive problem for most clay materials. The effects of high-pH tailings effluents, on the other hand, remain largely unexplored, although Morrison (1981) notes that strong caustic solutions can cause shrinking of some clays and complete loss of fluid from laboratory permeameters within a few minutes after initial contact.

Design of Clay Liners

From a design standpoint, the decision concerning whether or not to use clay to line an impoundment often rests on the local availability of suitable material, where "suitable" usually refers to a minimum allowable permeability specified by regulations. Regulatory guidelines typically call for compacted clay having a maximum permeability in the range of 10^{-6}–10^{-8} cm/sec. Such requirements based on clay permeability alone are not necessarily related to liner effectiveness in reducing seepage and constitute an inappropriate criterion for liner design.

The degree to which a clay liner reduces seepage depends not on its absolute permeability but on its permeability relative to the foundation materials it overlies and the tailings it retains, in addition to the relative thicknesses of these materials in the flow sequence. A simple exercise using Darcy's Law similar to that presented in Figure 11.3 will quickly show this to be the case. In general, for a clay liner to reduce seepage from an impoundment significantly, its permeability must be at least 10 times less than that of the tailings. Thus, depending on whether sands or slimes are being retained, the limiting liner permeability may be 10^{-5} cm/sec, or 10^{-7} cm/sec. Similarly, the permeability of the liner must be at least one to two orders of magnitude less than the foundation materials to significantly reduce seepage under saturated flow conditions, depending on the distance of flow in the subsurface. Here, required liner permeability is specific to the characteristics of each site.

Clearly then, clay permeability alone cannot be an adequate liner design criterion. The degree to which seepage quantity may be reduced is a function of the permeabilities and thicknesses of all materials in the flow sequence, including tailings, liner, and underlying foundation materials. Intelligent evaluation of clay liners therefore argues for design criteria specified in terms of an allowable seepage rate or required percent reduction in seepage brought about by the liner, rather than criteria based on permeability of the liner alone.

In practice, the design of clay liners consists principally of determining liner thickness. Liner thickness is ordinarily determined empirically on the basis of past experience or regulatory requirements, and usually ranges from about 6 in. to 3 ft. The high cost of clay liners, however, would seem to argue for a more rational approach.

Rational determination of liner thickness depends on there being a clearly defined seepage objective. If an allowable seepage rate from the impoundment is specified, then simple saturated flow analogies, such as those shown in Figure 11.3b, or unsaturated analyses discussed in Chapter 10, can be used to determine the thickness of the liner in conjunction with the depth of overlying tailings. On the other hand, if it is specified that no seepage move beyond the bottom of the liner, then procedures described by Bureau (1981) based on the simplified McWhorter and Nelson (1979) unsaturated flow model can be used to determine liner thickness such that the wetting front is retained within the liner during the impoundment life. Or if it is required that no contaminants leave the impoundment, then unsaturated flow principles can be supplemented with geochemical analyses such as those described in Chapter 10 to determine the necessary thickness of the liner. In fact, Griffin et al. (1976, 1977) suggest that, because of the effectiveness of clays in attenuating many types of contaminants, the primary value of clay liners should be considered their role as a geochemical "sponge" rather than in reducing seepage quantities per se. They present methods for determining clay liner thickness based on geochemical principles.

Liner thickness and/or seepage can often be reduced if the liner is overlain by a drainage system. The effect of drains, as shown in Figure 11.5, is to reduce the head acting on the liner. For instance, suppose the drains in Figure 11.5 were applied to the example clay liner shown in Figure 11.3b such that the maximum head on the liner were reduced to 5 ft. In this case, seepage would be reduced to 0.35 ft^3/yr per unit area, nearly a tenfold reduction.

To the extent that liner seepage under saturated conditions is directly proportional to applied head, effective drains can also drastically reduce necessary liner thickness, often to the minimum practical construction thickness of about 6 in. to 1 ft. This can be an important advantage when natural clays are in short supply, particularly considering that an effective and inexpensive drainage blanket can often be constructed over the entire impoundment bottom from cycloned sand tailings. In addition to their seepage-

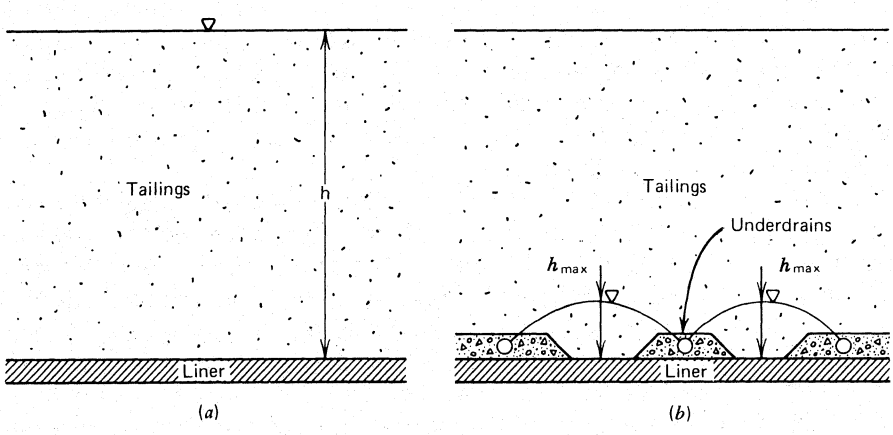


Figure 11.5 Comparison of head acting on clay liner. (*a*) Without underdrains. (*b*) With underdrains.

reduction function, drains underlying the tailings deposit have been investigated as a means of accelerating consolidation for tailings exhibiting slow sedimentation–consolidation characteristics (Rickel et al., 1982). However, the water collected from the drains must go somewhere, and if the collected water cannot be recycled to the mill, the source of the seepage problem may merely be shifted from the tailings impoundment to other water evaporation or treatment ponds.

Another factor influencing liner thickness is the ability of the liner to accommodate subgrade settlements without cracking or shearing. Clay liners, or any other liner for that matter, should not be placed on a soft impoundment bottom, over loose mine waste, or upon other compressible or moisture-sensitive materials. Although there are no definite rules for allowable liner settlement, a reasonable guideline is that the clay liner thickness should not be less than about one-half the magnitude of the total predicted foundation settlement under full-impoundment conditions.

One of the most important considerations for clay liner design is the configuration of the impoundment bottom as it affects desiccation and construction of the liner. For a completely flat impoundment bottom, the liner will presumably be completely covered with water as soon as discharge is initiated. However, for the more typical sloping impoundment bottom, a clay liner may not become covered with water for many months or years, even if the liner is constructed in segments slightly ahead of the rising pond level. Under these conditions, the effects of desiccation and frost action may be severely damaging to the liner. Data presented by Daniel (1981) suggest that the effects of even small cracks may be to increase liner permeability by two orders of magnitude or more. The extent to which these cracks may be sealed by swelling upon subsequent saturation is not known but will depend on the detailed shrink–swell characteristics of the material. It is likely, however, that desiccation cracks, once induced, will never be completely self-healing. The end result is that the effectiveness of clay liners may be questionable for sloping impoundment bottoms that do not quickly become covered with water. In addition, for very steep impoundment sides, practical installation of any type of liner, either clay or synthetic, may be unfeasible from a construction standpoint.

