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Final Report

EVALUATION OF LONG-TERM STABILITY OF URANIUM MILL TAILING DISPOSAL ALTERNATIVES

by

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Thomas A. Shepherd

for
Argonne National Laboratories

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Prepared For

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ABSTRACT

Potential failure mechanisms which could result in a release of radiation from uranium tailings impoundments were studied. Modes considered were elemental failure and failure due to natural phenomena. Elemental failure was considered to result from failure of an element of the tailings impoundment such as the cap, liner, embankment, revegetation, or water diversion structures. Natural failure modes studied included earthquake, flood, wind, tornado, glaciation, and fire. The effects of these failure mechanisms were considered for short (a few hundred years), medium (a few thousand years), and long (up to 100,000 years) long-term periods. A methodology is presented which can be used to quantitatively compare site alternatives for potential release of radioactivity during each time frame due to the effects of the particular failure mechanisms. The methodology was applied to various uranium tailing disposal plan alternatives as defined by Argonne National Laboratories. Several uranium tailings sites were visited during the investigation and are discussed in relation to the potential failure mechanisms.

PREFACE

This report is the result of a concentrated team effort. The nature of the task required large quantities of literature to be located, assimilated and applied to the problem of long-term stability of uranium tailings. A diversity of disciplines was organized and brought to bear in the analysis.

The research team was organized in the Geotechnical Engineering Program of the Civil Engineering Department. The principal investigators were John D. Nelson and Thomas A. Shepherd. They were assisted throughout the research by the members of the research team, Wayne A. Charlie, Mohammed Hamid and John D. Welsh. A committee of consultants was formed consisting of S. A. Schumm, Earth Resources Department; R. D. Heil, Agronomy Department; H. G. Olson, Mechanical Engineering Department (Nuclear Engineering); J. E. Johnson and T. E. Borak, Radiation Biology Department. This committee provided valuable assistance and direction within their particular disciplines. They reviewed the draft report and their comments and opinions have been taken into account in this final report.

Appreciation is also expressed to members of the Hydrology Program, the Hydraulics Program, the Atmospheric Sciences Department, and the Agronomy Department at Colorado State University who provided valuable advice on various aspects of the project.

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I. INTRODUCTION

This report has been prepared to provide insight and information regarding the potential long term stability aspects of uranium mill tailings disposal. The central focus of the study has been to identify and describe the potential failure modes which, over long time periods, could cause release of radioactive components of the tailings. The analysis of these potential failure mechanisms includes a description of the failure mechanism itself, a discussion of the natural or geotechnical processes that control it, an assessment of the magnitude of release that could result from a failure and the likelihood that the failure would occur within long time periods. The time periods considered range from a few hundred years up to 100,000 years.

An integral part of the analysis is the evaluation of site and design characteristics that could influence the magnitude and likelihood of failure for each mechanism. Monitoring, maintenance and remedial measures that may be appropriate are discussed. Of particular importance was the identification of predictive models, quantitative or not, that are available and could be used to evaluate actual likelihood of occurrence or magnitude of release for a failure mode. Throughout the investigation, quantification was attempted. When this was impossible semi-quantitative measures were applied to describe the likelihoods and magnitudes of failure.

For each failure mode identified, an analysis was made for three time periods. The first period included about 100 or 200 years after abandonment, the second up to a few thousand years after abandonment, and the third extended for about 100,000 years after abandonment. The likelihood and magnitude of failure was investigated for each period.

A methodology was also developed which was used to evaluate the performance of various disposal plan alternatives. This evaluation is based on the long-term response of the potential failure modes to the specific design, site and geographic location of the plan. To illustrate the use of the methodology and the information developed for long term failures, the methodology was applied to the base case and alternatives which were defined by Argonne National Laboratories.

In the analysis of the potential failure modes the design, site, and natural processes that control performance were identified and analyzed. The interaction between failure modes was also addressed.

The task undertaken was large and complex. Within a very limited amount of time large quantities of information had to be analyzed, integrated, and brought to bear on the problem of long term stability. In the time available, information from published sources and personal experience only could be used. Many references were compiled but the list is not complete. A detailed and critical review of all available literature was not possible. Decisions had to be made continuously about appropriate levels of investigation. The authors generally chose in favor of briefer treatment of a wider base of information. Therefore, in many aspects, the state-of-the-art may not be fully described. However, a useful basis for evaluation has been provided.

It was not possible in many cases to fully reference the sources of information or research. Also, some areas have not been fully covered. In those cases personal judgment and experience of the project team were called upon. In many cases collective subject judgments by the research team was employed.

It is believed that the information and analysis presented provides a significant advance in the determination of long-term performance of uranium mill tailings. By no means does it complete the work necessary. It does suggest areas where research and data is deficient, as well as the areas that are most in need of further research.

II. GENERAL CONSIDERATIONS

A. GEOMORPHOLOGY

When considering the safe and stable storage of uranium mill tailings for long periods of time (100,000 years), serious consideration must be given to the natural processes, specifically geologic and climatic, that have created the present day landscapes. The most fundamental concept of geology is the principle of uniformitarianism or uniformity which can be stated simply as "the present is the key to the past." This means that an understanding of present geological processes and the laws of physics and chemistry provide a basis for interpretation and understanding of the history of the earth. Viewed in another way, this concept can be restated as follows: "The past is a guide to the future." Therefore, the history of the last 100,000 years should indicate what can be expected during the next 100,000 years. The predictions made in this way may not be correct, but they do provide a basis from which plans for long-term storage of materials can be evaluated.

The event that dominates the history of the last 100,000 years is obviously the advance and retreat of the Wisconsin age continental ice sheet. During this time, much of the U.S. north of the Ohio River and north and east of the Missouri River was significantly affected by ice erosion and deposition. Elsewhere alpine glaciers were active, and clear evidence of glacial modification of the landscape in the western mountains can be found as far south as the San Francisco Peaks, Flagstaff, Arizona.

The direct effects of glacial ice was great, but even more important was the global changes of climate that brought about the ice ages. Therefore, significant changes of climate during the past

100,000 years have drastically changed the hydrologic cycle and the erosional and depositional processes acting on the landforms. For example, large pluvial lakes occupied the closed basins of Utah, Nevada and southern California. Throughout the world, river activity changed and reflected the altered runoff and sediment regime of the drainage basins.

Furthermore, the vast quantities of water stored in the ice sheets caused a 300 to 400 foot fall of sea level which exposed the continental shelves and caused the major rivers to cut deeply into sediment and bedrock.

During the past 100,000 years volcanic activity and faulting has had significant if local effects. For example, Sunset Crater near Flagstaff erupted in 1067. Recent fault scarps in easily erodible sands and gravels in Nevada, California and Utah indicate continued mountain building activity and the annual one inch per year migration of western California to the north along the San Andreas Fault is clear evidence of the instability of the earth's surface.

Precise releveling by the Coast and Geodetic Survey has revealed significant changes of the surface of the U.S. during the past few decades. Attempts to estimate rates of mountain building and rates of denudation have led to the conclusion that uplift can occur at an average rate of 25 feet per 1,000 years, and order of magnitude greater than denudation rates, which nevertheless can be as much as 3 feet per 1,000 years. Using these rates and assuming continued uplift and denudation uplift could total 2,500 feet and denudation 300 feet during 100,000 years. These are average values, and the rates could be expected to be much greater locally. In fact, uplift on the order of 900 feet has occurred in the Hudson Bay region during the last 10,000 years, as a result of the melting of the ice sheet and subsequent response of the earth's crust to the release of the tremendous load of ice.

The above brief and very general summary emphasizes the difficulty in predicting the response of tailings impoundments on or near the earth's surface for 100,000 years. It is unwise to extrapolate measured rates of denudation or uplift for 100,000 years, but it is possible to conclude that the last 100,000 years of earth history have been eventful and there is little reason to expect the next 100,000 to be less so.

To recapitulate, the past 100,000 years have been a period of: 1) major climatic change with associated changes in erosion rates and processes, vegetation density and type, and major extinctions of Pleistocene fauna and the formation of large lakes in presently arid areas; 2) major glacial modification of the northern part of the continents and the western mountains; 3) major sea level fluctuations with accompanying river incision and deposition; 4) continuing displacement of the earth's surface by faulting and isostatic adjustment to the addition and removal of ice loads.

If the past is indeed a guide to the future, it appears that long-term erosional stability cannot be assumed. Even where glacial activity and faulting are improbable, climate change and the resulting change in river behavior as well as change in the rates and mechanics of hillslope erosion prevent secure storage of earth materials. For example, an initially secure surface storage site can be rendered insecure by either an increase of precipitation which will increase mass wasting (sliding, slumping) or a decrease of precipitation which will cause reduction of vegetative cover and increase surface erosion by raindrop impact, overland flow and rilling.

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B. CLIMATE

Climate is one of the very important driving forces and determinants of the rate and directions of the geomorphic process. Predictions about future climates are integral to the evaluation potential of long term stability of structures placed on the earth's surface. One scenario of future climates, advanced by Calder (1978) and based in large part on the Milankovitch theory (Calder, 1978; Lamb, 1972), has the earth heating up in the next few hundred years. This heating, due to the "greenhouse" effect will end after a relatively short period and the world will proceed towards a new ice age. Leet and Judson (1971) do not take a position about the direction of climatic change. They do concur that climates are changing and emphasize the profound effect this change, regardless of direction, will have on man's long-range future.

Thus, climate will have a pronounced influence on the geomorphological processes. Although it is fairly certain that climatic changes will occur within the long time periods considered herein the direction that they will take is not predictable. This emphasizes the uncertain nature of evaluations of long-term stability of tailings impoundments.

C. RADIOACTIVITY FROM MILL TAILINGS

Uranium mill tailings contain only about [illegible]% of the Uranium-238 (and 235) that was present in the original ore. However Thorium-230 still maintains a decay chain of radionuclies from ^{226}Ra through ^{210}Po. These daughter radionuclides are gamma-ray emitters. Schiager (1970) estimates that the exposure rate above a tailings pile may be estimated as

$$\dot{X}(\text{Exposure Rate in } \frac{\mu R}{hr}) \cong 2.5(C_{Ra})\text{pCi/g}$$

where C_{Ra} is the ^{226}Ra concentration in the tailings.

The first decay product (daughter) of ^{226}Ra is ^{222}Rn, an inert gas which diffuses out of the tailings. The ^{222}Rn decays to other daughter products and their activity in air above and downwind of the pile is the chief contributor to lung radiation dose. Inhalation of ^{222}Rn daughter products is considered to be the critical radiological hazard from the entire uranium mining and milling process. The ^{222}Rn flux may be estimated directly as well from the ^{226}Ra concentration in the tailings. Covering the pile with approximately 2 feet of earth will reduce the gamma-ray flux to essentially background levels but will only reduce the radon flux by 25%. Radon-222 daughter concentrations downwind of the pile may be calculated by standard atmospheric dispersion methods.

Radiation dose may also be produced from radionuclides dispersed from the pile by wind erosion, runoff due to surface water and leaching into ground waters. In the latter cases radiation dose is calculated by the movement of the principal radionuclide through terrestrial and aquatic food chains.

Sears et al. (1975) present models for the rate of seepage into surface and ground waters given appropriate hydrology parameters. They also present radiation doses from model uranium mills both during operation and after decommissioning.

III. POTENTIAL FAILURE MODES

Long term failure of a tailings impoundment can occur as a result of the long term behavior or failure of some element of the impoundment; or it may occur as the result of a natural phenomenon that either was not anticipated in the design or that is of a magnitude larger than for which the impoundment was designed. An example of the former would be failure of the cap due to various factors. An example of the latter would be the occurrence of an earthquake of magnitude 8 whereas the impoundment may have only been designed on considerations of an earthquake of magnitude 6.

The likelihood of failure and consequent severity of failure of these two categories of failure would be considered somewhat differently. Consequently, the potential modes of failure to be considered herein have been categorized into elemental failures and failures due to natural phenomena. The list of failure modes/mechanisms that were considered in this investigation is shown in Table 1. This list has been compiled on the basis of observations, experience of the investigators, and discussion with personnel from Argonne National Laboratories and the Nuclear Regulatory Commission.

The discussion of these potential failure modes in the following section considers also the magnitude of release of radionuclides and the decrease in attenuation of gamma-ray emission rate. Although the release of radionuclides is a different mechanism than the attenuation of gamma-ray emission, the general term "magnitude of release" will be used in subsequent discussion to refer to either one or both mechanisms unless it is necessary at that point to differentiate between them.

TABLE 1. FAILURE MODES TO BE CONSIDERED

A. ELEMENTAL

1. CAP

a) Differential settlement
b) Gullying
c) Water sheet erosion
d) Wind erosion
e) Flooding
f) Chemical attack
g) Shrinkage

2. LINERS

a) Differential settlement
b) Subsidence of subsoil and rock
c) Chemical attack
d) Physical penetration

3. EMBANKMENT

a) Differential settlement
b) Slope failure
c) Gullying
d) Water sheet erosion
e) Wind erosion
f) Flooding
g) Weathering and chemical attack

4. REVEGETATION

a) Fire
b) Climatic change

5. WATER DIVERSION STRUCTURES

a) Slope failure
b) Obstruction

B. NATURAL PHENOMENA

1. Earthquakes
2. Floods
3. Windstorms
4. Tornadoes
5. Glaciation
6. Fire and Pestilence

IV. DISCUSSION OF FAILURE MODES

A. POTENTIAL FAILURE OF IMPOUNDMENT ELEMENTS

1. CAP

The cap on a tailings impoundment is provided primarily for the purpose of reducing radon emanation from the tailings. The cap may consist of a single material such as overburden, topsoil or native soil mixed with an additive such as cement or clay; or it may consist of an initial liner of relatively impermeable material such as clay or asphalt which is then covered with rock or overburden to provide stabilization. Increased release or radioactivity or gamma-ray emission may result from failure of either or both elements. If both a liner and cover exist, the interaction between the two must be taken into account in assessing the severity of a failure.

a. DIFFERENTIAL SETTLEMENT OF UNDERLYING MATERIAL

i. Causes and description

If differential settlement of either the foundation material or the tailings occurs, displacements may be induced in the liner that would cause failure. If the foundation soils beneath the impoundment are irregular either in thickness or compressibility the differential settlement may be relatively large and may occur across a short distance. In this case a shear type failure such as shown in Fig. 1 would result and failure would be localized.

Alternatively, differential settlements may occur across large distances (i.e., several hundred feet). Settlement may be due to compression of both the foundation material and the tailings themselves. Differential settlement across a valley is to be expected because the alluvium and tailings would be deeper near the center of the valley than toward the outer edges.

When settlement is considered in relation to its effect on the cap, compression that has occurred prior to placement of the cap is obviously of little concern. During construction of the impoundment the coarse tailings placed near the embankment would compress almost immediately. Consolidation of that material would, however, be nearly completed before placement of the cap.

The fine tailings in the slimes zone, however, are much less permeable and would continue to consolidate after placement of the cap. Consequently, differential settlement between the outer edges of the impoundment and the slimes zone would be expected.

Differential settlement due to compression of the tailings and fairly uniform alluvium would occur fairly continuously across the width of the impoundment. The result would be a general cracking of the material due to tensile stresses as shown in Fig. 1b. The degree of cracking would be expected to be greater near the slimes zones because of the larger settlement there. That may be important because the radium concentration in the slimes would be expected to be greater than in the relatively coarser tailings sand (IAEA, 1976; Borrowman and Brooks, 1975). The resulting radon flux from the slimes may be more than an order of magnitude greater than from the sand (Dames & Moore, 1977).

Differential settlement would be a continuously occurring process. The greatest amount of settlement would be expected to be completed within a period of time that could range from only a few years (5 or 10 yrs) up to 100 or 200 years. In mountain areas the alluvium would be expected to be somewhat permeable (10^{-3} to 10^{-4} cm/sec.) allowing consolidation to occur fairly rapidly. The time required for the tailings

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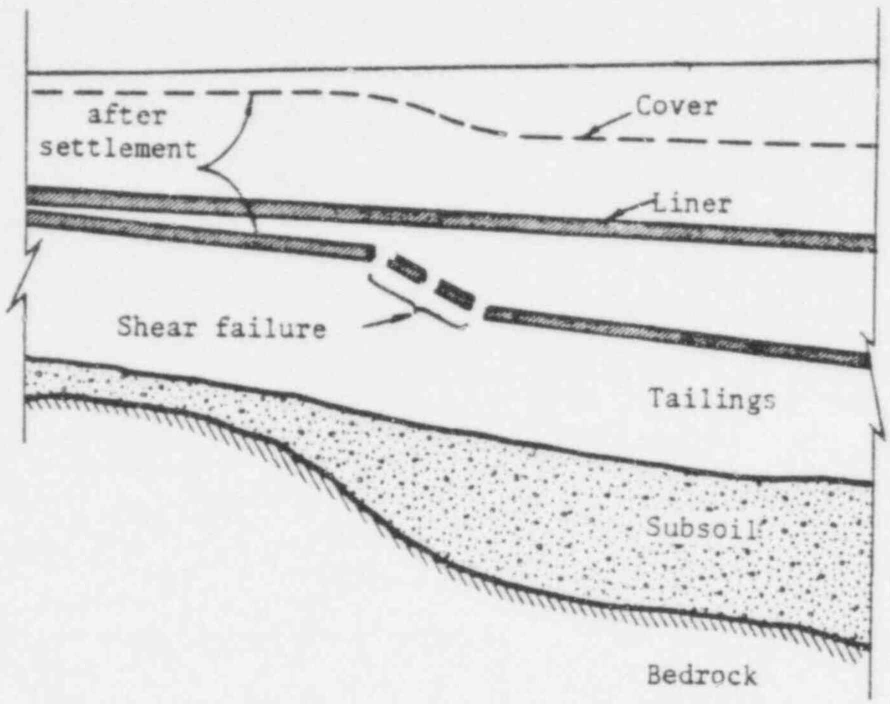
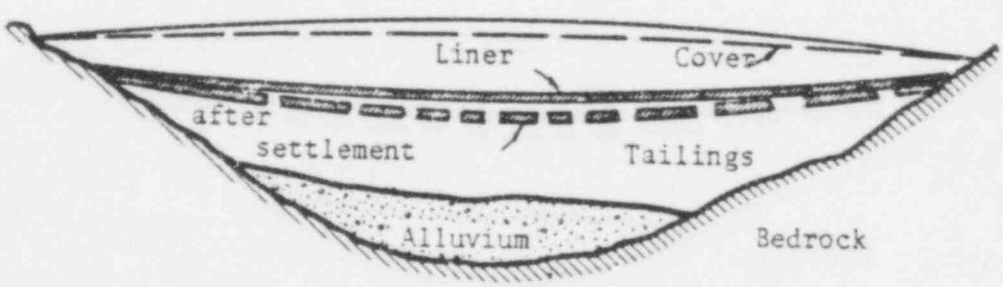


(a) Shear Displacement



(b) Tension Failure

Fig. 1. Failure of cap due to differential settlement.

to consolidate will depend on the rate at which pore pressures can dissipate through both the underlying and overlying liners and caps. A period of 100 to 200 years, however, is a relatively long time for consolidation to occur in a natural soil of the general dimensions and permeability to be expected in a tailings impoundment.

Other factors that may contribute to differential settlement would be the existence of collapsing soils or subterranean features that may contribute to localized subsidence. These factors are discussed in more detail with regard to the liner under the tailings.

Differential settlement due to those factors would probably occur rapidly and could occur at any time after abandonment. For example, slowly occurring compression of some foundation soils may increase seepage rates that may in turn introduce more water into an area containing collapsing soils. The collapse of the soils may then cause greater seepage rates that may further increase the differential settlement and area of influence.

Differential movement may also result from upward movement of the foundation soil due to swelling of expansive clays or clayshales. The net effect of the cap due to heave would be similar to that of compression.

ii. Interaction with other failure mechanisms

Failure of the underlying liner could accelerate the rate of consolidation of tailings but would not influence the magnitude of compression of tailings to be expected. If collapsing soils or soluble rocks (e.g., limestone) exist below the tailings, failure of the liner could also contribute to subsidence and differential movement of the cap. Other than those factors it is not expected that other potential failure mechanisms would contribute to differential settlement.

Differential settlement, however, can change the drainage characteristics over the impoundment and may contribute to either water or wind erosion. Also shrinkage cracks may interact with cracks induced by settlement as discussed in Example 1.

iii. Methods of prediction

The amount and rate of differential settlement to be expected can be analyzed on the basis of one-dimensional consolidation theory. Methods of analysis are well known in the field of soil mechanics and are discussed in most textbooks on soil mechanics (e.g., Lambe and Whitman, 1969; Terzaghi and Peck, 1967; Wu, 1976; Sowers, and Sowers, 1970, etc.).

More sophisticated methods of analysis utilizing two-dimensional consolidation theory and testing methods that duplicate the stress path followed in loading the foundation soils are also discussed in the literature (Lambe, 1964; Schiffman, Chen & Jordan, 1967).

Secondary consolidation and creep should also be considered in assessing the potential for long term differential settlement. Secondary consolidation is normally not of great magnitude in sandy soils or soils of low plasticity such as are frequently encountered in mountain alluvium. However, a significant amount of secondary consolidation has been observed in some tailings and has been attributed to the very angular nature of tailings from hard rock ore (Nelson, Shepherd and Charlie, 1977). In areas where the foundation material has a high plasticity such as clay or clayshale, creep may contribute to differential movements over a long period of time (Nelson and Thompson, 1977; Crawford, 1965).

The amounts of secondary consolidation can be predicted on the basis of laboratory consolidation tests (Lambe and Whitman, 1969). Creep measurements, however, require long time periods and the prediction of creep displacements, particularly where shear deformations enter in, require more sophisticated methods of analysis (Nelson and Thompson, 1977). Within the present state of the art, however, sophisticated methods of analysis are probably not warranted and it is doubtful that much accuracy could be obtained for prediction of displacements over a period of time much in excess of about 100 or 200 years.

iv. Likelihood

The likelihood of differential settlement occurring is great. However, the likelihood of differential settlements being of a magnitude that will allow the release of radioactivity will be very site specific and will depend to a large extent on the foundation soils and the nature of the tailings. During the design phase of an impoundment, predictions will be made of the amounts of settlement that will occur and the liner will undoubtedly be designed to accommodate relative displacements of the amounts anticipated. Failure would therefore consist of the occurrence of differential settlement of an amount greater than that predicted. The likelihood of failure will be influenced by the factor of safety that is employed in the design, the variability encountered in the foundation soils and tailings, and the confidence level of the analyses employed.

v. Magnitude of release

The potential magnitude of release due to failure of a cap as a result of differential settlement is very site specific and will be influenced greatly by the interaction between the cap liner and the cover. The magnitude of release will depend on the extent to which cracks can remain open for the entire thickness of the cap. Embankment dams constructed of brittle cohesive material of low plasticity have been observed to be particularly susceptible to cracking and can support cracks several feet deep (Sherard, 1973). On the other hand, cohesionless sandy soils will not support open cracks.

. Cracks in pavements have been observed to provide little resistance to radon emanation (private communication with H. G. Olson). It may, therefore, be assumed that relative to the tailings or cap material the crack would provide negligible resistance to release of the radon to the atmosphere. Thus, if a crack develops in the cap radon would be released from tailings on either side of the crack for a distance of the magnitude of the relaxation length. Sears (1975) gives the half thickness of sand with a low water content as being about 4 feet for radon. Thus, for purposes of comparison it will be assumed in subsequent discussion that if a crack develops, the release through the crack would be equivalent to the full release of radon from tailings extending to a distance of about 4 feet on either side of the crack. The actual equivalent distance will depend on the grain size and water content of the tailings and may vary somewhat from the assumed value of 4 feet depending on the particular tailings being considered.

The factor governing the release of radon is, therefore, not the width of the cracks but the spacing of cracks. If a shear type displacement occurs (Fig. 1a), the cracks would tend to be concentrated more in that area and the area of release would be the general area of distress plus a distance to each side of about 4 feet.

If the differential settlement is fairly uniformly distributed (Fig. 1b), the cracks would tend to be fairly uniformly spaced. Example 1 derives a method of analysis and provides some general computations on which to draw conclusions. In Example 1 it is shown that cracking due only to differential settlement would cause a minimal release of radon. It is also shown however that cracking caused by shrinkage must be considered together with cracking due to differential settlement and in the case of shrinkage significant release of radon may occur. That will be discussed more fully in a later section.

EXAMPLE 1--Computation of Magnitude of Release due to Cracking of Cap

For purposes of this example it has been assumed that the impoundment will be placed in a valley having the general cross section shown in Fig. 2a. The settlement at the center of the impoundment due to consolidation of the alluvium and the tailings is ρ. At the edge of the impoundment the settlement would be zero. The difference in surface elevation between the center of the impoundment and the edge is designated by a. The initial length of a cap liner across the impoundment can be obtained by integration of an equation representing its contour. After some settlement of the amount ρ has taken place, a new equation describing its shape can be written and integrated to determine the length that the liner would require. The difference between the final length and initial length would be taken by tensile strain and the development of cracks. If shrinkage of the cap occurs due to dessication the shrinkage would contribute to a shortening of the liner. Shrinkage will be discussed in a later section but is included here for completeness.

In order to compute the initial and final lengths of a typical cap it was assumed that the surface was parabolic of the form $y = a\left(\frac{x}{L}\right)^2$ *at*

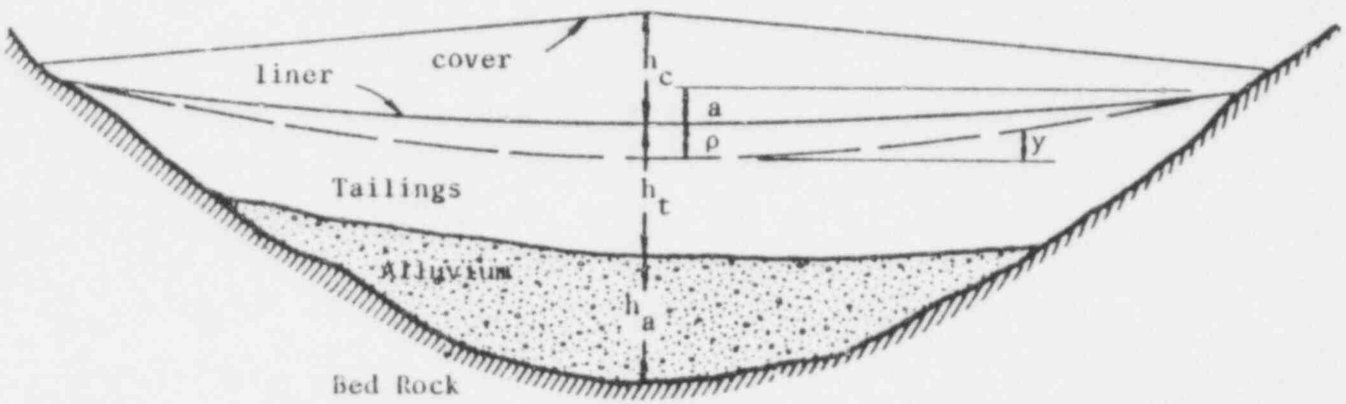


Fig. 2a. Cross section assumed for Example 1.

the time the liner was placed and after settlement it was parabolic of the form $y = (a+\rho)\left(\frac{x}{\ell}\right)^2$ where:

x = the distance from the center of the impoundment,

2ℓ = the width of the impoundment, and

y = the difference in elevation between the lowest point at the center and the surface.

The length of the liner, s_o, at the time of placement is

$$s_o = \int_{-\ell}^{+\ell} \left[\frac{4a^2x^2}{\ell^4} + 1\right]^{1/2} dx \qquad (1\text{-}1)$$

and after settlement

$$s_f = \int_{-\ell}^{+\ell} \left[\frac{4(a+\rho)^2x^2}{\ell^4} + 1\right]^{1/2} dx \qquad (1\text{-}2)$$

To demonstrate the general magnitudes of quantities to be expected s_o and s_f were computed for a value of "a" of 10 feet in a distance "ℓ" of 1000 feet and for a settlement ρ of 10 feet. Using these values s_o was computed to be 2000.14 feet and s_f was 2000.54 feet.

The total width of cracks must be equal to

$$s_f - (s_o + \varepsilon_t s_o - \varepsilon_s s_o) = s_f[1 - \frac{s_o}{s_f}(1 + \varepsilon_t - \varepsilon_s)]$$

where ε_t is the tensile strain at failure of the cap and ε_s is the strain induced by shrinkage due to dessication. If the strain is uniformly distributed across the surface, the number of cracks, N, that would be formed would be

$$N = \frac{s_f\left[1 - \frac{s_o}{s_f}(1 + \varepsilon_t - \varepsilon_s)\right]}{w} \qquad (1\text{-}3)$$

where w is the average crack width.

The average spacing of cracks L_c *would then be*

$$L_c = \frac{s_f}{N + 1} \qquad (1\text{-}4)$$

$$L_c = \frac{w}{\left[1 - \frac{s_o}{s_f}(1 + \varepsilon_t - \varepsilon_s) + \frac{w}{s_f}\right]} \qquad (1\text{-}5)$$

In relation to the other terms $s_o/s_f \approx 1.0$ *and* $w/s_f \approx 0$. *Equation (1-5) then becomes,*

$$L_c = \frac{w}{(\varepsilon_s - \varepsilon_t)} \qquad (1\text{-}6)$$

L_c *is plotted as a function of* ε_s *in Fig. 2b for various values of* w *and* ε_t. *If* $\varepsilon_s \approx 0$, *Eq. (1-5) must be used to evaluate* L_c *because the magnitude of* s_o/s_f *becomes significant. Using Eq. (1-5) and values previously computed for* s_o *and* s_f *it can be shown that if* ε_t *is greater than about 0.00025 no crack would form in the liner for crack widths up to about 1.0 in. Even if* $\varepsilon_t = 0$ *the maximum spacing of cracks would be about 350 feet for those values of* s_o *and* s_f. *In that case if the area contributing to release of radon is about 5 feet on either side of the crack the magnitude would still be considerably less than 1.0.*

However, if shrinkage of the cap is also considered, the curves shown in Fig. 2b would apply. The spacing of cracks that would result in different magnitudes of release have been computed on the assumption that an area extending to 5.0 feet on either side of the crack would contribute to release of radon. Those values of L_c *have been labeled on Fig. 2b.*

From Fig. 2b it is evident that even for a fairly wide crack (1.0 in.) and a relatively large tensile failure strain (0.01) a shrinkage of about two percent would result in the maximum release of radon.

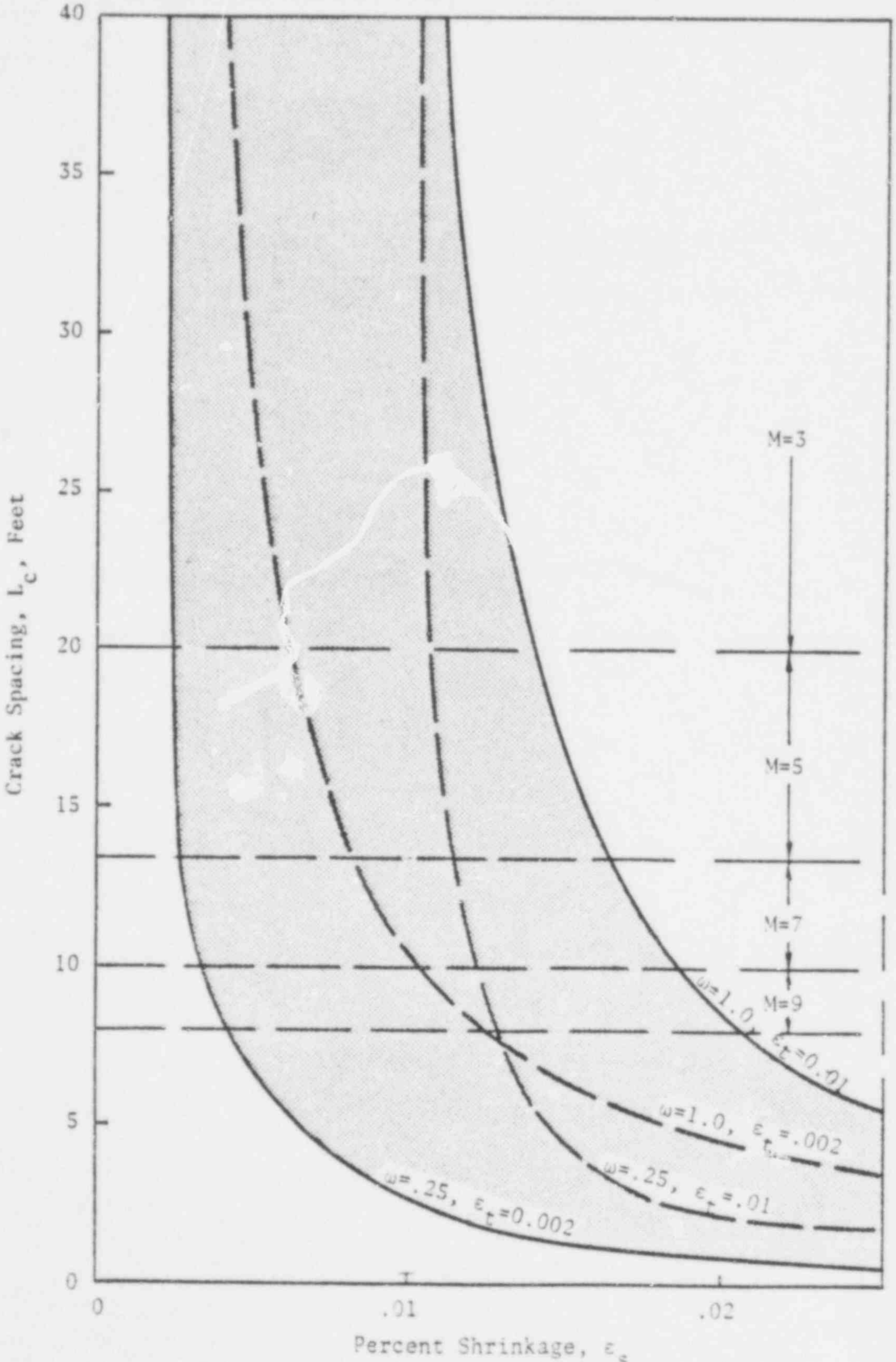


Fig. 2b. Crack spacing on cap.

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vi. Site considerations

Site considerations that will influence the differential settlement are primarily the subsurface soil and rock profiles. For an impoundment of relatively low height (approximately 100 ft) the nature of the soil lying over the bedrock will govern the settlement that will occur. For very high impoundments (several hundreds of feet) the compressibility of the bedrock may have to be considered as well. The worst situation would exist where the thickness of the subsoil and the tailings varies greatly across the site. If the subsoil profile is erratic, such as may be caused by old erosion channels in the bedrock, shear displacement of appreciable magnitude could result. Both the variability and the erratic nature of the subsoil must be considered.

If collapsing soils are found to exist in the subsoil, they should be treated as discussed below with regard to design. If water soluble rocks, such as limestone, or if underground cavities, such as in karst topography or from mining, are found to exist, consideration should be given to relocation of the site.

vii. Design considerations

If differential settlement is considered to be a problem, recommendations can be made in the design to either remove or stabilize the foundation soil such as by compaction or vibroflotation. Also, operational policies could be instituted to load potentially troublesome areas early in the project so that consolidation may occur before the cap is placed.

In situations where collapsing soils could result in large settlements due to wetting by seepage, the foundation area can be prewetted prior to construction of the embankment or deposition of tailings. This method has been utilized in construction of several water retention earth dams with success. At the Medicine Creek Dam in Nebraska prewetting was used to stabilize collapsing soils of depths in excess of 40 ft. (U.S. Bureau of Reclamation, 1974, pp. 250-254).

If design procedures cannot be instituted to decrease the expected differential settlement and if the severity in the particular case is large, the thickness of the cap and the nature of the material used must be adjusted so as to accommodate the degree of settlement expected. The design of the cap to account for differential settlement would consist primarily of using materials that are "self-healing" and that would be capable of withstanding differential movements. For example, the use of clay as the cap liner material and granular overburden for the cover would provide a material that could flow into discontinuities that were created by the relative displacements. Their thickness should also be of sufficient magnitude to accommodate differential shear displacements.

The use of clay soils for a liner component of the cap has the advantage of being relatively impermeable and therefore a smaller thickness would be required to minimize radon emanation. However, the more plastic clays that are the least permeable also exhibit the greater degree of shrinkage upon drying. From the results presented in Example 1 it appears that cohesive soils may be undesirable for use as a cap. Cohesionless soils that will not crack appear to be much more reliable.

viii. Monitoring, maintenance and remedial measures

Except for collapsing soils or subsidence due to underground cavities, differential settlement would be expected to take place over a relatively long period of time and would not occur suddenly. Monitoring schemes that would be applicable would consist of direct observation and aerial photography. If infrared photography is used at a site in conjunction with other failure modes, changes in drainage patterns and vegetation may provide indications of differential settlement. Periodic radon air concentration measurements may be used to detect cap failure.

Maintenance and remedial measures are not effective in reducing or eliminating differential settlement if it occurs. Remedial measures in the form of addition or reworking of cover material are possible on the cap to replace sections or repair discontinuities that may form. However, failure of the cap due to differential movement may manifest itself as another failure mechanism before it is identified. For example, drainage patterns may be changed due to differential movements resulting in increased erosion or gullying at points. Maintenance and remedial measures would then be those that are applicable for those mechanisms.

ix. Time dependence

Primary consolidation and hence the greatest differential settlement would be expected to occur within a time period ranging from only a few years (5 or 10 years) up to about 100 or 200 years. In some cases, where large deposits of soft, relatively impermeable foundation materials or where the tailings contain a significant amount of clay (e.g., Florida phosphates) the time required for consolidation may increase to several hundred years.

Secondary consolidation and creep, however, may continue for considerably long periods of time. Examples of this are seen in the Leaning Tower of Pisa that continued to tilt even after several hundred years. A large monument (Chedi) in Nakhorn Pathom, Thailand, is appearing to continue to settle approximately 400 years after its construction.

Evidence of movement of a slope over periods close to 1000 years have been observed (Crawford and Eden, 1967). Although those examples are somewhat different than tailing impoundments they indicate that deformations due to secondary consolidation and creep may occur for many years.

If deformation is due to creep of the foundation materials and potential shear instability of the foundation materials (e.g., bearing capacity or slope stability) accelerated movements may occur even thousands of years after construction of the impoundment (Crawford and Eden, 1967). The design, therefore, should provide for stresses low enough that accelerated creep would not occur (Nelson and Thompson, 1977).

In summary, failure due to differential settlement would be expected to be a short term phenomenon (hundreds of years) with potential for movements to continue close to the medium term (thousands of years). Over a period of 100,000 years it is expected that the displacements would have either slowed down to the point that they would be of little consequence or else failure would have occurred. More importantly, however, differential settlement is a subsurface phenomenon and the surface phenomena contributing to instability would far outweigh the effect of differential settlement on the long, long-term scale.

b. GULLYING

i. Causes and description

The causes of gully formation on the cap of a tailings impoundment would be a combination of steep slopes, the concentration of runoff, and an erosive soil. If gullies were to form, it is possible that relatively large quantities of cap material could be transported. If the gullies are deep enough tailings material could be transported also.