In summary, while clay liners have certain clear-cut advantages and disadvantages relative to other types, both their performance and effectiveness are subject to considerable uncertainty. Perhaps the most obvious factor governing the usefulness of clay liners is that clays must be present as a borrow source on or near the site. Depending on local conditions, a haul distance in excess of about 5–10 mi will generally favor the use of synthetic liners instead of clay on the basis of economics. While there is no doubt that clay liners can reduce seepage, their permeability and resulting effectiveness are difficult to predict accurately. It may well be the case, however, that the major advantage of clay liners is in their function as a contaminant-absorption medium, making the exact quantity of seepage they admit relatively less important.

Synthetic Liners

Although synthetic liners have been used for some time, their introduction as a tailings impoundment seepage-control method is relatively new. Kays (1977) provides a good overview of synthetic liner fabrication and manufacture, while Small (1980) and Kays (1978) describe the application of traditional liner design and selection methods to tailings impoundments.

The various types of synthetic liners include:

- Rigid liners.
  - Concrete.
  - Gunite (shotcrete).
  - Asphalt.
- Sprayed membranes.
- Synthetic rubber membranes.
  - Butyl rubber.
  - Ethylene propylene diene monomer (EPDM).
- Thermoplastic membranes.
  - Polyvinyl chloride (PVC).
  - Chlorosulfonated polyethylene ("Hypalon").
  - Chlorinated polyethylene (CPE).
  - High-density polyethylene (HDPE).
  - Elasticized polyolefin (EP).

Of these types, rigid liners are not widely used for tailings impoundments principally because of high cost and susceptibility to cracking in response to small settlements. Concrete and gunite may be susceptible to acid and/or sulfate attack, and asphaltic concrete may have questionable weathering and sun-aging characteristics (Kays, 1977).

Sprayed membranes include several new materials, such as catalytic air-blown asphalt and asphalt–polymer mixtures (Buelt and Barnes, 1981; Chambers, 1980). While these materials show promise on a research basis, practical problems remain to be resolved, including sunlight-aging characteristics and achieving uniform thickness in field applications.

The most common synthetic rubber membranes are butyl rubber and EPDM. Ordinarily reserved for linings of potable water reservoirs and related applications, the cost of these materials relative to other types of membranes is usually prohibitive for most tailings impoundment applications.

Thermoplastic membranes of the types listed above are the most common liners considered for tailings impoundments. The individual materials are discussed briefly below.

Thermoplastic Membranes

As discussed by Kays (1977), the chemical formulation of different liner materials results in differences in their ability to withstand weathering, sunlight exposure, and chemical attack by the effluent. While detailed tests can be performed to determine weathering resistance and chemical compatibility of specific liners and effluents (U.S. Environmental Protection Agency, 1980), data on chemical compatibility provided by liner manufacturers usually provide a reasonable guide. Nearly all thermoplastic membranes have good resistance to acids, bases, and salts in concentrations normally encountered in mill effluent. Aromatic hydrocarbons, such as toluene and benzene, highly damaging to many thermoplastic membranes, are not usually present in mill effluent. Some petroleum-derived flotation reagents, such as kerosene, may be moderately detrimental to some types of liner material, but these reagents are usually present in such low concentrations that they do not pose a major problem. The exception is where the liner is exposed to direct contact with floating oils, such as on impoundment sideslopes. With these generalizations, some of the specific characteristics of the various materials are:

Polyvinyl Chloride (PVC). PVC is among the least expensive liner materials, perhaps one of its major attributes. However, its weathering resistance is poor, and it must be protected by a water, tailings, or soil cover to avoid degradation by sunlight. PVC is relatively sensitive to petroleum derivatives unless an oil-resistant resin formulation is used. PVC in tailings impoundment applications is commonly used in 20–30 mil thicknesses.

Chlorosulfonated Polyethylene (''Hypalon''). ''Hypalon'' is perhaps the most common membrane in tailings impoundment applications. Available and commonly used in 30 and 36 mil thicknesses, it is resistant to most chemicals in mill effluent, including petroleum derivatives in low concentrations. It has good aging characteristics, and, significantly, it can undergo direct sunlight exposure, precluding the need for a soil cover on exposed sideslopes. Its cost per unit area is very roughly twice that of PVC. A ''Hypalon''-lined tailings impoundment is described by Lubina et al. (1979).

Chlorinated Polyethylene (CPE). CPE is similar to ''Hypalon'' in most characteristics, but at a cost intermediate between that of ''Hypalon'' and PVC.

Elasticized Polyolefin (EP). EP is a relatively new material. It has good sun-aging characteristics and need not be covered. It is roughly comparable in cost to ''Hypalon'' but does not have high resistance to petroleum derivatives.

High-Density Polyethylene (HDPE). Another relatively new material, HDPE has good chemical, weathering, and sunlight resistance. It is available in thicknesses ranging from 20 mils to 140 mils. Where thicker materials are used (80–100 mils), puncture and tearing resistance is improved compared to other thinner liners, but at a proportionally higher cost per unit area. Small et al. (1981) describe an application of HDPE to a tailings impoundment.

Usually an impoundment is lined with only one material, but in some cases, it is possible to join liners of two different types. For example, PVC and ''Hypalon'' can be joined with a factory splice, making it possible to line an impoundment bottom with less expensive PVC and the sideslopes, where sunlight exposure is a concern, with ''Hypalon'' or other similar materials. Webb et al. (1980) describe one such hybrid application.

Synthetic membranes have only recently been constructed for tailings impoundments, so there is some experience with their installation but little with operation. Considering that an impoundment membrane alone may cost several million dollars, it becomes evident that cost is a major factor in the selection of liner materials. Differences in tearing and puncture resistance are usually more a function of liner thickness than material type, and for equivalent thickness, the strength of the various liner materials is more or less similar. Because design constraints for membrane liners are not primarily site specific, economics play a major role in material selection.

Design of Membrane Liners

After selection of a material type that is compatible with the mill effluent, the most important issue in membrane liner design becomes selection of material thickness, and in this regard, the term "design" is somewhat of a misnomer. Liner thickness is determined on a largely empirical basis from experience with water-impounding reservoirs. In general, a liner thickness that has performed adequately in water-retention applications is ordinarily assumed to be suitable for tailings impoundments. However, past precedent must be invoked with considerable caution because of several unexplored areas in the mechanical behavior of liners under tailings deposits. First, tailings, having a higher unit weight than water, will apply higher unit vertical stress to the liner and often with greater depths. Second, tailings, unlike water, exert shear stresses on inclined portions of the liner. The behavior of liner materials under applied shear stresses has not been evaluated on either a theoretical or experimental basis, although Andrawes et al. (1980) present the results of research applying to embankments that incorporate liners. Until adequate theoretical models are available, it is likely that design of liner thickness will continue to be empirical, incorporating actual operating experience with tailings impoundments as it becomes available.

Other elements of liner design involve earthwork. Because it is impractical to install liners on slopes steeper than about 3:1, the presence of a liner can control the steepness of the upstream slope of an embankment, potentially adding considerable volumes of fill. Also, the liner must be underlain with a smooth bedding of sandy soil, which may not be readily available. More than a few failures of membrane liners by puncturing have been caused by inadequate bedding. For impoundments which contain liquid effluent only, it is sometimes necessary to provide vents or underdrains to remove subgrade gas that might otherwise lift the liner. Baldwin (1983) describes the use of a geotextile sublayer below the liner for gas venting purposes.