Gullies generally progress in a headward direction as a result of concentrations of runoff in particular areas. They begin at a location of localized erosion, generally at points of abrupt changes in slope. Because gullying is more severe in areas of steep slopes, it is unlikely that significant gullying of the cap would occur independently of serious gullying or instability of the retaining embankments. If gullies form directly on the cap without having been started in the embankment the effects would probably be local. Material eroded from one location in the cap would probably be redeposited within a short distance. Local thinning of the cap would occur but removal from the impoundment would be unlikely.

The release of radioactivity by gullying in the cap formed by the headward progression of gullies in the embankment is considered at a later point with regard to failure of the embankment.

ii. Interaction with other failure mechanisms

Gullying in the cap may concentrate runoff at particular points in the embankment and cause either major gullying of the embankment or slope instability due to an increase in water content. Minor gully formation on the cap could therefore result in more serious problems in the embankment.

Differential settlement could cause cracks to form in the cap which would act as channels for runoff and result in gully formation if slope and runoff were sufficient. Differential settlement could also result in the formation of general depressions in areas or change the original grading on the cap. That may result in the concentration of runoff or slopes steep enough to initiate gully formation.

Any increase in precipitation, decrease in infiltration capacity of the soil, or loss of vegetation on the cap could initiate gully formation.

On the other hand, cementation of the cap material by natural weathering could retard the formation of gullies.

iii. Methods of prediction

No mathematical models are in existence that predict gully formation. Some research regarding gully formation has been done in the Piceance Creek drainage basin in northwestern Colorado (Schumm, 1977). In this study a relationship between the critical valley slope and drainage area was established to predict gully formation. This relationship is shown in Fig. 3. This relationship, however, does not pertain to drainage basins smaller than about 5 square miles, because variations in the vegetative cover prevent recognition of critical threshold slopes on a smaller scale. Brice (1966) found a similar but less well-defined relationship for valleys in Nebraska.

It may be tenuous to attempt to define a relationship between critical slope and drainage area (or runoff) that could be applied to the cap and embankment of a tailings impoundment. However, observation of gullying in adjacent landscapes may assist in prediction if similar soils are used for the cap. Care must be exercised in extrapolating from a natural situation to the impoundment because of differences between natural soils and the cap. Potential differences in vegetative cover will also influence gullying. The concept that a critical threshold may exist between a stable and unstable situation is well worth exploring and is an area that deserves further research.

iv. Likelihood

The likelihood of gully formation in the cap of an impoundment other than due to headward progression of embankment gullies will generally be low. The likelihood of gullies progressing from the embankment is considered in a later section regarding embankments.

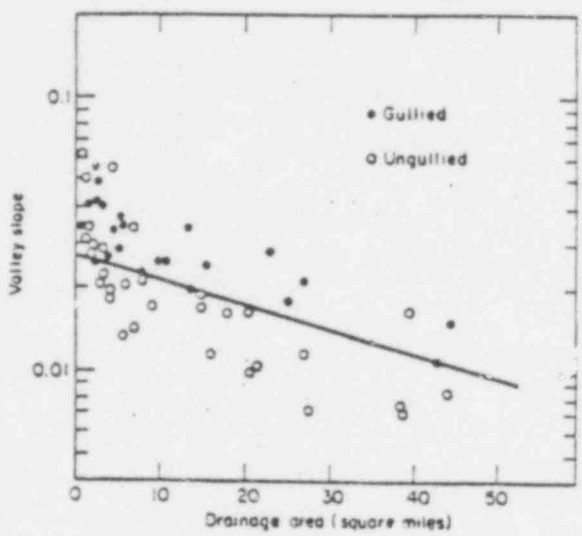

Fig. 3. Relation between valley floor slope and drainage area for small drainage basins, Piceance Creek area, Colorado. (From Schumm, 1977, after Patton and Schumm, 1975.)

The low value for likelihood of failure is based on the following assumptions:

- The slopes of the cap are gentle.
- The cap was graded during construction to minimize the concentration of runoff.
- The drainage area of the impoundment is isolated by diversion structures from regional drainages.
- The cap is protected by either vegetation or some erosion resistant material.

If these assumptions are not valid for a particular case the likelihood of failure would increase. The amount by which it may increase would be site specific.

v. Magnitude of release

The potential magnitude of release due to gullying on the cap would be a function of the amount of cover material and tailings removed. The mode of release would most likely be an increase in radon emanation due to a reduction in cap thickness in certain areas. Severe gullying of depths greater than the cap thickness could result in erosion of the tailings as well. The latter would be an extreme case which would probably be caused only by the unlikely existence of steep cap slopes or the progression of embankment gullies into the tailing.

Discontinuous gullies could be formed that would erode cover material from one location and deposit it at another, still within the impoundment. The increase in the release of radon gas to the atmosphere would be a function of amount of cover removed and the extent of the gullying.

The magnitude of release would be site specific and would increase with slope and local drainage area on the cap. The procedures used to estimate magnitude of release for cracking of the cap due to differential settlement could be employed to quantify the magnitude of release for a particular site.

vi. Site considerations

The site considerations that will influence the likelihood of gully formation on the cap is primarily precipitation. The annual amount, frequency, intensity, and duration are the factors of primary influence. High intensity, infrequent storms such as are common in arid or semi-arid regions do not support vegetation and are likely to produce the kind of runoff that would result in gully formation. Topographic location would be important if the impoundment is not isolated from the local drainage basin by diversion structures. In that case drainage patterns, drainage area, and location of the impoundment within the drainage area become important.

vii. Design consideration

Important design considerations to prevent gully formation are

- Avoidance of long steep slopes.
- Grading of the surface to avoid concentration of runoff, or to divert runoff into areas that can be stabilized against gullying.
- Establishment of vegetation cover or placement of erosion resistant cover such as rocks.
- Avoidance of highly erodible soils on the surface.

- Maintenance of soil infiltration capacity as high as practicable.
- Diversion of external runoff from the impoundment.

Observation of gullying in the local natural landscape can provide information on gully potential and design requirements.

viii. Monitoring, maintenance and remedial measures

The potential for gullying will exist as long as the impoundment is intact. Changes in climate, loss of vegetation, regional uplift or subsidence, or failure of another element of the impoundment could create an environment conducive to gullying. Long-term monitoring and maintenance may be necessary therefore to prevent or correct gully formation.

Monitoring schemes would include direct observation by on-site personnel or aerial photography.

Maintenance and remedial measures would not be difficult. Regrading of the surface to correct drainage features or to adjust slopes and slope lengths, or the replacement of cover material would generally be the extent required. The amount of maintenance would depend on the severity of gullying. However, it is anticipated that most maintenance and remedial measures would be straightforward and routine.

ix. Time dependence

The potential for the formation of gullies on the impoundment cap is time dependent in the sense that the likelihood of gully formation increases for longer time periods. If environmental and geomorphological conditions do not change and if the cap has been constructed in a manner to remain stable it would be safe to assume that gullying will not be a problem. However, over medium long-term time, and even within short intervals, it is unrealistic to expect conditions not to change.

The climate is likely to change over long periods. Drought may occur for several years followed by years of abnormally high precipitation. A change in precipitation and temperature will affect sediment yield and rainsplash erosion (Schumm, 1977, p. 28; Mabbutt, 1977, p. 69). Figure 4 demonstrates the relationship between climate and sediment yield. Figure 5 shows the effect of temperature on the relationship between mean annual runoff and mean annual precipitation. These data clearly demonstrate that the erosional process will change with changing climatic conditions, and the potential for initiating increased gullying will be a function of the environment before change and the direction of change.

Major climatic changes may not occur within the short long-term period but could certainly be expected within a medium long-term period. For example, changes in climate and a consequential decrease in suitability of the area for agriculture has been advanced as an explanation for the desertion of the Mesa Verde area about 1000 years ago.

Another important consideration is that gully formation as well as other erosional processes may not be continuous but episodic in nature. Schumm (1977) discusses this in relation to thresholds and responses of the system to continuous processes such as denudation.

Over long periods of time being considered herein, regional effects become more important in predicting the changes likely on the site. The scenario developed for future climates and the specific location of the impoundment become the most important factors in predicting future responses of the cap to gullying.

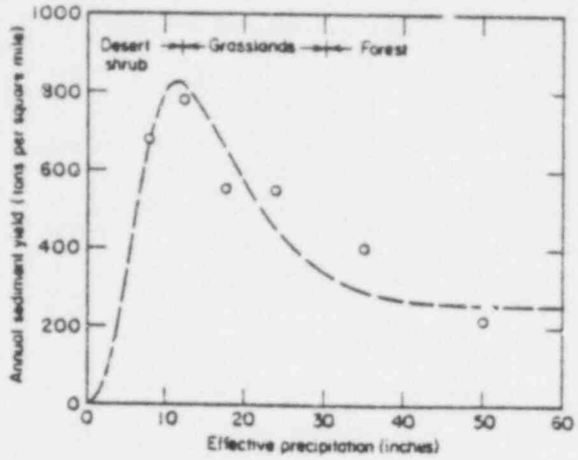

Fig. 4. Variation of sediment yield with climate as based on data from small watersheds in the United States. (From Schumm, 1977.)

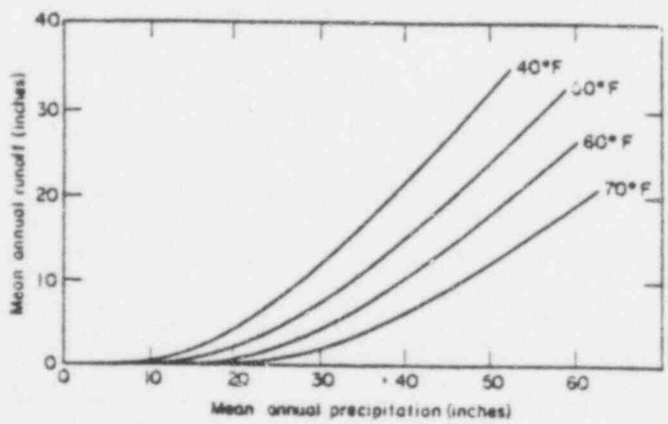

Fig. 5. The effect of average temperature on the relation between mean annual runoff and mean annual precipitation. (From Schumm, 1977.)

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c. SHEET EROSION BY WATER

i. Causes and description

Water erosion on the cap of a tailings impoundment will be caused by either raindrops striking the surface or water flowing over the surface. The combination of these two phenomena can detach and transport significant quantities of soil from an area. The slope of the land, the nature of the soil, the duration and intensity of the rain, and the ground cover are among the most important factors influencing the amount of erosion that will occur (Stallings, 1953, p. 1).

Soil may be transported off the area or it may be dislodged from one part of the cap to be redeposited in an adjacent, lower location. Over a period of time large quantities of material could be removed from the surface reducing the cap thickness or in the extreme case removing the cap entirely. If erosion is extensive, tailings material might be exposed and removed. Erosion of that extent, however, would require erosion of the embankment or confining structure as well.

Soil and drainage characteristics of the surrounding area may be important in evaluating total erosion potential of the site. The total drainage area above the impoundment may supply surface runoff to the site or it could introduce sediments to the impoundment if the location provides a depositional environment.

ii. Interaction with other failure mechanisms

Surface erosion will be affected by runoff from the surrounding area if it is allowed to reach the cap. Therefore, the operation of diversion structures has an important influence on erosion. In this regard, off-site erosion can affect the operation of diversion structures by supplying silt which may clog them. Also, long-term changes in drainage patterns in the surrounding area may introduce additional runoff to the site.

Settlement within the impoundment can change surface characteristics resulting in changes in the erosion potential. Climatic changes will affect erosion if changes in precipitation occur. These effects may be manifested in terms of both vegetation response and amount of runoff.

iii. Methods of prediction

The amount of soil loss due to erosion can be predicted by use of the Universal Soil Loss Equation (USLE). This equation was developed to predict soil loss due to sheet and rill erosion from agricultural lands.

The following description of the USLE is taken from U.S. Environmental Protection Agency publication "Preliminary Guidance for Estimating Erosion on Areas Disturbed by Surface Mining Activities in the Interior Western United States" (SCS, 1977).

THE UNIVERSAL SOIL LOSS EQUATION

The Universal Soil Loss Equation (USLE) is an empirically developed formula historically used to estimate soil loss on agricultural lands.

The soil loss equation is $A = R\ K\ L\ S\ C\ P$, where:

A, is the computed soil loss expressed in tons/acre/year.

R, the rainfall factor, is the number of erosion index units in a normal year's rain. The erosion index is a measure of the erosive force of specific rainfall.

K, the soil erodibility factor, is the erosion rate per unit of erosion index for a specific soil in cultivated continuous fallow, on a nine percent slope, 72.6 feet long.

L, the slope length factor, is the ratio of soil loss from the field slope length to that from a 72.6 foot length on the same soil type and gradient.

S, the slope gradient factor, is the ratio of soil loss from the field gradient to that from a nine percent slope.

C, the cover or cropping management factor, is the ratio of soil loss from a field with specified cropping and management to that from the fallow condition on which the factor K is evaluated.

P, the erosion control practice factor, is the ratio of soil loss under specified soil management practices, to that with straight rows, up and down the slope.

Many other sources also exist which describe the USLE. Several of these are Soil Conservation Society of America (1976), National Cooperative Highway Research Program Project 16-3 (UWRL, 1976), and Wischmer (1976). It should be noted that the description of the USLE presented by Utah State University (1976) in the NCHRP Project 16-3 redefines the CP factors. This report uses VM , which equates to vegetation and mechanical (or chemical) means of erosion control. The CP factors relate specifically to agricultural practices and the use of VM is intended to demonstrate the relationship to non-agricultural methods of control.

The USLE has had wide application in predicting soil losses due to water erosion. It must be remembered, however, that the equation was developed to estimate gross erosion from rainfall on farmlands east of the Rocky Mountains and that it is an empirical formula (UWRL, 1967, p. 18). The USLE is potentially useful as a technique for estimating soil loss from lands of the Interior Western U.S. disturbed by mining and construction activities (SCS, 1977, p. 1). However, a basic part of predicting soil losses based on the USLE is an understanding that the data that is generated is only a best estimate (Heil, 1977). It can be used to compare alternative conservation plans but actual quantities of soil loss are estimates. Furthermore, the factors that are used in the equation (R, K, L, S, C, P) must be analyzed for their applicability to the specific site and their accuracy (Wischmer, 1976, p. 6). In most cases factors must be calculated rather than determined experimentally.

Average losses are estimates and do not represent the probability of losses during short-term storms or during periods of various soil and vegetation conditions that might occur under various use intensity conditions (Heil, 1977). Also, it includes a specific set of management and climate related factors which may not adequately reflect changing conditions over very long time periods.

iv. Likelihood

The likelihood that water erosion will occur is almost a certainty. The amount of soil loss will vary depending upon the particular site, soil conditions and nature of the cover.

It is conceivable that deposition rather than erosion could occur. This is a specific site condition and can not be assumed without analysis. Erosion is the most likely expectation, especially if the cap is graded to eliminate impoundment of water.

v. Magnitude of release

The potential release of radioactivity will be a function of the amount of erosion that occurs and the reduction of cap thickness. The principle radiation release for erosional process failures of the cap will be radon gas emanation. The effectiveness of a cover to reduce radon diffusion is a function of the thickness of the cover, the radon content of the tailings, the water content of the cover soil, and the type of soil used (Sears et al., 1975). The equation shown below describes the diffusion through a cover from a plane source such as tailings (Tanner, 1964; Culot, 1973)

$$C_{(x)} = C_p \, e^{-(\sqrt{\lambda v/D}x)}$$

where: $C_{(x)}$ = radon concentration to a distance X from the plane source

C_p = radon concentration in the plane source

x = depth of the cover

D = effective diffusion coefficient for radon through the fluid (air, water, etc.) in the void spaces between soil particles

λ = decay constant of radon -222 = 0.693/half-life = 2.1×10^{-6} sec^{-1}.

v = void fraction; the fraction of the total volume which is not occupied by soil particles or porosity of the soil

e = base of natural logarithms

The attenuation of radon emanation as a result of covering to various thicknesses is shown in Fig. 6. Also shown is the effect of moisture content and soil characteristics on attenuation.

For all failure mechanisms that result in a reduction of cap or cover thickness the magnitude of release will be related to a percentage change in release due to loss of thickness.

Extreme erosion could result in the removal of the entire cap. Erosion that extreme, however, would need to be accompanied by erosion of the embankment as well to maintain a slope on the surface.

The magnitude of release from the cap due to erosion would be expected to increase with time. In order to estimate the potential magnitude of soil loss over long-time periods it would be necessary to develop future scenarios including climate and management practices.

vi. Site considerations

The site considerations that will influence the potential for erosion are the climate and precipitation, the topographic

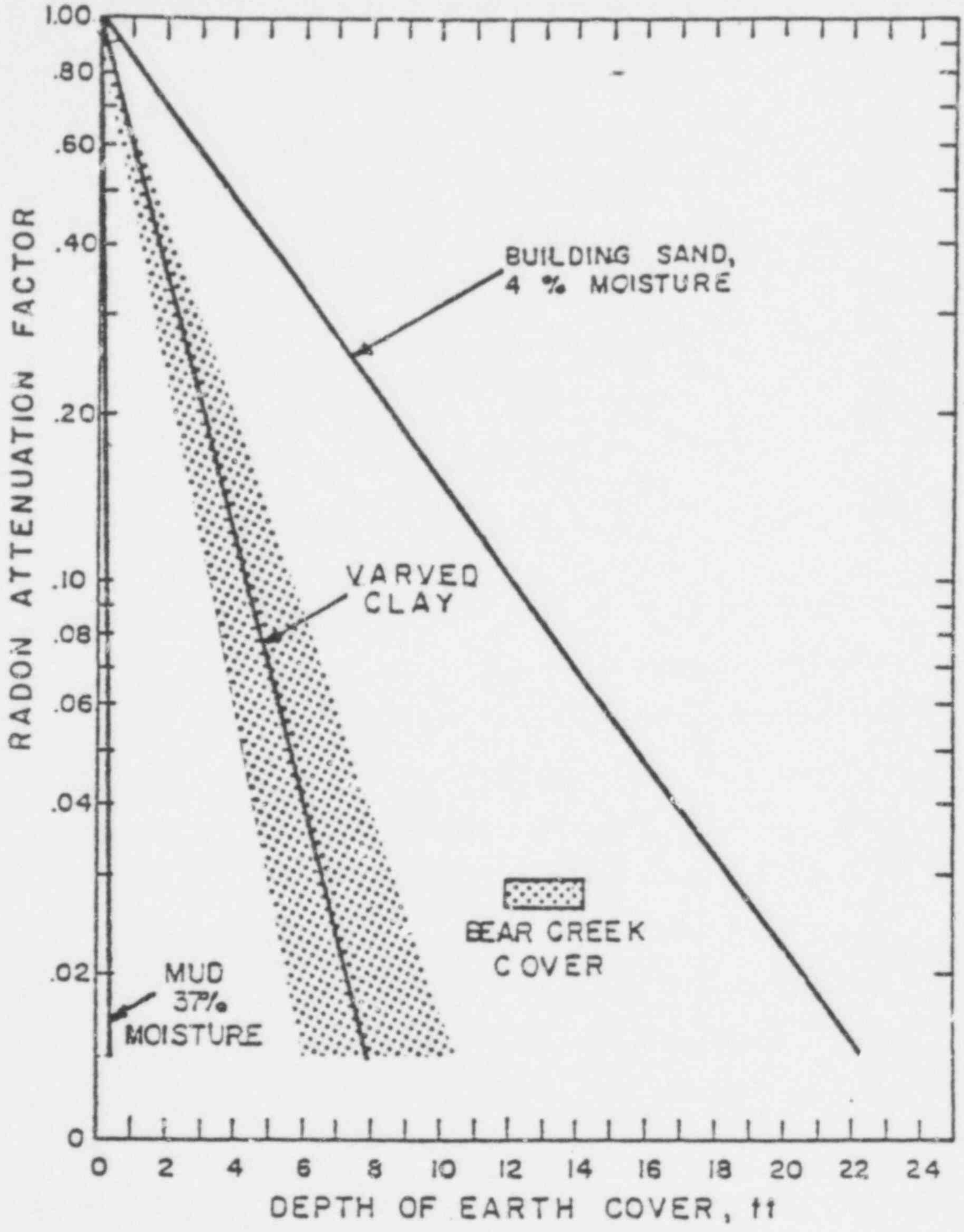

Fig. 6. Radon attenuation by earth cover over tailings piles.
(From Sears et al., 1975.)

location with respect to other drainages, and the local vegetation which would be expected to become established on the impoundment.

Some factors in the Universal Soil Loss Equation such as erodibility (K), rainfall (R) and cover (C) are site specific variables. Specific values for these factors and the methods of estimation or calculation are described fully in literature already cited. It should be noted that the K factor was developed for natural soils and existing data may not adequately define compacted soils (Heil, 1977). Also, the factor R is an average value for rainfall erosion and may not adequately represent the potential for an infrequent high intensity storm.

The local natural vegetation is important to consider because it reflects the nature of the vegetative cover that will ultimately exist on the site. Long-term erosion estimates must consider the evolution of vegetation and soil properties, unless significant commitments to maintenance are to be part of the disposal plan.

Topographic location is important in two respects. First, it must be determined if runoff from areas outside of the impoundment will supply water and therefore increase the erosion potential of the site. Secondly, consideration must be given to the potential for deposition of sediments on the impoundment from outside. If the impoundment can be located such that natural erosion of the surrounding area would deposit material on the cap particular benefits in the form of eventual burial could be realized.

vii. Design considerations

Considering the remaining factors of the USLE, LSCP (or LSVM), the design considerations that can influence the erosion are the slope characteristics, the conservation management practices, and the cover material.

Table 2, from the SCS (1977) report, describes the LS factor and illustrates the relationship between erosive power of water and slope length and steepness. Long or steep slopes should be avoided. Terraces and other conservation techniques common in agriculture can be employed to reduce slope length and steepness.

The CP or VM factors that describe the soil conservation practices used to control erosion losses are also an important design element. Good practices include the use of vegetative cover, mulching, mechanical or chemical soil preparation, runoff control, contouring and others (SCS, 1977; UWRL, 1976). These design considerations, however, imply a relatively high level of maintenance if their integrity is to be assured over extended time periods. This is especially true for special treatments such as terraces or mulching which could be destroyed or lost with no maintenance.

One design factor that may warrant particular consideration for erosion control is the placement of a coarse rock cover over the cap. This practice has been used with some success on the tailings impoundment in Shiprock, New Mexico. It would be similar to the phenomenon known as "armoring" with regard to geomorphological processes. Examples are presented in Mabbutt (1977) of desert landforms that have been stable for thousands of years that are covered by desert pavements or gravel armor. Alluvial surfaces exist in Death Valley that have been stable since late Pleistocene or about 20,000 years (Hunt and Mabey, 1966). It is apparent, therefore, that placement of a coarse rock layer over the cover or the use of coarse rock mixed with finer soil could be effective in minimizing surface erosion.

Table 2. Values of the topographic factor of "LS". (From SCS, 1977.)

Length of Slope (L) Ft.	Percent Slope (S) 0.2	0.3	0.4	0.5	1.0	2.0	3.0	4.0	5.0	6.0	8.0	10.0	12.0	14.0	16.0	18.0	20.0	25.0	30.0	40.0	50.0	60.0
20	.05	.05	.06	.06	.08	.12	.18	.21	.24	.30	.44	.61	.81	1.0	1.3	1.6	1.8	2.6	4	6	8	10
40	.06	.07	.07	.08	.10	.15	.22	.28	.34	.43	.63	.87	1.2	1.4	1.8	2.2	2.6	3.5	5	8	11	15
60	.07	.08	.08	.08	.11	.17	.25	.33	.41	.52	.77	1.0	1.4	1.8	2.2	2.6	3.0	4.5	6	10	14	18
80	.08	.08	.09	.09	.12	.19	.27	.37	.48	.60	.89	1.2	1.6	2.1	2.6	3.0	3.6	5.5	7	11	16	21
100	.08	.09	.09	.10	.13	.20	.29	.40	.54	.67	.99	1.4	1.8	2.4	2.9	3.5	4.2	6.0	8	13	18	23
110	.08	.09	.10	.10	.13	.21	.30	.42	.56	.71	1.0	1.5	2.0	2.5	3.0	3.7	4.5	6	9	14	19	25
120	.09	.09	.10	.10	.14	.21	.30	.43	.59	.74	1.0	1.6	2.1	2.6	3.3	4.0	4.6	7	9	14	20	26
130	.09	.09	.10	.11	.14	.22	.31	.44	.61	.77	1.2	1.6	2.2	2.8	3.4	4.1	4.9	7	9	15	20	27
140	.09	.10	.10	.11	.14	.22	.32	.46	.63	.80	1.2	1.7	2.3	2.9	3.6	4.3	5.1	7	10	15	21	29
150	.09	.10	.11	.11	.15	.23	.32	.47	.66	.82	1.2	1.8	2.4	3.0	3.7	4.5	5.3	8	10	16	23	30
160	.09	.10	.11	.11	.15	.23	.33	.48	.68	.85	1.2	1.9	2.5	3.1	3.9	4.7	5.5	8	10	17	24	31
180	.10	.10	.11	.12	.15	.24	.34	.51	.72	.90	1.4	1.9	2.6	3.3	4.1	5.0	6.0	9	12	18	26	33
200	.10	.11	.11	.12	.16	.25	.35	.53	.76	.95	1.4	2.1	2.8	3.6	4.4	5.3	6.3	9	12	18	27	35
300	.11	.12	.13	.14	.18	.28	.40	.62	.93	1.2	1.8	2.7	3.6	4.5	5.6	6.8	8	12	16	25	35	45
400	.12	.13	.14	.15	.20	.31	.44	.70	1.0	1.4	2.0	3.2	4.2	5.4	6.7	8.0	10	14	19	30	42	54
500	.13	.14	.15	.16	.21	.33	.47	.76	1.2	1.6	2.2	3.7	4.9	6.2	7.6	9.2	11	16	21	34	47	61
600	.14	.15	.16	.17	.22	.34	.49	.82	1.4	1.6	2.4	4.1	5.4	6.9	8.5	10.3	12	16	24	38	53	68
700	.15	.16	.17	.18	.23	.36	.52	.87	1.4	1.8	2.6	4.5	6.0	7.5	9.3	11.3	13	18	26	41	58	75
800	.15	.16	.17	.18	.24	.38	.54	.92	1.6	2.0	2.8	4.9	6.4	8.2	10.1	12.2	14	20	28	45	58	81
900	.16	.17	.18	.19	.25	.39	.56	.96	1.6	2.0	3.0	5.2	6.9	8.8	10.8	13.1	16	22	30	48	67	87
1000	.16	.18	.19	.20	.26	.40	.57	1.0	1.6	2.2	3.0	5.6	7.4	9.3	11.6	14.0	17	24	32	51	72	93
1100	.17	.18	.19	.20	.27	.41	.59	1.0	1.8	2.2	3.5	5.9	7.8	9.9	12.2	14.8	18	25	34	54	76	98
1200	.17	.18	.20	.21	.27	.42	.81	.10	.18	2.4	3.5	6.2	8.2	10.4	13.0	15.6	18	27	36	57	80	104
1300	.18	.19	.20	.21	.28	.43	.82	1.2	2.0	2.4	3.5	6.5	8.5	11.0	13.5	16.4	19	28	38	60	84	109
1400	.18	.19	.21	.22	.29	.44	.63	1.2	2.0	2.6	3.5	6.8	9.0	11.4	14.1	17.1	20	30	40	63	88	114
1500	.19	.20	.21	.22	.29	.45	.65	1.2	2.0	2.6	4.0	7.1	9.4	12.0	14.7	17.8	21	31	41	65	92	119
1600	.19	.20	.21	.23	.30	.46	.66	1.2	2.2	2.6	4.0	7.4	9.8	12.4	14.8	18.5	22	32	43	68	95	123
1700	.19	.21	.22	.23	.30	.47	.67	1.2	2.2	2.8	4.0	7.6	10.1	12.9	15.9	19.2	23	33	44	70	97	128
2000	.20	.22	.23	.24	.32	.49	.71	1.4	2.4	3.0	4.5	8.4	11.1	14.1	17.5	21	25	36	49	77	108	141

Contour limits - 2 percent 400 feet, 8 percent 200 feet, 10 percent 100 feet, 14 - 24 percent 60 feet. The effectiveness of contouring beyond these limits is speculative.

When the length of slope exceeds 400 feet and (or) percent of slope exceeds 24 percent, soil loss estimates are speculative as these values are beyond the range of research data.

viii. Maintenance, monitoring, and remedial measures

Monitoring schemes to detect erosion loss are fairly straightforward. Visual inspection with the aid of elevation markers or sediment catchments would be effective to monitor erosion. Remote methods could include aerial photos and, where vegetative cover was a key to the erosion control plan, infrared imagery would be effective. Sediment load monitoring in streams or rivers, at locations remote from the site, could provide an indication of upstream erosion changes.

Maintenance and remedial measures would need to be timely in order to minimize eventual need for extensive repairs. The addition of cover material, regrading, or stabilization of problem areas would normally be part of a routine maintenance program. Persistent problems may require more extensive treatment, such as major changes in drainage patterns or planting of different vegetation types.

Monitoring of radon emanation may also be an effective means of initially evaluating the performance of the cap and subsequently indicating changes which may be the result of erosional loss of cap material.

ix. Time dependence

Erosion is a continuing process and will continue and change as conditions change over time. It is impossible to predict increases or decreases over time without also describing the hydrologic changes that can be expected. As the time period under consideration becomes long, the natural erosional processes in the specific locales may provide the best clue to future potential erosional patterns.

In general, erosion will increase continuously with time unless natural processes such as armoring decrease the erosional rate. Prediction of soil loss may be possible up to the medium long-term period. However, for long long-term considerations it is nearly impossible to predict what changes in erosion might take place.

d. WIND EROSION

i. Causes and description

The three basic ingredients contributing to wind erosion are (Chepil, 1956):

- Loose, finely divided, dry soil.
- A smooth, bare soil surface.
- Strong wind.

Soil particles may move as suspended dust and be carried great distances or they may move relatively short distances by saltation or surface creep depending on size, surface condition and wind velocities (Chepil, 1958; Woodruff et al., 1972).

Wind erosion can remove significant quantities of top soil from a site. The effects of wind erosion can be the destruction of vegetation by abrasion, the accumulation of soil in culverts and diversion structures, and the removal of the productive fine grained silts and organic constituents of a soil.

Wind erosion can also take the form of blowouts in tailings embankments, similar to those common in beach dunes. These blowouts can continue to grow, thereby reducing stability and resulting in extensive movement of the tailings embankments. If left unchecked, wind erosion could cause movement of fine grained tailings material over great distances.

ii. Interaction with other failure mechanisms

An important interaction with other failure mechanisms is the potential effect wind erosion could have on diversion structures.

Diversion ditches can become filled with coarse soil fractions moved by surface creep and saltation. With diversion structures clogged, surface runoff that is intended to be diverted away from the impoundment may flow over the surface of the cap. Another important effect of wind erosion may be the potential loss of protective vegetation due to loss of soil fertility or destruction by abrasion of wind blown particles. The loss of vegetation by fire, drought or pestilence will increase the potential for wind erosion.

iii. Methods of prediction

Wind erosion can be estimated by using a soil loss equation similar to the USLE. This equation was developed by W. S. Chepil and is described in several reports (UWRL, 1976; Woodruff and Siddoway, 1965; Woodruff et al., 1972).

The wind erosion equation is:

$$E = f(ICKVL)$$

in which:

E = soil loss by wind in tons/acre/year

I = soil wind erodibility factor, related to the soil fraction greater than 0.84 mm. as shown in Table 3.

I_s = soil wind erodibility index, needed to compute erodibility for windward slopes, Fig. 7. For slopes, erodibility becomes $I' = I \times Is$ in tons/acre/year.

C = climatic factor

K = soil surface roughness

V = equivalent quantity of vegetative cover

L = unshielded field width measured along the direction of the prevailing wind.

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Table 3. Soil erodibility index I . (From UWRL, 1976.)

Percent of Dry Soil Not Passing a 20 Mesh Screen (Units)	0	1	2	3	4	5	6	7	8	9
	Non-crusted Soil Surface (tons/acre)									
0	-	310	250	220	195	180	170	160	150	140
10	134	131	128	125	121	117	113	109	106	102
20	98	95	92	90	88	86	83	81	79	76
30	74	72	71	69	67	65	63	62	60	58
40	56	54	52	51	50	48	47	45	43	41
50	38	36	33	31	29	27	25	24	23	22
60	21	20	19	18	17	16	16	15	14	13
70	12	11	10	8	7	6	4	3	3	2
80	2	-	-	-	-	-	-	-	-	-
	Fully Crusted Soil Surface (tons/acre)									
0	-	51.7	41.7	36.7	32.5	30.0	28.3	26.7	25.0	23.3
10	22.3	21.8	21.3	20.8	20.2	19.5	18.8	18.2	17.7	17.0
20	16.3	15.8	15.3	15.0	14.7	14.3	13.8	13.5	13.2	12.7
30	12.3	12.0	11.8	11.5	11.2	10.8	10.5	10.3	10.0	9.7
40	9.3	9.0	8.7	8.5	8.3	8.0	7.8	7.5	7.2	6.8
50	6.3	6.0	5.5	5.2	4.8	4.5	4.2	4.0	3.8	3.7
60	3.5	3.3	3.2	3.0	2.8	2.7	2.7	2.5	2.3	2.2
70	2.0	1.8	1.7	1.3	1.2	1.0	0.7	0.5	0.5	0.3
80	0.3	-	-	-	-	-	-	-	-	-

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The equation is a useful management tool for determination of potential wind erosion under existing conditions, and for determination of surface conditions and sheltering necessary to reduce wind erosion to a tolerable amount (Woodruff et al., 1972). The equation serves as a guide for determining the management conditions necessary to control wind erosion under conditions at a given site (Heil, 1977).

It is important to note that the computed soil loss is an estimate based upon specific site and management conditions. It can be valuable in showing the relative effectiveness of alternatives for wind erosion control. It is the best tool available for prediction of wind erosion losses and does define the factors that need to be considered when evaluating wind erosion potential. Its ability to predict long-term wind erosion losses, however, is limited by changes which the input factors may undergo over time.

iv. Likelihood

The likelihood that wind erosion will occur is high. The amount of soil material moved and the distance of movement is a function of the factors considered in the wind erosion equation.

v. Magnitude of release

The magnitude of release due to wind erosion of the cap will be a function of the reduction of cap thickness caused by soil loss. The expected mode of release would be increased radon emanation as described in water erosion of the cap, Section A1c.

If no cap exists, or if it is removed by other mechanisms, direct blowing of tailings material is possible. The fine grained portion of the tailings (slimes) would have the highest potential for long distance transport, while the coarser sands would move shorter distances by saltation or surface creep. Long-term exposure of tailings to wind erosion could disperse the radioactive particles over large areas.

vi. Site characteristics

Site conditions that are important factors in determining the wind erosion potential are the soil chosen for the cap, wind speed and direction, precipitation as it affects soil moisture and the nature of the vegetative cover that can be supported. The topographic location of the impoundment with respect to prevailing winds and the unsheltered length exposed to winds also are important to the effective control of soil loss by wind.

Long, open windward reaches will increase wind erosion. Any barrier on the windward side of the impoundment can be effective in reducing wind erosion. As with water erosion, it may be possible to locate impoundments to take advantage of deposition of wind transported soils. Studies of existing wind deposited material can provide guidance in this regard. Location of impoundments at the top of unprotected knolls or ridges, in valleys that channel prevailing winds or diurnal winds, or in areas susceptible to high seasonal winds, such as the Colorado Front Range area, may increase the wind erosion potential.

vii. Design considerations

The design considerations that are important for prediction of wind erosion potential are cap soil characteristics, the soil roughness, the slope of the surfaces, the unsheltered length, and the vegetation cover.

The surface roughness factor in the wind erosion equation is mainly intended for use on bare or fallow fields. It could be an important consideration especially during the early stages of vegetation establishment. In long-term application it would probably become a factor only in maintenance and remedial work.

The slope of the surfaces that face the windward side is very important. Figure 7 demonstrates the increase in erodibility due to slope. The avoidance of steep slopes and knolls on the windward side will reduce erosion potential.

Wide, unsheltered and smooth surfaces perpendicular to the prevailing wind enhance the erosion potential. Any barrier that can be placed to break up the flow of wind can be beneficial in reducing the wind erosion potential.

An important factor of design is vegetation. Good vegetative cover is an effective way to control wind erosion. The natural vegetation in the immediate vicinity will suggest what kind of cover can be expected. Grading or diversion to direct precipitation runoff over the cap without enhancing the potential for water erosion or stability problems may be a useful design feature. Changing climates that may affect both vegetation and wind must be considered in scenarios of future wind erosion potential.

Perhaps the factor over which the most control can be applied is the choice of capping soil. Figure 8 (Chepil, 1958) indicates that soil particles having an equivalent diameter of 0.1 mm. are most erodible. Chepil (1965) also points out that erodible soil particles can be protected from wind erosion by being sheltered by larger nonerodible soil particles. This protection occurs when nonerodible material projects above the surface in sufficient density to completely shelter the erodible fraction from the wind. This constitutes the phenomenon of armoring described previously.