Exposed liners on slopes must be protected against wind lifting, wave action, and, depending on the material, sunlight exposure or contact with floating oils. While gas vents may provide a solution to the wind-lifting problem, it is often the case for a variety of reasons that soil is used to cover exposed portions of the liner on slopes. The soil cover, in turn, must be overlain by riprap to prevent its being eroded. Obtaining and placing these materials without damaging the liner often poses a major design problem and results in major added expense over and above the cost of the liner itself.

Effectiveness of Membrane Liners

Leakage through membranes results from two sources: leakage due to pinholes and porosity in the material itself, and leakage through tears or seams joining individual sheets. Leakage through tears and seams is usually assumed to be the major source of seepage loss, and the quantity of seepage is primarily a function of the quality of field seaming procedures and the care taken to prevent punctures during installation. Even for well-constructed membranes, some leakage through seams undoubtedly occurs, but the quantity is impossible to predict.

Leakage by the other mechanism, flow through the intact material, is usually considered so small as to be negligible. Nevertheless, this flow may in fact be comparable to that of other types of liners. Although the permeability of membrane liners is exceedingly small, it is finite, and a few measurements have been reported in the literature. For example, assuming a 30-mil liner and using a permeability of 10^{-10} cm/sec, as cited by Buelt and Barnes (1981) for "Hypalon," calculations similar to those presented in Figure 11.3 can be made under the assumption that Darcy flow governs liner seepage. These calculations suggest that seepage through the intact liner material itself could be virtually the same as for the slimes liner and clay liner shown in Figure 11.3, not accounting for any tears that might be present.

Both the above factors suggest that some seepage can be expected through membrane liners but that the quantity is difficult or impossible to predict. Perhaps it is this lack of predictive methods that leads to the misleading application of such terms as "impervious" to membrane liners. This should not be taken as an indictment of membranes in particular, but it should emphasize the fact that, while seepage can indeed be reduced, achieving "zero discharge" from any type of liner—membrane, clay, or otherwise—is physically impossible.

A major issue in evaluation of membrane liners is often longevity. The service life of liners is somewhat clouded by the limited time frame of operating experience with individual materials. The data that are available suggest that properly protected liners compatible with the contained effluent experience little or no deterioration after 10 yr or more (Hickey, 1969). Laboratory tests that simulate accelerated aging usually indicate useful lives for most materials in excess of 20 yr. On the basis of these factors, it appears that most properly constructed membrane liners can be expected to have a service life at least as long as the active life of most tailings impoundments.

EFFLUENT MODIFICATION

Mitigating the effects of seepage can be brought about in two general ways: by engineered structural measures, or by changing the nature of the effluent to eliminate toxic contaminants. Considering the high cost of liner systems and the inherent uncertainties in their effectiveness, modifying the effluent should not be overlooked as an alternative where strict groundwater protection criteria would otherwise require liners or other expensive seepage-control systems. Eliminating contaminants from the effluent stream can result in considerable savings in tailings disposal cost if by so doing a Type I or Type II rather than a Type III system can be constructed.

Modifying the effluent can be accomplished by either pretreatment of the effluent or by modifications to the metallurgical process. A good example of pretreatment is cyanide-laden effluent from gold leaching. Here, relatively simple chlorination or oxidation treatment of effluent prior to discharge can allow disposal of tailings under considerably reduced seepage restriction requirements and at appreciably reduced impoundment cost. Also, impoundment design may be simplified by allowing for discharge of extreme flood inflows through spillways if a high-quality effluent can be obtained. Similarly, neutralization of low-pH effluents, such as uranium, can be an economical alternative to stringent impoundment seepage controls if the cost of neutralization can be shown to be less than the cost of impoundment liners or similar measures.

Metallurgical extraction processes have historically been designed to maximize extraction efficiency by using the most effective grinding processes and reagents. Tailings and effluent disposal costs have in the past had an insignificant effect on process design, and these costs have often been neglected in comparing alternative processes. With increasingly stringent regulations, however, disposal costs may have a significant effect on process alternatives where toxic or otherwise detrimental reagents are added. If disposal costs are included, it might be found economical, for example, to eliminate such flotation reagents as cyanide or pH modifiers. Substitution of less harmful reagents resulting in reduced seepage-control cost might be possible with little loss in extraction efficiency.

SUMMARY

Fundamental to evaluating seepage-control measures is defining their objectives and intent. Depending on the degree of groundwater protection necessary and the precise terminology used to define and measure the effects of seepage, control measures on a number of levels may be possible. On the least complex level, the nature of the mill effluent, the groundwater regime, and the effectiveness of geochemical processes may not justify special seepage-control measures. In other cases, partially effective control measures may be justified if the seepage effects are monitored during operation. In extreme cases of highly toxic effluent, high contaminant concentrations, and critical groundwater protection goals, liners of one kind or another are often required.

Table 11.2 summarizes the advantages and disadvantages of the various seepage control measures discussed in Chapter 11. Some methods, such as cutoff trenches and collector ditches, are relatively inexpensive but are effective in specific circumstances. Other methods, such as slurry walls and grout curtains, are usually expensive and may be effective or practical only under specific site conditions. While the most effective methods of seepage control are usually considered to be clay or synthetic liners, there are major costs and major uncertainties associated with both. With regard to the effec-

Table 11.2 Summary of Seepage Control Measures

Seepage Control Measure	Type	Advantages	Limitations
Seepage barriers	Cutoff trench	Inexpensive; installation can be well controlled	Not practical for saturated foundations; effective only for shallow pervious layers
	Slurry walls	Low-permeability barrier can be constructed	High cost; not well suited for steep terrain or bouldery ground; impervious lower boundary required; practical depth of about 40 ft
	Grout curtains	Barrier can be constructed to great depths; not affected by site topography	High cost; limited effectiveness due to permeability of grouted zone; cement grouting practical for only coarse soils or wide rock joints
Return systems	Collector ditches	Inexpensive; suitable for any type of embankment	Completely effective for only shallow pervious layers, but still beneficial in other cases
	Collector wells	Unlimited depth; useful as a remedial measure	Expensive; effectiveness depends on local aquifer characteristics
Liners	Slimes liner	Inexpensive; not susceptible to rupture or cracking; effectiveness comparable to other types of liners	Limited to tailings with fine fraction; not suited to high-runoff sites; requires cycloning and special spigotting procedures
	Clay liners	Effective in reducing seepage; effective in restricting contaminant movement by geochemical processes	Clay must be locally present; expensive; properties difficult to predict; poorly suited to steep terrain because of desiccation
	Synthetic liners	Simple to construct; effectiveness not dependent on site conditions; not affected by most mill effluents	Expensive; effectiveness sensitive to quality of construction; limited tailings operation experience; installation difficult on steep or variable terrain

tiveness of liners, two things are clear. One is that liners can reduce seepage relative to that from an unlined impoundment. However, they are by no means impervious. "Zero discharge," even with the use of impoundment liners, remains an elusive goal. Recognition of this fact emphasizes the point stressed by Taylor and Antommaria (1979): ultimately, it is the naturally operating geochemical and groundwater flow processes which determine the degree to which contaminants affect groundwater resources, and even expensive structural measures cannot substitute for a detailed understanding of the impoundment site and the chemical nature of the materials it contains.

12

Reclamation of Tailings Impoundments

By about 1900 the original Comstock Lode was dead. . . . A cyanide scavenger moved in, erected a dragline with a clamshell bucket to seize the tailing, cleaned it out, and dumped it back in the Gold Canyon stream bed, where it is now visible as a dead-white flat of detritus. Vegetation is only now beginning to creep over this flat, a generation after the last bucket was dumped.

Otis Young, *Western Mining*

Reclamation is a concept relatively new to the field of tailings disposal. Traditionally, the day of mine closure resulted in the establishment of a new mining ghost town, with the tailings pile being one of its chief artifacts. If the climate and chemical nature of the tailings were not too severe, volunteer vegetation might eventually establish itself. But more often than not, the old tailings piles, located mostly in or near streambeds, were washed away during floods before any natural vegetation could establish itself.

Such practices are obviously no longer appropriate in a present-day context. Reclamation of disturbed mine and tailings impoundment areas is often a major problem, particularly in arid and/or cold climates. Failure to address this problem may hamper regulatory approval of a proposed mine. Moreover, failure to incorporate reclamation considerations into tailings disposal planning from the start may result in an impoundment that is very difficult and expensive to reclaim later upon abandonment.

In the preceding 11 chapters, the disposal of tailings has been considered primarily in the context of planning and operation of a mine, and may be thought to end when active discharge ceases. This is in fact not the case. From a larger point of view, it is only tailings deposition, not disposal, that ceases. Tailings management must continue until such time as the deposited tailings are assured to be permanently stable and environmentally innocuous. Because serious efforts at reclamation of tailings impoundments have been so recent, there is very little experience by which to judge their likely long-term success. At present, reclamation in the short term is very often a process of trial and error, although advances in botany, agronomy, and detailed appreciation of tailings chemistry are adding useful knowledge. In the long term, however, evaluating the ultimate success of present-day tailings reclamation efforts will be done by future generations.

Insight into reclamation efforts can be gained by considering the objectives they are intended to achieve. While different emphases are often placed on various priorities, the following are usually considered to be fundamental objectives of reclamation:

Long-term mass stability of the impoundment.
Long-term erosion stability.
Long-term prevention of environmental contamination.
Eventual return of the disturbed area to productive use.

Various options are often considered to achieve these aims, including riprap, chemical stabilization, and vegetative stabilization. The remainder of Chapter 12 provides a discussion of these reclamation objectives and methods.

PURPOSES OF RECLAMATION

Reclamation costs for tailings impoundments commonly range from \$1,000 to \$5,000 per acre. Thus, for many typically sized impoundments, reclamation may cost from roughly \$1 million to \$10 million, a considerable sum in itself. In addition, as a result of the less than responsible behavior at many past mining operations with regard to reclamation it is understandable that government authorities commonly require the mine operator to post a reclamation bond to ensure that planned reclamation efforts are actually carried out after mine abandonment. Bonding costs are proportional to and magnify the estimated cost of actual reclamation. In light of the economic consequences involved, it is important that the purposes of reclamation be well understood.

Long-Term Mass Stability

Slope Stability

Chapter 8 discussed at length the issues involved in assessing embankment slope stability during operation. The principal difference in stability between the operating and abandonment cases is in phreatic conditions. Ordinarily when tailings discharge ceases and a continuous source of water no longer supplies the decant pond, the phreatic surface within the embankment drops dramatically, resulting in greater embankment slope stability after reclamation than during operation. Consequently, there is little concern for mass stability of embankment slopes after abandonment; any tailings embankment slope that was stable during operation will maintain its overall integrity after operations cease. This assumes, of course, that effective measures are taken during reclamation to prevent accumulation of water in the impoundment, as subsequently discussed.

Seismic Stability

During impoundment operation, liquefaction of loose deposited tailings in high seismic areas may raise the spectre of massive flow-type slides during earthquakes, as discussed in Chapter 9. However, the expected lack of saturation after abandonment and reclamation ordinarily precludes liquefaction, even under major seismic shock. Therefore, the seismic stability of abandoned tailings deposits is usually assured, as evidenced by the stability of abandoned deposits during the Chilean La Ligua earthquake discussed in Chapter 9, provided again that measures are taken to ensure that the deposit remains unsaturated.

Hydrologic Stability

Hydrologically induced failures are the major cause of mass instability of abandoned tailings deposits. Accumulation of runoff water in an impoundment, in addition to raising the possibility of slope or seismic instability, can cause direct impoundment failure by overtopping or by erosion at the embankment toe.

Although hydrologic stability is a major factor in designing a stable impoundment during operation, it is not necessarily the case that an impoundment that is safe from a hydrologic standpoint during operation will remain so after abandonment and reclamation. As discussed in Chapter 4, a major factor in hydrologic planning is selection of the appropriate design flood, and return-period methods often provide an acceptable basis for design over the operating lifetime of the impoundment. Consider, for example, an impoundment with a 30-yr life designed to handle the 1,000-yr return-period flood. As shown in Table 4.3, this impoundment would have a 3% failure probability during its active 30-yr life. This failure probability may represent a reasonable operational risk. If, however, the life of the impoundment after abandonment and reclamation were arbitrarily defined as 500 yr, the failure probability would increase to 40%, approaching a 50/50 chance that hydrologic failure will occur at some point over the 500-yr period in the absence of additional flood-control measures. Longer assumed postabandonment lifetimes will increase the failure probability proportionately, and an impoundment designed for any return-period flood after abandonment will fail eventually if the reclaimed impoundment has a perpetual life. This observation argues strongly for hydrologic design of abandoned impoundments on a Probable Maximum Flood (PMF) rather than a return-period basis if economically feasible, since the PMF theoretically has no chance of being exceeded within any foreseeable time frame.

As discussed in Chapter 4, design for PMF flood handling is often difficult, further emphasizing the need to minimize runoff inflow by judicious siting rather than by structural flood-handling measures. Diversion ditches can be used to divert long-term runoff, but constructing ditches for the PMF or a major fraction thereof is often very costly. Further, perpetual ditch maintenance may be required. Similar problems apply to the use of permanent abandonment spillways or culverts, with the additional disadvantage that frequent flood passage or continuous base stream flow through abandonment spillways may result in long-term saturation of the tailings deposit, raising the possibility of mass instability due to embankment slope or seismic factors.

For sidehill and valley-bottom impoundment configurations the ultimate surface of tailings deposited by peripheral spigotting will slope back from the embankment crest to the rear of the impoundment where the tailings surface joins the natural ground surface. In these cases, minor regrading of the tailings surface may allow runoff to pass around the rear portion of the impoundment rather than accumulate in it.

For all types of impoundments—cross-valley, sidehill, valley-bottom, and completely enclosed ring dikes—accumulation of water may be prevented by capping the impoundment with material that is peaked near the center of the impoundment and sloped to drain toward its perimeter. Mine waste may provide a convenient source of capping material. However, major quantities of material are required to produce a nominal 0.5–1% cap slope for drainage, even for relatively small impoundments. Also, additional material must be added to prevent water-ponding depressions that would otherwise result from settlement of slimes under the weight of the capping material. It is not uncommon that an additional 10 ft or more of capping material is required for settlement compensation over the slimes zone.

Long-Term Erosion Stability

An additional and essential purpose of reclamation is to prevent long-term erosion of the abandoned deposit by wind or water. Tailings are notoriously susceptible to gullying by water runoff erosion, and one need only consider the similarity between tailings and the sands that comprise migrating dunes to appreciate the need for protection against wind erosion.