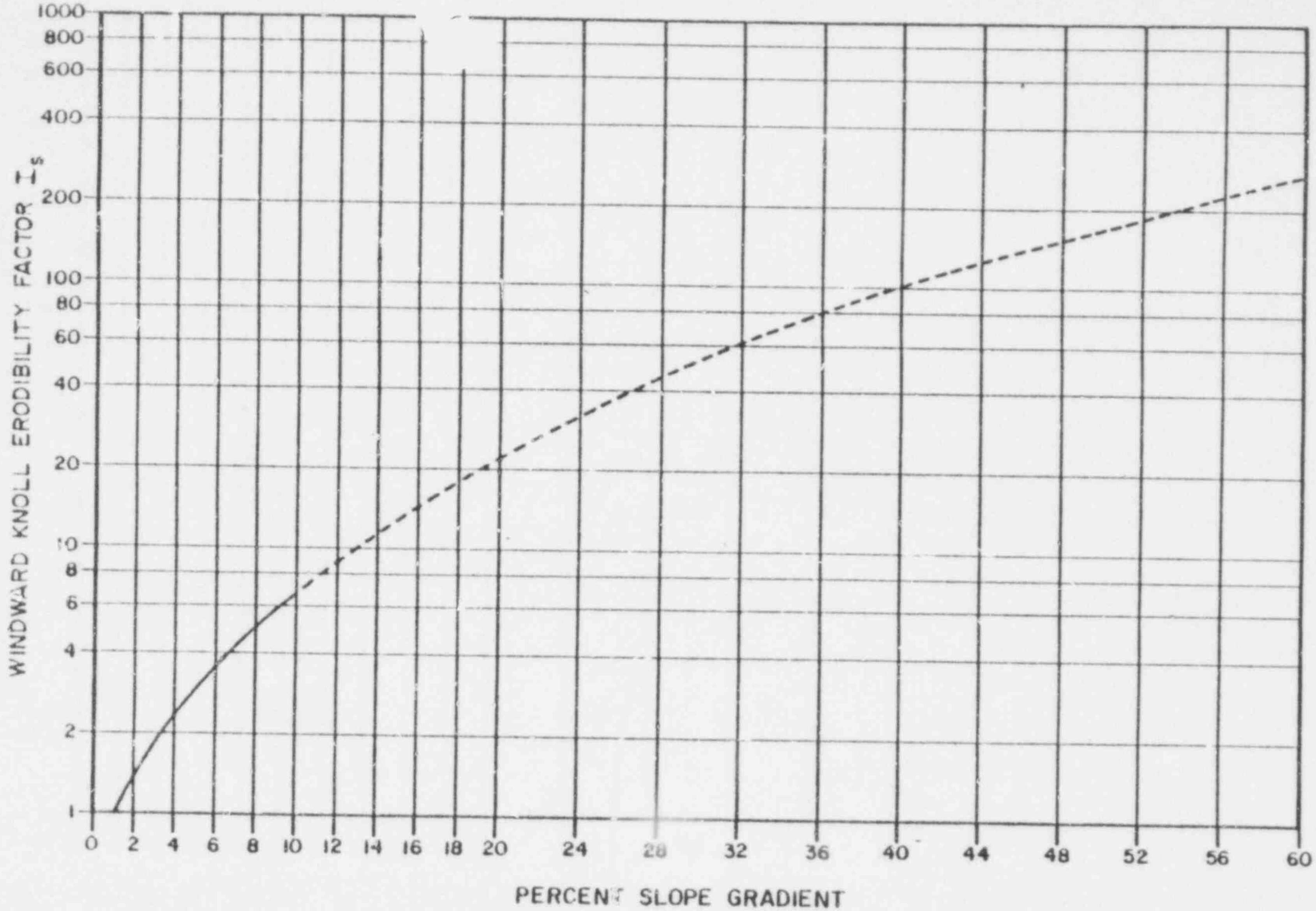

Fig. 7. Soil erodibility factor index for windward slopes, I_s . (UWRL, 1976.)

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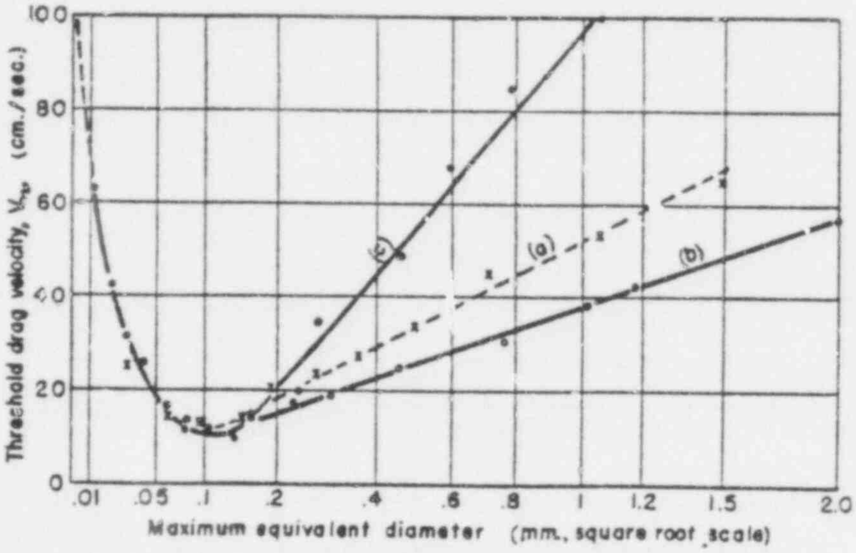

Fig. 8. Relation of the threshold drag velocity of the wind to the maximum equivalent diameter of the transported soil particles: a, Sieved fractions in which the ratio of minimum to maximum diameter varies as $1:\sqrt{2}$; b, sieved fractions in which the size of particles ranges from fine dust to the indicated maximum size; c, soil containing 15 percent of the nonerodible clods ranging up to 25 mm in diameter. (From Chepil, 1958, p. 7.)

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In many cases the nonerodible fractions are subject to abrasion and disintegration by the bombardment of the moving particles. In such a case the process of protecting projections never is completed and erosion continues. However, if pebble or rock can be incorporated in the soil matrix used for the cap, wind erosion potential could be reduced by the formation of a protective, nonerodible pavement.

viii. Monitoring, maintenance and remedial measures

Monitoring techniques similar to those discussed in water erosion could potentially be employed to observe wind erosion. Most useful are perhaps techniques that would identify changes in vegetation, both on and off the site.

Maintenance and remedial measures, likewise, are of the same nature as for water erosion. One difference would be that with wind dispersed materials, capture and cleanup may not be possible. Timely maintenance and identification of design deficiencies or problems related to changes in climatic conditions and subsequent correction are important to long-term wind erosion control.

ix. Time dependency

Wind erosion is possible throughout the entire time interval. Changing conditions or the occurrence of interacting failure modes can alter predictions about wind erosion potential.

Wind erosion may be expected to occur continuously. If armoring forms pavements on the surface, however, wind erosion can be checked for several thousand years.

e. FLOODING

i. Cause and description

Damage to the cap by flooding may result from either the routing of flood waters around or on the impoundment or directly from the precipitation on the cap. If flooding is due to extremely heavy precipitation in the immediate area of the impoundment, sheet erosion and gullying of the cap can result. This event would be a sudden occurrence that would accentuate gullying and sheet erosion as was discussed previously. The effects of those phenomena would then accelerate from that point on.

On the other hand, heavy flooding in the immediate area of the impoundment could result in a washout of portions of the cap. In that case, failure would be in combination with failure of other elements of the impoundment. The general nature of impoundment failure due to a major flood in the immediate area is discussed below, with regard to natural phenomena.

It should be noted that flooding is not necessarily the result of precipitation in the immediate area but may result from heavy precipitation at any point within the watershed upstream from the impoundment area. The occurrence of major floods is discussed more fully at a later point.

ii. Interaction with other failure mechanisms

Whereas other failure mechanisms would not influence the occurrence of a flood, the effects of a flood will depend to a large extent on the degree of failure that has already occurred on the impoundment due to other modes of failure. For example, gullying, sheet erosion and differential settlement may have altered the surface of the cap such as to cause water to be channeled into a particular area. Also, if differential settlement occurs of sufficient magnitude to cause water to be impounded somewhere on the surface, flooding could result in overtopping and severe erosion of the embankment and cap.

In order to assess the full extent of damage that may result from a major flood (i.e., a flood larger than the design flood) the effects that other factors may have had on routing flood waters over the surface of the cap must also be considered.

iii. Methods of analysis

The prediction of occurrence of a flood of magnitude greater than the design flood is discussed at a later point with regard to natural phenomena. The extent of damage that may occur to the cap may be analyzed on the basis of models of gullying and erosion as discussed in other sections of this report.

iv. Likelihood of occurrence

Reclamation and abandonment plans would undoubtedly attempt to incorporate some design features that would minimize damage due to an event of the design magnitude. Thus, failure of the cap due to flooding would generally be coupled with failure of some other element of the impoundment, such as the diversion structures, or else would be the result of the occurrence of an event of magnitude greater than the design magnitude. The probability of occurrence of an event of magnitude greater than the design magnitude within the long time frames being considered herein is relatively high.

The likelihood of damage to a cap due to flooding would also depend greatly on its location relative to the surrounding topography and whether the flooding is accompanied by heavy precipitation at the site.

Severe flooding must be expected. Therefore, the likelihood of failure will depend on whether or not the flood water can come into contact with the cap.

v. Magnitude of release

The magnitude of release resulting from flood damage to the cap will depend upon the amount of cap that may be eroded. It will depend on the location of the impoundment relative to the surrounding topography and the effect that heavy precipitation can have on erosion of the material.

Flood water of depths of several feet are capable of removing soil of a considerable thickness over a relatively short time (Simons et al., 1977). Thus, if major flood waters can come in contact with the cap, it may be expected that the entire depth of cap could be removed in the area of contact.

The magnitude of release would therefore be in proportion to the cap area that may come into contact with flood waters. The magnitude of release that may result from erosion due to the largest precipitation event conceivably would be a function of the amount of cover that is removed either by sheet erosion or gullying.

vi. Site considerations

Within the medium long-term or long long-term periods the likelihood of a flood of major proportions occurring is relatively high. This is discussed in more detail in part B of this chapter. It would be undesirable, therefore, to place the impoundment in a natural drainage area unless the upstream catchment area is small. For example, at Bear Creek, Wyoming, although the impoundment is being placed in a natural draw, it is located at the head of the draw very near to the divide between watersheds. Thus, the catchment area that can contribute to flooding is very small.

vii. Design considerations

The most effective design considerations to minimize flood damage are either to select the site such that it does not exist in a flood susceptible area or to utilize natural or man-made flood control structures that will divert flood waters away from the cap. Neither of these measures, however, will be effective against abnormally high precipitation falling over prolonged periods of time directly onto the cap. To design for that situation an erosion resistant cap consisting of either vegetation or material of sufficient grain size or erosion resistance would be applicable. In this regard, it may be desirable to introduce drainage features onto the cap that would direct runoff towards areas in which erosion protection could be accentuated. For example, the cap may be graded towards natural depressions that would be filled with coarse rock material. The more gentle slopes leading into the controlled drainage areas could be covered with vegetation or granular cover to minimize wind and water erosion.

viii. Monitoring, maintenance and remedial measures

Because of the sudden nature of a major flood, monitoring schemes other than direct observation are obviously not applicable to detect failure due to this occurrence. The only maintenance procedures that would be applicable to such a situation would be to maintain surface characteristics in accord with the design plans. If precipitation or floods that exceed those for which the impoundment was designed occur, remedial measures may be necessary to prevent radioactive release. Slopes that were steepened by erosion should be regraded and covered with stable material. Areas of the cap that have been washed out would need to be replaced.

ix. Time dependence

Although the likelihood of occurrence of a flood greater than the design flood is small at any particular time, the likelihood of occurrence will increase in proportion to the length of time period being considered. However, the probability of occurrence would be the same for short, medium or long long-term periods unless climatic changes should occur.

f. CHEMICAL ATTACK

i. Causes and description

No cap will be completely immune from chemical attack or weathering. Chemicals, sun, ozone, wind, drying, rain, moisture, heat and freeze-thaw cycle will act on the cap. Usually, guarantees on synthetic liner materials do not cover malfunctions due to chemical attack. Both acid and alkaline leach processes are employed in the concentration of uranium. Therefore, the extraction process will define the general chemical environment of the pond.

Sun, aging and heat appear to be the worst enemies of polyvinyl chloride and polyethylene. Asphalt concrete is also affected by heat because the heat slowly distills the volatile components of the asphalt. Freezing weather and freeze-thaw cycles influence mostly the rigid linings; concrete, shotcrete (Gunite) and asphalt concrete.

Acids attack Portland cement concrete, as do sulfates. Acids and sulfates react chemically with the hydrated lime and hydrated calcium aluminate in the cement. The sulfates of sodium, potassium and magnesium are commonly present in alkali soils and groundwater found in arid and semiarid regions (Troxell, Davis and Kelly, 1968). ACI (1966) discusses chemical effects on various materials, including unprotected concrete.

Chemical attack of a soil cap may take place. Throughout geologic time rock and soil masses have been acted on by chemical and mechanical processes. Crystal size and structure of soil and rock are important to the resistance of different minerals to weathering. The most commonly found minerals in soils are quartz and feldspars. Quartz is relatively stable but feldspars are more subject to weathering. In an acid environment the feldspars can change to clay minerals within the short long-term period. Quartz is subject to weathering in an alkali environment. Clay minerals, in turn, can go into solution or weather further in an alkali or acid environment. The end product will depend on the nature of the environment. Examples of chemical weathering are given below.

- Hydrolysis

Of the chemical processes, hydrolysis is probably the most important. Hydrolysis is the reaction between H^+ and $(OH)^-$ of the water and the ions of the mineral. Due to the small size of the H^+ ion, it is able to replace existing cations. The pH is very important in this process because it influences the amount of H^+ ions present. Also, the solubility of many minerals is a function of pH. Mitchell (1976, p. 51) gives the solubility of alumina and amorphous silica as a function of pH. An example of this type of weathering is the decomposition of feldspar minerals to form the clay mineral kaolinite.

Hydrolysis of a silicate mineral occurs when the hydrogenated surface layers become unstable and break off (Reiche, 1945). For the reaction to continue, H^+ ions must be introduced and removal of the soluble products by leaching or precipitation must occur.

- <u>Oxidation and Reduction</u>

An example of oxidation is given by the reaction of pyrite with water to form acid (Keller, 1957).

$$2\ FeS_2 + 6\ H_2O + 7O_2 \rightarrow 2\ Fe(OH)_2 + 4\ H_2SO_4$$

- <u>Solution by Carbonation</u>

Carbonation is the reaction of carbonate or bicarbonate ions with a mineral. Typically, atmospheric CO_2 is the source of carbonate. Carbonation is particularly detrimental to limestone.

- <u>Ion Exchange</u>

If an ion exchange reaction takes place in a clay cap or liner, the soil properties may change. In general, the physical properties of clays, such as plasticity, shrinkage, swelling, strength and permeability are modified by changes in exchangeable cations. The greater the ion exchange capacity of a clay, the larger the potential for property changes (Mitchell, 1976).

- <u>Chemical Weathering Products</u>

The rate of chemical weathering is dependent on time, temperature, moisture, pH and soil type. During weathering minerals tend to become more stable (Grim, 1968). Under conditions of active leaching, acidity favors the concentration of silica and the removal of iron and aluminum. Neutral or alkaline conditions favor the concentration of iron and aluminum (Grim, 1968, p. 518).

Youthful soils have, by definition, little material that has chemically weathered to clays. The intermediate weathering stage of soils introduces some material chemically weathered to illite and/or montmorillonite clays. The advanced weathering stages of soils contain kaolinite as the representative clay mineral (Mitchell, 1976).

A drastic change in the environment may cause chemical weathering or reversal of the weathering cycle. For example, illite was formed in a period of less than a year as a consequence of adding potash fertilizer to a soil containing kaolinite (Wood, 1941; Grim, 1968, p. 519). Under long-term exposure to acid conditions, the clay minerals in the cap will ultimately be broken down.

ii. Interaction with other failure mechanisms

It is doubtful that other potential failure mechanisms would contribute to chemical attack of the cap. Chemical attack on the cap could change its physical properties possibly leading to failure by shrinkage, changes in permeability, strength, erosion resistance or compressibility. Weathering to soils to a higher clay content could be beneficial in terms of radon emanation (Fig. 7).

iii. Methods of prediction

The amount and rate of chemical attack of the cap will be dependent on the type of cap, the possibility of the water in the tailings impoundment interacting with the cap, the pH of the liquid, temperature, degree of weathering and the environment in which the soil existed prior to being used as a cap. Methods for analysis of chemical weathering are discussed in Grim (1968) and Mitchell (1976). Types of materials present in a clay cap that would be most subject to chemical weathering can be predicted based on past experience and/or laboratory tests. Within the present state of the art the accuracy of short or long-term predictions of the amount of chemical attack on a cap is doubtful. At best it could indicate general trends.

iv. Likelihood

The likelihood of chemical attack on the cap is high. However, the likelihood of chemical attack being of a magnitude that would lead to failure and allow the release of radioactivity is probably small. The likelihood of failure would be site specific and will depend on the type and thickness of the cover, the nature of the tailings and the acidity or alkalinity of the tailings water. Some materials may be very subject to chemical attack under conditions that can exist in a uranium tailings impoundment.

v. Magnitude of release

The potential magnitude of release due to failure of a cap will be influenced greatly by the nature of the chemical change that may occur. However, as noted above, the potential magnitude of release is expected to be low.

Since repair to the cap would not be difficult to accomplish, the negative utility factor of a failure of the cap would not be very high.

vi. Site considerations

The considerations that will influence chemical attack on the cap are primarily the soil types available for use as a cap, the climate and the process used to leach the uranium.

vii. Design considerations

If chemical weathering of the cap is a problem, recommendations can be made in the design to neutralize the tailings, to use a thicker cap or import material more capable of withstanding the chemical attack.

viii. Monitoring, maintenance and remedial measures

Maintenance and remedial measures are not effective in reducing chemical attack. Remedial measures can repair damage and would consist of replacing or covering sections of the cap.

ix. Time dependence

The time required for chemical attack of materials varies. The rate of chemical change would increase as the thickness decreases, as the permeability increases, in warmer climates, and as the acidity or alkalinity increases. The type of cap, whether it is a synthetic or soil material, and the mineralogical composition of the soil, will influence change. Some change may occur within the short long-term period. After several thousand years it would be expected that no further change would occur.

g. SHRINKAGE

i. Causes and description

In arid and semiarid regions desiccation of the cap is almost certain to occur. If cohesive soils are used for the cap, shrinkage may be expected to accompany desiccation. Shrinkage is not expected to be of concern if sandy soils are used for the cover. However, because of the lower permeability, the tendency would be for clayey soils to be used.

Shrinkage results in clays due to a reduction in the bound water and the development of high attractive forces between soil particles during drying. In general, the more plastic clays exhibit a greater potential for shrinking and swelling than do less plastic clays (Holtz and Gibbs, 1956; Altmeyer, 1956; Seed, Woodward and Lindgren, 1962). If shrinkage occurs, cracks can form in the cap and radon can be released.

A clay cap would probably be placed at a water content close to the plastic limit of the clay. The shrinkage limit is defined as the water content below which further drying would produce no further shrinkage. The amount of shrinkage that may be expected to occur would depend, therefore, on the difference between the plastic limit and the shrinkage limit. For montmorillonite and illite clays this difference may be as great as about 30% or 40% (Lambe and Whitman, 1969, p. 33). Many of the natural clays in the western United States contain significant amounts of montmorillonite.

ii. <u>Interaction with other failure mechanisms</u>

Shrinkage cracks can influence the drainage characteristics over the impoundment and may contribute to either water or wind erosion. If water flows along cracks it may lead to the development of gullies.

iii. <u>Method of prediction</u>

The amount of shrinkage can be analyzed using methods outlined in soil mechanics textbooks. However, it has been shown that the amount of total shrinkage will depend mainly upon the proportion of the clay, type of minerals, exchangeable cations, orientation of clay particles and the degree of aggregation (DeJong and Warkentin, 1965), and the initial moisture content.

A greater degree of shrinkage would be expected for clays having relatively high plasticity. In semiarid regions such as west Texas desiccation of clays in the dry seasons proceeds to a depth as great as 20 feet. Within this depth the clay is broken up by shrinkage cracks (Terzaghi and Peck, 1967). In India natural deposits of black cotton soils are characterized by a general pattern of cracks, especially during the dry season (Singh, 1967). Cracks about 10 cm wide and over 1 m. deep are not uncommon and in deep deposits the cracks may extend to about 3 m.

The degree of shrinkage to be expected can be predicted on the basis of the lineal shrinkage of a sample of representative soil.

iv. Likelihood of failure

The likelihood of shrinkage depends primarily on the clay content of the cap material, the moisture content and the aridity of the regions. Wide and deep cracks would be likely to occur in arid zones but not in humid zones. The amount of shrinkage will be governed by the percentage of clay and the plasticity.

v. Magnitude of release

The potential magnitude of release due to shrinkage cracks of the cap will be influenced by the spacing of cracks as discussed previously with regard to cracking due to differential settlement. The general magnitude of release to be expected may be estimated from Fig. 2b. It is seen from Fig. 2b that if more than about 2% shrinkage occurs the maximum release of radon will occur. This magnitude of shrinkage is very possible for most cohesive soils. . cohesionless sands are used as the cap shrinkage is obviously of little concern.

vi. Site considerations

Site considerations that will influence the shrinkage will be the aridity of the zone and the nature of material available to be used for the cap.

vii. Design consideration

If shrinkage is considered to be a problem, recommendations can be made in the design of the cap with regard to the of clay present in the cap soil. Nonplastic soils can be mixed with clayey soils to decrease the potential for cracking.

Compaction of the cap material could also be taken into consideration since soils compacted dry of optimum would have flocculated structure. Soils so compacted would shrink less than soils compacted wet of optimum.

viii. Monitoring, maintenance and remedial measures

Applicable monitoring techniques would consist primarily of direct observation of the surface conditions.

Maintenance and remedial measures for severe cracking would consist of filling excessively large cracks with a form of grout or slurry. If slurries are used, however, they may exhibit shrinking when they dry. Alternatively, mixtures of dry sand and bentonite could be graded over the site to fill cracks. This technique is sometimes used in repairs of cracks in embankment dams.

ix. Time dependence

If desiccation will cause shrinkage cracks it is expected that they would form within the first 100 years after abandonment. The exception to that would be if climatic changes should cause a change from a more humid environment to an arid one.

2. LINERS

The liner of a tailings impoundment is provided for the purpose of reducing and controlling seepage from the tailings impoundment. Failure of the liner can be defined as any change in the liner's behavior or properties that allow seepage rates and possible radionuclide movement at rates faster than the design limits. To predict the severity of failure of a lining, the interaction between the tailings, the liner, the soil below the liner, underdrains and water pressure must be taken into account.

Linings may be classified as natural or man-made, flexible or rigid, impervious or semi-impervious, continuous or non-continuous. Compacted earth liners reduce and control seepage rates but are not impermeable. Man-made liners are manufactured to be impermeable (for practical purposes), but in the final installation are usually not impermeable Manufacturing defects, seepage at seams, mechanical damage, settlement of subgrade, and aging all contribute to some degree of leakage. Consequently, both compacted soil and man-made linings should generally be backed up with some type of well designed underdrain system. If there is a lining, it probably will leak.

Linings have been used for thousands of years. One of the earliest known applications occurred over 3000 years ago for the Tigris River embankment of Assur where layers of bitumen and clay were used (Asphalt Institute, 1965). The rapid growth of linings in the last few years has started a new technology dealing with methods and materials to control movement of water. Table 4 shows how common linings are classified. Linings of compacted clays are in widest use to reduce and control seepage because of their low cost, flexible nature and longest record of successful performance (Kays, 1977).

a. DIFFERENTIAL SETTLEMENT

i. Causes and description

Potential failure of the liner due to differential settlement would be similar to potential failure of the cap. However, the settlement of concern with regard to the liner is only that contributed by compression of the subsoil foundation materials. The foundation soils may be expected to be less uniform than the tailings, both in

Table 4. Lining classifications. (After Kays, 1977, p. 3.)

Flexible	Rigid	Miscellaneous
Plastics	Gunite	Bentonite clays
Elastomers	Concrete	Chemical treatments
Asphalt panels	Steel	Waterborne treatments
Compacted soils	Asphalt concrete	Combinations
	Soil cement	

Impervious	Semiimpervious
Plastics	Compacted soils
Elastomers	Gunite
Asphalt panels	Concrete
Steel	Asphalt concrete
	Soil cement
	Bentonite clays
	Chemical treatments
	Waterborne treatments

Continuous	Noncontinuous
Plastics	Compacted soils
Elastomers	Gunite
Asphalt panels	Concrete
Steel	Asphalt concrete
	Soil cement
	Bentonite clays
	Chemical treatments
	Waterborne treatments

material properties and in thickness. Also, strains within the tailings would distribute the displacement at the surface of the tailings, making the differential settlement somewhat more uniform in the cap. Consequently, there would be a greater likelihood for a shear displacement type of failure in the liner than in the cap (Fig. 1).

One particular concern in some areas may be that problems due to collapsing soils may be accentuated if the liner fails. Thus, whereas differential settlement would usually be a continuously occurring process, sudden differential settlement may result if collapsing soils are present. The sudden differential settlement may occur at almost any time after abandonment or during construction.

Swelling of expansive clays or clayshales under the impoundment may also contribute to differential movement. If the entire site is underlain by expansive soil the heave may be fairly uniform and may not be of much concern. Also, in that case the soil would probably be sufficiently impermeable so as not to require that a liner be placed. If, however, discontinuities exist, such as contact zones between different dipping strata, some areas may heave and others may not. If heave is of a magnitude that could decrease the effectiveness of the liner, special design precautions may be necessary. It should be noted that seepage from the tailings is not necessary to cause swelling of expansive subsoils. The elimination of evapotranspiration from the surface and diffusion of water from greater depths may cause heaving.

In general, the potential for differential settlement to affect the liner would depend almost entirely on the nature of the foundation subsoils and the degree of pretreatment prior to construction of the liner.

ii. Interaction with other failure mechanisms

Other failure mechanisms that may influence differential settlement of the subsoil foundation material are mainly those that may contribute water to the underlying soil. As such, any failure of the liner that may result in excessive seepage could contribute to the occurrence of differential settlement. However, not all soils exhibit settlement due to wetting and in some instances seepage may not contribute to settlement.

iii. Methods of prediction

Methods of predicting the amount of differential settlement to be expected in the liner would follow similar analyses as discussed for predicting differential settlement of the cap. It should be emphasized that accurate prediction of differential settlement is highly dependent upon an adequate subsoil investigation and soil testing program during the design phase of the impoundment. Particular attention should be paid to detection of the existence of collapsing and compressible soils and irregular subsoil profiles.

iv. Likelihood of failure

The likelihood of occurrence for differential settlement is very site specific. In general, the likelihood of differential settlement would increase as the depth to bedrock increases. Irregularity of the soil profile and erratic material properties will increase the likelihood of failure.

As for the cap, the liner will undoubtedly be designed to accommodate differential settlements of a magnitude predicted during the investigation. Consequently, the likelihood of failure will depend on the degree of conservativeness employed in the design and the variability of the subsurface soils and rocks.

Depending on the site characteristics, the likelihood of failure could range from a very low value up to a value reflecting almost certainty.

v. Magnitude of release

The release of radioactivity through a liner would be in the form of dissolved radioactive materials in seepage water.

Prediction of the magnitude of release through a cracked liner is difficult. Attempts to bound the quantity of flow through the crack can be done utilizing Darcy's law and flow net analyses (Cedergren, 1977; McWhorter and Sunada, 1977). An example of computations that may indicate the general magnitudes of seepage rates is shown in Example 2. On the basis of the computations shown therein it is expected that the magnitude of release would not be great. Even if the degree of cracking is an order of magnitude larger than that assumed in the example, it is expected that the release would fall within the lowest category of magnitude that is used.

Example 2. For purposes of computation it will be assumed that the crack formation would be of the general nature as shown in Fig. 9a. A square area of width L may be analyzed. Assuming that the mid-plane between cracks is a plane of symmetry would make that plane a flow boundary. It has been assumed that vertical saturated laminar flow through the tailings and the crack will take place, that the crack has filled with tailings, that the liner is impermeable and that the pore pressure is zero in the substratum. It is expected that these assumptions will provide a reasonable upper limit of leakage.

Application of Darcy's law to the situation depicted in Fig. 9b yields the following equation for leakage, q .

$$q = 2wk \frac{(1+p)L^2}{(2w+pL)}$$

where

$p = \frac{h_L}{h_T}$,

k = *the permeability of the tailings, and*

other terms are defined in Fig. 9.

Using a crack width of 0.4 ft in 2000 ft as computed in Example 1, a value of 0.05 for p *and a permeability of* $3x10^{-5}$ *ft/sec* (10^{-3} *cm/sec) for the tailings yields a leakage rate of* $2.5x10^{-7}$ ft^3*/sec per square foot of area. This rate of leakage is small relative to typical seepage rates that would occur through a foundation material without a liner (i.e., the magnitude is small).*

In the case of a shear type displacement the magnitude of release would depend on the magnitude of displacement and the material used for the liner. If a synthetic material is used, and the magnitude of differential settlement does not cause rupture of the liner, obviously no release would occur. However, if the magnitude of differential settlement is greater than what could be tolerated by the synthetic liner, the magnitude of release would be a function of the area exposed by failure of the liner.

In a natural material, such as compacted bentonite, the likelihood of a tension crack failure would not be great because of plastic flow within the clay. If a shear type displacement occurs of a magnitude greater than the thickness of the liner, the magnitude of release would be a function of the area of the discontinuity that is exposed (similar

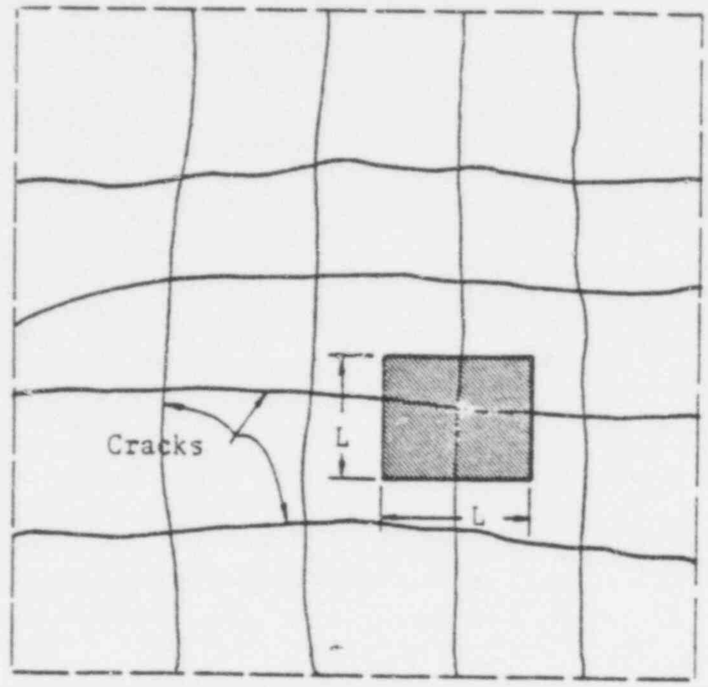

Fig. 9a. Crack pattern assumed in liner.

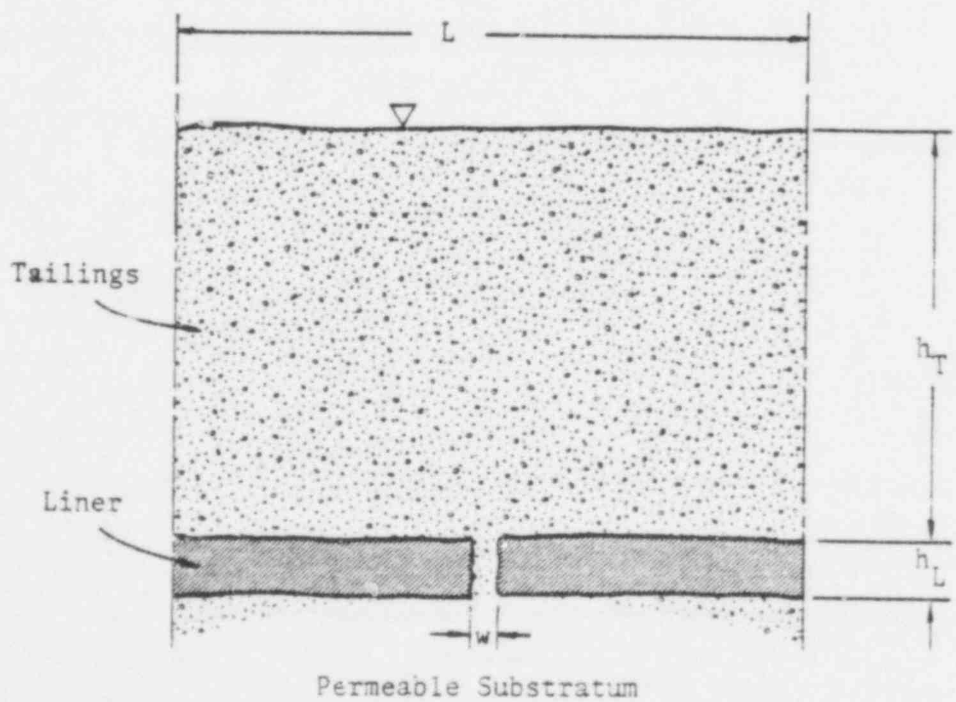

Fig. 9b. Cross section analyzed for seepage.

to Example 2). On the other hand, complete disruption of the liner may not occur but the effective thicknes. of the liner may decrease in the zone of differential settlement. In tha case, the quantity of seepage through the liner would be computed on the basis of Darcy's law for the thinner zone and compared to the release that would occur for the situation where no liner existed. In most cases the magnitude of release is not expected to be large.

vi. Site considerations

Site considerations that will influence the differential settlement are similar to those discussed for differential settlement of the cap. For impoundments of low height, only the subsoil conditions need to be considered. For higher impoundments the bedrock may need to be considered as well.

As discussed previously, differential settlement may be expected to be larger where the subsoil profile is irregular or where large differences in thickness and material types occur across the site.

vii. Design considerations

If differential settlement is potentially problematical, design recommendations can be made to either remove or stabilize the subsoil prior to construction of the impoundment. Because the liner must be placed prior to deposition of the tailings, the utilization of operational policies to load potentially troublesome areas early in the project is of little benefit.

The thickness of the liner and the nature of the material to be used must be selected so that it will not leak excessively under settlements of the magnitude to be expected. The thickness of the liner should be such that after differential settlement has occurred, a

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IMAGE EVALUATION
TEST TARGET (MT-3)

6″

IMAGE EVALUATION
TEST TARGET (MT-3)

6″

sufficient thickness would still exist to minimize the seepage to an acceptable value. In situations where differential settlement may exist, liners constructed of natural materials such as compacted bentonite have the advantage over synthetic liners in that they are "self-healing" and are not expected to fail if the thickness is greater than the shear displacement encountered. In addition, natural materials may flow laterally, under the vertical stresses imposed by the weight of the tailings above them. Tension cracks would therefore not occur in natural material liners due to uniformly distributed differential settlements. Clays of higher plasticity have both the advantage of being capable of flowing and also of being less permeable than less plastic clays.

viii. Monitoring, maintenance and remedial measures

Excessive seepage through a liner may be monitored through the use of infrared photography if changes in the vegetation are influenced by the seepage. If the seepage enters the groundwater system, the presence of radionuclides may be monitored by water quality sampling from test wells in the area. If, however, the seepage does not enter the groundwater system and is confined to the soil and rock below the impoundment, little can be done to detect excessive seepage.

Differential movements in the liners can be detected by the use of settlement points or markers to observe the elevation of the liner. Some electronic instrumentation is available in the form of LVDT's or soil strain gauges (e.g., Bison Instruments, Minneapolis, Minnesota). The use of electronic instrumentation, however, requires the presence of personnel and may not be considered reliable for the long time periods being considered herein. Also differential movements are of little concern unless seepage occurs. Therefore, monitoring schemes that do not monitor seepage would be of little interest.

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Very little maintenance of the liner can be accomplished after deposition of tailings. Remedial measures to correct failure of the liner due to differential settlements are virtually impossible. If the zone of differential displacement can be identified fairly acurately, some grouting with chemical or cement grout may be possible. The use of cement grout may be restricted, however, in the presence of high concentration on sulfuric acid because of potential sulfate attack on the cement.

ix. Time dependence

Because differential settlement of the liner is due only to compression of the foundation materials, consolidation will occur faster than for the cap. If large deposits of soft, relatively impermeable foundation materials (i.e., clay) exist, consolidation may require up to about 100 years to be completed. However, for sandy clays or silts primary consolidation will occur in only a few years after abandonment. If the subsoils are very permeable, consolidation may have been completed by the end of tailings placement. Secondary consolidation and creep, however, may continue for considerably longer periods of time. If the deformation is due to creep of foundation materials or potential shear instability of the foundation materials, accelerated movements may occur even thousands of years after abandonment. In those cases where creep failure could occur, the impoundment could be redesigned to reduce the stresses in the foundation material. Differential movement due to swelling of expansive clays may require considerably longer periods of time. In that case, however, it is expected that differential movements would be completed within the short long-term period.

If collapsing soils are present, differential settlements may occur suddenly at almost any time within the lifetime of the impoundment. If the liner fails due to other mechanisms, the introduction of large quantities of seepage into the collapsing soils may result in an accelerated collapse of the soil, and more rapid differential settlements occurring at that time.

For long long-term considerations, it is expected that displacements will have stopped. After several thousand years have elapsed it is not expected that much water would remain in the tailings and seepage would be of little concern.

b. SUBSIDENCE OF SUBSOIL AND ROCK

i. Cause and description

If localized subsidence of subsurface soil or rock below a tailings impoundment occurs, displacements may be induced that could cause failure of the liner. Subsidence at any site could develop from many causes, some of which are listed in Table 5.

ii. Interaction with other failure mechanisms

Subsidence may contribute to cracking and failure of the liner if differential settlement is relatively large and occurs across a short distance. Subsidence may also contribute to cracking and failure of the dam, decant lines, underdrains, etc. Subsidence can also change the drainage features on the surface of the impoundment. Cracking of the liner and increased seepage may reduce the shear strength of foundation material. This can result in stability problems for the impoundment.

Failure of the liner due to other factors, such as horizontal movement along a fault, differential settlement, stretching or malfunction of the liner itself could cause increased seepage which in turn could create subsidence. O a small crack forms, water moving

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through the crack (or cracks) may cause a further increase in seepage by enlarging the crack and subjecting the foundation material to wetting and erosion.

iii. Methods of prediction

The possibility of subsidence is discussed in textbooks on Soil Mechanis and Geology (e.g., Terzaghi and Peck, 1967; Mitchell, 1976; Sowers and Sowers, 1970; Bureau of Reclamation, 1974; Thornbury, 1969). However, methods to predict the amount and rate of subsidence from subterranean voids are unavailable. Methods to predict amount and rate of settlement of subsoil deposits from loading or removing water, oil or gas are also discussed in most textbooks in Soil Mechar s. Information about underlying geologic formations can suggest the chance for subsidence due to solution cavities. Knowledge of past underground mining activity in the site area could be used to predict potential for collapse due to loading the surface with impounded tailings.

Methods to predict amount and rate of movement from collapsing and swelling soils are in the process of development and only provide i-cations of the potential for collapse or swell. The same is true of tests on dispersive clays. Subsidence from earthquakes will be discussed at a later point.

iv. Likelihood

Localized subsidence, as differentiated from differential settlement, that could cause failure of the liner (or embankment) will be very site and design specific. It will depend to a large extent on the foundation rocks and soils, the size and depth of the tailing impoundment and the nature of the tailings. The design of a lin to accommodate differential settlement caused by subsidence was discussed in the previous section.

v. Magnitude of release

The potential magnitude of movement of radioactive materials as a result of failure of the liner by subsidence is very site specific. The amount of release will be limited by a number of factors such as the location of the failure, the nature of the tailings, the backup control measures (e.g., underdrains below the liner to collect seepage), the permeability and types of natural soils and rocks below the liner, or the type of subsidence (total collapse of a large cavern forming a sinkhole versus uniform surface settlements).

vi. Site considerations

Site considerations that will influence subsidence are primarily the subsurface soil and rock profile, level and change in level of the groundwater surface, the area and depth of the impoundments, and most conditions presented in Table 5.

vii. Design consideration

Extensive search should be made for man-made causes for subsidence such as old mine shafts, wells and open drill holes. If they are found, their effect and possible sealing should be studied. Economic ore deposits under or surrounding the proposed site which might be mined at some later date should be investigated. The importance of adequate foundation investigation is emphasized since approximately 40 percent of all water retaining earthfill dam failures are attributed to failures of the foundations (Bureau of Reclamation, 1974).