While wind erosion is primarily a factor on flat, unbroken surfaces, water erosion is most often a problem on embankment slopes. Although no adequate analytical methods are available to model water erosion potential as a function of embankment slope steepness, experience indicates that tailings embankment slopes flatter than about 3:1 are usually required for reasonable erosion resistance and establishment of vegetation. Slopes flatter than 5:1 are sometimes preferred (U.S. Nuclear Regulatory Commission, 1979). If these embankment slopes will ultimately be required for erosion protection purposes after abandonment, there may be little reason for elaborate design efforts to justify steeper embankment slopes for the impoundment during operation. In effect, reclamation requirements rather than operating stability considerations may ultimately control steepness of embankment slopes.

Environmental Contamination

While seepage ordinarily diminishes and eventually ceases after termination of active tailings discharge, special reclamation-related precautions may be necessary in some cases. In particular, pyrite oxidation is greatly enhanced in the unsaturated zone, as discussed in Chapter 1. Thus, as the phreatic surface drops in an abandoned impoundment, oxidation of pyrites, if they are present in the tailings, will greatly accelerate, producing reduced pH and increased liberation of metallic contaminants. These oxidation-produced contaminants, potentially much more noxious than those present during impoundment operation, may be leached into the groundwater by infiltration of precipitation on the impoundment surface. To prevent leaching of contaminants in such cases, a clay cap is often required over the impoundment surface upon abandonment and reclamation, in conjunction with regrading to prevent ponding of runoff. In the particular case of pyrite-rich tailings, the option of maintaining saturated conditions in the impoundment after abandonment to prevent long-term oxidation should be considered.

Uranium tailings present another special case. Here, radioactive decay of radium-226 produces potentially harmful daughters of gaseous radon-222. Diffusion of radon gas does not occur for saturated tailings, but after abandonment, reduced saturation may make radon-reduction measures necessary. Radon diffusion and placement of soil covers over the impoundment to prevent it have recently been widely researched (Re et al., 1980; Rogers and Nielson, 1981; Nelson, Gee, and Oster 1980). The cover thickness required depends on the allowable rate of radon emanation, the Ra-226 content of the tailings, and the soil type and long-term moisture content of the cover material. While thinner covers may be adequate based on the application of this and similar research to specific cases, the U.S. Nuclear Regulatory Commission (1979) requires a minimum 10-ft cover over uranium tailings impoundments to prevent long-term radon emanation after abandonment and reclamation.

Return of Land to Productive Use

While the final objective of reclamation—to return the tailings impoundment area to productive use—is certainly a desirable goal, the term "productive use" holds different meanings for different people and provides a continuing subject of debate. To a mining company, productive use in the future may mean allowing for the possibility of remining the tailings deposit to glean unrecovered mineral values; to others, development of housing subdivisions on the deposit may be desirable. The definition of "productive use" considerably influences the manner in which the tailings deposit is eventually reclaimed.

Ordinarily, productive use is defined in the context of land use patterns that existed prior to development of the mine, and this definition argues for a return of the area to some semblance of its premining configuration and vegetative cover. Even here, however, goals may be unclear. Revegetation of the deposit with introduced grasses may be suitable for productive use as livestock grazing range. On the other hand, these introduced grass species may not be suitable for wildlife habitat, which requires a more diverse cover of native species (Jones, 1982). While controversy surrounds debates over productive use, definition of specific goals is essential if reclamation is to be carried out in a systematic and planned way. Liaison with regulatory authorities is an essential element in defining an approach that is acceptable to all parties, including local citizens and environmental groups (Palmay and Gwilym, 1980).

STABILIZATION METHODS

Of the reclamation purposes discussed above, stabilizing the impoundment against long-term wind and water erosion is often the most technically problematic and difficult to achieve. While assuring mass stability, preventing environmental contamination, and returning the area to productive use may require substantial effort at considerable cost, most concern over reclamation effectiveness usually centers around achieving long-term erosion stability. Here is where difficult technical issues arise.

Immediately after tailings discharge ceases, the impoundment surface will usually consist of a relatively firm and reasonably dry above-water sand beach, a soft and saturated slimes surface, and a submerged area covered by the decant pond. Assuming that permanent impoundment of water is to be prevented for reasons previously discussed, the first step in accomplishing stabilization is to allow the entire surface of the impoundment to dry. Preparatory drying may require long periods of time depending on climate, the size of the decant pond, and the nature of the tailings. The object is to achieve a surface firm enough to support the weight of equipment used in stabilization efforts. Drying of the decant pond may be by evaporation or by drainage to an effluent treatment plant depending on local climatic characteristics, with the time required depending on pond size, treatment capability, and net evaporation rates. Desiccation and consolidation of the slimes surface may take considerable time, which can be estimated on a preliminary basis using methods developed by Krizek et al. (1977) for dredged fill. In the extreme case of phosphate slimes deposited without flocculants or other means of accelerating consolidation, it may take many months for the surface to become firm enough to support the weight of a man and many years to support lightweight equipment.

Once a reasonable degree of drying and desiccation of the impoundment surface has been achieved, stabilization efforts can proceed, usually according to three basic options: riprap, chemical stabilization, or revegetation. These options are not mutually exclusive and can be used in combination. For example, riprap may be used to stabilize embankment slopes, with vegetation used on the impoundment surface. Similarly, chemical stabilization can be used in conjunction with revegetation. These three basic options are discussed individually in the following sections.

Riprap

The use of riprap for erosion stabilization follows from its conventional use for engineering purposes as channel protection and slope protection to prevent water erosion. For wind and water stabilization purposes, "riprap" includes not only conventionally sized rock fragments but also gravels. Smelter slag has also been used as a stabilizing cover for tailings impoundments (Dean et al., 1974). The effectiveness of riprap to stabilize soils against erosion is evidenced in nature by the development of *desert pavement,* a surface layer of pebbles remaining after wind erosion of sandy soils that forms an effective erosion-resistant surface for long periods of time.

Rules for sizing riprap and determining the thickness of riprap cover for conventional engineering applications are referenced in Chapter 4. These rules, however, may not apply to riprap for erosion stabilization. For example, because velocities of water flowing over reclamation riprap on relatively flat surfaces are generally low and because a major function of riprap particles is to break the impact of falling raindrops, only a thin layer of riprap, essentially enough just to cover the surface, may be necessary for stabilization. Also, filter layers beneath coarse riprap are probably not required for reclamation purposes except possibly on steep slopes. In fact, research has shown that for gravel-sized materials, the finer the gravel, the less thickness of cover required to prevent wind erosion (Chepil et al., 1963).

Riprap for stabilization purposes may be readily available at many mine sites in the form of mine waste or stripped overburden. However, the cost to completely cover a tailings impoundment, even with a relatively thin layer, may be high. Furthermore, riprap does little to enhance vegetation and in some cases may severely retard revegetation of the underlying tailings by natural processes, delaying or preventing return of the land to productive use. While it has been suggested that riprap may be conducive to deposition of windblown soil particles forming a suitable habitat for vegetation growth (Dreesen et al., 1978), observation of riprap-like talus deposits in nature provides little support for this view.

Chemical Stabilization

A number of sprayed-on chemicals have been used for dust control, giving rise to their limited application for wind-erosion stabilization in a reclamation context. Chemical stabilizing agents used for temporary tailings erosion control include elastomeric polymers, calcium lignosulfate (a paper mill waste), asphalt emulsions, sodium silicates, and resinous adhesives. In general, chemical stabilization cannot be considered a permanent reclamation measure. Reapplication of the chemical agents is usually required on an annual basis at a cost ranging from roughly $200 to $500 per acre.