The lining is not a structural member itself and is only designed to reduce and control sewage. The foundation soils and rocks must support the impoundment regardless of whether the foundation is dry or saturated. Therefore, consideration should be given to all parts of the

Table 5. Some causes of subsidence.

1. Subterranean Voids
    a. Rock solution (limestone or dolomite areas)
        Sinkhole--vertical
        Caverns --vertical or horizontal
                --may occupy one or more levels
    b. Mines
    c. Wells and drill holes
    d. Volcanic deposits--gas vents and voids
    e. Cracks in foundation soils and rocks
2. Collapsing Soils--low-density loess upon wetting
3. Dispersive Clay
4. Piping (erosion tunnels)
5. Soft Clay Deposits
6. Organic Soil Deposits
7. Removal of Oil and Gas Deposits
8. Lowering Groundwater Level
9. Loose Sands--Earthquakes
10. Movement along Faults
11. Loading from the Impoundment Itself
12. Collapse of Underdrains, Decant Lines, etc.

impoundment, including the lining, underdrain systems, the foundations, etc. The other parts should be designed assuming that some failure of the lining has occurred and seepage is occurring.

Design of the liner itself to account for differential settlement has been discussed previously. Collection systems (i.e., underdrains) should be designed to carry excess seepage, but not soil, to collection areas.

viii. Monitoring, maintenance and remedial measures

Failure may occur suddenly as a collapse of a large underground void or occur over a relatively long period of time. Monitoring schemes that could be employed are direct observation of water levels in the tailings impoundment, flows and radioactivity of underdrains, and monitoring wells. Kays (1977) notes that detection methods for seepage are not well developed. Maintenance of the underdrain collection system may be needed as long as there is water in the impoundment. Maintenance of the natural groundwater level to prevent lowering may be effective in reducing subsidence. Remedial measures include grouting and/or pumping at locations surrounding and under the area of liner failure. Limitations to grouting would be the same as discussed previously.

ix. Time dependence

Subsidence due to collapse of underground voids is possible at any time during the short or medium long-term period. If it has not occurred within those time periods it would not be expected to occur during the long long-term period.

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c. CHEMICAL ATTACK

i. Causes and description

No liner will be completely immune from chemical attack or weathering. The causes and nature of the chemical changes would be similar to those described previously for the cap.

ii. Interaction with other failure mechanisms

A potential failure of the liner or underdrain system from any cause would contribute to potential chemical attack of the soils below. Possible changes in the subsurface material could lead to changes in permeability, strength or compressibility. Chemical attack can change the physical properties of the liner leading to a possible failure of the liner by shrinkage and/or changes in plasticity. This can lead to the development of cracks. On the other hand, the seepage of acidic water into the subsoils may cause a chemical change to clay minerals thereby decreasing the permeability. In that case the weathering would have a beneficial effect.

iii. Methods of prediction

The amount and rate of chemical attack of the liner will be dependent on the type of liner, the pH of the water in the tailings, and the temperature. If the liner is made of clay, the type of clay minerals are important. Methods for analysis of chemical weathering are discussed in Gram (1968) and Mitchell (1976). Within the present state of the art it is possible to predict the trends or effects of chemical attack on linings below saturated tailings but the accuracy of such predictions are uncertain.

iv. Likelihood

The likelihood of chemical attack of the liner would be high. A change of liner properties due to chemical changes is probable for liners containing tailings with high or low pH. The extent to which these changes would cause failure, however, will depend on the material or soil type, the thickness of the liner and the nature of the chemical change. Some liners may be very subject to chemical attacks under conditions occurring in an impoundment, whereas in others the effects may not be detrimental.

v. Magnitude of release

The magnitude of release will depend on the nature of the failure. If the chemical changes result only in a change in permeability, the seepage rates could be computed on the basis of Darcy's law and would be a function of the change in permeability. If the chemical change destroys the liner the amount of release would depend on the size of the failure. The magnitude could vary from 0 (no change or decrease in permeability) to 100% of release by seepage (complete disintegration of the liner).

Since repair to the liner would be difficult to accomplish, the negative utility factor of a significant failure of the liner would be high.

vi. Site considerations

The site conditions that will influence chemical attack of the liner are primarily the soil types available for construction, and the climatic conditions. The permeability and types of subsurface soils would also be important, since failure of the liner could subject them to chemical weathering. A site underlaid by limestone, for example, would be very subject to chemical attack of the seepage was acidic.

vii. Design considerations

If chemical weathering of the liner is considered to be a problem, recommendations can be made in the design to neutralize the tailings or use a thicker liner of imported materials more capable of withstanding the chemical attack. The underdrain system should also be designed to withstand chemical attack.

viii. Monitoring, maintenance and remedial measures

Maintenance and remedial measures are very hard to accomplish after covering the liner with tailings. If an underdrain exists, then its outlets should be maintained.

Monitoring techniques would be the same as previously described for differential settlement failure of the liner.

ix. Time dependence

The time needed for chemical attack of a liner (or any material) varies greatly. The thinner and more permeable that the liner is, the warmer the climate is, and the further the pH of the water is away from neutral, the faster the chemical attack will take place. In general, the time dependence would be the same as for chemical changes of the cap. It would be expected that all chemical changes would be completed within the short or medium long-term changes.

d. PHYSICAL PENETRATION

i. Causes and description

Any factor which increases the seepage rates above the design rates can be defined as a failure. Physical penetration or mechanical damage to the liner can occur in a number of ways and may depend on the type of liner.

Synthetic liners have been found to generally exhibit some defects. In the manufacture of the lining, defects such as pin holes or thin areas may occur. The installation of the lining requires a complex process. Physical penetration or mechanical damage during shipping, unpacking or handling may occur. The lining is joined together in the field under variable conditions (heat, wind, dust, moisture, inexperienced people and subgrade imperfections) and defects in the joints can easily occur.

Natural liners of compacted soils are variable in soil properties and permeability. A number of problems have occurred when installing and using compacted soils. Many of the problems can be traced to improper soil selection and installation. Therefore, soil linings must be controlled by proper choice of soil, moisture content, compaction and type of compaction equipment. Various causes of physical penetration are listed in Table 6.

Physical penetration would be most critical during construction of the liner and during initial deposition of tailings. After covering with the cap, the liner would be fairly well protected except for burrowing animals, surface drilling, aging and shear stresses induced by settlement of the tailings on foundation soils (Kays, 1977).

ii. Interaction with other failure mechanisms

The interaction of physical penetration with other failure mechanisms is low except to the extent that excessive seepage may trigger the other failures. Once the liner has been covered by tailings it is doubtful that further physical penetration of the nature shown in Table 6 would occur.

Table 6. Principal physical penetration mechanisms.

A. Voids--water pressure forces the lining into the void and stretching may cause failure.

B. Wave action--waves may cause floating objects, such as trees and ice, to penetrate the membrane and erosion of soil liners.

C. Animals--rodents and large animals may walk on, burrow under or eat through the liner.

D. Weed growths--weeds, above or below the liner, may get started with roots getting through at pin holes, seams, or other damaged areas.

E. Maintenance cleaning--workmen and equipment may penetrate the liner.

F. Sharp objects below liner--rocks, rock outcrops, differential settlement, cracks, or breaks in pipes in the underdrain may penetrate the liner.

G. Reverse hydrostatic uplift--if water pressure below the liner is greater than the pressure above, movement may occur.

H. Tension failure--tailings deposited on stee slopes tend to develop shear stresses and may slide. The movement of the tailings may pull the liner along with it or fail the liner.

I. Vandalism

J. Others--Murphy's Law: If there is a lining, it probably will leak!

iii. Methods of prediction

Physical penetration of the liner is linked to design, construction and outside causes. In past failures of liners, the question has arisen of which came first, a lining failure or foundation failure. A weak design assumption, construction methods, or outside forces will create situations where failure will most easily occur.

iv. Likelihood

The likelihood of physical penetration is great. In order to design a liner that is capable of performing well after physical penetration, materials that are "self-healing" such as clay materials with filters and underdrains would appear desirable.

v. Magnitude of release

The potential magnitude of release due to a failure of the liner will be influenced by the interaction between the liner, the underdrain system, the tailings, and the foundation.

After the impoundment is completed, there is no practical way to identify the locations of leaks. Increased seepage rates can be estimated on the basis of failure area or size of cracks expected. A general formula that defines leakage is given as:

$$Q = f(A_1, H, k_1, t) + g(A_2, H, k_2, t)$$

where

Q = leakage

A_1 = lining area

H = maximum water depth

k_1 = permeability of liner

A_2 = area of liner failure

k_2 = effective permeability of system

t = time.

The first set of terms defines seepage without penetration and the second set of terms defines excess seepage due to the penetration of the liner.

vi. Site considerations

Site considerations that will influence physical penetration depends on the type of liner, the foundation profile, the thickness of a prepared subgrade, the material used in the subgrade (e.g., the presence of rocks would be undesirable), the presence of structures, such as decant towers, spillways, inlets or drains, that may fail at the transition zone with the structure and lining, the presence of voids below the liner, and climate.

vii. Design considerations

To reduce the likelihood of penetration, rock outcrops can be removed or cushioned with a prepared subgrade, large sharp rocks should be removed, and voids below the liner should be filled before the liner is placed. If differential settlement occurs below linings near structures, the lining may tear at the structure-liner connection. Therefore, differential settlement between structures and the liner should be controlled. However, some physical penetration must be expected and underdrains can be provided to handle this flow. The best design will assume that some seepage will occur and will include backup systems to control it. The thickness of the lining may be increased to reduce the probability of physical penetration and filters may be used between the lining and drains.

The site should be selected and the lining designed so that if leakage does increase, soil will not be removed, additional settlement will not occur, side slopes will not sluff or fail, or cracks will not open.

viii. Monitoring, maintenance and remedial measures

Any penetration during construction should be immediately repaired. Maintenance and remedial measures are very hard to accomplish after covering the liner with tailings. As a tailing impoundment is being filled, close inspection of the liner above the water level is needed.

Monitoring schemes would be the same as for other liner failure mechanisms.

ix. Time dependence

During construction and filling physical penetration is most likely to occur. After abandonment burrowing animals, future drilling operations, physical movement of decant lines or other substructures could cause penetration. The chance for penetration would increase with time, but remains low.

3. EMBANKMENT

a. DIFFERENTIAL SETTLEMENT

i. Causes and description

Differential settlement of the embankment of a tailings impoundment may occur due to compression of either the foundation soil or the soil in the embankment itself. Differential settlement within the foundation soil will result from either uneven subsurface profiles or from variations in material properties.

The material from which the embankment itself is constructed would be of fairly uniform characteristics. However, differential settlement can occur within the embankment material if its height varies due to an uneven bedrock. For example, if the abutments for the embankment consist of steep or irregular bedrock such as a valley wall or a canyon wall, differential movements would be likely.

Damage to the embankment from differential settlement usually take the form of cracking of the embankment. In some instances the cracks may be visible on the surface, but in other cases they may exist in the lower regions of the embankment and may not be visible.

In water impoundment dams, cracking is of particular concern because, if it leads to piping, total failure of the dam can result. In tailings dams the tailings themselves provide a filter over the crack and protect against piping. However, for uranium tailings dams, dissolved radionuclides can escape through the cracks. In addition, if the cracks continue to the surface, surface water erosion may be increased around the cracks.

Experience has indicated that cracking is more pronounced in dams that are constructed in areas having rainfall less than about 15 inches (Sherard, 1973). The main reason for the greater incidence of cracking in arid regions is because the borrow of which the embankment is constructed is found in a dry condition and is hard to wet uniformly and adequately. As a result the embankment is nonuniform and brittle. The embankment soil cannot flow or creep as differential settlement takes place. In addition, the foundation materials in these areas are usually partially saturated and subject to settlement when wetted. Cracking of the embankment can also be caused by lateral displacement of the foundation soils under the weight of the embankment. Such cracks, however, are usually longitudinal cracks running parallel to the axis of the embankment. Those cracks may influence the stability of the dam, but contribute little to seepage.

ii. Interaction with other failure mechanisms

Excessive seepage through the liner could induce settlement in foundation soils that were partially saturated initially. Also, erosion and gullying could cause stress concentrations in zones of the embankment that could then cause cracks to form. The latter effect, however, would be small.

If precipitation enters cracks pore water pressure can be induced that could reduce the stability of the embankment slopes.

iii. Methods of prediction

Differential settlement due to consolidation of foundation materials can be predicted using one-dimensional consolidation theory. This has been discussed previously. Similar analyses can be applied to compute the vertical compression of the embankment. However, cracking in the embankment can occur even for relatively low values of crest settlement.

Attempts have also been made to predict displacement within embankments both vertically and horizontally by the use of the finite element method (Clough and Woodward, 1967; Lee and Shen, 1969). Such analyses provide the capability of determining displacements and stresses within the embankment for various material constitutive relationships. They do not however, predict cracking within the dam. The magnitude and extent of cracking must be inferred from the computed displacements and engineering judgement regarding the nature of the embankment material.

Several case histories and some statistical results indicating the cracking of dams as related to construction practices and material properties are discussed in Sherard (1973). At the present time no mathematical model for prediction of locations and magnitudes of cracks exists.

iv. Likelihood of failure

Because most uranium tailings impoundments are constructed in areas of relatively low annual rainfall, the likelihood that cracking will occur is relatively high. On the other hand, the extent to which such cracks would be continuous across the width of the embankment is uncertain. Furthermore, the deposition of coarse tailings near the face of the embankment helps to minimize the effect of cracking. Thus, although cracking will probably occur, the likelihood of release is lower. The likelihood of failure may be classified as moderate.

v. Magnitude of release

Dissolved radionuclides will be released along with any seepage that would exit from cracks in the embankment. Some small amount of suspended tailings could also be carried out. If the cracking is of a high frequency and the cracks are relatively large, considerable amounts of seepage may be released. On the other hand, the tailings may fill the cracks and decrease the amount of seepage loss. Piping through cracks would not be expected to occur. Therefore it is possible that the introduction of water into cracks that do form may soften the material around the cracks, thereby providing a self-healing mechanism to close the cracks.

The computation of a magnitude of release from cracking is difficult and largely subjective. It is expected that the magnitude of release through cracking would, in most cases, be low.

vi. Site consideration

Differential settlement may be expected to be of concern over sites in which deep deposits of soft or potentially collapsing foundation soils exist. Sites in which the embankment would be constructed between steep bedrock abutments, particularly if steps or benches in the bedrock exist, are more contributory to cracking. A major site consideration would be the existence of adequate borrow material to construct an embankment that is not particularly susceptible to cracking. Desirable characteristics of the borrow material will be discussed below with regard to design consideration.

An additional site consideration is climate. It was noted earlier that cracking is of more concern in areas where annual rainfall is low.

vii. Design considerations

Cracking of embankments can be accentuated by the presence of zones of radically different materials within the embankment. Adequate transition zones should be provided between the core and shell of zone embankments. In homogeneous embankments this is of less concern.

The types of materials used for construction may also influence cracking. Residual soils and soils which assume a brittle nature after construction exhibit greater amounts of cracking (Sherard, 1973).

During construction the embankments should be compacted at water contents at or above the optimum water content for standard AASHTO compaction. Care should be taken to insure uniform mixing of water with the embankment materials. Uneven compaction such as may occur due to the use of hand compaction around outlet structures, such as decant lines or around abutments, can enhance differential settlement.

viii. Monitoring, maintenance and remedial measures

Monitoring of differential settlements of the crest of the embankment can be accomplished through the use of surveying techniques, monuments, and settlement points. However, cracking may result even for relatively small amounts of differential settlement, particularly if brittle materials are used for construction. Release can be monitored by the detection of radioactive materials downstream from the embankment. The use of infrared photography can also be used to detect changes in vegetation or environment.

Maintanance procedures to decrease or control differential settlements are virtually impossible. However, if cracks appear within the embankment, attempts would be made to keep the cracks closed for stability reasons.

If cracks do occur, remedial measures may consist of grouting the cracks with either cement or chemical grout. If cement grout is used, caution should be exercised to minimize sulfate attack on the cement either from the presence of sulfate in the tailing waters or in the soil. An alternative to grouting is the filling of cracks with sands and dry bentonite mixtures or to fill the cracks with slurry. Slurry method is used generally only where cracks are very wide and deep. It has been used successfully in many instances by the Soil Conservation Service (Sherard, 1973). However, some shrinkage of the slurry could occur when it dries.

If the cracks are large, it may be necessary to remove the material around the cracks in a trench and recompact material into the trench.

ix. Time dependence

Differential settlements of the embankment and foundation soils would be expected to be completed within a short long-term period. Experience has shown that if cracking is to be a problem it usually occurs earlier in the lifetime of the dam, within approximately ten years. In medium and long-term time periods creep may cause the soil to flow and close the cracking of the embankment. Also, after the tailings have dried, the seepage through cracks would not be of concern. Stability of the embankment, of course, would continue to be a potential problem.

b. SLOPE FAILURE

i. Causes and descriptions

Slope failure of the embankment can occur as a result of several different factors. During operation of the tailings impoundment the primary cause of slope failures is the existence of a high phreatic line. After abandonment, however, the phreatic line would become lower. The exception to that would be if water could collect over the impoundment and seep into the tailings.

Another potential cause of slope failure would be the buildup of pore water pressures by precipitation entering cracks in the embankment.

A third cause of potential slope instability would be creep failure occurring within the slopes (Nelson and Thompson, 1977). Creep failure results from continued deterioration of the soil strength as a result of creep strains that occur within the material.

Potential slope instability can also be caused by seismic events. Seismicity is discussed in section B1 below.

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ii. Interaction with other failure modes

Embankments should be designed to have flat slopes in order to minimize wind erosions. Flat slopes enhance slope stability as well and decrease the likelihood of occurrence of a creep failure within the embankment.

Differential settlement that could cause cracking of the embankment can promote progressive failure of embankments by the introduction of water into cracks.

Wind and water erosion can remove materials near the toe causing an increase in the steepness of the slope. If erosional processes continue to remove material, the stresses within the embankment can be increased to the point of failure.

In general, factors that decrease the probability of other failure mechanisms also enhance the stability of the embankment. Failure mechanisms that can remove material and cause changes in the geometry of the embankment can contribute to slope instability.

iii Methods of analysis

The stability of the embankment after construction can be analyzed to determine the factor of safety by a variety of methods of analysis. These methods of analysis are discussed in several books and publications (for example, Lambe and Whitman, 1969; Janbu, 1973; Sherard et al., 1963; Bureau of Reclamation, 1974).

To aid in predicting the location of the phreatic line for use in analyzing slope stability, finite element models are available (for example, Kealy and Busch, 1971). The data required, however, for prediction of the phreatic line are difficult to determine with accuracy for the in situ condition. For that reason, the application of these models is somewhat limited unless field data is available for comparison.

The models are useful, however, in predicting changes that may occur as a result of changes in embankment height, water level or other alternatives.

A finite element method has been used for prediction of creep failure of slopes (Nelson and Thompson, 1977). This theory, however, is relatively new and needs considerable experimental data for verification.

The methods of analysis discussed above can all be used to predict stability of slopes, but they do not predict the extent of damage that will occur if slope instability does result. They may, however, indicate the general region within which failure will occur (i.e., the critical failure surface). On that basis, the amount of material that may be influenced can be estimated.

iv. Likelihood of failure

The likelihood of a slope failure occurring due to a high phreatic line would be very small provided that the embankment was kept drained. The likelihood of failure resulting from water entering the cracks would depend on the extent of maintenance on the site. However, even with minimal maintenance the likelihood of failure would be low to moderate. The failure of the embankment due to erosion at the toe is site specific. In that case likelihood of failure would be slightly less than the likelihood of occurrence of erosion.

The likelihood of failure occurring as a result of creep depends on the magnitude of stresses existing within the embankment. If stresses within the embankment are everywhere less than the residual strength of the soil, the probability of failure due to creep is very small. If, however, stresses in particular zones exceed the residual strength of the soil, progressive creep failure may occur after sufficiently long periods of time (Nelson and Thompson, 1977). If the residual strength of the embankment soil is used for design the likelihood of creep failure would be minimal.

v. Magnitude of release

The magnitude of release resulting from slope failure of the face of the embankment would not be large. It is doubtful whether a slope failure would actually intersect the tailings except in the case of liquefaction during an earthquake. If instability of the embankment does occur, the face would slump and the unstable mass would assume a more stable configuration. The release would be small. If, however, further progressive failure can continue as a result of geometry changes caused by the initial slope failure. a larger release may be possible.

One instance in which relatively large amounts of radionuclides could be released would be if the tailings are saturated and the slope failure intersects them. Under those situation, liquefaction of the tailings can occur. This phenomenon would be similar to failure by overtopping of the embankment. Such failures have been observed to continue over several days. The flowing tailings can travel for distances on the order of miles depending on the slope of the downstream terrain.

vi. Site considerations

The embankment may be designed and constructed around site features. Therefore, site considerations are usually not a limiting factor for embankment design, provided that construction material is available in the area.

If the foundation soils are weak, the stability of the embankment would be decreased unless the subsoils are stabilized. Another site consideration that may influence slope stability is the ability to route flood waters around the embankment so as not to cause erosion of the toe. Also, embankments should not be constructed over faults.

vii. Design considerations

Design that is conducive to stability of the embankment utilizes flat slopes and the use of berms at the toe if necessary. From the standpoint of long-term creep failure the slopes should be designed such that the residual shear strength of the embankment soil is not exceeded. Although stresses in excess of the residual strength may be tolerated in some small zones, the embankment should not be designed on the basis of the peak strength of the soil.

viii. Monitoring, maintenance and remedial measures

Monitoring schemes to monitor slope movement would consist primarily of methods that could detect lateral movement on and around the slopes. The placement of monuments and aerial photography may be used in some cases to detect displacements and potential instability. The observation of cracks on the embankment would indicate the onset of progressive failure.

Inclinometers and other electronic instrumentation is available to detect slope movements and strains within the soil. Electronic instrumentation may require some maintenance over long time periods. However, slope indicators, for example, require only that a tube be placed in the embankment. In order to observe movements an instrument would be lowered into the tube. Maintenance required on such a system is minimal.

Maintenance required on the slope to prevent instability depends on the slope cover. Vegetation or coarse granular material should be placed on the slopes. Some maintenance to minimize cracking of the slope may be required.

If slope failures do occur, remedial measures are possible in the form of regrading and restabilization of th slide area. This could include the placement of berms and replacement of cover over the slide area to minimize release.

ix. Time dependence

If slope instability has not occurred by the time of abandonment of the si'r, it is not expected that slope instabil'ty would occur within the short long-term period. However, for medium and long long-term considerations, creep displacement can induce failure. Although creep may occur for time periods up to the long long-term (100,000 years), it is expected that other potential failure modes would be of more concern within that time period.

c. GULLYING

i. Causes and description

The cause of gully formation on the embankment of a tailing impoundment will be a combination of steepness of slope, concentration of runoff and erodibility of the soil. The most likely place for gullying to begin would be at the crest of the slope. At this point runoff from the cap would increase in erosive power as the gradient steepens. Gullying of the embankment could also be the result of the headward migration of gullies started from outside the impoundment. This may occur if runoff is concentrated around the embankment into a drainage along which the gully could advance.

Gullying of the embankment could result in significant radionuclide releases. Gullies are capable of moving significant quantities of material (Mabbut, 1977; Schumm, 1977). If it is allowed to grow, a gully could advance through an embankment into the tailings.

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If the embankment is breeched, only the specific site conditions could predict the amount of tailings that would be carried downstream.

ii. Interaction with other failure mechanisms

The interaction of gullying of the embankment with other failure mechanisms is the same as for gullying of the cap. Differential settlement may cause cracking or depressions that would concentrate runoff at specific points on the embankment increasing the potential for gullying. Gully formation on the cap could concentrate runoff and cause gully formation on the embankment. The reverse is also true. Any change in climate or vegetation cover could increase runoff or decrease stability and lead to gully formation on the embankment.

If the toe of the embankment is eroded and steepening of the slope results, the risk of gully formation will increase. The interaction of gullying with slope stability must also be considered. If enough material is removed by gullying, local slope instability could be created. This could remain a local problem or it could spread throughout the entire embankment.

iii. Methods of prediction

As discussed previously, there is no mathematical model that predicts gully formation. Schumm (1977) and Brice (1966) have suggested that by study of natural gully formation in a specific area some indication of a stable versus an unstable condition can be established. In addition, evaluation of calculated soil losses using the Universal Soil Loss Equation for different conditions could provide some insight into the potential for gullying.

iv. Likelihood

The likelihood of gully formation on the embankment will be higher than for the cap because of the steeper slopes. In order to adequately predict the likelihood of gullying for short long-term periods, the site and design plan must be evaluated. This evaluation would consider

- Slope length and gradient.
- Vegetation cover or other stabilization measures.
- Embankment soil erodibility.
- Climate.
- Occurrence of gullying in similar natural settings in the immediate vicinity.

Over longer periods of time the likelihood for gully formation on the embankment will be relatively high.

v. Magnitude of release

The magnitude of release will be a function of the size and extent of the gully or gullies formed. Small gullies that are either repaired or reach a state of equilibrium might release little, if any, radionuclides. At the other extreme an extensive gully that continued to grow, cutting through the embankment and into the tailings, may result in extensive instability of the embankment. Extensive amounts of radioactive material could be released either by mobilization of tailings or by radon emanation. The magnitude of release is site specific and depends on the particular design of the impoundment. It can range from a low value to a high value depending upon the specific disposal plan.

vi. Site considerations

Site considerations that will influence this potential failure mechanism are similar to those discussed for gully formation on the cap. In addition, consideration should be given to the area below the embankment. Existing slope, soil, and runoff conditions for that area should be analyzed for potential gully formation that could advance headward into the embankment. Also, changes that could occur in the area as a result of the impoundment should be analyzed.

vii. Design considerations

The design of gently embankment slopes and the avoidance of sharp breaks in slope between the cap and embankment are probably the most important design considerations for the reduction of gully potential. Grading to avoid concentration of runoff and construction of adequate diversion structures to eliminate off-site runoff are also important. If steep slopes are necessary because of topography or other design constraints, shortening of drainage paths by the use of terraces and diversion of runoff off the slope should be considered. Vegetation or the inclusion of stones or cobbles in the soil matrix of the embankment could increase stability. Highly erodible soil should be avoided.

viii. Monitoring, maintenance and remedial measures

The conditions discussed for gully formation on the cap are applicable here also. However, the gully formation on the embankment is more likely, is potentially a faster process, and the consequences are more severe. Therefore, increased monitoring, maintenance, and remedial efforts may be appropriate. If maintenance is a continual necessity the basic design should be analyzed for deficiencies.

This is an important monitoring consideration. All failures or problems should be analyzed to determine if design inadequacies are causing the problem. Small failures or problems may be a forewarning of the presence of major difficulties.

ix. Time dependence

Gullies can form at any time during the history of the impoundment. There is no basic time dependence except that relative to likelihood. The longer the time span the greater is the chance that gullies will form. This is true for a non-changing environment, but even more important if climatic changes can be introduced with time. A change in climate can cause a stable situation to become unstable. The likelihood of climatic changes or variations should be considered to the extent that they could influence a particular disposal plan.

d. WATER SHEET EROSION

i. Causes and description

The general causes of water sheet erosion of the embankment are similar to those discussed for the cap. The important difference is the slope of the embankment. On the cap, the slopes will be relatively gentle unless topographic constraints are encountered. For the embankment, however, the slopes are likely to be steeper and therefore more susceptible to water erosion. The LS factor of the USLE (Fig. 9) illustrates this point.

ii. Interaction with other failure mechanisms

Any failure mechanism that results in increased runoff or a concentration of runoff will interact to increase the water erosion potential of the embankment. These failure mechanisms include differential settlement, diversion structure failures, vegetation failure and climatic change. These factors are discussed in more detail with regard to water erosion of the cap. In addition, flood waters that come into contact with the toe of the embankment have the potential to erode and steepen the slope. If steepening occurs increased erosion is likely.

iii. Methods of prediction

The Universal Soil Loss Equation is, at present, the only method available to predict soil loss due to water erosion. The application of this predictive model is discussed previously with regard to erosion of the cap.

iv. Likelihood

The likelihood of water erosion of the embankment will be the same as for the cap. Erosion will occur continuously. The likelihood of failure will increase continuously with time reflecting the time related nature of this failure mechanism.

v. Magnitude of release

Erosion losses of soil from the embankment will increase radon emanation and gamma-ray emission to a lesser degree than with erosion of the cap. The reason for this is that the relative thickness of the embankment will be greater and the more highly radioactive slimes portion of the tailings will generally be located in the central portions of the impoundment away from the embankment. However, if erosion is allowed to progress to the point where slope instability results, the potential release would be greater, as discussed in the previous section.

vi. Site considerations

The important site considerations are the climatic conditions that relate to precipitation, and the runoff potential for upstream watersheds. These both relate to the amount of water that can come in contact with the embankment as runoff or precipitation. For long time spans the natural vegetation in the area is important because those types of vegetation will be established on the embankment.

vii. Design considerations

Embankment design to control erosion potential should take into account the slope length and steepness. The use of terraces and water diversion schemes should be considered. Soil properties and vegetative cover will influence erosion. On steep embankment slopes where vegetation is more difficult to establish, consideration should be given to using rock and gravel to serve as a filter and riprap. This type of protection can be effective against water as well as wind erosion. The shape of the slope may also be an important design element. Research done on slope shapes conclude that on concave slopes sediment loss will be lower and the erosion depth will be shallower than with either a uniform or convex slope (Meyer et al., 1969).

viii. Monitoring, maintenance and remedial measures

Monitoring measures will be the same as described for all previously discussed erosional processes. Most effective will likely be site inspections. Remote sensing techniques, such as aerial photography, both visual and infrared, could identify actual failures or provide indications of potential failures. Monitoring climatic changes could also indicate precipitation or temperature changes that may alter the erosion regime.

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Maintenance and remedial measures would consist of maintaining slope grades, vegetation cover, runoff control structures, and timely repair of any erosion failure.

ix. Time dependence

As discussed previously, there is no basic time dependency related to water erosion failure. Erosion is a continuous process occurring throughout the time spans considered.

e. WIND EROSION

i. Causes and description

Wind erosion failure of the embankment is the result of wind detachment and transport of soil materials. The process elements and description are summarized by the wind soil loss equation presented previously with regard to the cap. Wind erosion of the embankment, however, may be more severe due to steeper slopes. The actual extent of erosion potential must be analyzed for each specific site and disposal plan. Wind erosion of the embankment can be localized in "blowouts" common in beach dunes, and similar in failure extent to water formed gullies.

ii. Interaction with other failure mechanisms

If climatic conditions change resulting in higher winds than predicted, reduction or loss of vegetative cover, and/or loss of soil moisture, wind erosion will increase. If blowouts occur they have the potential to become large and interact with surface drainage, causing channeling. This could lead to enlargement by water erosion and the potential for significant losses of embankment and cap material, and tailings.

As with water erosion, any failure mechanism that increases the slope of the embankment, such as stream undercutting, increases the wind erosion potential. Furthermore, any failure that results in loss of vegetation or other protective covers (mulch, crusted surfaces, or riprap) from the embankment will interact to increase the wind erosion potential.

iii. Methods of prediction

The only method for predicting soil loss potential due to wind erosion is the wind soil loss equation discussed previously with regard to the cap.

iv. Likelihood

The likelihood that wind erosion will occur on an embankment is high and increases with time. The magnitude of release reflects the overall effect.

v. Magnitude of release

The magnitude of release due to wind erosion on the embankment will be based upon an estimate of embankment thickness lost and consequences of that loss. Losses are calculated using the wind soil loss equation already discussed. The potential soil loss and consequences of blowouts can only be subjectively evaluated.

Since calculated soil losses are the result of site specific evaluation only a general discussion of magnitude is possible. As for water erosion, wind erosion of the embankment is potentially greater than on the cap because of slope. However, this increased loss is mitigated by the greater thickness of soil in the embankment and the greater distance from the more highly radioactive slimes zone of the impoundment. In short or medium long-term periods erosion losses are expected to be relatively low. For the long long-term periods, wind erosion could remove the entire embankment and tailings pile if no maintenance is applied.

vi. Site considerations

The most important site consideration is the direction and force of prevailing winds. If possible, the impoundment should be located so as to reduce exposure of embankment slopes to prevailing winds.

vii. Design considerations

Design considerations for the control of wind erosion would be the use of gentle slopes, windbreaks, incorporation of gravel or cobbles in the soil matrix to provide armoring, and vegetation cover. In extreme cases, irrigation of the surface to control dryness or the use of chemical cementing agents may be required.

viii. Monitoring, maintenance and remedial measures

Monitoring could be accomplished by site inspections or the use of remote sensing methods to identify developing wind erosion problems or to identify changes that would predict future problems, such as loss of vegetation cover.

Maintenance and remedial measures would include repair and stabilization of blown out areas, regrading of surfaces, repair of wind control structures, and reestablishment of vegetation.

ix. Time dependence

Wind erosion is a continuous process occurring throughout the time span being considered.

f. FLOODING

i. Causes and description

The general causes and descriptions of flood failure of the embankment are the same as the cap and have been discussed in that section. The basic natural phenomena of flooding is discussed in Section B.2 of this chapter.

ii. Interaction with other failure mechanisms

Failure of the embankment due to the occurrence of floods greater than the design flood for the impoundment will be influenced by several other failure mechanisms. The failure of diversion structures, erosion or gullying of the cap, gullying of the embankment, or differential settlement in the cap or in the embankment could result in concentration of flood waters. This concentration and increase in flow could result in overtopping and washout of the embankment. Erosion or differential settlement of the cap could form impoundments of water near the crest of the embankment. Flood waters so contained could raise the phreatic surface or cause overtopping at times of abnormally heavy precipitation.

Erosion or gullying on the embankment itself, as well as differential settlement or minor slope stability problems could remove protective vegetation or riprap. In that case, flood waters coming in contact with the toe or lower slopes of the embankment could result in serious erosion and undercutting of the slope. If the slopes are steepened from this erosion, gullying or slope stability failures could be accelerated.

iii. Methods of prediction

The prediction of occurrence of a flood of magnitude greater than the design flood is discussed in section B.2. Sediment carrying capacity of streams and erosional processes have been discussed in an ASCE manual on sedimentation (ASCE, 1977). However, there is no method of analysis that can predict the amount of embankment material that may be removed during a flood. Estimates can be made from comparisons with damage to other types of embankments during flooding but the predictions will be subjective.

iv. Likelihood of occurrence

The likelihood of occurrence of a flood greater than the design flood for the impoundment is discussed with regard to the occurrence of the natural phenomena. The likelihood of occurrence for the short long-term period would be low because of design, maintenance and storm probabilities. For medium long-term considerations the likelihood of failure will be somewhat higher and will depend on the design considerations. For long long-term periods, however, it will be almost a certainty that extreme flooding would occur. If the impoundment is located in an area that is susceptible to flooding, the likelihood of failure is high.

v. Magnitude of release

The magnitude of release will be a function of the degree of damage to the embankment, the size of the flood, and the routing of the flood water. Moderate slope failure due to undercutting of the embankment could result in very little release. If flood waters are confined to flood plains and diverted away from the impoundment, a small amount of radionuclides would be released. Conversely, if large quantities of water either flow over or around the embankment, major portions of the tailings impoundment could be removed. In general, it can be assumed that any portion of the embankment that comes into contract with the flood waters will be removed. The consequences of that with regard to magnitude of release must be evaluated for each site on an individual basis.

vi. Site considerations

Location of the site with respect to upstream drainage areas, size of the catchment areas above the site, climatic conditions, and location within flood plains are the basic considerations. In general, optimum locations would be high above potential flood plains or at the top of watersheds close to drainage divides. At the latter location the catchment area and runoff potential from higher portions of the drainage would be minimized.

It may be possible to locate the site so that floods will deposit rather than remove material. Careful analysis, however, would be needed to determine their potential.

vii. Design considerations

Design consideration that could reduce the potential for flood damage to the embankment would be:

- The use of riprap or other "armor" to protect slopes against flood erosion.
- Location at sites that are not susceptible to flooding.
- The use of flood diversion structures.

viii. Monitoring, maintenance and remedial measures

Monitoring climate and precipitation/runoff could provide information about the adequacy of original flood protection design. Such monitoring would identify changing conditions or add to the existing data base used to evaluate flood probabilities. If failure due to flooding did occur it would occur suddenly. Therefore, there is no directly applicable monitoring scheme that could predict potential flood damage.

Maintenance and remedial measures would include the maintenance of vegetation or mechanical cover of the embankment and flood diversion structures. They could include design changes of diversion structures if monitoring of climate and watersheds indicate a change in flood potential.

ix. Time dependence

Although the likelihood of occurrence of a major event greater than the design magnitude is small, such an event can occur at any time during the lifetime of the impoundment. The likelihood of occurrence will increase in direct proportion to the length of time period in consideration. However, at any particular time, the probability of occurrence in the year immediately following is the same whether it be the initial one hundred or initial one thousand year period.

g. WEATHERING AND CHEMICAL ATTACK

i. Causes and description

Weathering and chemical attack of the embankment soils may alter its chemical and physical properties. These processes may be physical, chemical or biological. Changes occur because a soil is unstable if certain conditions change. Those changes may occur in temperature, pressure, chemistry or pH. Basically there are two types of weathering:

Physical weathering is a grinding or shattering action that reduces particle size, but does not change mineral composition. Physical processes include:

- loading
- temperature expansion
- crystal growth (e.g., ice and salt in cracks of rocks)
- colloid plucking (e.g., lichens curl up and dry and pull out pieces of rocks)

Generally, silt, sand and gravel sized particles are formed by physical weathering.

Chemical weathering yields an increase in bulk of rock, smaller particle size, and a more stable mineral composition. Chemical processes include:

- hydration--the chemical addition of water to minerals (this must be distinguished from the free water that causes physical weathering). An important hydration example is the decomposition of feldspar mineral in granite to form the clay mineral, kaolinite.
- oxidation--addition of oxygen ions to the minerals composing the rock. Rocks containing iron are greatly affected by oxidation.
- reduction--the opposite of oxidation. The oxygen ions are removed from the minerals in rock.
- solution by carbonation--solution of the rock material by water containing a considerable amount of carbon dioxide. This is particularly detrimental to limestone.

Generally, clay sized particles are formed by chemical weathering. Factors influencing the rate of weathering and nature of the products include climate (temperature and rainfall), time, mineralogical composition, vegetation, drainage, and bacterial activity.

Physical and chemical weathering of the embankment would be greatest near the surface. It may decrease the permeability, increase compressibility and decrease strength. On the other hand, cementation of particles may increase the strength of the embankment. Leaching and ion exchange from water flowing through the embankment may also alter the physical properties of the embankment. Weathering of illites and the release of potassium may result in cracking of the soil.

The effect of low level radiation on the exchange of capacity and behavior of clays is unknown. Physical and chemical weathering of the tailings may occur and lead to physical property changes. Chemical weathering has been discussed in more detail with regard to the cap and the liner.

ii. Interaction with other failure mechanisms

Seepage from a failure of the lining behind an embankment may contribute to increased weathering of the embankment. Weathering of the embankment may lead to changes in stability, settlements, and increased or decreased wind erosion. Stability reduction or settlement may lead to movement and cracking of the liner or cap and may cause failure of the liner or cap.

iii. Method of prediction

Some minerals are particularly susceptible to rapid weathering. Materials that commonly degrade are shales, siltstones, mudstones, feldspars, and pyrite. The long-term characteristics of weathering materials is difficult to determine, but accelerated degradation tests could indicate the general trend.

It is usually considered good engineering practice to use embankment materials with a low susceptibility to weathering.

iv. Likelihood

The likelihood of occurrence of weathering is great. However, with proper selection of embankment material the likelihood of physical weathering being of the magnitude great enough to allow major amounts of undesirable soil property changes is small.

v. Magnitude of release

The magnitude of release will depend on the changes that occur due to weathering. For example, if the permeability changes, the amount of seepage will change in proportion to the change in permeability. If the weathering contributes to slope instability the magnitude of release would be as discussed previously with regard to stability failures.

vi. Site considerations

Site conditions that will influence weathering of the embankment are primarily the soil types available at the site embankment construction, the climate and the process used to leach the uranium.

vii. Design considerations

Design considerations that will influence weathering of the embankment are the selection of soils for construction with weathering property as a criteria, testing to anticipate weathering problems and incorporation of test information into the embankment design.

viii. Monitoring, maintenance and remedial measures

Maintenance and remedial measures are not effective in reducing weathering or chemical attack. Remedial measures to correct damage would consist of the addition of material to an embankment where excess weathering has occurred. Site inspection may be the only applicable monitoring schemes applicable to the detection of weathering failures.

ix. Time dependence

The time needed for weathering of soils varies greatly. The warmer that the climate is, the more moisture that is available and

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the further the pH of the seepage is from neutral, the faster that weathering will occur. Freeze-thaw cycles will also increase the rate of physical weathering. The type of soil will greatly influence the rate of weathering. Failure of the embankment due to weathering is expected to be a long long-term phenomenon with little effect over shorter periods of time.

4. REVEGETATION

a. FIRE

i. Causes and description

Fires can be caused by either human activity or natural events. Lightning is a natural fire-causing agent and is the most probable long-term cause of fires that can affect the impoundment. Fire potential is a function of climate, thunderstorm frequency, and vegetation. The direct effect of fire is the destruction of living and dead vegetation. Indirectly, fire will affect wind erosion and water erosion potential.

ii. Interaction with other failure mechanisms

By itself, destruction of the vegetative cover may not be significant. It is significant, however, to the extent that it affects water erosion, wind erosion and runoff. Any change, removal or reduction in vegetative cover can increase erosion and runoff.

Fire destruction must be considered for the entire watershed in which an impoundment is located. Removal of vegetation by fire can affect the regional runoff, sediment yield, and flood characteristics. These changes could have significant impacts on the impoundment area.

iii. Methods of prediction

Methods exist for predicting the occurrence of fires (Deeming et al., 1977; Borrows et al., 1977). These methods analyze fire occurrence in terms of lightning risk, climate and available fuel. Agencies such as the U.S. Forest Service have gathered large quantities of data on fire potential and fire occurrence. Specific fire occurrence estimates are site specific, but can be made.

iv. Likelihood

The likelihood of fire related failure of vegetation will be high for the long periods of time being considered.

v. Magnitude of release

In order to evaluate the magnitude of release related to failure of vegetation by fire, estimates must be made of the potential erosional and flood consequences. From the estimates, the potential magnitude of release can be established. Magnitude of release by erosional mechanisms has been discussed previously.

Generally, the magnitudes of release will be small if erosion rates are low and natural revegetation can occur quickly. If the fire is followed by extreme precipitation or wind storms, however, erosion rates could be high.

vi. Site considerations

In order to reduce the potential for lightning striking the impoundment it should not be located on a high place relative to the surrounding topography. The fuel quality of native vegetation will indicate whether a site would have a high fire potential. Microclimates, such as southern versus northern exposures, will relate to moisture regimes and therefore influence fire potential. The likely path along which a fire could spread should also be considered.

vii. Design considerations

The short long-term periods of fire potential may be influenced by the selection of fire resistant or low fuel revegetation species. Over the longer time spans, however, native species would invade the impoundment and control the fuel quality. Since fire is caused by lightning in a natural setting, lightning control measures may be an effective design element. Construction of lightning rods or other lightning control devices may be warranted on the site. The likely effect that such control measures may have on adjacent areas must be considered. Fire breaks can also be constructed to isolate the impoundment from fires that may start elsewhere.

viii. Monitoring, maintenance and remedial measures

The condition of vegetation can be monitored in terms of fuel quantity and quality. These parameters can be used to evaluate fire potential. Site inspections as well as visual and infrared photography could be used to judge changes in fuel quality and quantity.

Maintenance and remedial measures would entail management of the vegetative cover to minimize fire potential. If vegetation is destroyed, stabilization and revegetation may be necessary.

ix. Time dependency

The likelihood that a fire will occur and destroy impoundment vegetation increases as the time period of consideration increases. The calculated fire occurrence defines the long-term probability that a fire will occur. As with floods, the likelihood that fire will occur on any given day will be the calculated probability.

b. CLIMATIC CHANGE

i. Causes and description

Climatic factors such as moisture, temperature, light and wind exert a limiting effect on plant life. Although they can stand some variations, all plants have an upper limit and a lower limit of tolerance to a change in climatic factors (Odum, 1959).

Two levels of climatic change exist. First there are very short-term changes such as drought which occur frequently but last for only a few years within a region. Second are long-term gradual changes in world climate which may cause glacial advances or changes in sea level.

Vegetation is most sensitive to variations in temperature and moisture. These two factors interact within an area to produce the limits of climate which a plant species must tolerate. Lack of moisture or drought is the biggest problem associated with climatic change.

Cyclical or short-term drought will change the vigor and percent cover of vegetation. It may also influence the mixture of species that will develop on a short-term basis. Cyclical drought could be severe enough to destroy significant portions of the living vegetation. Long-term drought will result in permanent changes in the vegetation. At any given point in time the effect of either drought type will be the same with respect to interactive failure mechanisms. However, permanent changes in vegetation for the medium and long long-term period have potentially more serious consequences.

Vegetation has lower limits of tolerance to climatic change on disturbed soils than on natural soils (Curry, 1975). This phenomenon is caused by the destruction of the fabric of the topsoil which alters the biological, physical, and chemical characteristics of the soil. Climate input from man such as irrigation may establish species that cannot adapt to variations in the natural climate over short-term periods or may not succeed in the same manner as natural species over long-term periods.

ii. Interaction with other failure mechanisms

Failures of vegetation caused by climate change interact with wind erosion, water erosion and flood failure potentials. Drought caused by permanent climatic change is most serious because this results in permanent changes in the erosional process. The likely interactions are fully discussed with regard to failure of vegetation due to fire.

A climatic change could result in the succession of a deep rooted plant species which could result in uptake of radionuclides by the plants. Thick vegetation could also cause an obstruction of water diversion facilities.

iii. Methods of prediction

Short-term climatic cycles can be predicted from historic records on a regional basis. These predictions or forecasts must be evaluated in light of confidence limits for the data base used.

Long-term global climatic changes which cause major glacial advances may be predictable with the "astronomical" or Milankovitch theory (Calder, 1978). The long-term projection is for the earth to warm due to the "greenhouse" effect and then to cool into another ice age over the long long-term periods due to variations in the earth's orientation with the sun.

The best basis for evaluation of the effects of these changes may be the development of future scenarios, evaluating the consequences of each.

iv. Likelihood

The likelihood that failures in vegetation due to climatic change will occur over an extended time frame is high. The extent and duration of a drought and its resulting effect on vegetation must be determined based upon local climatic conditions, historical drought cycles and scenarious about future climate.

v. Magnitude of release

The potential of release due to a climatic change which effects vegetation comes from two sources: 1) failure of the vegetative cover may cause other failure mechanisms to occur, or 2) success of species which can concentrate radiation in its cells.

The magnitudes of release due to failure of vegetation because of cyclical drought is the same as described for fire caused failure of vegetation. In the case of drought caused by permanent climate change, the magnitudes for long long-term periods can be relatively high. In this situation, loss or change of permanent vegetation can potentially result in significant increases in erosion rates and flood potential and magnitude. Reliable estimates of magnitude, changes in erosion rates and flood potential must be evaluated on the basis of predicted climate and vegetation changes.

It has been stated that certain deep rooted plants such as pines can concentrate uranium within its cells (Odum, 1959). Although the magnitude of release is small, this type of release may come more directly into contact with other life.

vi. Site considerations

Consideration of site characteristics that favor establishment of drought resistant native vegetation is appropriate.

Site considerations with respect to erosion and flood potential are also important because of the interaction of vegetation with these failure mechanisms. Any site feature that minimizes drought effect, such as exposure, elevation or relief may be effectively combined to reduce drought impact.

vii. Design considerations

Revegetation planning should consider the use of drought resistant species. This may be effective for the short long-term period. However, for long time periods native vegetation will invade the area and be predominant. Grading to conserve moisture and to capture runoff could be used to reduce drought effects. All design considerations that were presented for wind erosion, water erosion, and flood should be considered because of the high interaction of vegetation failure with these failure mechanisms.

viii. Monitoring, maintenance and remedial measures

Generally the measures discussed in the previous section with regard to fire are applicable here. However, monitoring of weather records is of special importance for drought. Cyclical drought predictions may be confirmed or changed as data is added to the historic base. Likewise, indications of permanent climatic changes may be identified as weather trends are monitored and anayzed.

Because drastic disturbance results in a partial destruction of the natural fabric and chemistry of the soil strata, short-term maintenance will be required in most cases to prevent small climatic variations from killing revegetation effort. This type of maintenance will normally require reseeding, fertilization and irrigation. It should be done in moderation so that the species developed will be as closely suited to the extremes of natural climate as possible.

ix. Time dependency

Drought may occur at any point in the time span considered. Cyclical drought frequency may be identified and its timing predicted. Over long periods, droughts must be expected with varying intensities. Variation will be based upon the fluctuation of climate.

5. WATER DIVERSION STRUCTURES

a. SLOPE FAILURE

i. Causes and descriptions

Water diversion structures would have been constructed and been in use since the beginning of deposition of tailings in the impoundment. Therefore, in most cases, the short-term stability would have been proved and only long-term stability of slopes above the water diversion structures and canals would be of concern.

Many cases have been cited in the literature of slopes having failed several years after construction. The time period for these long-term failures ranges from less than 100 years (Casagrande, 1949; Skempton, 1964; Lambe and Associates, 1973) to about 1000 years (Crawford and Eden, 1967). A review of several long-term instability considerations in case histories is presented in Nelson and Thompson (1974).

Long-term instability may result from one or both of two mechanisms. When a slope is cut in overconsolidated clay or clayshale, negative pore water pressures may be induced. These negative pore water pressures result in an increase in the effective stress within the slope and an increase in stability. Dissipation of the pore pressures occurs over some time period resulting in a gradual decrease in the shear strength of the soils (Eigenbrod, 1975; Vaughan and Walbancke, 1973). The other factor that can cause instability is the existence of creep strains occurring within the soil. Accelerated creep and subsequent failure may occur after sufficient deterioration of the soil strength (Campanella and Vaid, 1974; Nelson and Thompson, 1977).

These mechanisms are of concern in clays and clayshales. In sandy soils long-term stability does not differ appreciably from short-term stability except as it is affected by erosion.

ii. Interaction with other failure mechanisms

Other failure mechanisms would not contribute to long-term failure of slopes around diversion ditches. However, failure of diversion ditches could disrupt flood routing and could enhance erosion of the impoundment.

iii. Methods of analysis

Methods of analysis for prediction of long-term instability are in their infancy. Eigenbrod (1975) presented a method for analysis for pore pressure equilibration based on two-dimensional consolidation theory. Nelson and Thompson (1977) indicated how slope failure due to creep may be predicted if the creep properties of the soil are known. In general, it has been observed that the peak strength of soil deteriorates over long periods of time. Therefore, for medium long-term considerations and longer, slopes should be designed such that the shear stresses within the slopes do not exceed the residual strength.

iv. Likelihood of failure

The likelihood of failure due to slumping of the slopes above the diversion ditches is of concern primarily in over-consolidated clays and clayshales only. Its occurrence will depend on the rate of dissipation of pore water pressures and the creep characteristics of the materials. If the structures are adequately designed, however, it is expected that the likelihood of failure would be small.

v. Magnitude of radiation release

No direct release could result from failure of water diversion structures. The release would result only because this failure mechanism is coupled with other potential failure mechanisms. Thus, the magnitude of release must be considered in context with the potential failure mechanism (e.g., erosion of the embankment toe, erosion of the cap, impoundment of waters, etc.) that it accelerates.

vi. Site considerations

This failure mechanism is obviously of concern only for sites requiring relatively large slopes above diversion ditches or extensive water diversion systems. Furthermore, it is of concern only in areas where materials conducive to long-term instability exist. Those materials are relatively impermeable materials. Their properties are characterized by a high peak shear strength relative to the residual shear strength. Materials of this type are commonly overconsolidated clays and clayshales.

vii. Design considerations

In order to achieve stability in long long-term periods, slopes around diversion structures should be designed so that either the stresses within the slopes are everywhere less than the residual shear strength of the soil,or else,such that conditions can be reached,through creep,wherein the stresses are everywhere below the

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residual shear strength (Nelson and Thompson, 1977). The stresses to be considered in the design are those that will exist after drainage and complete dissipation of the excess pore water pressures.

viii. Monitoring, maintenance and remedial measures

Monitoring of progressive failure and slumping of diversion ditches can be accomplished through various surveying techniques or photogrammetry. Some success has been achieved in the use of infrared photography for the prediction of the onset of progressive failure using color slicing techniques (McKean, 1976). This technique, however, is currently in an initial stage of development.

Monitoring of slopes at the Ft. Peck dam spillway has been done by the U.S. Army Corps of Engineers utilizing tiltmeters and inclinometers. The occurrence of progressive failure is noted by the acceleration of rates of movement of the slopes. Those monitoring schemes, however, require continuous observation by personnel.

If relatively rapid movements in slopes are observed, maintenance or remedial measures would consist of excavation to flatten the slopes. If a slope failure does occur, regrading and reconstruction of the diversion ditch is possible.

ix. Time dependence

Equilibration of pore water pressures would probably occur within a short long-term period. On the other hand, creep failures may occur at any time from the short long-term up to the long long-term time periods. Furthermore, since it has been shown that the creep rates will decrease to a particular minimum value prior to the onset of the accelerated creep (Campanella and Vaid, 1974), the fact that slopes may be decreasing in rate of movement does not indicate that potential long-term instability does not exist. However, after the onset of accelerated creep, actual slope failure does not occur until some time later (Nelson and Thompson, 1977). Therefore, if accelerated movements are detected, there does exist some time period within which remedial measures and maintenance can be taken.

b. OBSTRUCTION

i. Causes and description

Obstruction failures of water diversion structures may occur from three major sources; soil, ice and vegetation. Each of these mechanisms can cause a partial or total restriction, thus reducing the water carrying capacity of the facility.

Soil obstructions can occur in two ways, either as a buildup in a sediment or as a landslide into a diversion channel. Sedimentation is caused by more soil particles being transported into the channel than is removed by erosion by the water in the channel. The soil can be transported into the channel by the bedload of an intercepted stream, by mechanical weathering, by erosion along the banks of the channel, or by windblown sediments. The accumulation of sediments is a function of the amount and particle size distribution of the sediments and the velocity of flow. The velocity of flow is a function of the quantity of water, the area of the channel, the slope of the channel, and the roughness of the channel perimeter.

Landslides which involve a mass soil movement into a channel can occur as a result of bank slope instability as discussed in the previous section or as a geologic phenomenon in rocks or soils above the facility.

Ice and snow accumulation can also cause obstruction to water diversion structures. Ice blockage can occur as a buildup in facilities such as pipes, open channels, or spillways. The buildup can cause a blockage during extended cold periods or can cause blockage as ice jams when snow melting occurs. In cold climates snow accumulations can cause almost complete flow restriction during the early weeks of the annual spring thaw. Glacial activity could be an additional source of ice obstruction in water diversion facilities.

The third source of obstruction is vegetation. Both living and dead vegetation are potential problems. Trees and shrubs can root and grow along channel banks. During extended periods of low flow, they can develop within the channel bottom. Within the channel, they reduce flow and provide places for waterborne objects such as dead vegetation to accumulate. Grasses and other ground cover can also increase the roughness of a channel and reduce flow.

Dead vegetation can be carried into water diversion works by wind, water, or by animals and man. Floods typically carry large amounts of dead trees and assorted vegetation into water courses which can cause blockage. Banana leaves carried by flood water posed a problem at one tailings dam in the Phillipines (Brawner, 1978). Animals such as beavers and muskrats can build dams in diversion canals from vegetation and mud. Man-caused blockages can be caused by fences, inadequate culverts or crossings, and the placement of fill or discarded material in channels.

ii. Interaction with other failure mechanisms

Obstruction of water diversion facilities can cause failure of tailings impoundments due to gullying, sheet erosion or overtopping.

iii. Methods of prediction

Methods of predicting sedimentation and erosion of channels have been discussed by Einstein (1950); Colby (1956); Colby and

Hubbell (1961); and Simons and Albertson (1958). These techniques are based on modeling the velocity of flow, the amount of solids and the settling velocity of the particles. The amount of solids and particle size distribution can be predicted by comparitive analysis of bedloads carried by streams in the area. Velocity prediction is based on the estimated quantity of flow, the slope of the channel, the area of the channel cross-section, and roughness of the channel. A model can then be developed based on an energy balance in the stream or canal.

Slope stability analysis and prediction was discussed in the previous section. Landslide potential above the ditches can be evaluated by a geotechnical engineering analysis of the slopes above the facilities.

Short term ice and snow obstruction potential can be predicted from a knowledge of local climate, projected weather patterns, and experience of others operating water diversion facilities such as highway maintenance personnel or local irrigators.

A field study of natural vegetation recovery along old roads, canals, and abandoned disturbances can be useful tools in predicting the types and quantities of vegetation that will invade new diversion canals in the short and long-term. A field assessment is also useful in predicting vegetation that may be transported into the facilities.

iv. Likelihood

The likelihood of obstruction of water diversion facilities is great. Inspection and maintenance at frequent intervals will be required to prevent failure. Depending on the design and site conditions and the secondary effects of flood routing, failure of water diversion facilities may or may not cause a release of tailings or radionuclides. Thus, the likelihood of failure could assume either a high or a low value.

v. Magnitude of release

The potential magnitude of release due to a water diversion obstruction failure is site and event specific. The release will depend on the exact location of the obstruction, the quantity of water that is diverted to an unplanned location, and the susceptibility of that location to water-induced failure mechanisms. The largest potential for release would be during the occurrence of a larger flood than the tailings impoundment can store or bypass by alternate means. An overtopping failure and removal of the embankment could release 50 to 60 percent of the tailings during the initial event and eventually all of the tailing if the dam breach was not corrected.

vi. Site considerations

Site considerations that will influence obstruction potential of diversion facilities are climate, topography, geology, and hydrology. The interrelationships between these factors at each site will determine the obstruction potential of a planned diversion facility.

In general, the smaller the drainage area above a tailings impoundment in relation to the size of the impoundment, the safer the impoundment site against overtopping. Flatter terrain is most often less susceptible to landslides. Drier climates are less conducive to vegetation obstruction, but may be more susceptible to windblown sedimentation.

vii. Design considerations

If diversion canals are a permanent part of the abandonment plan, they should be designed to be self-cleaning as much as possible. This would mean designing the canals so that the water velocity would purge the excess bedload from the canal periodically. This can be done only if the original bottom and sides of the canal are protected from erosion. Methods commonly used are:

- Concrete lining and gabion protection. Both of these methods are also effective in reducing plant encroachment. Synthetic linings have been used to prevent seepage but are usually not considered to be abrasion resistant. The potential for long-term degradation of synthetic linings is not known. Concrete is also subject to deterioration from both chemical and physical attack.
- Buried conduit instead of open canals can be used to minimize slope stability obstruction potential for water diversion canals. Although this is an effective short-term provision, maintenance around inlets to the buried conduit is still required.
- Sedimentation basins in diversion canals are recommended when canals intercept streams (Neuhauser, 1974). These basins would also require cleanout in long-term usage.

viii. Monitoring, maintenance and remedial measures

Inspection and maintenance will be required to some degree on all water diversion works. The degree of effort which will be required after 50 or 100 years will be a function of the number and severity of mechanisms that can cause obstruction or related failure.

ix. Time dependence

Sedimentation and deterioration of water diversion structures may occur continuously, but other mechanisms could cause obstruction at any time after construction. The likelihood of an obstruction occurring in the short to medium long-term periods is high unless some maintenance is performed.

B. NATURAL PHENOMENA

1. EARTHQUAKES

a. Description of Phenomenon

Large earthquakes are basically caused by "global plate tectonics" which involve the movement of plates comprising the surface of the earth. Global plate tectonics is linked to the theory of continental drift. The majority of large earthquakes occur along the edges of plates where the plate movement has resulted in the development of faults in the rock masses. Earthquakes can also be caused by other mechanisms, such as crustal readjustment from loading or unloading, filling of large reservoirs, volcanoes, liquid injection along pre-existing faults, and nuclear explosions.

Movement along one side of a fault results in elastic strain energy slowly accumulating in the crustal rock on either side of the fault. When the stress developed along the fault exceeds the strength of the rock, rupture occurs and spreads in all directions along the fault in a series of erratic movements. The elastic strain energy is liberated in the form of elastic stress waves that propagate outward from the fault and comprise the earthquake.

The interaction of the earthquake with the tailings impoundment may be accelerations, and hence stresses, induced in the impoundment structure or actual relative displacement of parts of the impoundment. Accelerations and induced stresses may cause slope instability, cracking of embankments, caps or liners, differential settlement within the tailings or liquefaction and subsequent mobilization of the tailings material. Relative displacement may be caused by construction close to or on faults. Location on active faults should be avoided if the fault can be identified during the geological reconnaissance of the site.

b. Interaction with other failure mechanisms

Table 7 lists a number of large reservoirs that may have induced earthquakes due to fillings. The two likely causes for the triggering effect are the extra load on the earth's crust or an increase in the water pressure along fractures below the reservoir. For tailings impoundments, if the lining fails and excessive seepage can enter faults, the probability of an earthquake may be increased.

An earthquake can damage any or all components of the impoundment. If the impoundment is placed above an active fault, movement along the fault may cause differential settlement, strain and possible weak shear zones or cracks in the liner, cap or impoundment. Structures such as spillways or decant lines crossing the fault could be damaged or failed.

However, groundshaking is probably the largest earthquake hazard because it occurs over wide areas and at large distances from the fault movement. Damage and failure to a tailings impoundment from groundshaking may include damage to structures and equipment, ground failure, oscillation of water and wave generation in the impoundment or failure of water retention structures upstream from the tailings impoundment. Ground failure could cause liquefaction of tailings or embankments or it may involve differential settlement of foundation failure (Seed, 1967; Youd, 1973).

c. Methods of prediction and analysis

Prediction of earthquake-related geologic and seismic hazards can be based on the regional geology, available data on past earthquakes, aerial reconnaissance to locate faults, subsurface exploration, and laboratory testing. An estimate of the maximum credible earthquakes may be made for a selected fault based on the fault length

Table 7. Man-made reservoirs with induced seismicity. (From Horn and Scott, 1977.)

Location (dam-country)	Dam height (m)	Capacity ($m^3 \times 10^9$)	Basement geology	Date impounded	Date of first earthquake	Seismic effect
L'Oued Fodda, Algeria	101	0.228	Dolomitic marl	1932	1/33	Felt
Hoover, USA	221	38.3	Granites and Precambrian shales	1935	9/36	Noticeable (M = 5)
Talbingo, Australia	162	0.92		1971	1972	Seismic (M < 3.5)
Hsinfengkiang, China	105	11.5	Granites	1959		High activity (M = 6.1)
Grandval, France	78	0.29		1959/60	1961	MM intensity V in 1963
Monteynard, France	130	0.27	Limestone	1962	4/63	M = 4.9
Kariba, Rhodesia	128	160	Archean gneiss and Karoo sediments	1958	7/61	Seismic (M = 5.8)
Vogorno, Switzerland	230	0.08		8/64	5/65	
Koyna, India	103	2.78	Basalt flows of Deccan Trap	1962	1963	Strong (M = 6.5) 177 people killed
Benmore, N. Zealand	110	2.04	Greywackes and argillites	12/64	2/65	Significant (M = 5.0)
Kremasta, Greece	160	4.75	Flysch	1965	12/65	Strong (M = 6.2) 1 death, 60 injuries
Nurek Tadzik, USSR	317	10.5		1972 (to 100 m)	1972	Increased activity (M = 4.5)
Kurobe, Japan	186	0.199		1960	1961	Seismic (M = 4.9)

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and type of faulting. The predominant frequency and duration of strong motion, and the peak acceleration are important factors in assessing damage and failure to structures, soils, and slopes. Some important factors and characteristics of earthquakes are discussed below.

i. Earthquake records

Horizontal acceleration is usually recorded in two directions. The vertical component can also be recorded. Maximum accelerations are widely quoted and published after earthquakes. However, duration of accelerations above a certain level and the frequency spectrum must also be considered. The maximum acceleration recorded for most earthquakes is about 0.6g in the horizontal direction and less in the vertical (Bolt, 1973).

ii. Earthquake intensity and magnitude

Two different scales to describe earthquake magnitude are commonly employed in earthquake engineering. These two scales are the Modified Mercalli Scale and the Richter Magnitude.

The Modified Mercalli Scale is a subjective intensity scale which ranks earthquakes according to the damage incurred. A given earthquake can have many different intensities according to this scale depending on the location at which the earthquake is described. Table 8 describes the Modified Mercalli Scale.

The Richter Magnitude is a measure of the total energy released during an earthquake. A given earthquake has only one magnitude. An increase in Richter Magnitude by 1 means a 30 fold increase in energy released as seismic waves. The relationship between the energy release and the magnitude M is:

Table 8. Modified Mercalli (MM) intensity scale of 1931. (From Horn and Scott, 1977.)

I	Not felt except by a very few under especially favorable circumstances.
II	Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing.
III	Felt quite noticeably indoors, especially on upper floors of buildings, but many people do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibration like passing of truck. Duration estimated.
IV	During the day felt indoors by many, outdoors by few. At night some awakened. Dishes, windows, doors disturbed; walls make cracking sound. Sensation like heavy truck striking building. Standing motor cars rocked noticeably.
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.
VI	Felt by all, many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight.
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 motor cars.
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 motor cars disturbed.
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.
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.
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. Practically all works of construction are damaged greatly or destroyed. Waves seen on ground surface. Lines of sight and level are distorted. Objects are thrown into the air.

$$\log E = 11.4 + 1.5M$$

where E = energy in ergs.

Some examples of Richter Magnitude are given in Table 9.

iii. Location of faults

Some information on fault locations can be obtained from geologic maps of an area of interest. Detailed maps of faults have been prepared for areas near nuclear power plants (Anon, 1973). If reliable maps are not available, field work, photo interpretation and/or geophysical techniques can be employed to locate faults. The location of faults is particularly important since tailings impoundments should not be located on an active fault.

iv. Maximum credible earthquake; design earthquakes

After faults have been located in an area, the magnitude of the largest earthquake that is likely to occur should be determined. The magnitude of a large earthquake is proportional to the fault rupture length and to the square of the fault offset (Bonilla and Buchanan, 1970). Studies by Albee and Smith (1967) suggest that a fault does not rupture over its entire length. Generally the area of rupture is less than about 50% of the fault length (Albee and Smith, 1967). Using curves developed by Tocher (1958), Iida (1965), Albee and Smith (1967), Bonilla (1967), or Bonilla and Buchanan (1970), an estimate can be made of the maximum credible earthquake. The probability of a large earthquake occurring on a small fault is very low. Table 10 lists approximate relationships between Richter Magnitude and fault rupture length. Table 10 can be used to make a preliminary estimate of the maximum credible earthquake on a fault. The maximum credible earthquake is defined as the maximum earthquake that appears reasonably capable of

Table 9. Examples of Richter magnitude (after Horn and Scott, 1977, p. 306).

Year	Region	Deaths	Mag.	Comments
June 12, 1897	India, Assam	1,500	8.7	
September 3 & 10, 1899	Alaska, Yakutat Bay		7.8 & 8.6	
April 18, 1906	Calif., San Francisco	700	$8^1/_4$	
December 28, 1908	Italy, Messina	120,000	$7^1/_2$	
January 13, 1915	Italy, Avezzano	30,000	7	
December 16, 1920	China, Kansu	180,000	$8^1/_2$	
September 1, 1923	Japan, Kwanto	143,000	8.2	Great Tokyo fire
December 26, 1932	China, Kansu	70,000	7.6	
May 31, 1935	India, Quetta	60,000	7.5	
January 24, 1939	Chile, Chillan	30,000	$7^3/_4$	
December 27, 1939	Turkey, Erzincan	23,000	8.0	
June 28, 1948	Japan, Fukui	5,131		
August 5, 1949	Ecuador, Pelileo	6,000		
February 29, 1960	Morocco, Agadir	14,000	5.9	
May 21–30, 1960	Southern Chile	5,700	8.5	
September 1, 1962	Northwest Iran	14,000	7.3	
July 26, 1963	Yugoslavia, Skopje	1,200	6.0	See text
March 28, 1964	Alaska	131	8.6	Prince William Sound, Tsunami
August 31, 1968	Iran	11,600	7.4	Surface faulting
May 31, 1970	Peru	66,000	7.8	$530,000,000 damage. Great rock slide. See text
February 9, 1971	Calif., San Fernando	65	6.5	$550,000,000 damage. See text
December 22, 1972	Nicaragua, Managua	5,000	6.2	See text
February 4, 1975	Liaoning, China	Some	7.4	Much damage
Septemer 6, 1975	Lice, Turkey	2,400	6.8	12,000 homes damaged
February 4, 1976	Guatemala	23,000	7.9	Motagua fault break
May 6, 1976	Friuli, Italy	1,000	6.5	Extensive damage. Aftershocks
July 27, 1976	Tangshan, China	600,000?	7.6	Great economic loss
March 4, 1977	Roumania	1,500	7.2	Felt Moscow to Rome

Table 10. Relationship between magnitude and fault rupture length (Horn and Scott, 1977, p. 27).

Magnitude (M) (Richter)	Rupture Length (km)
5.5	5 - 10
6.0	10 - 15
6.5	15 - 30
7.0	30 - 60
7.5	60 - 100
8.0	100 - 200
8.5	200 - 400

occurring under conditions of the presently known geological framework and earthquake history. The assignment of a design earthquake (of magnitude less than or equal to the maximum credible earthquake) must be based on the consequences of failure and the level of acceptable risk.

v. Duration of strong ground shaking

The duration of strong motion may be the most important factor in causing damage and failure to structures, soils and slopes. The velocity of propagation of a fault break is approximately one to two miles per second. Hence, if the length of the fault break is known, the duration of an earthquake can be calculated. The duration of strong motion is approximately equal to the length of the fault divided by the velocity of propagation.

vi. Estimation of rock motion for design purposes

The significant characteristics of rock motions for purposes of analysis include the maximum amplitude of the accelerations, the predominant frequency or period of the strong motions, and the duration of the motion. The rock motions at any particular site will depend on the amount of energy released along the fault during the earthquake and the distance of the site from the zone of energy release.

In general, both maximum acceleration and the frequency decrease with increasing distance from the zone of energy release. The attenuation of the maximum acceleration and predominant frequency is due to geometric and material damping. Schnabel & Seed (1972) and Bolt (1973) have developed graphs which relate maximum acceleration, and Richter magnitude with distance from the causative fault (Fig. 10).

Estimates of duration of strong motion have been made by B. A. Bolt (1973, 1977) from available strong motion records and assumed attenuations. These are shown in Table 11.

Table 11. Duration of ground motions (seconds) (acceleration > 0.05 g; frequency > 2 Hz) versus magnitude and distance from fault rupture (after Bolt, 1973, 1977).

Distance (km)	Magnitude						
	5.5	6.0	6.5	7.0	7.5	8.0	8.5
10	8	12	19	26	31	34	35
25	4	9	15	24	28	30	32
50	2	3	10	22	26	28	29
75	1	1	5	10	14	16	17
100	0	0	1	4	5	6	7
125	0	0	1	2	2	3	3
150	0	0	0	1	2	2	3
175	0	0	0	0	1	2	2
200	0	0	0	0	0	1	2

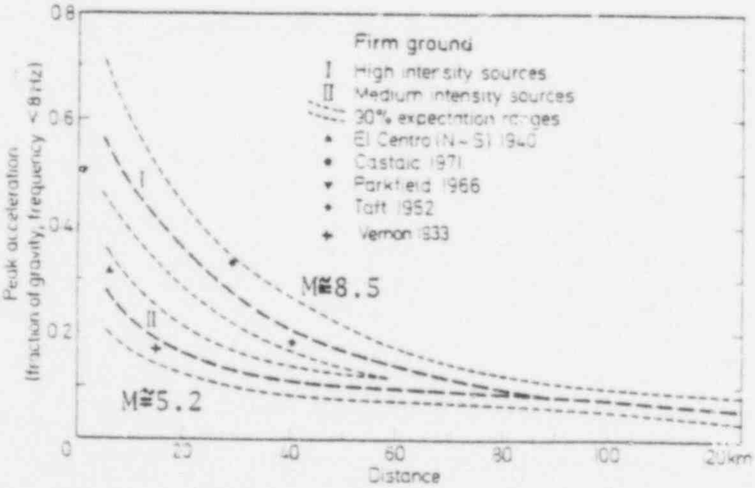

Figure 10. Attenuation of maximum acceleration of ground shaking with distance from fault rupture. The ground conditions are rock or firm overburden (after Bolt, 1973).

Seed, Idriss and Kiefer (1969) have developed relationships between the predominant period, magnitude and distance from the causative fault. These are shown in Fig. 11. S. T. Algermissen (1969) has developed an effective peak acceleration map of the United States as shown in Fig. 12. This map should be used with caution on a regional scale since local factors may increase or decrease actual acceleration.

These relationships can be used to estimate the peak acceleration, duration of strong ground motion and frequency of shaking resulting at a site due to movement of a fault located a particular distance from the site.

vii. Influence of soil conditions on ground motions

The ground motion is controlled by the rock motion and the nature and depth of the soils underlying the site. The predominant period of ground motion increases as the ground conditions become softer.

Soil conditions can modify both the amplitude and frequency characteristics of the underlying rock motion. Analytical methods to predict ground motion, employing laboratory determined dynamic soil properties and predicted rock motions, have been developed by Idriss et al. (1973).

viii. Soil structure interaction effects

The presence of a massive impoundment may cause ground motions at the base of the impoundment to be different from the predicted free field motions. Also, the response of the impoundment itself resulting from the ground motion is necessary for analysis.

ix. Earthquake induced liquefaction of sands and tailings

During an earthquake, a soil element is subjected to a series of alternating shear stresses. If a sample of saturated sand

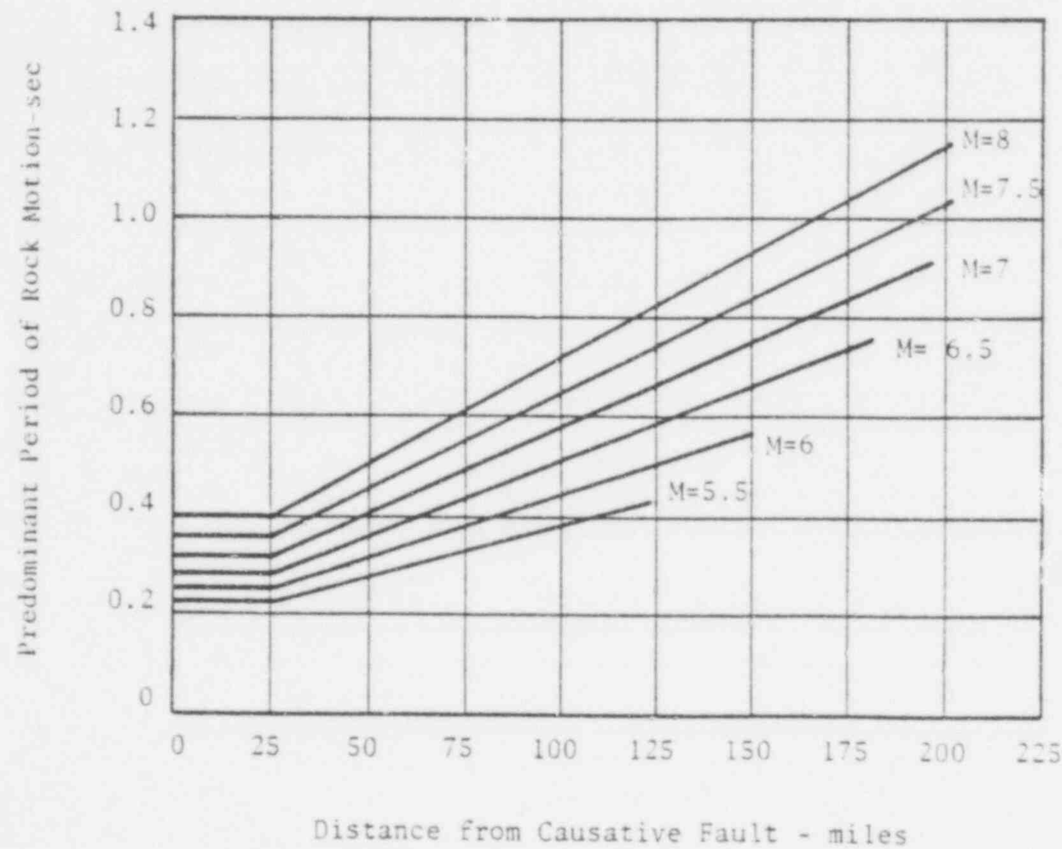

Fig. 11. Predominant periods for maximum acceleration in rock. (From Seed et al., 1969.)