Although insufficient for long-term reclamation, chemical stabilization can still be of value. Morrison and Simmons (1977), for example, describe the use of a petroleum resin emulsion to minimize blowing dust from exposed portions of operating tailings ponds. Also, chemical stabilization may be used to augment vegetative stabilization during initial periods, when seedlings are most susceptible to sandblasting or burial by blowing tailings (Dean and Havens, 1970).

Vegetative Stabilization

Vegetation is by far the most common and usually the preferred stabilization option for tailings impoundments. If a self-perpetuating vegetative cover can be established, not only can wind and water erosion be minimized, but also the impoundment can be returned to some semblance of its original appearance and land use. In favorable climates and for tailings of favorable chemical composition, revegetation may require only modest effort or may occur by natural processes during a reasonably short period of time. However, in arid climates or for tailings having low pH, or high concentrations of heavy metals or salts, establishment of vegetation may be a lengthy, difficult, and costly process. The factors involved in revegetation are discussed separately in a subsequent section.

Impoundment Layout Factors in Reclamation

Regardless of which of the above stabilization options is used, difficulty and cost can be minimized by incorporating reclamation-related factors as an integral part of tailings impoundment planning.

In Chapter 5, considerable discussion is devoted to optimizing impoundment layout by considering trade-offs between impoundment area and depth. It is evident that the cost of reclamation is directly proportional to the area disturbed, and to this extent, reclamation problems can be minimized in degree if not difficulty by deeper impoundments that cover a smaller area.

By including reclamation costs in comparisons of alternative impoundment layouts, a balance can be struck that incorporates all the factors contributing to impoundment cost, including embankments, liners (if any), and reclamation.

As also discussed in Chapter 5, segmented impoundments may offer significant reclamation advantages compared to a single, larger impoundment. Stabilization of impoundments upon abandonment, particularly by vegetative methods, is to a large degree a trial-and-error process. If impoundment segments can be constructed, filled, and reclaimed sequentially, a valuable opportunity is offered to determine the most effective and least costly reclamation method during project operation rather than after. Performance of field-scale reclamation experiments on individual impoundment segments can minimize costly mistakes. In addition, prompt and successful reclamation of individual impoundment segments can decrease the overall environmental impact of tailings disposal by reducing the area disturbed by active tailings impoundments at any one time.

TAILINGS REVEGETATION

As previously noted, revegetation is ordinarily the prime option for reclamation stabilization of tailings impoundments. Although revegetation has been planned for many currently active impoundments, relatively few tailings reclamation programs have been carried out to successful completion. This section discusses the wide and complex array of factors involved in revegetation strategies for tailings impoundments.

Vegetation Requirements

Vegetation growth is dependent on two principal factors: climatic characteristics and the nature of the growth medium. The success of revegetation requires that the program be suited to constraints imposed by both.

The strong effect of climate on vegetation growth is intuitively apparent, and from a revegetation standpoint, it often seems that mines somehow have a perverse tendency to be located in climates that are arid, cold, or otherwise inhospitable to vegetative growth. Low rainfall often presents major difficulties in revegetation, although vegetative stabilization has been successfully carried out in arid climates (Ludeke, 1973). The short growing season and such problems as frost heave in cold climates also hamper vegetative growth, although again reclamation efforts can be successfully designed specifically for cold climate (Brown et al., 1978). Perhaps the most difficult case combines cold climate with arid conditions, a characteristic of many alpine and arctic tundra mine sites (Brown, 1974).

The medium within which vegetative growth takes place also has an important influence on revegetation. Soil characteristics that influence vegetative growth include:

Texture.
Fertility.
Toxicity.

Texture refers to the grain size of the soil as well as the degree of aggregation or agglomeration of individual particles. Fine-textured soils may hold excessive moisture when wet or may be compact when dry, inhibiting root penetration. Coarse soils, on the other hand, may not be able to hold sufficient moisture to sustain plants, even in the presence of adequate rainfall. Healthy plant growth is promoted by a well-aggregated soil composed of agglomerates of finer textured particles; the finer grained individual particles that comprise the aggregates hold water and nutrients, while the aggregates themselves allow drainage and root penetration.

Soil fertility refers to nutrients required for plant growth, including nitrogen, potassium, and phosphorus, as well as to necessary bacteria and fungi.

Toxicity of the growth medium will stunt or kill developing plants. While heavy metals such as iron, manganese, zinc, and copper are necessary in very small quantities for healthy plant growth, the presence of the same constituents in higher concentrations may slow or preclude plant development. Although threshold levels vary, total metal concentrations exceeding 0.1% for individual elements often prove to be toxic (Bradshaw and Chadwick, 1980). Outside relatively narrow limits, pH and salinity can also be harmful.

Judged by these criteria, many types of tailings provide a poor growth medium on the basis of texture, fertility, toxicity, or all three. Tailings do not have ideal texture. Tailings sands have poor moisture-retention characteristics, while slimes are poorly aerated and become compacted on drying—both factors that inhibit root penetration (Ripley et al., 1978). From a nutrient standpoint, tailings are usually found to be deficient in nitrogen particularly, but also in potassium and phosphate (Dean et al., 1974; Nielson and Peterson, 1972). Nitrogen-fixing bacteria are also absent.

Chemical toxicity sometimes poses major problems. Low pH can have a direct effect on seeds and plants by acid attack. In addition, metallic ions liberated by low pH—particularly iron, copper, zinc, and lead—are often toxic to plants (Peterson and Nielson, 1973), and metal toxicity problems may develop at very inopportune stages in plant development. For example, tailings that contain pyrites may originally be suitably low in metal content to support initial seed germination, but may then become highly toxic from a pH and metal standpoint as a result of pyrite oxidation (Nielson and Peterson, 1972). The acidification process, which may take from a few months to a few years, may occur just in time to kill a well-established stand of young plants. Excessive salinity may also kill plants by inducing an osmotic gradient through roots that results in dehydration and withering (Dean and Havens, 1970).

To revegetate tailings directly without topsoil requires that inadequacies related to climate and growth medium be resolved. These may pose difficult but not insurmountable problems. In dry climates, irrigation may be used, at least temporarily. Such techniques as drip irrigation (Bach, 1973) and condensation traps (Hodder, 1979) have been adapted to tailings irrigation, in addition to conventional spray irrigation. Care must be exercised, however, in irrigating slopes to avoid erosion by concentrated irrigation runoff (Bengson, 1979). Improving deficiencies in texture and nutrients often go hand in hand with such techniques as straw mulching, hydroseeding, and use of additives such as sewage sludge (Day and Ludeke, 1979; Dean et al., 1974). Ameliorating chemical toxicity of tailings can be more difficult, but lime neutralization of low-pH tailings has been performed to increase pH to about 5.5–6.5 (Sutton, 1973; Sorrell, 1974). However, the permanence of liming may not be assured and periodic reapplication may be required if acid is brought to the surface by moisture in the tailings. In some cases, preleaching by water infiltration may reduce surface toxic metals or salts to tolerable levels.

It is apparent that considerable effort may be required to modify the tailings to provide conditions conducive to direct revegetation. The alternative is to cover the entire impoundment with topsoil, an option increasingly required by regulations. Topsoil is commonly applied to the impoundment surface in depths ranging from 6 in. to 3 ft. However, as noted by Ripley et al. (1978), topsoiling is not a cure-all and often involves great expense. To be effective, topsoil must be stripped from the impoundment surface and stored in stockpiles throughout the active life of the impoundment; to borrow topsoil from areas outside the impoundment would mean denuding a borrow pit essentially equivalent in area to the impoundment, a procedure producing little if any net benefit. Stockpiling topsoil for periods of many years may rob it of nutrients, bacteria, fungi, and texture-enhancing organics, potentially leaving a material almost as barren as the tailings it is intended to cover.