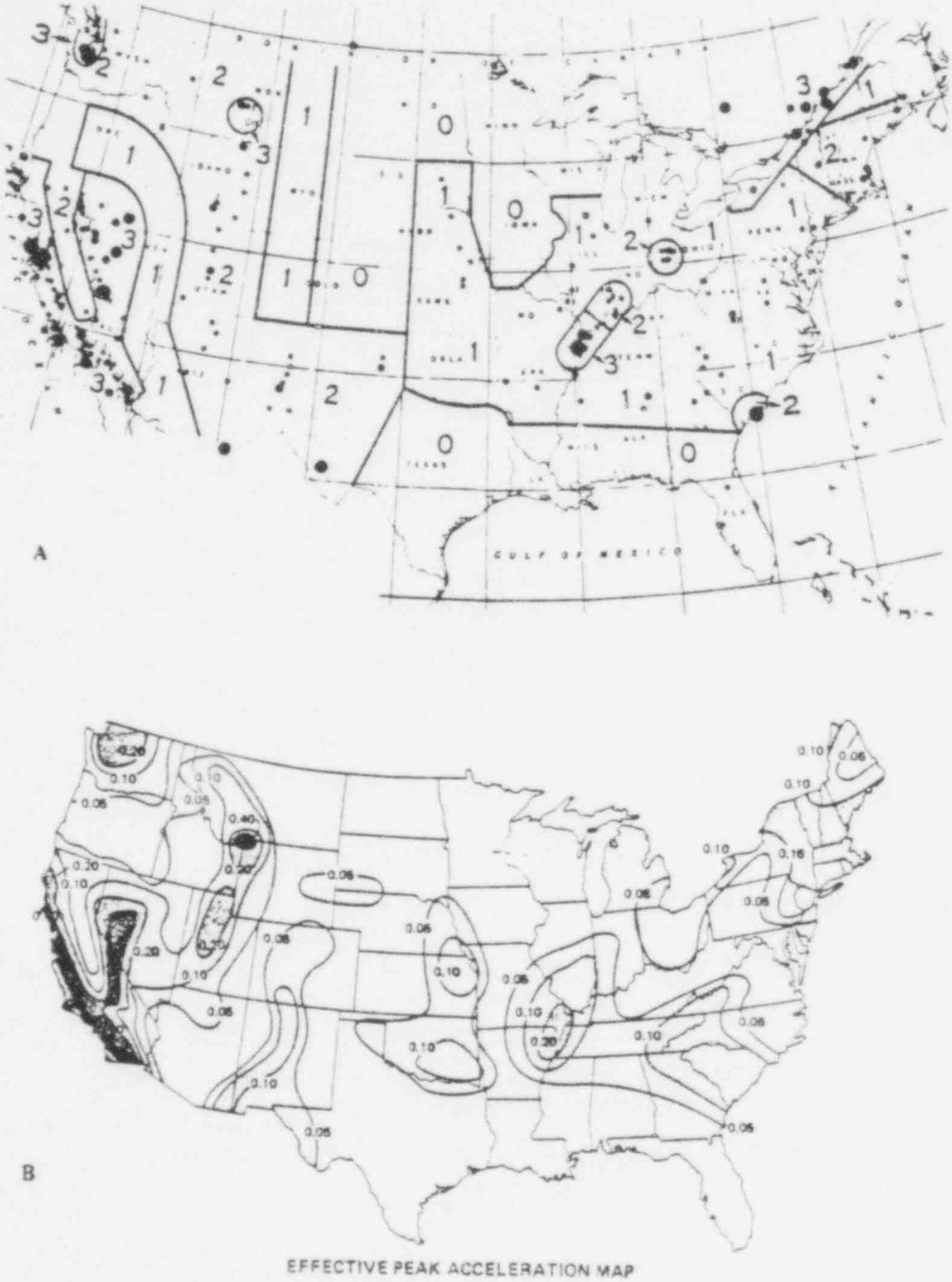

Figure 12. Two forms of seismic risk maps (from Horn and Scott, 1977, p. 16).

is subjected to cyclic loading, it may remain stable for some number of cycles and then suddenly become unstable. The loss of strength results from the buildup of pore water pressures in the sand due to a tendency for the soil volume to decrease during shaking. The point of instability is reached when the pore water pressure becomes equal to the total stress. At this point the effective stress in the soil and its strength becomes zero.

The liquefaction potential of a soil deposit from ground motions depends on many relationships. A review of case histories of earthquake-induced soil liquefaction has shown the following factors to be significant.

- Soil Type--Liquefaction occurs in saturated cohesionless soils. Uniformly graded soils appear to be more prone to liquefaction than well graded soils (Ross et al., 1969; Lee and Fitton, 1969). For uniformly graded soils, fine sands tend to have a higher liquefaction potential than coarse sands, gravelly soils, silts or clays. An increase in the clay fraction appears to reduce the liquefaction potential (Seed, 1967).
- Initial Relative Density--For other factors remaining constant, an increase in the relative density decreases the liquefaction potential (Seed and Idriss, 1971). Soils with a relative density less that about 60% are most susceptible to liquefaction.
- Initial Effective Stress--The liquefaction potential is lower for higher initial effective stress (Lee and Fitton, 1969).

- Intensity of Ground Shaking--As the intensity of ground shaking (accelerations or stress changes) increases, the liquefaction potential increases (Seed and Idriss, 1971).
- Duration of Ground Shaking--As the duration of shaking (i.e., more strain cycles) increases, liquefaction potential increases (Seed and Idriss, 1971). One large stress cycle or many smaller stress cycles can cause liquefaction.
- Initial Shear Stress--Liquefaction will be induced more easily under level ground conditions than in sloping zones of a deposit (Seed, 1967).
- Pore Water Pressure--Liquefaction can only persist as long as high excess pore water pressures persist in a soil. Therefore, the permeability of the saturated soil, the nature of soils above and below it and the presence of drains are important to the onset and duration of liquefaction.

x. Analysis of soil liquefaction potential

Any method for evaluating liquefaction potential should take the above significant factors into account. Seed and Idriss (1971) proposed a method for evaluating liquefaction potential from earthquakes. That method provides an indication for the potential of a deposit to liquefy mainly on the basis of relative density and intensity. Standard penetration test data can be used to estimate the relative density and therefore the liquefaction potential of a deposit.

Finite element analyses have also been used to predict the onset of liquefaction in deposits or embankments (Seed, 1968; Schnabel, Lysmer and Seed, 1972). Those methods provide a basis on which to estimate the liquefaction potential of an impoundment. They are generally conservative. However, considerably more research is needed before accurate predictions of liquefaction of tailings impoundments can be made with much confidence. Also, considerably more research is needed on the liquefaction potential of materials having typical gradations and grain sizes characteristic of tailing-

d. LIKELIHOOD OF FAILURE

The number and magnitude of earthquakes that have occurred in an area provide information on the probability of occurrence of earthquakes in the future. Along with regional geology, regional tectonics and detailed site studies, recurrence intervals for earthquakes of various magnitudes can be developed (Wallace, 1970; Blume, 1973). The procedure for determining recurrence intervals is similar to that used for hydraulic design. As with any natural phenomenon, the confidence that can be placed on the predicted recurrence interval increases with the time period over which reliable earthquake and faulting information is available. An example of a curve showing the recurrence interval of earthquakes of different magnitudes is shown in Fig. 13. That curve is based on the data shown in Table 12.

On the basis of the nature of the potential source of earthquakes in that area (i.e., distance from a fault, length of the fault, etc.), the maximum credible earthquake for the St. Vrain site was considered to be of magnitude 5.5 and is marked on Fig. 13.

Data on recurrence intervals of earthquakes have been collected for all nuclear power plant sites. Therefore, typical data is available for many areas around the United States.

The likelihood of a failure due to the occurrence of an earthquake would depend on the likelihood of occurrence of an earthquake of magnitude

Table 12. Earthquake magnitude versus frequency at the C. H. Green Observatory, Colorado.

Richter Magnitude	Average Recurrence Interval	Mean Annual Frequence (Number of Earthquakes per year
0.2	1 day	365
2	7 days	50
3	50 days	7
4	1 year	1
5.3	7 years	0.1
5.5	50 years	0.02

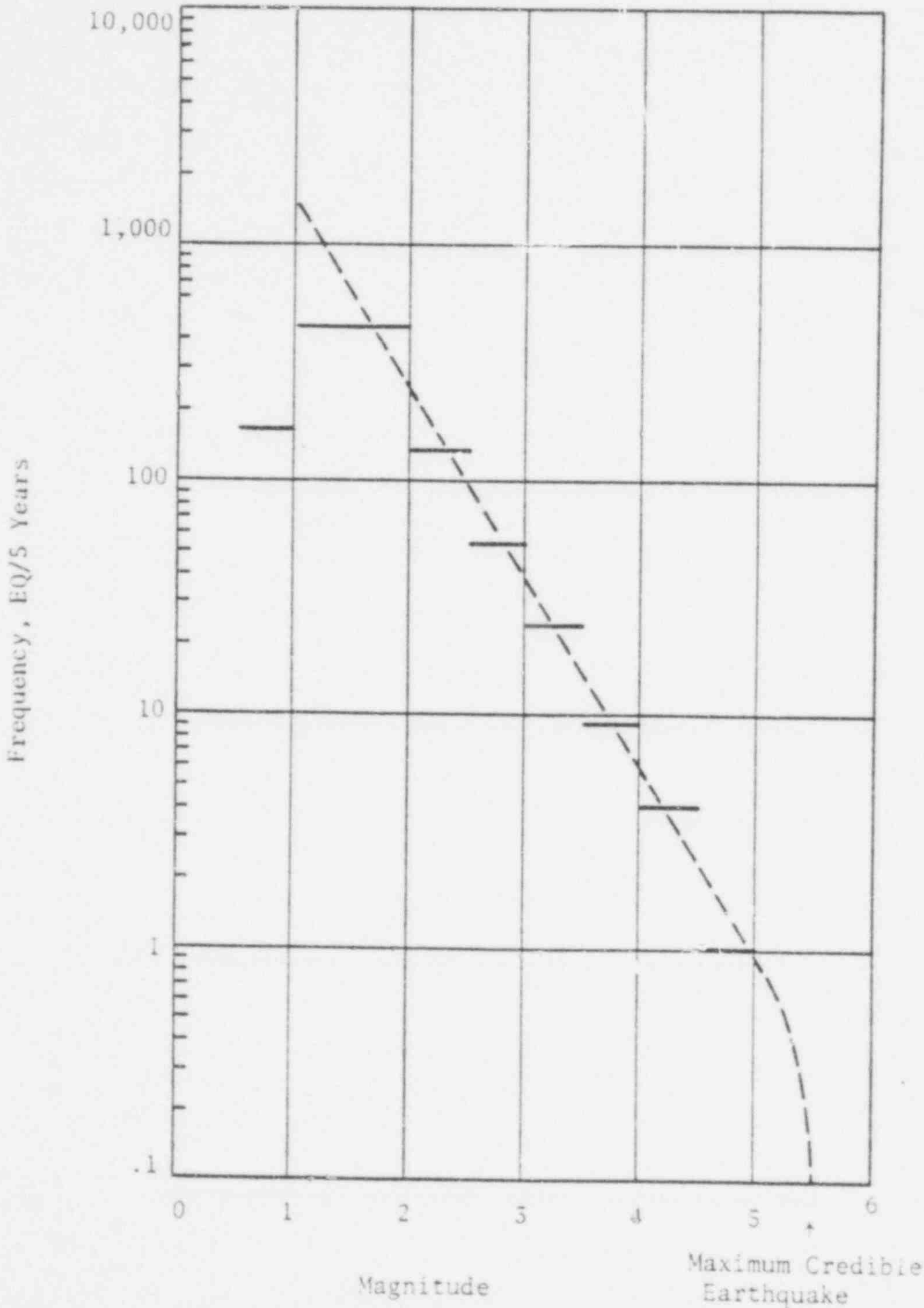

Fig. 13. Frequency distribution of earthquakes for the Fort St. Vrain, Colorado, nuclear generating station. (From Public Service Company of Colorado.)

greater than the design earthquake. The probability of occurrence can be obtained from a curve similar to that shown in Fig. 13. However, even if a disposal plan was designed on the basis of the maximum credible earthquake there does exist a finite probability of occurrence of a larger earthquake. The likelihood of that happening is small for short time periods and increases as the time period being considered increases. The determination of that likelihood is somewhat subjective and depends on the confidence level of the data available for the site.

If an earthquake does occur at a site, the likelihood of release of radioactive material is a function of the nature of the disposal plan. If clay embankments are used and if the embankment is not saturated, the likelihood of liquefaction is very low. For medium long-term and long long-term considerations, it may be assumed that the tailings would no longer be saturated. Thus, liquefaction would be of concern only for short long-term periods.

If disposal plans other than deposition of the tailings behind an embankment are used, the likelihood of failure will depend on the susceptibility of the impounding structures to earthquake damage.

In general, the likelihood of failure due to earthquakes would be high. The magnitude of release, however, may not be great. That is discussed below.

e. MAGNITUDE OF RELEASE

The potential magnitude of release due to failure of any portion of the impoundment as a result of earthquakes will be influenced by the magnitude of the earthquake, the distance from the impoundment to the earthquake epicenter, the soil conditions under the site, the nature of the tailings, and the disposal plan employed. Tailings impoundments in seismically active areas will be designed for some level of ground motion. Consequently, only earthquake intensities that exceed the design magnitude should cause failure.

The release can be quantified on the basis of the type of failure that would be induced by various magnitudes of earthquakes. A very large earthquake may cause an impoundment to fail. If the tailings liquefy, they may move large distances downstream. Dobrey and Alvarez (1967) discuss liquefaction of tailings dams during an earthquake. They presented examples in which liquefied tailings flowed 10 to 20 miles downstream. Other examples also have been observed where liquefied tailings have flowed long distances (10 to 20 miles), but those have not been published in the literature. The magnitude of release would be high if liquefaction occurs.

In order to liquefy to the extent mentioned above, however, the tailings must be saturated. That may be the case for short long-term periods. However, for periods of time greater than about several hundred years, it is not expected that the tailings would remain saturated.

Release due to the occurrence of a major earthquake, therefore, would be more likely to occur through cracks in the cap or liner or through areas of slope instability of the embankment.

f. SITE CONSIDERATIONS

Site considerations that will influence the response to earthquakes are the magnitude and distance from a probably epicenter (i.e., a major fault) and the subsurface profiles. The potential for liquefaction of subsurface foundation soils may influence the stability of the impoundments.

The size of the impoundment may also influence the ground motion. For an impoundment of relatively low height the predicted ground motion will not be changed appreciably by the stresses induced by the tailings. However, for impoundments that are several hundred feet high the interaction between the impoundment and the subsurface materials may modify the ground motions. Very high impoundments (several hundred feet) may also induce seismicity.

If the surrounding topography is steep, the possibility of large earthquake induced landslides of natural soil and rock falling into and displacing the tailings should be considered. The location of the impoundment with respect to upstream water retention structures or tailings dams that may fail during earthquakes should be considered with respect to their possible effect on the tailings impoundment.

g. DESIGN CONSIDERATIONS

If liquefaction of the subsoil is possible at a site, recommendations should be made in the design to either remove or stabilize the potentially liquefiable deposits in some way such as compaction or vibroflation. The design of the liner and cap to enable them to withstand differential movements caused by an earthquake would involve the use of materials that are self-healing and capable of withstanding possible movements. For example the use of a clay liner and cap.

The use of drains within the embankment or tailings to facilitate dissipation of pore water pressures induced during an earthquake may decrease the liquefaction potential.

Methods employed in the design and construction of dams in seismically active areas to withstand stresses caused by acceleration or liquefaction would be applicable (Lee and Roth, 1977; Lee, 1974; Seed et al., 1975). In general, the design of impoundments in seismically active areas would utilize nonliquefiable materials and would consider the stresses induced by the acceleration of the earthquake. To attempt to consider effects that may result from an earthquake of magnitude greater than a design earthquake or the maximum credible earthquake may be difficult and not practical. For long term considerations, however, the possibility of such an occurrence must be recognized. Because the greatest problems may result from liquefaction and subsequent mobilization of the tailings, it is desirable to facilitate drainage of the tailings. Consequently, the major design consideration would be the removal of water from the impoundment as soon as would be practical after abandonment. This could be accomplished by drainage into evaporation ponds or by natural seepage. Although the seepage of water from tailings is not considered to be desirable at this time, the existence of large impoundments of saturated, fine grained, hydraulically placed tailings for periods of hundreds of years may pose potential earthquake hazards. If the tailings impoundments can be drained, their hazard from the standpoint of earthquake loading would be decreased appreciably.

h. MONITORING, MAINTENANCE AND REMEDIAL MEASURES

Monitoring schemes to detect potential radionuclide release due to damage to a particular element of the tailings impoundment would be the same as discussed previously with regard to that particular element of the impoundment.

In seismically active areas the relationship between recurrence interval and probable earthquake magnitude may change due to the existence of more complete data and more site specific data. For that reason, seismic records taken either on-site or at a nearby site may be useful in updating the recurrence interval versus magnitude relationship and may result in some revision of the maximum credible earthquake. Seismographs, however, would require a certain amount of of continual maintenance and may not be practical for medium or long-term periods.

Maintenance and remedial measures to repair damage to various elements of the impoundment structure may or may not be possible (e.g., cracks in the cap may be filled but liners would generally be inaccessible). However, if liquefaction and mobilization of the tailings occurs, remedial measures to contain the tailings would be extensive and impractical. Some clean-up operations may be possible but the effectiveness of the remedial action would depend on the extent to which the tailings were mobilized.

i. TIME DEPENDENCE

The probability of occurrence of an earthquake decreases as the magnitude of the earthquake increases. The occurrence of an earthquake larger than the design magnitude would always be possible and the likelihood of occurrence would increase as the time period of consideration increases.

The seismicity of an area is not constant and will change with time. Earthquakes of significant magnitude may occur in areas that have not experienced appreciable earthquakes previously. Earthquakes may recur in those areas for periods of time and then cease. Such was the case for the Denver area in the early 1970's. (In that situation the seismicity was said to have been induced by down-hole pumping of waste.) In the last ten years the Chicago area and central Illinois, which are not seismically active areas, have experienced two earthquakes of magnitude near 5. Thus, the seismicity of an area is difficult to predict over medium and long long-term periods. It may increase or decrease with time. The state of the art of earthquake prediction does not allow for changes in seismicity to be predicted with much accuracy. It must be considered that earthquakes may occur at any time after abandonment of the impoundment.

However, after a sufficient period of time the water content of the tailings will decrease. Liquefaction potential of the tailings would, therefore, be of concern only over short long-term periods. For medium and long long-term periods earthquake caused damage would be of less severe nature.

2. FLOODS

a. DESCRIPTION OF PHENOMENON

Flooding can result from large rainstorms, rapidly melting snow or from localized cloudburst storms. Rain floods characteristically have high peak flows and moderate volume and duration. Flooding from rapidly melting snow is characterized by moderate peak flows, large volume, long duration and diurnal fluctuation of flow. Cloudburst storms can be expected to occur frequently during summer in the western region. The rainfall intensity of cloudburst storms is high and the resulting runoff is characterized by high peak discharge, short duration and small volume. Sometimes snowmelt runoff may be augmented by rain which leads to very high peak flows and severe flooding. Flooding is more severe when the ground is frozen and infiltration is minimal.

The failure mechanism associated with floods for tailings impoundments is erosion. Floods can damage tailings structures by erosion along the toe of an embankment or by overtopping. Any erosion of tailings during flooding will result in the transportation of solids for great distances. Tailings impoundments built across natural drainages are usually more susceptible to flood damage.

b. INTERACTION WITH OTHER FAILURE MECHANISMS

Floods can interact with many of the other failure mechanisms, especially those which involve erosion or slope stability. Small floods associated with high intensity rainfall can cause gullying, sheet erosion, increased sediment transport into diversion structures, and damage to vegetation on reclaimed surfaces. Large floods can result in embankment instability due to erosion of the toe, overtopping and washout of diversion facilities, or overtopping and washout of tailing embankments.

Increases in soil moisture which may be associated with a flood may also contribute to instability of slopes. Landslides often occur in natural slopes following periods of heavy rainfall.

c. METHODS OF PREDICTION OF DISTRESS

The potential for flood distress to a tailings impoundment can be predicted with conventional streamflow and weather prediction techniques. Actual flood caused distress can be observed by physical inspections or by aerial photography. Some flood warning devices could be employed for short-term warning applications, but would not be effective for long-term usage. In general, it may be assumed that any portion of the impoundment that extends above the level surface would be completely washed away by a major flood.

d. <u>LIKELIHOOD OF OCCURRENCE</u>

Prediction of floods involves a determination of the amount of water that may occur at a point in a watershed and a statistical prediction of how often that same event is likely to occur. The determination of the quantity is based on historical streamflow records, precipitation records, and an analysis of the topography of a particular site.

Frequencies are normally expressed as an "x-year" flood. This implies that over a long time period as many floods of "x-year" magnitude or larger can be expected to occur as there are "x-year" long periods within the time span considered. For example, a 100-year flood implies that there may be 10 floods of equal or greater magnitude that may occur over the next 1000 years. It should not be taken to mean that the time period between the 10 floods would be equally distributed at every 100 years over that time period.

A maximum probable flood is the largest flood that is expected to occur at a particular site. The magnitude of this flood is based on hydro-meteorological data rather than historical streamflow records. The maximum probable storm for the watershed is evaluated for this analysis. The conditions that are assumed to produce the maximum probable flow are subjective. Over the extremely long time periods considered herein, there is a definite possibility that even the maximum probable flood can be exceeded.

As more data is collected and as climate may change over long time periods, the magnitude of certain frequency floods and the maximum probable flood may have to be adjusted.

e. MAGNITUDE OF RELEASE

The magnitude of release due to flooding is related to the type of erosional process which is involved. Erosion of the cap leads to an increase in the emanation of radon gas and could lead to erosion of the tailings underneath if deep gullying occurred without maintenance.

The largest magnitude of release would result from overtopping the embankment. During overtopping, the water quickly becomes channeled into a few deep gullies which progress headward. If the slimes are unconsolidated and uncemented, it is likely that 80-90% of the slimes (-200 mesh) would be released. Although most of the coarse material would not be lost during the initial event, continued erosion could occur unless remedial action was taken.

f. SITE CONSIDERATIONS

Regional assessment of hydrological conditions is the first step in tailings dam siting with respect to floods. This analysis will include streamflow records of major streams, precipitation records, and assessment of other relevant factors such as topography, geology, and type of vegetation. This assessment can be refined for the assessment of potential damage from flooding for each disposal site alternative within that region.

Depending on regional location, flood flow originates from rainfall, from snowmelt or from a combination of varying amounts of rainfall and snowmelt. In southwest Colorado, for example, flooding results from large frontal type rainstorms approaching from the southwest, from rapidly melting snow, and from localized cloudburst storms. Flooding from rain can generally be expected from mid-June through December, but records show that rainstorms producing major floods occur most frequently during September and October.

In determining the critical design storm rainfall estimate for a particular drainage area, it is necessary to consider the size, configuration and runoff characteristics of the basin as well as the meteorological characteristics of major storms in the area or region. As this is done for each particular disposal site, design floods would be developed. The effects of the design flood at each site can then be analyzed to minimize the risk of tailings release.

Generally, minimizing the catchment area above a tailings dam will reduce the flood potential. Factors such as slope and vegetation will also influence flood magnitude.

Generally, a tailings dam constructed across a drainage has a greater potential for flooding than a perimeter type dam which is more isolated from large drainages. A cross-valley type tailings disposal sites, water diversion facilities such as spillways, decant lines, and bypass canals are often required to prevent flooding. These structures will normally require some maintenance even for short-term uses.

If a tailing site is located such that the toe of the retaining embankment can be eroded during the flood stage of an adjacent stream, this portion of the retaining embankment should be protected with riprap.

g. DESIGN CONSIDERATIONS

The magnitude of the design flood is the primary design consideration for flooding. The selection of the design flood should be based on the evaluation of the relative risks and consequences of the tailing failure under both present and future conditions.

The design flood should be based on records which are truly representative of the average condition and, to provide useful answers, it must be based with a data series that is relevant, adequate and accurate. Adequacy refers primarily to the length of the record. Sparsity of data is often a problem. If the collected data is too small the probabilities derived cannot be expected to be reliable.

Other design considerations will involve methods to minimize erosional effects such as establishment of vegetation and placement of riprap on exposed slopes. If diversion facilities are required, they should be designed so that minimal maintenance is required.

h. MONITORING, MAINTENANCE AND REMEDIAL MEASURES

Maintenance needs will be site specific depending on the climate and design of the facilities. If water diversion is required, some degree of maintenance for long-term flood protection will likely be required. Remedial measures may or may not be possible depending on the nature of the failure.

i. TIME DEPENDENCE

Large floods are not time dependent. Two large floods can happen in successive years if conditions are right. However, the likelihood or probability of this occurring is small.

The effects of floods can be time dependent because of additive effects if the need for maintenance or remedial action is not satisfied.

3. WINDSTORMS

In general, the winds at a site are controlled by:

- Large-scale and regional atmospheric pressure distribution.

- Local effects produced by the topographic configuration of the surrounding area, surface friction, structures, warming and cooling of mountain slopes, large bodies of water and thermal instability.

Extensive studies of wind velocities related to geographical location have resulted in detailed wind velocity maps. Wind probability maps for various mean recurrence intervals have been developed. The 100-year mean recurrence interval wind velocity is typically used for design when an unusually high degree of hazard to life and property exists in case of failure. Wind velocity maps for various mean recurrence intervals are given by Thom (1968).

Wind velocities reported by Thom (1968) are for a height of 30 ft and it is necessary to modify the velocity for other heights. In addition, the above velocities may be greater due to wind gusts. Generally, wind velocities near the ground will be lower than those at a 30 foot height.

Typical wind information available from the U.S. Weather Bureau is peak gust, 1 and 3-hour mean speeds and directions (vectors), mean daily wind vectors, and prevailing wind directions. To determine the wind speed versus return period, the highest mean hourly wind speed occurring each year is recorded to provide a set of maximum values which is then analyzed using extreme value statistics. For any particular location where annual extreme data is available, the wind speed relating to any chosen probability can be determined (Harris, 1970).

Generally wind velocities near the ground would be lower than those reported for the 30-ft height. Wind damage by waves and ice movement could be severe to a tailings impoundment during operation of the mill.

However, after abandonment, if the pond has been drained and capped, the effect of strong winds would be much less. If a cap or cover exists, no actual release of radionuclides would be likely to occur. Rates of wind erosion as a function of wind velocity and other factors is considered with respect to erosion. In those sections of the report, isolated wind storms were considered in the Wind Soil Loss Equation. Consequently, they are not evaluated separately here.

Wind erosion, the major problem related to winds, is discussed fully in sections A.1d and A.3e of this chapter. For consideration of long-term wind erosion potential it may be important to develop predictions about the future wind environment at a site. Radionuclide release due to high winds over an abandoned impoundment would be small for all time periods for impoundments having a suitable cover or cap. If the tailings are exposed (e.g., the Base Case), dispersed radionuclides may be transported for short periods of time.

4. TORNADOES

In general, tornadoes are caused by regional atmospheric pressure distribution and thermal instability in the air. Until recently very little was known regarding wind speeds within a tornado. The intensity of tornadoes may vary and, therefore, the expression of probability of occurrence of a tornado must also take into account the intensity of the tornado.

In 1971 a rating scale for tornadoes was developed by Fujita (1971) that was based on observed damage in the path of the storm. The rating is called the F-scale. It depends on the severity of damage and relates to maximum windspeed. Ratings vary from F 0 to F 6. Since 1971 all tornadoes have been rated according to the F-scale. Pertinent data on tornadoes is compiled at the National Severe Storms Forecast Center of the National Weather Service in Kansas City, Missouri.

McDonald, Minor and Mehta (1973) have used that data to predict the probability of occurrence of tornadoes with windspeeds exceeding a particular magnitude, V . The curve so obtained for the geographical area east of the Continental Divide is shown in Fig. 14. From Fig. 14 it can be seen that the probability of occurrence of a tornado with winds exceeding about 250 mph is less than 10^{-5} for a 50-year period. However, in their article they state that the values in Fig. 14 are for the probability of winds exceeding a given value in a year, not 50 years as listed on the figure. The probability of tornado occurrence for the one-degree square in which the Sweetwater uranium project is located is roughly 1.6×10^{-4}/year (U.S. Nuclear Regulatory Commission, 1977). The point probabilities of tornado occurrence for the Fort St. Vrain nuclear generating station is roughly 1×10^{-3}/year (Public Service of Colorado). Similar site specific data can be used to predict the recurrence of tornadoes for a specific impoundment site in a manner similar to that used for earthquakes. Figure 15 shows the mean annual frequency of occurrence for the entire U.S. for the period 1953-1962

Generally, however, tornado activity is lower than that shown in Fig. 14 for most areas in which uranium is currently being milled.

Tornado damage could be severe to a mill or the tailings impoundment during operation of the mill. However, after abandonment, if the pond has been drained, the effect of a tornado would be much less. The high winds in the tornado could cause wind erosion. However, the duration of the winds would be relatively short and the tornado would not, in itself, remove much material. It may, however, cause some local erosion that could be magnified by other erosional processes.

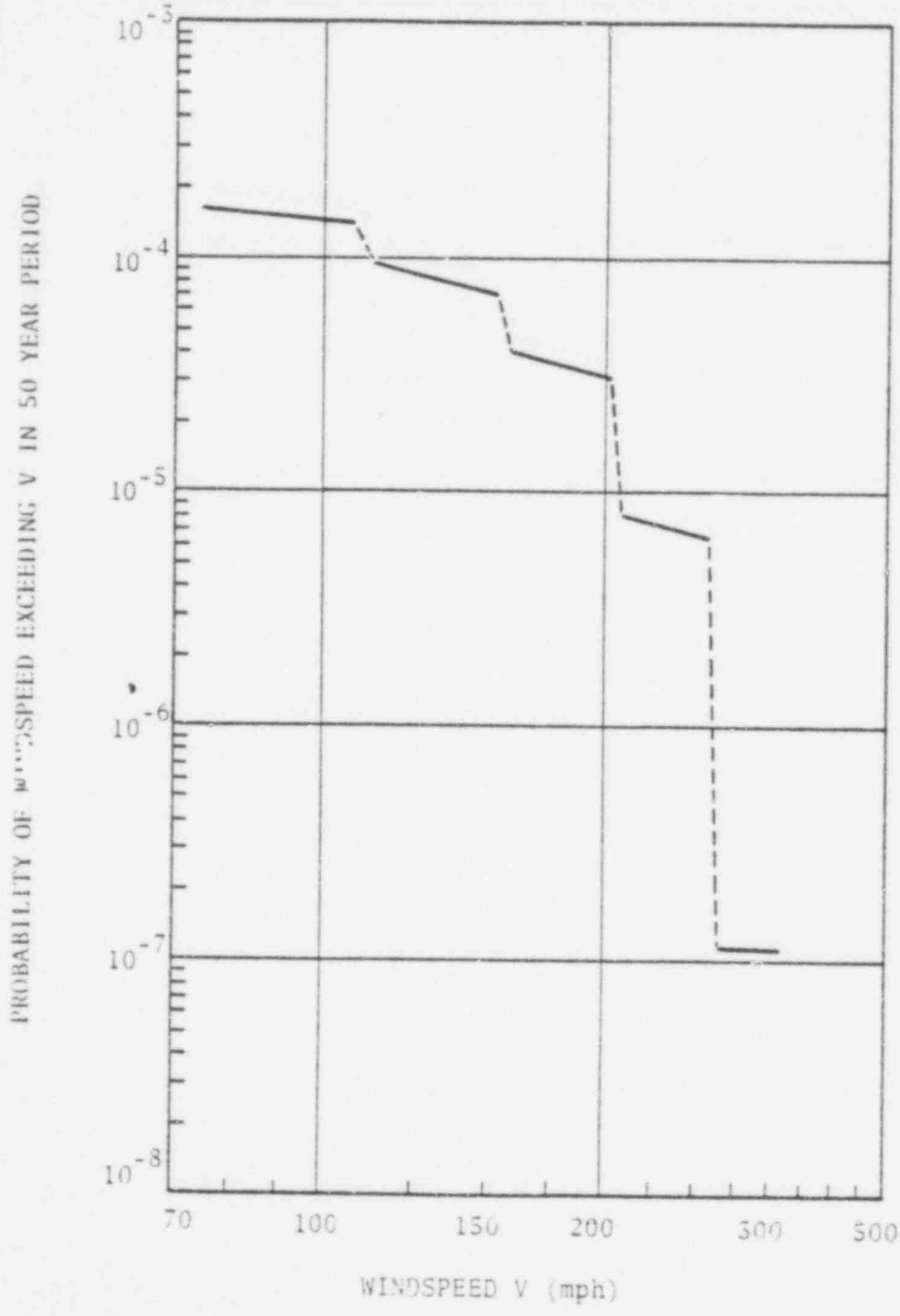

Fig. 14. Recurrence interval for tornadoes. (After McDonald, Minor and Mehta, 1973).

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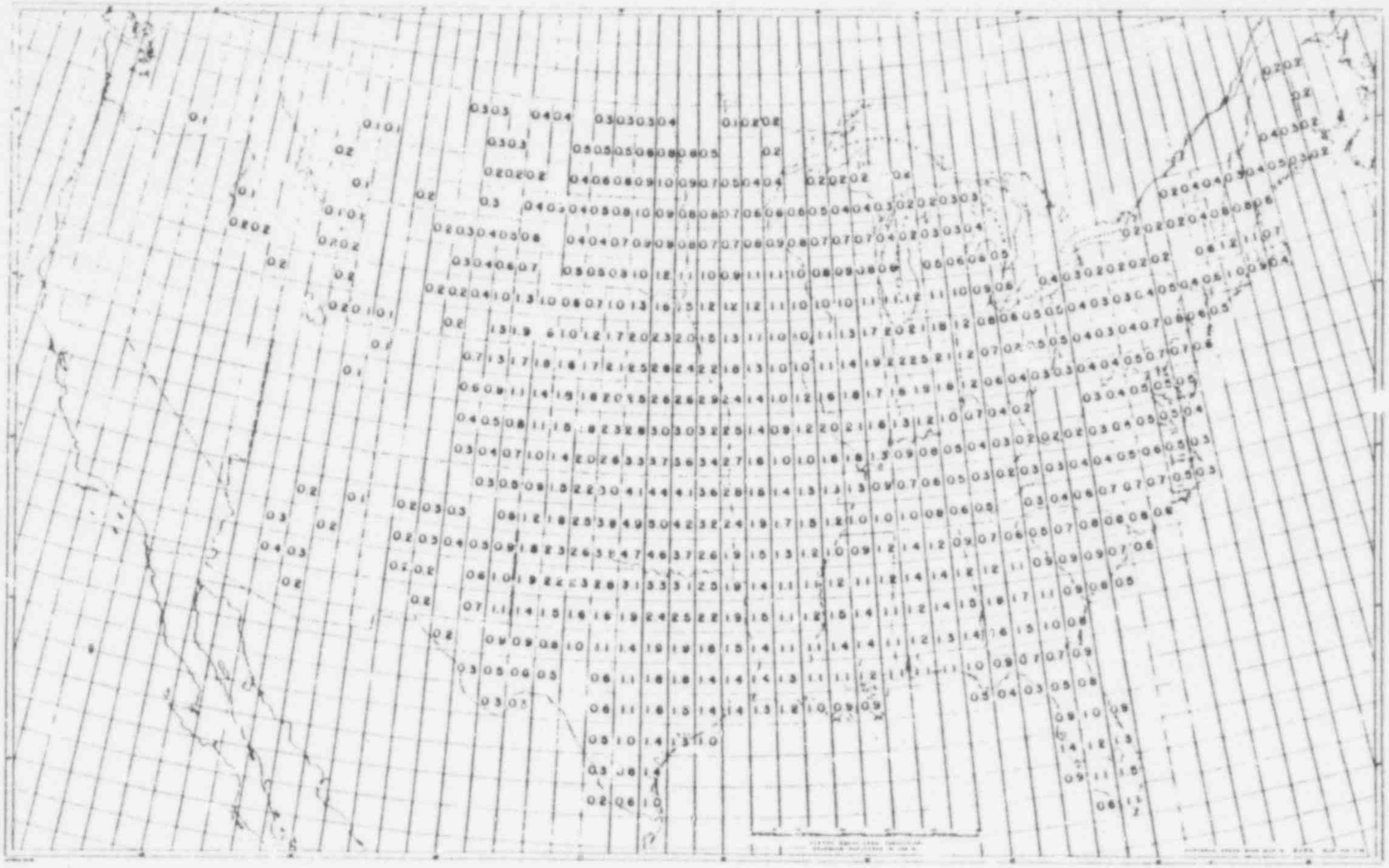

Fig. 15. Mean Annual Frequency of Tornadoes, 1957-62 (from Thom 1963).

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If the tornado is able to pick up radioactive material, it may be transported over large distances. However, on an abandoned impoundment the only relatively small amounts of tailings could be dispersed by tornadoes. Consequently, the release of radioactive material would be minimal. If a cap or some form of cover exists on the tailings no actual release of radioactive material would occur.

It may be concluded, therefore, that release due to a tornado passing over an abandoned impoundment would be of little concern even for the long time periods considered herein.

5. GLACIATION

a. DESCRIPTION OF PHENOMENON

A glacier is a mass of ice that is moving or has moved due to the force of gravity. Glaciers are formed where the climatic conditions favor snowfall precipitation in quantities exceeding the quantities depleted by sublimation or melting. As the depth increases, the snow recrystalizes into ice crystals and then into solid glacial ice. When the weight of the mass exceeds the static resisting forces, the mass begins to move and a glacier is formed.

Glaciers occur in mountain valleys and as ice sheets. Valley glaciers are formed at high elevations and flow down existing valleys to an elevation where the ice flow rate is in equilibrium with the rate of dissipation. Since these rates are functions of climatic variation, the equilibrium front may advance or recede with slight changes in the climate (Leet and Judson, 1971). Ice sheets cover broad areas and flow radially from a central mound. The largest ice sheets are the continental glaciers of Greenland and the Antarctic.

The characteristic movement of glacial ice would be the phenomenon which could damage a tailings impoundment. The individual sand grains would be picked up by the glacier and transported to the zone of wastage where they would either be deposited or be transported by water.

b. INTERACTION WITH OTHER FAILURE MECHANISMS

The movement of a glacier of even a small size would trigger several other failure mechanisms of a tailings dam. Blockage or destruction of diversion canals could cause a failure due to overtopping, erosion, or flood. The weight of the glacial ice could cause differential settlement or embankment instability. All vegetation could be destroyed, leaving any remaining tailings subject to erosion by wind and water. Because of the magnitude of the forces associated with glaciation and the related failure mechanisms that could occur due to a glacier, it is very doubtful that any portion of a tailings impoundment could survive even a small, relatively short-term glacier.

c. LIKELIHOOD OF OCCURRENCE

Climatic conditions favoring glaciation are the reduction of mean temperature and an increase in precipitation in the form of snow. Climatic changes may occur due to a change in the solar energy which reaches the surface of the earth (Lamb, 1972). Variations in solar energy which have effected the earth's climate over geologic history may have been caused by volcanic pollution, dust, or by cyclic changes in solar activity. Other theories concerning the causes of glaciation involve continental drift, variations in the orbit of the earth, and fluctuations in the oceanic circulation patterns (Leet and Judson, 1971). Although the causes of long term climatic changes are not known, there will be changes in the future.