These disadvantages notwithstanding, topsoiling may be the only solution if the chemical conditions of the tailings are too severe to be ameliorated by other means. Barth and Martin (1981) note that it may be necessary to place a filtered gravel layer between the tailings surface and topsoil layer to prevent upward migration of contaminants into the topsoil by capillarity, a procedure that usually involves great expense. Also, for pyrite-rich tailings, topsoiling with a low-permeability soil may be mandatory to prevent acid leaching by infiltration.

Environmental Factors

In addition to climate and growth medium, the environmental characteristics of plant habitat influence vegetation establishment and regeneration. These environmental factors occur on a very small scale and may vary over short distances.

For revegetation directly on tailings, wind movement of sand may inhibit seedling establishment by sandblasting or burying the young plants before their root systems are able to have any stabilizing effect. Seeds may also become desiccated or transported to undesirable growth areas. Mitigating measures to prevent sandblasting include temporary chemical stabilization, water sprinkling to maintain a wet surface, use of windbreaks, heavy mulching, and furrowing.

Temperature and radiation are other elements of the vegetative microenvironment that may be severe on tailings impoundment surfaces. Most plants are productive between temperatures of about 40–100° F. However, sustained surface temperatures on natural soils in excess of 150° F are not uncommon, and temperatures on dark-colored and/or south-facing impoundment surfaces may be even higher. In some cases, high surface temperatures have been identified as the major factor restricting mine waste revegetation (Ripley et al., 1978). On the other hand, while a light-colored tailings surface may be cooler than a dark one, it may not be any more hospitable to plant growth. Light-colored surfaces reflect far more solar radiation, having direct destructive effects on plant foliage or indirect effects via the increase in heat load on leaves. Mitigation measures to correct temperature and radiation extremes are limited to shading and in some cases providing a heavy straw cover, although the best solution is probably to select species adapted to the microclimatic conditions. Thus, for example, species suited to north-facing impoundment slopes may be different from desirable species on south-facing exposures.

Microclimate can be modified to some degree by surface preparation techniques—for example, deep furrowing or *gouging*. These techniques involve constructing depressions or basins that provide a degree of seedling protection from radiation and wind sandblasting while at the same time acting as a trap to collect surface moisture (Hodder, 1979).

A final but noteworthy environmental factor related to vegetation establishment is grazing, by either domestic livestock or wildlife. Initial stages of seedling germination and growth are critical to the success of revegetation, and fences or other measures are often mandatory to protect the vegetated area from the effects of grazing. Fedkenheuer et al. (1980) note that damage to seeded areas on tailings impoundments by rodents may also inhibit seedling establishment.

Species Selection

An ongoing debate among some reclamation specialists involves the selection of species used in revegetation, and in particular whether native or introduced plant species should be used. Issues related to productive use are involved in this debate; for example, revegetation with natural species may provide cover for wildlife habitat, whereas introduced species may be better suited for eventual forage and grazing. To understand some of the issues involved in species selection, it is necessary to explore the concept of plant community succession, a process that occurs in nature and that will also come about during impoundment revegetation.

When an area has been devastated—for example, by fire, flood, avalanche, or tailings impoundment construction—a series of biota replacement occurs, which is termed *succession*. Pioneer species that initially invade the area will be those best suited to the new soil and microclimate conditions. After some period, these species will enhance soil development and change the microclimatic characteristics to the extent that other species better suited to the changed conditions will invade, overcome the pioneer community, and establish themselves. Natural upland succession ordinarily progresses through a series of more or less well-defined stages—for example, from annual grasses and forbs, to perennial herbaceous species, to shrubs, to shade-intolerant trees, to shade-tolerant trees. The process may take from a few years to a few centuries. Not only does community succession apply to natural areas, but studies of mine waste disposal areas up to 50 yr old provide evidence that definite successional trends also occur on revegetated areas (Harthill and McKell, 1979).

Species diversity is an important consideration in plant community succession. Pioneer communities usually consist of a limited number of species, those few that are suited to the relatively harsh initial conditions. As soil and microclimate conditions are modified by successive generations of plant growth, additional species become established that increase the diversity of the community. Eventually a *climax* community becomes established, one with a wide diversity of species that enables it to withstand attack by insects, disease, drought, or other normal environmental perturbations. The climax community exists in a long-term dynamic equilibrium with the environment; the species mix may change from time to time in response to environmental changes, but through its diversity the climax community has the ability to eventually reestablish itself in more or less its initial makeup.

Applying these concepts to tailings impoundment reclamation, it can be seen that establishment of a stable and self-perpetuating climax community is the ultimate goal of revegetation. The most common means of achieving this end is to use introduced species for initial revegetation. Ordinarily, a variety of introduced species is available, allowing the selection of those plants that can quickly stabilize the surface by shallow soil-holding root systems, rapid growth rates, and high seed production. The use of introduced species also offers the opportunity for use of special salt-resistant or metal-resistant varieties, an important and sometimes crucial factor in attempts to revegetate the surface of toxic tailings directly without topsoiling (Bradshaw et al., 1978). The use of introduced species is often preferred because of flexibility in selecting those plant varieties that have characteristics compatible with initial impoundment soil and microclimate conditions.

The alternative is to use native species for revegetation. Johnson and

Putwain (1981) discuss advantages and disadvantages of native versus introduced species. They provide several case histories of the use of seminatural revegetation on iron, bauxite, manganese, nickel, copper, and other types of tailings demonstrating that revegetation with natural species, while often costly and difficult, can be successful in establishing a self-perpetuating cover. If successfully carried out, revegetation using native species may speed the establishment of the climax community and shorten the time required to return the disturbed area to its former appearance and land use. In dry climates, low water requirements of some native plants may make initial irrigation unnecessary or at least less critical. Partly for these reasons, revegetation with native species is being increasingly required by regulations.

However, revegetation with native species is often difficult. Attempts to revegetate with native species that comprise the climax community on adjacent lands may be a futile attempt to short-circuit the natural and necessary processes of plant community succession. The initial microenvironment and soil conditions on the impoundment surface will usually be quite different from those on adjacent undisturbed lands. In fact, the impoundment environment may never be the same, giving little reason to believe that the climax community on the reclaimed area should necessarily consist of the same native species that existed prior to its disturbance.

Many native species may not be well suited to the critical need for preventing wind and water erosion during the important initial stages of vegetation establishment; if this erosion is not quickly checked, it may be impossible for conditions conducive to plant growth to develop later. For example, native climax species often have deep, water-seeking root systems, low seed production, and slow growth rates, whereas species best suited to initial stabilization should ideally have shallow, spreading roots, rapid growth, and high seed production.

Establishing initial cover with native species is often complicated by soil profile requirements and seed availability. In using native species, it is usually mandatory that topsoil be used, and it is often necessary to strip, stockpile, and replace meticulously and separately the various topsoil horizons. In addition, adequate seed sources for native plants are frequently unavailable, requiring that seed plants or seedlings be raised in greenhouses or cultivated plots, sometimes many years in advance of reclamation for slow-growing plants.

Generally, the use of native species for initial revegetation is possible in many cases, but it may be difficult, costly and risky. Cases where native species are suitable are most often those where topsoiling is feasible and where it has been determined that reclamation objectives related to appearance or wildlife habitat outweigh short-term erosion protection and economic considerations.