The likelihood of occurrence of continental glaciation in the Western United States even within a long long-term period is remote.

Although there have been four major advances of glaciers in the past one million years (Pleistocene Epoch), there is no evidence of continental glaciation south or west of the Missouri River.

There is, however, a possibility of increased valley glaciation in the mountainous regions in the West. A climatic change over a much smaller area could result in an increase in valley glaciers or small ice caps at higher elevations. There exist several glaciers at high elevations in the Rocky Mountain area today. Heavy glacial activity existed in the mountains as recent as 10,000 years ago. A significant increase in valley glaciation is considered to be remote within the short long-term periods, possible within medium long-term periods and likely within a long long-term period.

d. MAGNITUDE OF RELEASE

The magnitude of release of tailings due to glaciation would depend on the location of the impoundment in relation to the equilibrium front at the forward edge of a valley glacier. If the equilibrium front did not reach the impoundment, release water could be diverted around the structure without release of tailings. If the glacier moved onto or past the tailings impoundment, the entire impoundment would probably be released by one of the failure mechanisms described above. However, because of the slow annual movement of glaciers, it may be possible to collect and reimpound radioactive material being released by the glacier.

e. SITE CONSIDERATIONS

Previously glaciated mountain valleys would be less desirable for tailings disposal sites than non-glaciated sites such as flat terrain or valleys created entirely by erosion.

f. DESIGN CONSIDERATIONS

At the present time, technology does not exist to design any facility to withstand forces of glaciation.

6. FIRES AND PESTILENCE

The discussion of failure due to the natural phenomena of fire or pestilence parallels that found in Section A.4, Revegetative Failure. Fire is referred to specifically in this section, while pestilence, defined as any disease or organism induced failure of vegetation, would result in similar effects. The ability to predict or anticipate a pestilence failure is based on an analysis of the vegetation type, its resistance to disease or predation, and the climate. Future vegetation types that result because of changes in climate or invasion of native species will form the basis for evaluation of potential future problems.

The likelihood, magnitude of release and interactions would be identical to those identified and discussed in Section A.4. No general predictive model appears to be available for pestilence-caused failure of vegetation.

V. A METHODOLOGY FOR THE COMPARATIVE EVALUATION OF DISPOSAL ALTERNATIVES

This methodology has been developed to allow comparative evaluation of alternative uranium tailings disposal schemes on the basis of a weighted score of all potential modes of failure. The weighted score is an ordinal scale which represents the undesirable expected outcomes as high values desirable outcomes with low values. The synthesis of the scores and the theoretical basis for the methodology are described in the following.

A. Theoretical Framework

The theoretical basis for the methodology lies in the assumption that the level of long-term risk of a disposal alternative can be analyzed as a function of the expected and predictable consequences of a set of potential modes of failure. The consequences of these potential modes of failure for each alternative can be valued in several different ways for comparison purposes. The most useful and defensible scale for measuring performance would be an ordinal scale. This scale describes quantities in terms of greater or smaller, but does not imply distances between values. Such a scale will be used for measurement in this methodology. However, quanitification of pertinent variables will be used to define the position along the ordinal scale to which a particular item belongs.

The severity of failure is defined as a function of the likelihood of failure, the expected magnitude of failure, and the negative utility of failure (Fig. 16).

The expected severity of failure for each potential failure mode will be scored and added for each disposal alternative. The scores can then be compared to suggest relative differences in the aggregate performance of alternatives (Fig. 17).

$$\underbrace{\text{Likelihood of Failure} \times \text{Expected Magnitude of Failure} \times \text{Negative Utility of Failure}}$$

$$\downarrow$$

Severity of Failure

Fig. 16. Severity of failure

Potential Modes of Failure	Expected Severity of Failure
F_1	S_1
F_2	S_2
F_n	S_n
	$\sum_{1}^{n} S_i = A_1$

Ranking of Scores of Different Alternatives

$$A_1 > A_2 > A_3 > A_4 > A_n$$

Fig. 17. Scoring of alternatives

B. Likelihood of Failure (L_i)

The likelihood of failure is a function of design, engineering, material, site, climate, maintenance, and monitoring characteristics of a disposal plan alternative and probability of occurrences of failure mechanism. In this report the likelihood of failure is described by an ordinal scale.

The likelihood of failure is a term that will be used to describe, in as quantitative terms as possible, the probability that a particular mode of failure will occur. In some instances the occurrence of a natural phenomenon may in itself constitute a form of failure (e. g., tornade, earthquake of magnitude "R", flood larger than maximum probably flood, glacier advance, etc.). In those cases the probability of occurrence can be fairly well defined in quantitative terms if statistical data is available.

On the other hand, phenomena such as creep failure of embankment slopes, slumping of diversion ditches, etc., are not easily quantified and the determination of a probability of failure is, to a large extent, subjective. Consequently, to assign a value of "probability" may be misleading and the term "likelihood" is chosen as being less specific.

For sudden occurrences such as are listed above, a likelihood of occurrence may be defined. However, if a quantitative value is to be used the interval values that may be assigned should reflect the confidence with which the likelihood can be determined. It is intended at this time to describe likelihood on the basis of a scale ranging from 0 to 10 with 0 representing a probability of occurrence of 0.0 and 10 representing a probability of occurrence of 1.0. However, because of the subjectiveness involved in assigning many probabilities it is intended to only use the numbers 1, 3, 5, 7, 9 and 10. This implies that the confidence interval corresponds to a probability of 20 percent and the selection of that interval is in itself somewhat arbitrary.

For continuous processes such as wind erosion of the cap, gullying of the embankment, etc., the probability of occurrence is 1.0. However, the extent to which the continuing process may result in an increased radiation release, within the time frame being considered, will define the severity of failure.

The severity may be expressed at different points of time as well. For example, over a period of 100 years a cap may be reduced in thickness by a predictable amount resulting in a particular amount of increased radon gas diffusing through the cap. Over a 1000 year period the amount of cap loss may change and interaction with other failure modes such as seepage or differential settlement may increase radon gas emanation. The result is that the severity at the end of the 100 years and 1000 years may be different.

In assessing the severity for continuous processes, the maintenance and monitoring programs may have a pronounced influence.

C. Expected Magnitude of Failure (M_i)

This value describes the expected amount of radioactive material that could be released if a particular mode of failure should occur. A value for magnitude of failure was assigned for each potential mode of failure.

Four discrete modes of release are described. It was considered that radioactive material or gamma-ray emission could be released in the form of randon emanation, generally through the exposed or covered surface of the impoundment, dissolved radionuclides escaping in seepage from the impoundment, undissolved radionuclides escaping by physical transport, i.e., mass movement, wind-blown dust, flood transport, and a reduction in gamma-ray attenuation. Although these four modes of release are not independent of each other they were considered separately for quantification of release.

For each mode, the magnitude was determined as a percentage of the maximum amount of radioactivity (or reduced gamma-ray attenuation) that could be released via that mode. So as not to imply a confidence level of greater accuracy than exists, only values of 0, 1, 3, 5, 7, 9 and 10 were assigned. The magnitude was then presented as a matrix as shown below.

$$M_i = \begin{bmatrix} w \\ x \\ y \\ z \end{bmatrix}$$

w = radon emanation

x = dissolved radionuclide release

y = undissolved radionuclide release

z = reduction in gamma-ray emission attenuation

D. Negative Utility of Failure Mode (U_i)

The negative utility is a weighting factor which considers the extent of the problem posed by a particular failure mode. Because pathways of escape of radiation are different for different potential modes of failure, they imply different kinds of hazards, different levels of hazard, and different control problems. These are considered in the development of the negative utility factor. To a large extent the determination of U_i is a subjective evaluation.

For example, slope failure and mass movement of material from a waste embankment may result in much material movement. However, in terms of control and problems created, it may not be great. The material that is moved may not travel far and it will be relatively easy to recapture. The failure can be anticipated, and is easily identified and remedied once it occurs. A slope failure, without liquefaction would, therefore, have a low value of negative utility.

By way of comparison, a seepage failure may result in large quantities of radiation release that may be difficult to detect. In addition, maintenance and remedial measures are very difficult. Therefore, the negative utility would be high.

The subparts of the negative utility function would be

1) Consequences of potential failure (C_i),
2) Response to and ease of maintenance (M_i), and
3) Response to and ease of monitoring (MO_i).

Maintenance and monitoring are assumed for the purpose of defining the negative utility to be independent of the disposal alternative. It should be noted that maintenance and monitoring are part of the disposal alternative affecting the likelihood of failure.

The negative utility will be the same for all the potential failure mechanisms within a failure mode. The negative utility factors are shown in Table 13. The values presented were arrived at as the best estimate of the project team.

Table 13. Negative utility of failure.

Potential Failure Mode	Negative utility, u
Cap	0.5
Liner	2.0
Embankment	1.0
Revegetation	0.25
Diversion Structures	0.75
Earthquakes	2.0
Floods	1.75
Winds	1.5
Tornadoes	1.75
Glaciation	1.0
Fire and Pestilence	0.25

E. Discussion of Methodology

A point that should be noted is the relationship between likelihood and probability of failure. These are essentially the same thing. However, because there may not be a statistical basis for describing a probability of failure for all failure modes and in order to avoid the suggestion of a statistically accurate quantity, the term "likelihood" of failure would be the best wording.

The entire methodology is summarized in Fig. 18. The methodology is designed to provide a means of comparing alternative disposal plans on the basis of expected risk for the aggregate failure modes. The application of this methodology provides a valuable tool for management decision making. It provides insight into the sensitivity, and to some extent, cost effectiveness of design, siting and operational options that may be available for each disposal alternative. The decision maker can evaluate the effect of changes within his control for a particular alternative. The methodology also provides a consistent basis for evaluating the effectiveness or performance of alternative disposal plans. The scores or indexes are based upon ordinal values and subjective judgments, but within the state of the art they provide a realistic and appropriate framework for judgment and decision making. The methodology also provides a clear description of procedure in order that new information can be integrated into the process as it is developed.

STEP 1

$$\left.\begin{array}{l}\text{Engineering Design}\\ \text{Site Selection}\\ \text{Climate}\\ \text{Maintenance}\\ \text{Monitoring}\\ \text{Natural Phenomena}\end{array}\right\} \rightarrow \text{Mode of Failure} \Rightarrow \text{Likelihood of Failure } (L_i)$$

$$\text{Likelihood of Failure } (L_i) \times \text{Magnitude of Failure } (M_i) \times \text{Negative Utility Factor } (U_i) \Longrightarrow \text{Severity of Failure } (S_i)$$

STEP 2

$$\sum_{i=1}^{n} S_i = \text{Alternative Index, } A_j$$

n = Number of Potential Failure Modes

STEP 3

A_j for various alternatives may then be compared. (Low A_j desirable)

Fig. 18. Methodology for Comparison of Disposal Plan Alternatives

VI. APPLICATION OF METHODOLOGY TO BASE CASE AND ALTERNATIVES

In this chapter the methodology for assessment of the long-term reliability of tailings impoundments (Chapter V) was applied to the Base Case and the eight alternatives presented by the Argonne National Laboratories. These various alternatives are shown in Figures 19 through 27. In the following discussion, general comments pertaining to the application of the methodology for each potential failure mode are followed by a short discussion or computation to justify the selection of a particular value for Likelihood or Magnitude of failure. The values are shown in Table 14.

For each failure mode four values of magnitude of failure are shown. These four values represent the four separate release modes discussed in the previous chapter on methodology. Corresponding to these four values, four severity of failure values are developed. In most cases not all the release modes are significant. Therefore some mode values will be zero. An example of this would be cap failures which result only in changes in radon emanation and gamma emission. In contrast would be a flood failure which results in release due to all four modes.

For each potential failure mode the negative utility factor would be the same for the Base Case and all eight alternatives. The selection of a negative utility factor is subjective and was arrived at by discussion among members of the project team. The values used herein are presented in Table 13 and are not discussed at this point any further than was presented in conjunction with Table 13. Further experience, developments in technology or differing opinions of investigators may indicate the desirability of changing the negative utility factors. The values shown in Table 13 represent only the best estimates for present conditions of the project team.

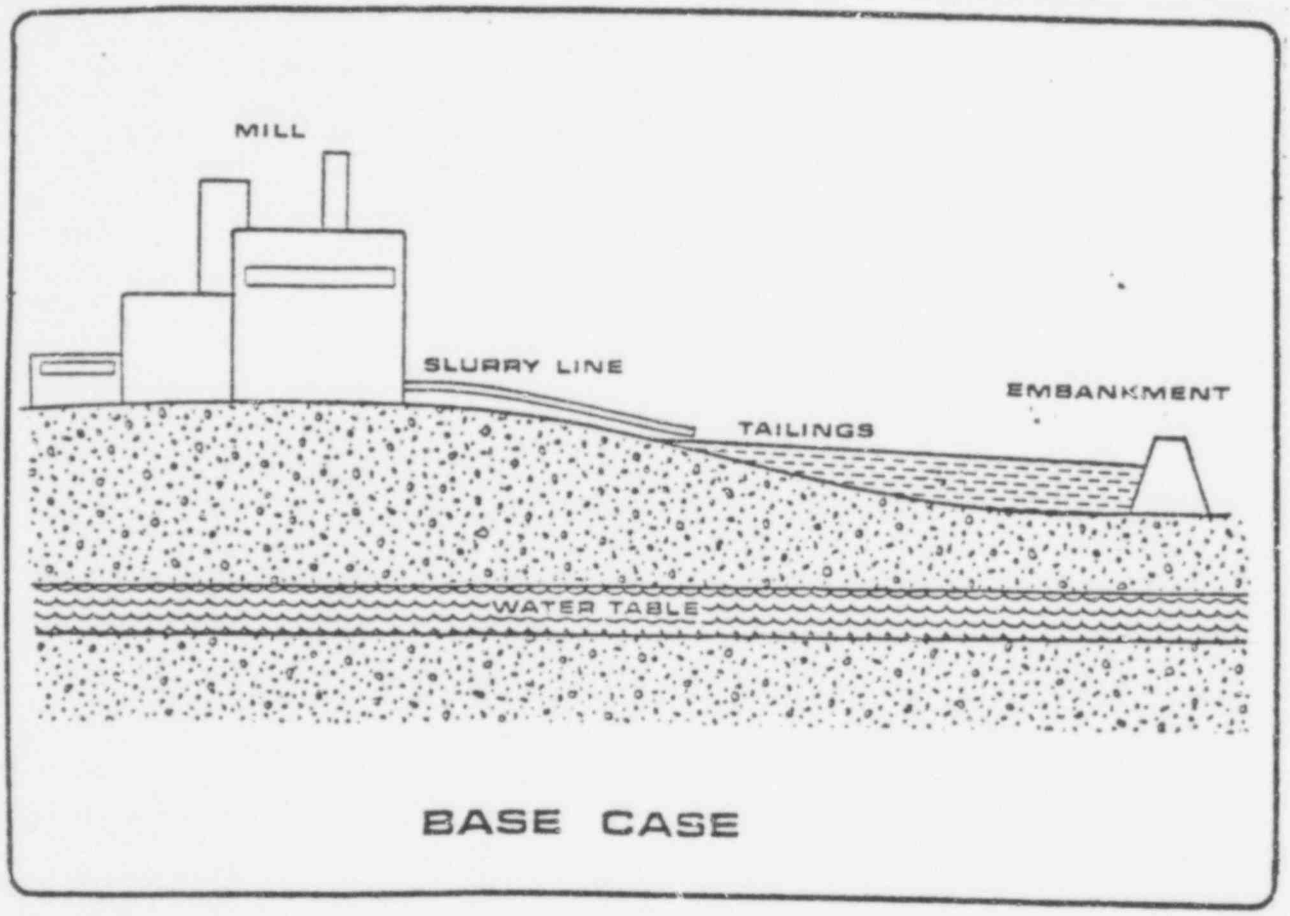

Fig. 19. Untreated tailings deposited on unprepared ground and retained by embankment(s).

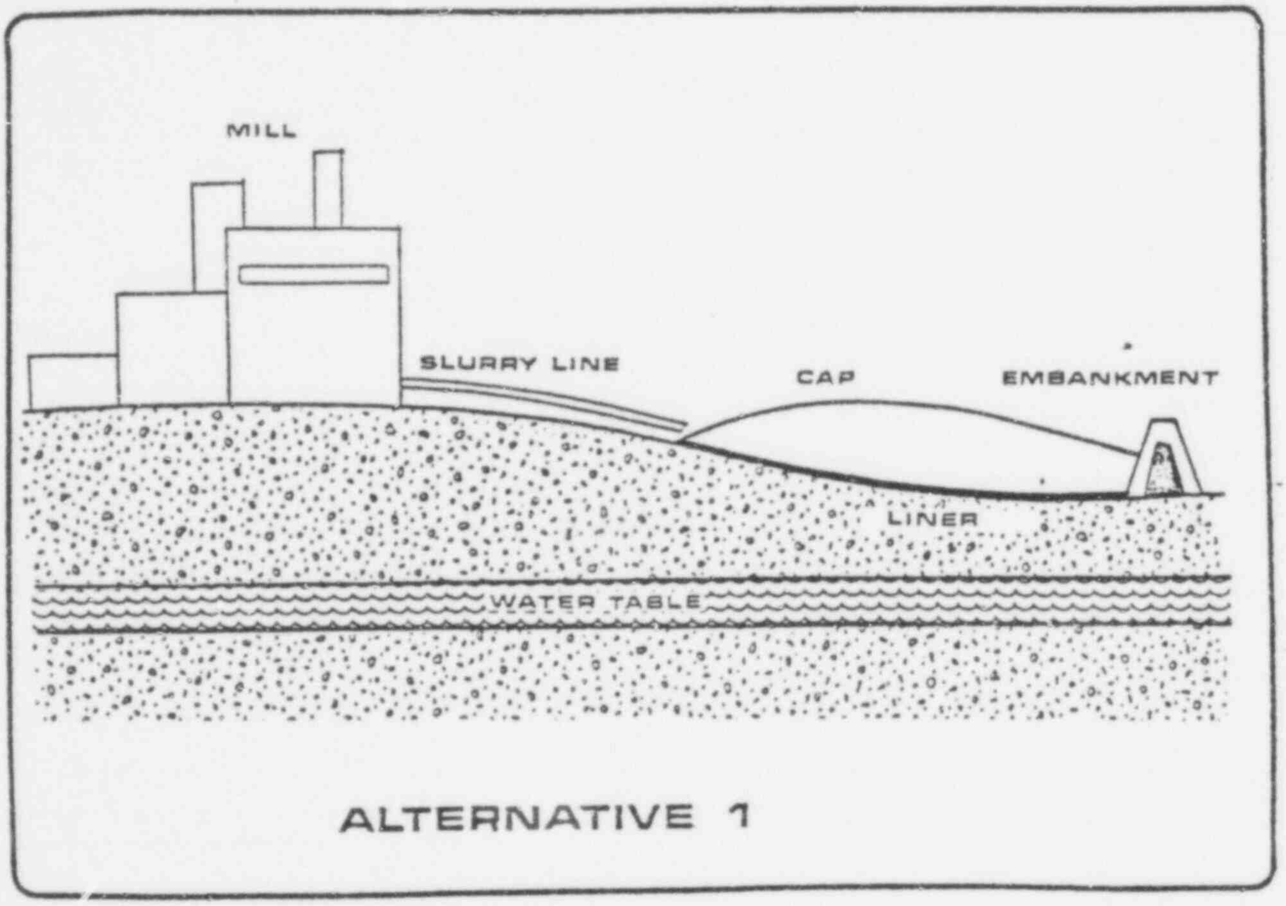

Fig. 20. Untreated tailings deposited on prepared ground, capped, and retained by embankment(s).

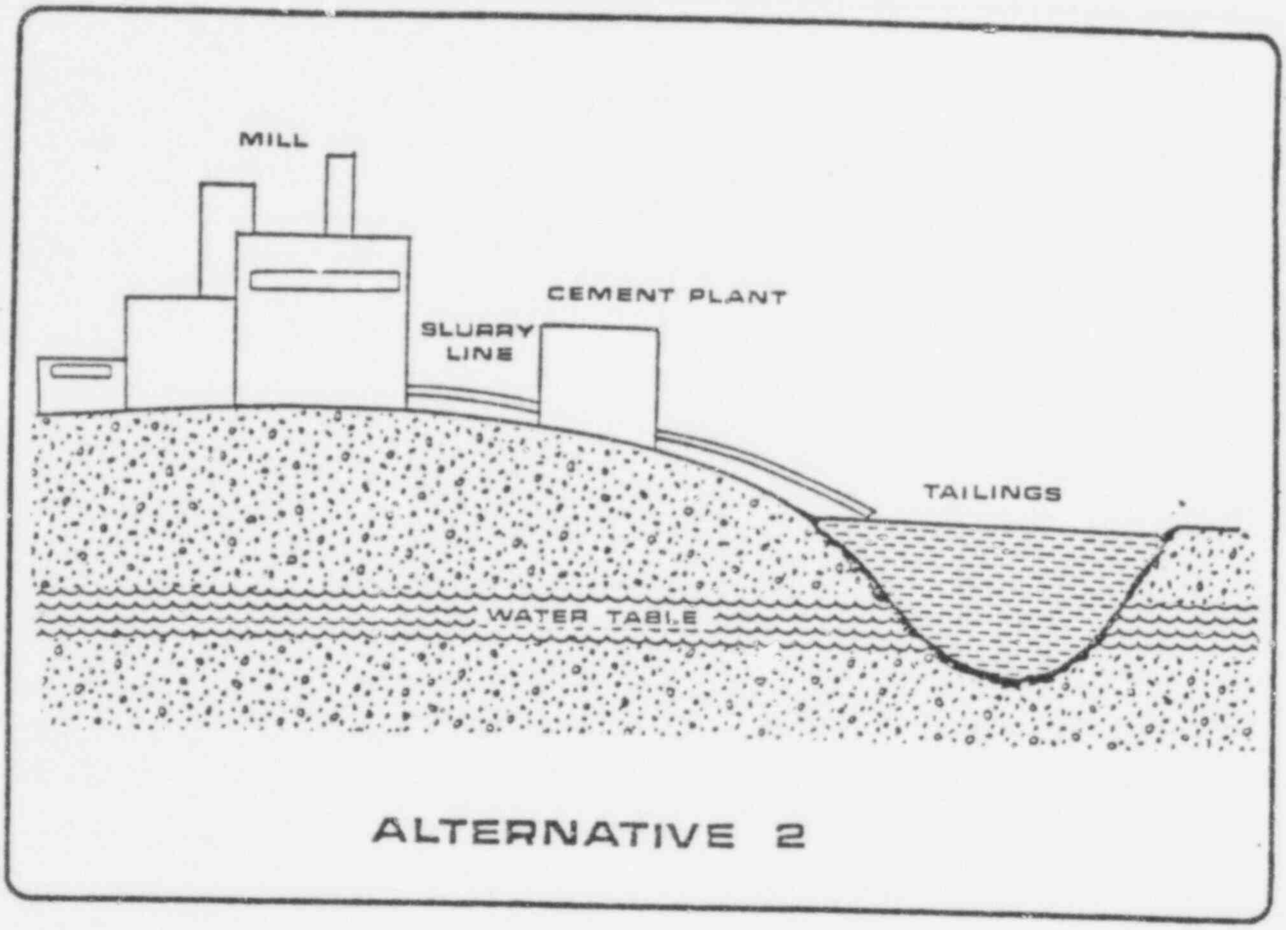

Fig. 21. Fixed tailings deposited in unlined open pit mine, suitably covered.

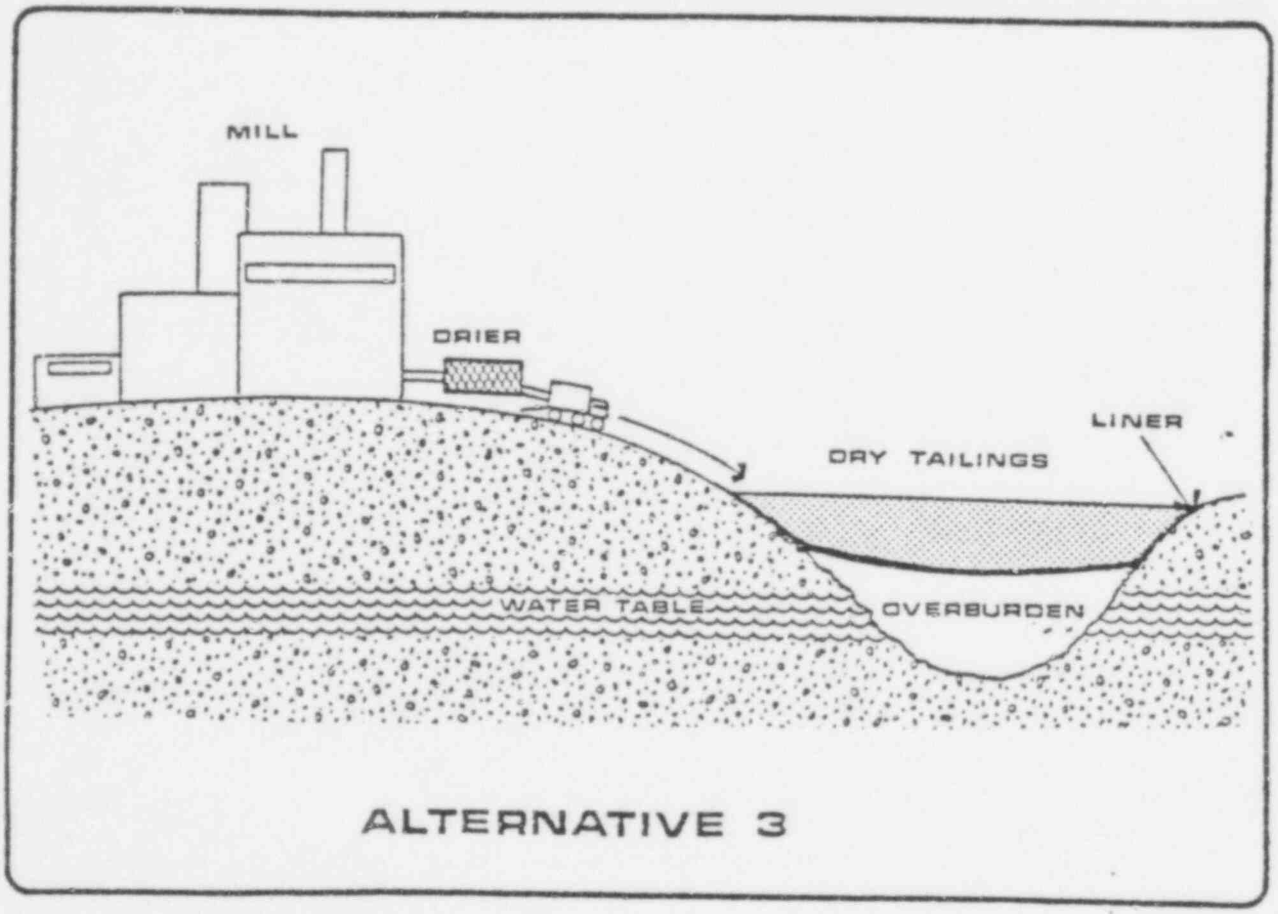

Fig. 22. Dried tailings deposited in filled and lined open pit mine, suitably covered.

432 110

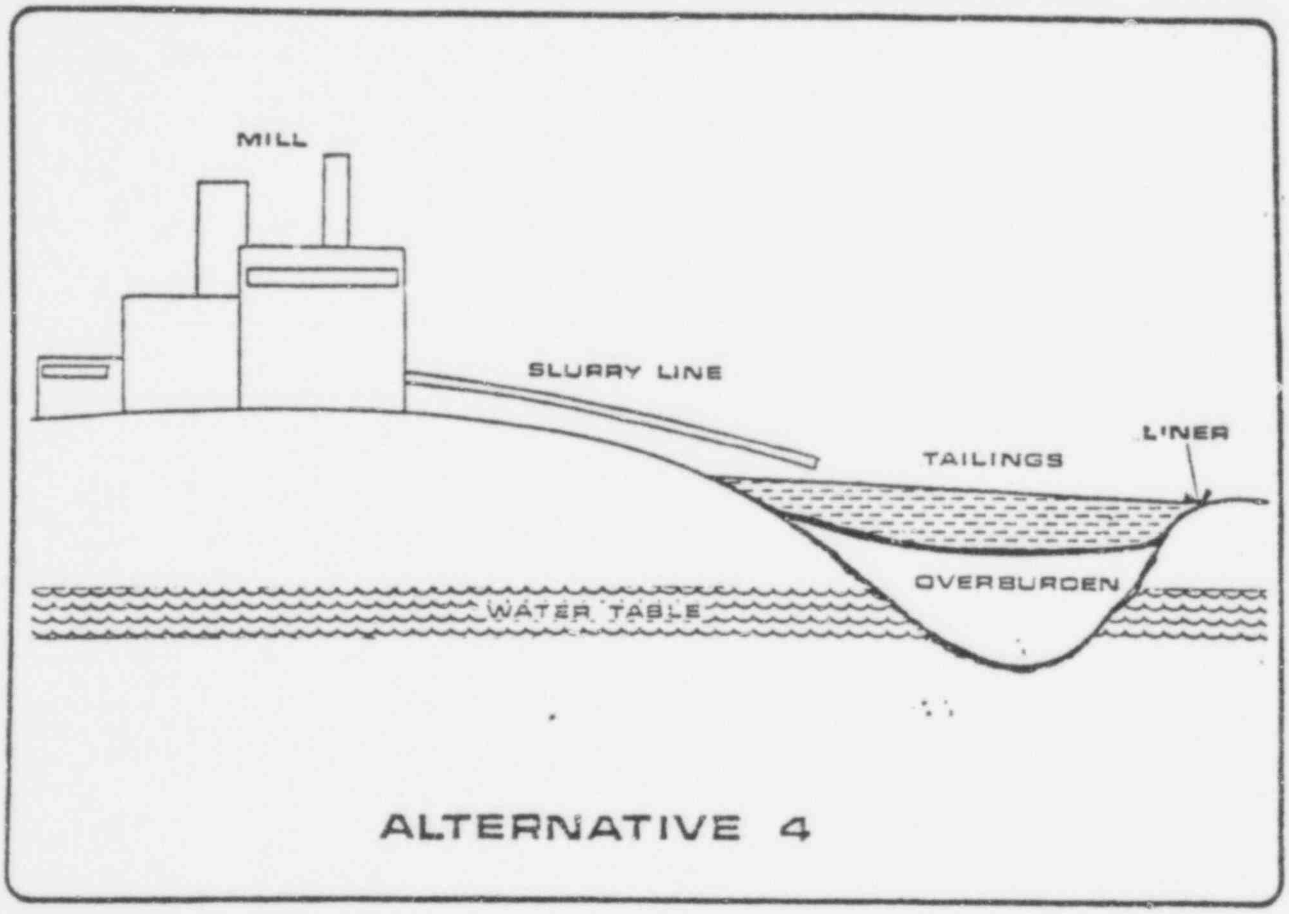

Fig. 23. Untreated tailings deposited in filled and lined open pit mine, suitably covered.

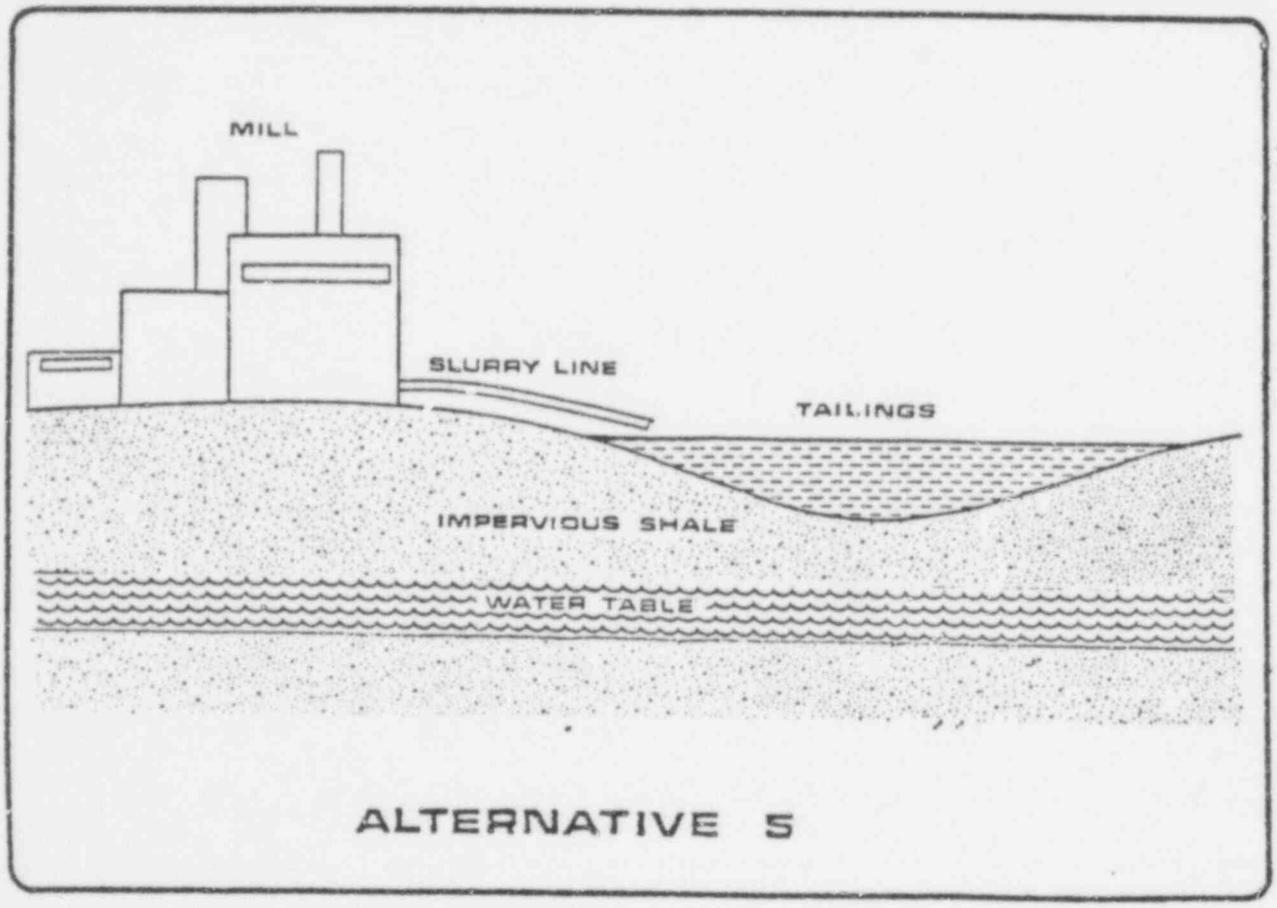

Fig. 24. Untreated tailings deposited in excavated pit in impervious formation (if available), suitably covered.

632 182

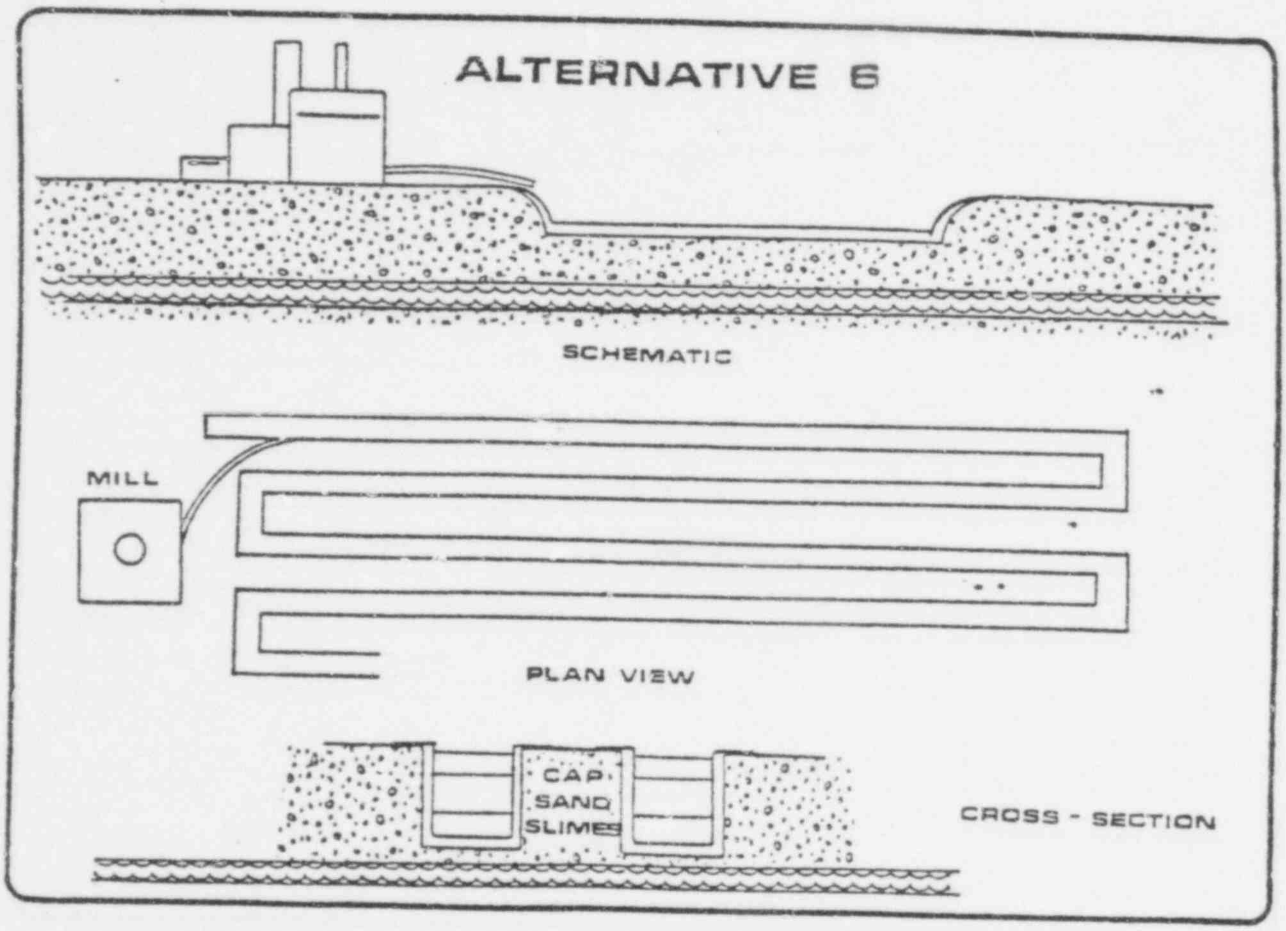

Fig. 25. Untreated tailings deposited in excavated lined trench, suitably covered in landfill type operation.