Regardless of whether native or introduced species are used, it is important in species selection that site factors and species characteristics be compatible. Those species chosen for revegetation should be matched to the soil characteristics, erosion potential, and microclimate of the tailings impoundment. Species characteristics that govern suitability to site conditions include ease of propagation, required planting methods and schedule, root development and depth, and the potential for competition between species in the mix (Schiechtl, 1980). Species adaptable to a broad range of conditions should be preferred to those that can survive only under limited circumstances, in order to minimize the risk of mismatching species characteristics and site conditions. Table 12.1 summarizes some of the species used for short-term revegetation in moderate climates and illustrates attributes that should be considered in species selection.

In summary, the goal of revegetation should be to arrive ultimately at a stable climax community, which may or may not be the same as that on adjacent undisturbed land. In some circumstances, and through unusual care and expense, it may be possible to establish a near climax community of native species. The most common way of arriving at this stable community, however, is to start with a rapid-growing pioneer community of introduced species adapted to the specific soil, microclimate, and moisture conditions of the impoundment—a community that later gives way to a more diverse and adaptable climax community through natural successional trends.

Revegetation Sequence

Typically, revegetation efforts follow a series of steps according to the principles and techniques discussed above. While the specific procedures are unique to each tailings impoundment and climatic regime, the following are usually representative elements of the process:

1. *Seedbed Preparation.* Seedbed preparation is necessary to set the stage for establishment of the short-term community. Initial operations may include grading, furrowing, or gouging to enhance microclimate. If direct revegetation of the tailings is being attempted, seedbed preparation may also require liming, mulching, and fertilizing. Alternatively, placement of a suitably thick layer of topsoil over the impoundment surface may form a favorable seedbed.
2. *Short-Term Revegetation.* It is common practice in both humid and dry environments to rely largely on grasses for the primary initial source of short-term ground cover, although legumes (for example, alfalfa) are sometimes favored because of their nitrogen-fixing ability (Day and Ludeke, 1981). Usually several species are included in the initial seeding mixture to increase diversity and reduce the chance of total community failure. Because seeding requirements may be different for those species within the mix,

species selection, seeding density, and seeding date must be determined from a thorough knowledge of local site conditions and the individual species (Ripley et al., 1978).

Seeding practices include drill sowing, broadcasting, or transplanting seedlings if shrubs are used. There is usually a seeding practice best suited to each species, and use of multiple species often requires compromise methods. Hydroseeding is a useful technique that applies seed, fertilizer, and surface mulch rapidly and uniformly in a single operation and is especially useful in soft areas or slopes with difficult access.

In arid climates, temporary irrigation may be necessary to ensure rapid establishment of the short-term vegetation cover. Irrigation may be by spray or drip systems, depending primarily on the species selected.

3. *Long-Term Vegetation.* To achieve the ultimate goal of attaining a self-sustaining and stable community, a transition between short-term and long-term vegetation must occur. In some cases, this may be left to invasion by native species after short-term vegetation is assured and soil development is well under way. In other cases—for example, when irrigation has been used temporarily to establish the short-term community—it may be necessary or desirable to enhance the natural succession process by replanting with a more diverse mix of species suited to the next stage of community succession, such as shrubs. The need for artificial enhancement of the successional process will depend on the success of previous short-term efforts and on the ultimate intended land use of the reclaimed area. All revegetation efforts, however, should work toward self-regeneration and minimum management in the long term.

SUMMARY

The basic objectives of tailings impoundment reclamation are to achieve long-term mass stability, long-term erosion stability, prevention of environmental contamination, and return of the impoundment area to productive use. The emphases placed on these goals will differ according to the nature of the tailings, site conditions, and regulatory–environmental objectives. However, all four factors must be addressed in reclamation plans, and this is best done during initial impoundment planning. If reclamation is integrated into impoundment planning and design from the start, ultimate costs can be minimized by optimizing impoundment layout, embankment sideslopes, hydrologic design measures, and other factors with both short-term operational criteria and long-term reclamation objectives in mind.

Table 12.1 Factors in Selection of Short-Term Revegetation Species (after Foote et al., 1970)

Parameter	Classification	Examples
Texture	Fine	Kentucky bluegrass, bent grass, red fescue
	Coarse	Smooth bromegrass, reed canary grass, timothy
Growth, height	Short	Buffalograss, Kentucky bluegrass, red fescue
	Medium	Redtop, perennial rye grass
	Tall	Smooth bromegrass, timothy, switchgrass
Growth, turf	Bunch	Timothy, big bluestem, sand dropseed, perennial ryegrass
	Sod former	Quackgrass, smooth bromegrass, Kentucky bluegrass, switchgrass
Origin	Introduced	
	Native	
Way of spread	Seed	Red clover, alsike, sand dropseed, rye, perennial ryegrass, field bromegrass
	Vegetatively	Prairie cordgrass, some bentgrasses
	Both	White clover, crownvetch, quackgrass, Kentucky bluegrass, smooth bromegrass

Life	Annual	Summer	Corn, sorghum, rabbit clover, oats, soybeans
		Winter	Rye, hairy vetch, field bromegrass
	Biennial		Sweetclover
	Perennial	Long-lived	Birdsfoot trefoil, crownvetch, Kentucky bluegrass, smooth bromegrass
		Short-lived	Timothy, perennial ryegrass, red clover, white clover
Establishment	Hard		Big bluestem, crownvetch
	Medium		Perennial ryegrass, smooth bromegrass
	Easy		Rye, oats
Maintenance	Hard		Tall fescue, reed canarygrass
	Medium		Kentucky bluegrass, smooth bromegrass
	Easy		Crownvetch, white clover
To maintain stand	Hard		Timothy, alfalfa, perennial ryegrass
	Medium		Kentucky bluegrass, white clover
	Easy		Birdsfoot trefoil, crownvetch, big bluestem, switchgrass, reed canary grass
Root system	Shallow	Strong	Timothy, Kentucky bluegrass
		Weak	Sand dropseed, crabgrass, foxtail, white clover
	Deep	Strong	Alfalfa, big bluestem, switchgrass, reed canarygrass
		Weak	Many weeds

Long-term mass stability usually requires primarily consideration of long-term hydrologic factors; embankment slope stability and seismic stability ordinarily are sufficiently enhanced by the postabandonment drop in phreatic levels to be of little long-term concern. In cases involving long-term potential for environmental contamination by tailings oxidation or radon emanation, covering the entire impoundment with low-permeability soil may be mandatory.

Usually, the factor of primary concern in reclamation is achieving long-term wind and water erosion stability. Three basic methods are available for erosion protection: riprap, chemical stabilization, and vegetative stabilization. These methods may be used in combination, either in different areas of the impoundment or together to supplement each other. Of these methods, however, vegetative stabilization is the most common, with the ultimate goal of achieving a self-sustaining community that will protect the impoundment from erosion while returning it to some form of productive use.

Revegetation efforts must account for regional climate, growth medium characteristics, and environmental factors. Because all these elements are unique to each tailings deposit and because they also change over time, it is not surprising that revegetation is sometimes a trial-and-error procedure. However, as noted by Wali (1975), a systematic approach may minimize time, cost, and failures in reclamation efforts. Such an approach requires a detailed knowledge of factors that will limit or constrain vegetation establishment, plans for mitigative measures to correct deficiencies, and understanding of short-term and long-term successional processes.

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