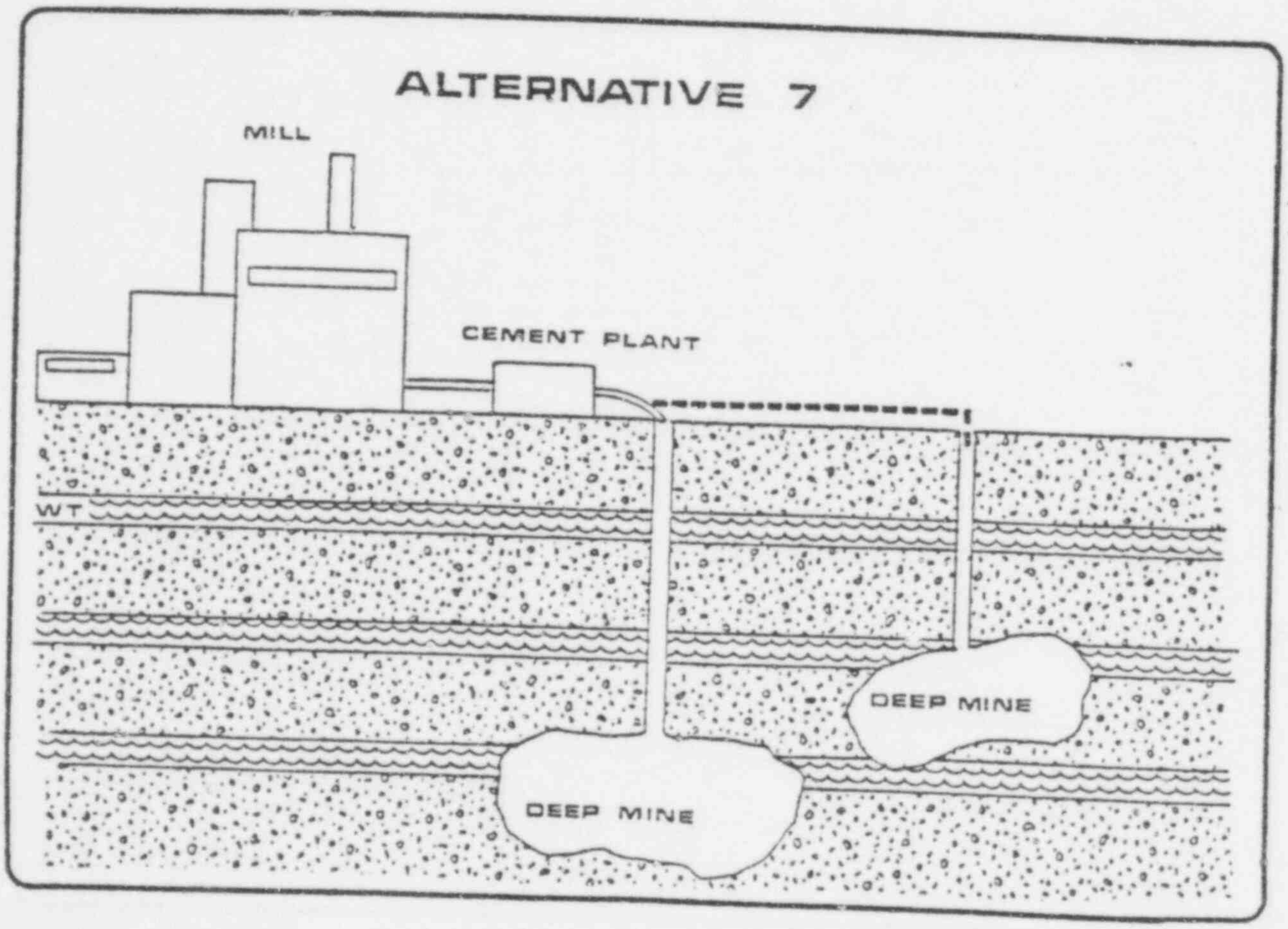

Fig. 26. Fixed tailings deposited in unlined deep mine.

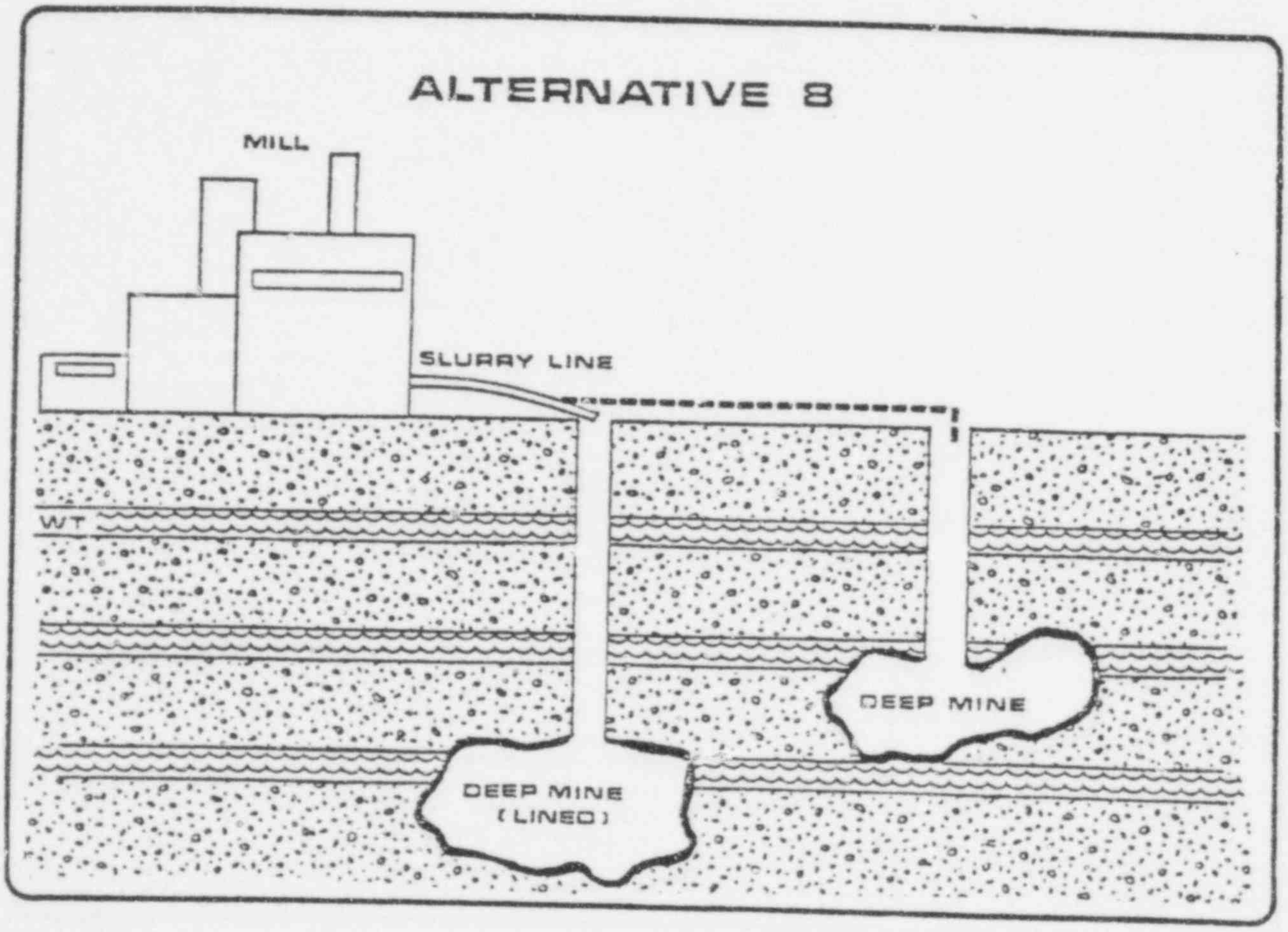

Fig. 27. Untreated tailings deposited in lined deep mine.

The depiction of tailings disposal in Figures 19 through 27 are considered to be schematic. For example, it is assumed that deposition of tailings would be done, with or without the use of cyclones, to ensure that the coarse tailings are placed near the embankment and that the slimes zone is near the middle of the impoundment. Also the embankment (where shown) may exist over any number of sides (1 to 4) of the impoundment. For disposal plans that may require maintenance the required maintenance is considered to be a part of the design plan. It is also assumed that the maintenance is carried for the short long-term period only.

The value of severity listed in Table 14 was arrived at by multiplying the likelihood, magnitude and negative utility factors. Four values of severity are developed because there are four values for magnitude of failure. These four values are not added to arrive at total severity. This could be done but caution must be exercised in the use and interpretation of such a total. The individual values for those factors were ordinal and the product of ordinal numbers may not result in another ordinal scale. For that reason some subjective adjustment of the severity was made in certain cases to more reflect the severity on an ordinal scale. The values of likelihood, magnitude and utility factor were not changed, however. In order to reflect the accuracy of predicting likelihood and magnitude, only values of 0, 1, 3, 5, 7, 9 or 10 were used. The negative utility factor was purely subjective. Therefore, the value of severity computed was rounded off to the nearest whole number. Caution should be exercised in determining what difference in severity actually constitutes a significant difference. The writers make no attempt to quantify that.

In the following discussion some repetition exists between failure modes. That repetition was left in the text to minimize the amount of cross-referencing needed if the reader is considering only a particular failure mode.

1. Cap Failure Due to Differential Settlement (Ala)

   a. General Comments

The general thickness of the alluvium is about 150 to 300 ft thick. The soil association to which this belongs is the Petula-Tomahawk Association which is underlain by sandy clay loam and fine sandy loam. These soils are typically fairly free draining and in the geologic setting shown for the UMOGES would contain layers of clay, silt, and combinations of these soils. The clay content however, is expected to be fairly low because it is a relatively recent soil. Because the site is in a fairly wide floodplain, isolated pockets of soil having variations in properties are expected to occur within the 250 acre area of the tailings impoundment.

The release modes of importance for all cap related [illegible]res are radon emanation and gamma ray emission. Each release is attenuated by a soil cover. However, radon emanation is much more significant because its half thickness (the thickness of soil cover required to reduce emanation by 50%) is 4 feet compared to the gamma ray half thickness of 4 inches. Only in case where no cover exists or is entirely removed is gamma ray a release of concern.

b. Base Case

i) Likelihood -

Because no cap will be used the likelihood of failure is 10 for all time periods. The absence of a cap is tantamount to certain failure of a cap.

ii) Magnitude -

Because no cap will be used the magnitude of radon emanation and gamma ray emission is 10 for all time periods. The absence of a cap is tantamount to release of the maximum amount of air-born activity possible.

c. Alternative 1

It is assumed that the cap for Alternative 1 will consist of approximately 10 inches of clay and several feet of overburden varying in depth from shallow (3 to 6 ft) over the coarse tailings near the embankment to deep (10 to 20 ft) over the slimes zone. It is assumed that the thickness of the cap has been designed to permit some differential shear displacement of a predicted amount. Because all settlement will occur during the short long-term period, the likelihood will not change for other time periods.

i) Likelihood -

Because of the existence of pockets of different soils in the alluvium some differential settlement will occur. However, much of the settlement will have been completed before placement of the cap. Consolidation of the tailings will result in uniformly distributed differential settlement contributing to tension type cracking. The likelihood of occurrence of settlement of an amount greater than allowed for in the design was estimated to be 7.

ii) Magnitude -

On the basis of the computations shown in Example 1 of the discussion of this failure mechanism, the magnitude of release is considered to be low. A value of 1 has been assigned. This value could increase if a brittle cap liner with no additional cover is used. For example, if an asphalt or concrete cap of inadequate thickness is used, the magnitude may increase to 3 or greater if large shear type differential settlement can occur.

d. Alternative 2

i) Likelihood -

Because the weight of tailings will be less than the weight of the overburden removed from the open pit mine, differential settlement of the foundation subsoils will be small. Some elastic rebound due to unloading may occur but should be minimal. Settlement due to compression of fixed tailings will be minimal. Total differential settlement will be minimal and a value of 1 was assigned.

ii) Magnitude -

Similarly to Alternative 1, the magnitude of release would be small (1) if failure did occur.

e. Alternative 3

i) Likelihood -

Some settlement of the overburden placed in the open pit may occur. The dried tailings would consolidate during placement and before the cap is placed. Because of the more uniform nature of the overburden and the dry tailings, shear type differential settlement would be small. Differential settlement would have a minimal likelihood (1) of causing failure.

ii) Magnitude -

Similarly to Alternative 1 the magnitude of release would be minimal (1).

f. Alternative 4

i) Likelihood -

Differential settlement in the overburden at the bottom of the pit would be small but could occur. Consolidation of the untreated tailings will occur after placement of the cap. Differential settlement between the slimes zone and the coarse tailings is expected but large shear type displacements are not expected. The likelihood of differential settlement causing failure is less than for Alternative 1. A value of 5 was assigned.

ii) Magnitude -

Similarly to Alternative 1 the magnitude of release would be small (1).

g. Alternative 5

i) Likelihood -

There will probably be some predictable differential settlement of the untreated tailings between the slimes zone and the coarse tailings. The likelihood of differential movement of the impervious shale is remote. The likelihood of failure due to differential settlement is therefore somewhat greater than Alternatives 2 and 3 but less than Alternative 4. A value of 3 was assigned.

ii) Magnitude -

Similarly to Alternative 1, the magnitude of release would be small.

h. Alternative 6

i) Likelihood -

Because of the uniform nature of the deposition of tailings differential compression of the tailings will be small. Placement of tailings into a lined, excavated trench would make settlement of the foundation soils minimal. The likelihood of failure is therefore minimal (1).

ii) Magnitude -

The magnitude of cracking and hence, release would be minimal (1) particularly in view of the shorter distances over which uniform settlement would occur.

i. Alternatives 7 and 8

Because the tailings are stored in the deep mine, no cap is necessary and failure of a cap does not apply to these alternatives. Values of zero have been applied to the severity.

2. Cap Failure Due to Gullying (A1b)

a. General Comments

Because there exists no predictive model other than the threshold concept discussed in sections A1b and A3b, the prediction and evaluation of likelihood and magnitude for this mode of failure are the result of a subjective analysis of the site and design factors.

Radon emanation and direct gamma-ray radiation are the two modes of release considered significant. They are assumed to be of about the same proportionate release magnitude. Gully failure is assumed to remove cover material down to the tailing surface.

b. Base Case

i) Likelihood -

Because no cap or cover is specified the likelihood of failure is 10 for all the time periods. The lack of a cap is tantamount to certain failure of a cap.

ii) Magnitude -

Because no cap will be used the magnitude will be 10. The absence of a cap is tantamount to release of the maximum amount of activity possible.

c. Alternative 1

i) Likelihood -

The likelihood that a gully would form on the cap is moderate for short-long term periods and increases with time as shown in Table 14.

ii) Magnitude -

For the short long-term M will be low. The level is a function of the maintenance, monitoring, and remedial effort applied. It is assumed that a reasonable level is provided for this period. The magnitude is assigned a value of 1. For the medium and long long-term periods the magnitude could increase. For those time periods no maintenance, monitoring, or remedial effort was assumed. Significant portions of the cap and tailings could be exposed and removed by gullying. Severe storms can occur at the model site and could cause severe gully problems. The magnitude of release with no maintenance, therefore, was set at 5 for medium long-term periods and 9 for long long-term periods.

d. Alternatives 2 through 5

i) Likelihood -

These alternatives do not include an embankment, which could contribute significantly to the potential for gullying on the cap. The likelihood of gullying on the gentle slopes on the cap will be reduced with respect to Alternative 1 for all time periods. A likelihood of 3 was assigned.

ii) Magnitude -

If gullying did occur the magnitude of release would be the same as for Alternative 1.

e. Alternative 6

This alternative describes a highly engineered disposal plan. It is likely that a greater lineal extent will be required by the impoundments. This fact could increase the chance for gullying because it increases the chance for crossing or coming in contact with a drainage or potentially susceptible area. Those two considerations would be compensating and the result would be the same as for Alternative 2 through 5.

f. Alternatives 7 and 8

There is no possibility that surface erosion related failures would influence the underground disposal. The severity was set at zero.

3. Cap Failure Due to Water Sheet Erosion (Alc)

a. General Comments

The base case does not include a cap and therefore radon emanation and gamma emission is considered to be 100 percent. The magnitude is set at 10 and serves as the base reference for all other plans.

Because the different half thickness for these two radioactive sources are so different, 4 feet for radon and 4 inches for gamma-ray, gamma-ray emission only becomes important when significant thicknesses of cap are removed. This will be the case only in the long long-term period.

Where a cap is employed it will be assumed to vary from about 3 - 6 feet over the sands to about 10 - 20 feet over the slime and have a 10 inch liner of clay. This is similar to the Bear Creek Project (NRC, 1977a) and the Sweetwater Project (NRC, 1977b). The universal soil loss equation (USLE) was used to evaluate erosion reduction of cap thickness. This reduction was then used to estimate the magnitude of release based on the relationship of radon emanation and gamma-ray emission to cover thickness. Soil and climate factors used in the USLE were estimated on the basis of the model site description. Maintenance, monitoring and remedial measures will influence the amount of erosion loss as will long-term changes in climate. Repair to the cover was not accounted for in the USLE calculations. Important to the application of the USLE is the assumption that soil loss is constant over the entire site. That will probably not be the case. Some areas may receive material while others may erode. The most appropriate use of the USLE may be to show relative differences in erosion control plans (Heil, 1977), not to calculate absolute magnitudes of soil loss. However, it is the best tool available and can be useful if judiciously applied.

b. Estimate of Soil Loss using the Universal Soil Loss Equation

As described in Section Alb of Chapter IV, the USLE is:

$$A = RKLSP$$

From the SCS (1977) guide the factors are:

R--rainfall factor--will be taken as 40 for the average condition, over 75 percent of the time. For twenty percent of the time R will increase to 100, and for the remaining 5 percent R will be 150. In this manner infrequent but highly erosive storms were included.

K--soil erodibility factor--the cover soil was assumed to be a clay loam. K was assumed to equal 0.37. No attempt was made to estimate short term changes in K due to disturbance.

LS--topographic factor--slopes on the cover were assumed to be gentle, approximately 2 percent and the maximum slope length was 1650 ft. LS would then be equal to 0.46 (Table 2).

C--cover factor--for the semiarid environment, natural cover will probably be sparse. C factors for pasture or rangeland can range from 1.0 with zero percent cover to 0.003 for 100 percent cover with grasses. The value of C was considered to be 0.25 for covers of approximately 20 percent.

P--erosion control practices factor--it is assumed that no continuous erosion control practices are employed because of gentle slopes. P will be 1.0.

Using these values, the calculated soil loss is:

$$A = KLSCP = \text{tons soil loss/acre/year}$$

$$A_1 = 75\%(40)(0.37)(0.46)(0.25)(1) = 1.28$$

$$A_2 = 20\%(100)(0.37)(0.46)(0.25)(1) = 0.85$$

$$A_3 = 5\%(150)(0.37)(0.46)(0.25)(1) = 0.32$$

Total Soil Loss = 2.45 tons/acre/year

For a 250-acre impoundment:

Soil loss/year = 612.5 tons

If the bulk density of the soil is assumed to be 120 pounds per cubic foot the loss in cubic feet per year would be 10,200 ft^3. Over 250 acres the depth of soil loss would be 0.011 inches/year.

For the time periods considered herein the rate of loss would result in total soil losses of 2.2 inches in 200 years, 22 inches in 2000 years and 93 feet in 100,000 years.

c. Base Case

i) Likelihood -

Since no cap exists, failure is a certainty. Therefore, L is 10 for all periods.

ii) Magnitude -

Since no cap is specified the magnitude of release is set at 10.

d. Alternatives 1 through 6

i) Likelihood -

The likelihood of erosion is high (10).

ii) Magnitude -

Based on the estimate of soil loss for the short long-term period the magnitude of release for radon and gamma-ray was taken to be zero. Even if the rates of soil loss doubled the estimated reduction in cover thickness would not be significant. For the medium period, the estimated soil loss is 22 inches. This loss of thickness results in an increase in the radon attenuation factor to a value of less than 0.01 near the center and a value less than 0.20 using the data for varved clay shown in Figure 6. For a release of radon between 0 and 20 percent of the activity, a value of 1 would apply. With no maintenance and interaction with other failure modes this value could increase to 3. Gamma-ray emission remains zero. For the long long-term period total removal of the cap and probable removal of tailings is indicated. A value of 10 was used for both radon and gamma-ray. All of these estimates assume that site relief can accommodate complete removal and that there is no deposition on the impoundment from external sources.

e. Alternatives 7 and 8

There is no possibility of erosion failure for underground disposal, therefore severity is zero.

4. Cap Failure Due to Wind Erosion (A1d)

a. General Comments

The likelihood of the occurrence of wind erosion at the cap of the impoundment is not dependent upon the design concepts of the disposal plan. It is a function of climatic conditions. A value of 10 is assigned based upon the assumption that wind erosion will occur. It does not imply the seriousness of the potential. That is a function of magnitude.

The magnitude of release due to radon emanation and gamma-ray emission will be a function of the reduction in thickness of the cover due to removal of soil by wind. The amounts of reduction will be predicted by use of the wind soil loss equation discussed in Chapter IV. The specific magnitude will be estimated based upon the relationship of cap thickness to diffusion of radon gas and gamma-ray emission. Radon emanation is predicted using Figure 6. Gamma-ray emission will only be important if significant thicknesses of cap are removed. This is because of its low (4 inch) half thickness.

b. Wind Soil Loss Equation

The wind soil loss equation calculates the tons/acre/year of soil loss from a specific site based on the parameters discussed in Chapter IV. In order to apply the equation to demonstrate its application, values of pertinent factors were based upon judgment and model site information. The equation and the factor values are:

$$E = f(I, C, K, V, L)$$

where

I = soil wind erodibility factor. The cap is assumed to be a compacted clay loam. It has reasonably good aggregate characteristics, which should improve with time after initial placement. The wind erodibility group would be six (Table 3) and an estimated 40 percent of the dry soil is coarser than 0.84 mm. I, therefore, equals 47 (Table 3).

I_s = windward knoll erodibility factor. I_s would equal 150 percent based upon a 2 percent maximum grade (Fig. 7).

C = local wind erosion climatic factor. This factor was assumed equal to 60 percent based upon general wind and climate of model site (UWRL, 1976).

K = soil surface roughness factor equal to 0.6, an estimated value.

V = vegetative factor equal to approximately 8000 equivalent pounds per acre (Fig. 28a). This is based on herbage and mulch of approximately 2000 lbs. Data for this estimate is for similar precipitation areas in the western U. S. (USDA, 1968).

L = unshielded wind fetch distance. This value would equal 1650 ft. based on a square 250 acre impoundment with no wind barriers.

Based on these values the soil loss is determined by the method described in Woodruff (1965) and UWRL (1976) and presented in Figure 28b of this section, next page.

Yearly soil loss due to wind erosion using Fig. 28a is calculated to be less than 0.5 tons per acre. For a bulk density of 120 lbs/cu ft the soil loss would be less than 0.002 inches/year.

For the time periods under consideration the cummulative totals of the thickness lost would be less than 0.5 inches in 200 years, less than 5 inches in 2000 years and about 20 feet in 100,000 years.

The correction factor for knoll erodibility has been neglected because initial soil loss was below the readable limits of Figure 28a. The procedure cannot predict the possibility of blowouts forming.

c. Base Case

i) Likelihood -

No cap exists, therefore failure is a certainty and L was set equal to 10 for all time periods.

ii) Magnitude -

Since no cap or cover exists, magnitude is set at 10.

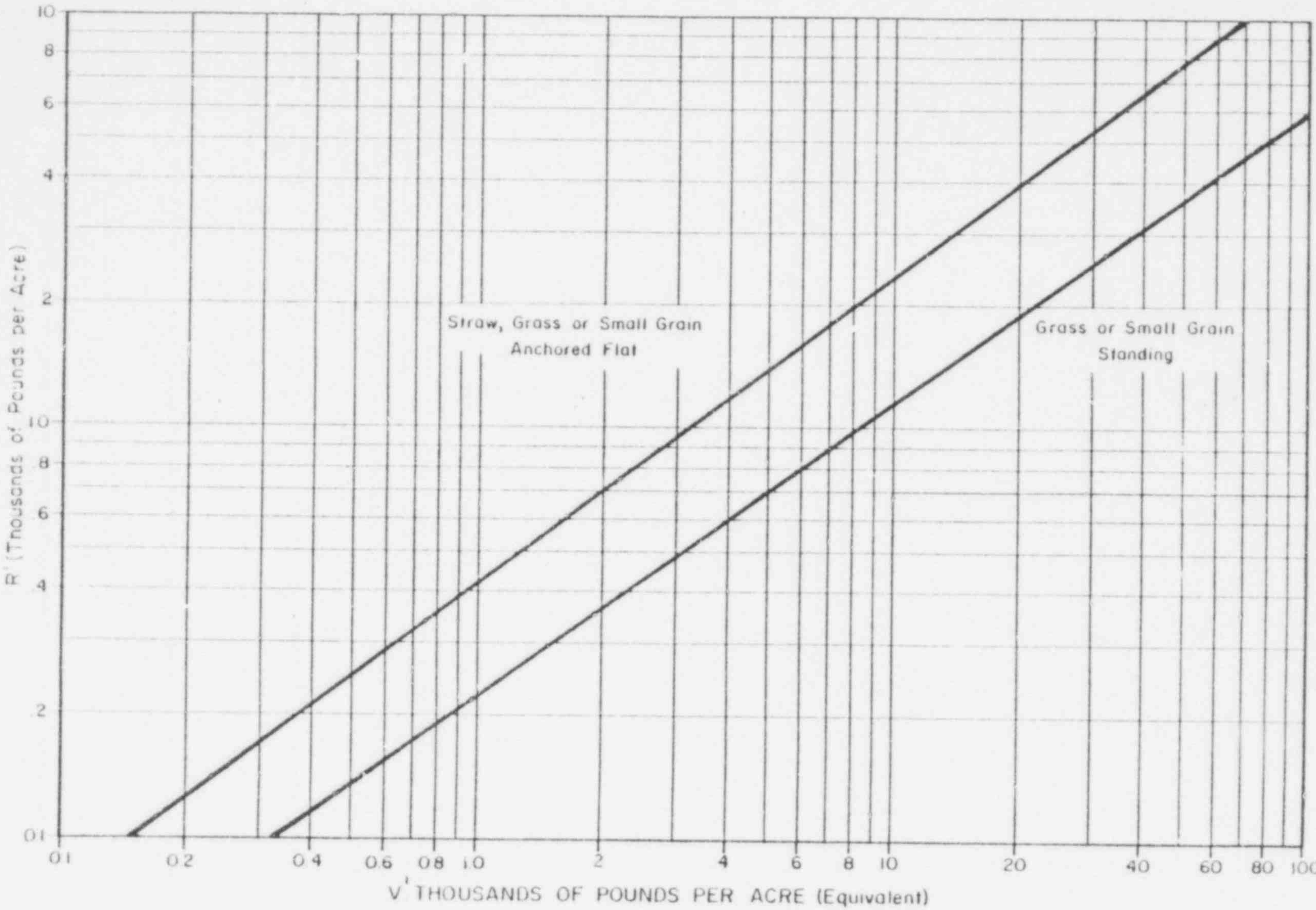

Fig. 28a. Equivalent vegetation - pounds per acre (UWRL 1976)

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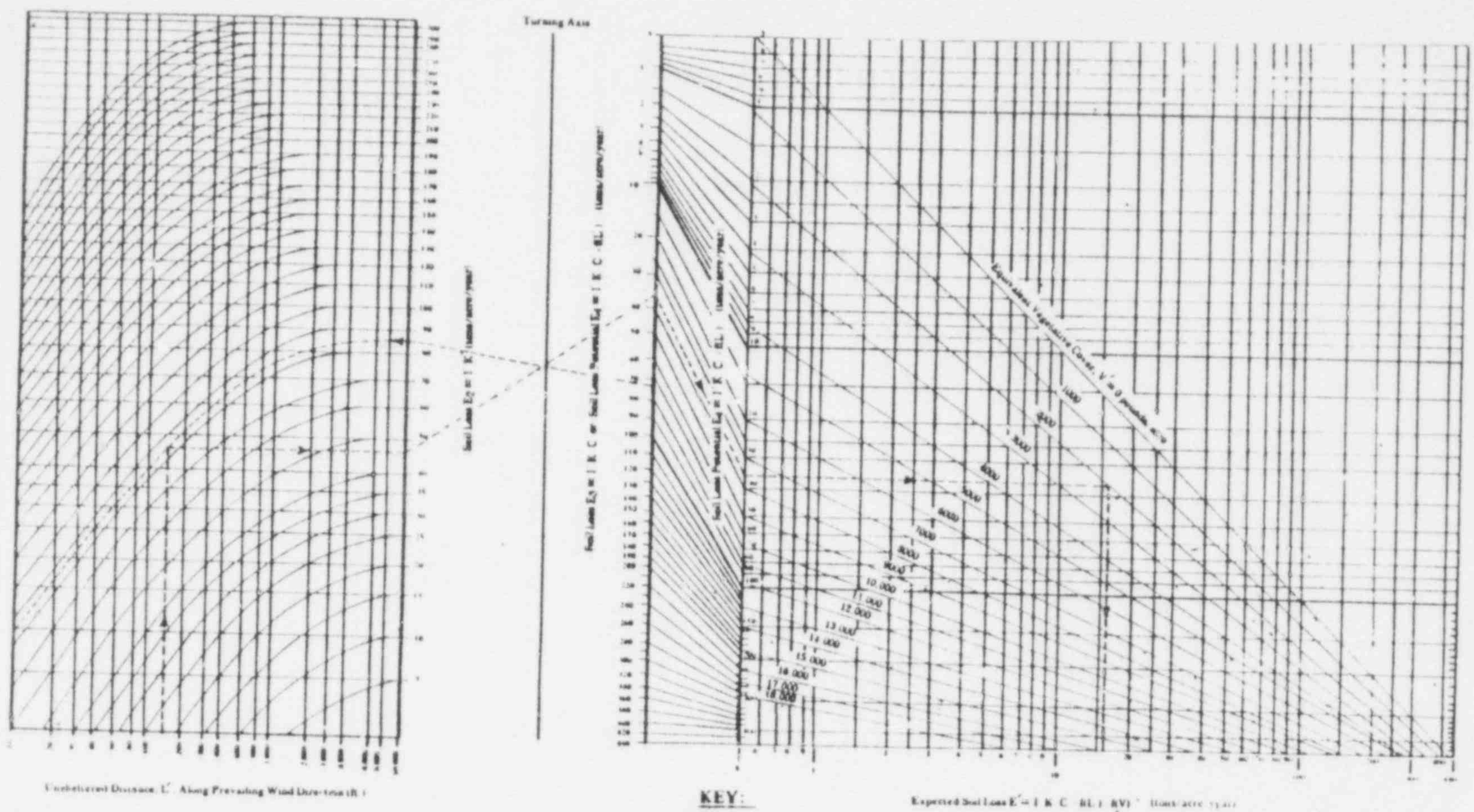

KEY:

Step 1. Locate I'K' value on vertical scale to the left of turning axis and construct curved dotted line as indicated.

Step 2. Move vertically upward from L' value to intercept curved line (Step 1) then horizontally to E_2 scale.

Step 3. Mark I'K'C' value on vertical scale to the right of turning axis.

Step 4. Draw line from I'K' value (Step 1) to I'K'C' value (Step 3) and mark intercept on turning axis.

Step 5. Draw line from value on E_2 scale (Step 2) through point on turning axis (Step 4) to E_4 scale, then diagonally to second E_4 scale, horizontally to intercept with appropriate V' value, and then vertically downward to read expected soil loss on E' scale.

Fig. 28b. Wind soil loss equation nomograph (from UWRL 1976b).

d. Alternatives 1 through 6

i) Likelihood -

The likelihood is the same for these alternatives as for the base case.

ii) Magnitude -

The magnitude of release is based on the soil loss calculated above.

For the short long-term period wind erosion caused soil losses are very low. The magnitude was considered to be unity reflecting the chance for an extreme soil loss due to an extreme event.

For the medium period losses are also low. From Figure 6 it can be seen that the increase in radon emanation resulting from a 5 inch soil loss is negligible. Increases in gamma-ray emission would also be negligible. A minimal value of 1 was used.

Over the long long-term period wind erosion could remove the entire cover and possibly some tailing. For that period the magnitude was taken to be 10.

e. Alternatives 7 and 8

Wind erosion cannot effect underground disposal. Severity is zero.

5. Failure of Cap Due to Floods (Ale)

a. General Considerations

Flood damage to the cap consists of the erosion and removal of major portions of the cap due to inadvertent routing of large quantities of flood water over the cap. It is differentiated from sheet erosion by the fact that major quantities of flood water would remove all of the particular cap material with which it comes into contact.

On the geologic cross section for the model site it is indicated the tailings impoundment is place approximately 200 ft higher in elevation than the tributary river floodplain. It is assumed, therefore, that flood waters from the tributary river would not come into contact with the impoundment. Flooding from ephemeral streams feeding into the tributary river, however, is possible. If a flood larger than the design flood or failure of diversion structures occurs, the magnitude of the failure will depend on the extent of flood water contact. For purposes of this application it has arbitrarily been assumed that flood waters could contact about 25 percent of the cap and erode 25 percent of the impoundment. Those assumptions are arbitrary and site specific considerations may change them.

The modes of release that are considered as radon emanation and gamma-ray emission. This is based on the assumption that only the cap material is removed and no other failure modes interact to cause significant removal of tailings, such as an embankment failure. This is highly idealized, other interactions must occur, but for purposes of this evaluation the assumption is necessary.

b. Base Case

Because no cap is placed both the likelihood and magnitude have values of 10 which is tantamount to release of the maximum amount of activity possible.

c. Alternatives 1 through 6

i) Likelihood -

The likelihood of failure depends on the occurrence of a flood of magnitude greater than the design flood and/or failure of flood control measures. It is assumed that the design flood would be larger than a 100-year flood, and hence, the likelihood of short long-term failure was assigned a value of 1. Over a medium long-term period there is a considerably higher probability of occurrence and a value of 5 was assigned. This value could change depending on the recurrence interval data for the area. Over a long long-term period it is highly probable that a flood higher than the design magnitude and failure of flood control structures would occur. Recurring floods would also compound the damage. A value of 9 was assigned.

ii) Magnitude -

The release by failure of 25 percent of the cap would be 25 percent of the potential activity. A value of 3 was therefore assigned for the short long-term. That value can increase with time as further interaction with other failure mechanisms occurs or if the events are repeated without remedial measures. For the medium long-term and long long-term the magnitudes are therefore increased to 5 and 9, respectively.

d. Alternatives 7 and 8

Because the tailings are deposited in the deep mine and no cap exists there can be no failure. The severity was then taken as 0.

6. Failure of Cap due to Chemical Attack (A1f)

a. Base Case

Because no cap exists both the likelihood and magnitude are equal to 10, which is tantamount to maximum possible amount of radon emanation and gamma-ray emission. For all alternatives that include a cap gamma-ray emission is reduced to a level equivalent to a zero release magnitude.

b. Alternatives 1, 4, 5 and 6

i) Likelihood -

The likelihood of chemical attack on the cover occurring is great. However, the likelihood that chemical attack would be of a rate and nature that would lead to failure is probably remote for short long-term periods. Therefore, the likelihood of failure for the short long-term will be assigned a value equal to unity. Chemical changes may continue to occur. However, as the water in the tailings drains over long time periods (the liner will leak to some extent) the contact of chemicals from the tailings will decrease. It is probable, therefore, that the chemical attack on the cap would diminish. The likelihood of failure was considered to be 1 for all time periods.

ii) Magnitude -

The magnitude of radon emanation will depend on the nature of the chemical changes that occur. If sandy and silty soils are changed into clay soils the permeability may decrease and radon emanation may actually decrease. If leaching and ion exchange occur the permeability of the cap may increase. A value of 1 for all three time cases was assumed. This value is conservative and may in fact be zero.

c. Alternatives 2 and 3

i) Likelihood -

The likelihood of chemical attack on the cap would be less than for Alternative 1 because the tailings are fixed or dry in these alternatives. The chemicals in the tailings would, therefore, be less mobile and less water to facilitate transport of ions would be present. A minimal value of unity was assigned to the likelihood.

ii) Magnitude -

The magnitude of release for these two alternatives would be similar to that for alternative 1. A value of unity was therefore assigned.

d. Alternatives 7 and 8

Because the tailings are deposited in the deep mine and because there is no cap or release due to failure of a cap, the severity of this failure mode is zero.

7. Failure of Cap due to Shrinkage (A1g)

a. General Comments

With respect to a shrinkage failure of the cap, all alternatives which incorporate a cap can respond in the same manner and are therefore discussed as a group. The base case does not include a cap, and as discussed previously, the likelihood and magnitude were set at 10 for all three time periods.

b. Alternatives 1 through 6

i) Likelihood -

The likelihood of a shrinkage failure of the cap is moderate in the short long-term period. The value will increase with time rising to a maximum at the medium and long long-term periods. This reflects the judgement that the cap would be desicated within several hundred years. The characteristics of the material used for the cap would influence the likelihood that it would shrink.

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If cohesive material is used for the cap and cover the likelihood of shrinkage is high. For this discussion a cohesive material is assumed to be used. This represents the worst and most conservative case. If a noncohesive material is used the likelihood and magnitude of a shrinkage failure is zero.

ii) Magnitude -

The mechanism of release would be increased emanation of radon gas through shrinkage cracks in the cap. If a cohesive material is used, as assumed, the likelihood that lineal shrinkage of about 2 percent is great. This is fully explained in Example 1, Chapter IV. If this magnitude of shrinkage occur emanation of radon would be at almost as great a level as in the base case. Therefore, a value of 9 is applied for all time periods. If a non-cohesive material is used shrinkage would not occur. It is estimated that gamma-ray emission will not increase due to shrinkage of the cap and cover.

c. Alternatives 7 and 8

Because no cap exists and because there is no release of airborn activity the severity is zero.

8. Failure of the Liner Due to Differential Settlement (A2a)

a. General Comments

It is assumed that the liner under an impoundment on the ground or in a pit consists of a compacted clay liner approximately three feet thick that consists of bentonite mixed with the natural alluvium and recompacted. In the deep mine it was assumed that the liner was shotcrete or similar material. The only mode of release considered significant for all liner failures is the release of dissolved radionuclides in seepage. The amount of dissolved radionuclides released is considered to be proportional to the amount of seepage increase due to failure.

b. Base Case

i) Likelihood -

Because no liner exists and the ground is permeable and not prepared, the likelihood of failure was assigned a value of 10.

ii) Magnitude -

The magnitude of seepage released is equal to the maximum possible and was assigned a value of 10.

c. Alternative 1

i) Likelihood -

Differential settlement in the alluvial subsoils is to be expected for the same reasons discussed with reference to differential settlement of the cap. If the liner thickness has been chosen so as to be thicker than the amount of differential settlement with an appropriate factor of safety, the likelihood that excessive differential settlement will occur is moderate. Because consolidation would be completed in the short long-term period a value of 5 was assigned to the likelihood for all three periods of time. This value could increase if a thin, brittle liner is installed such as asphalt or if a thin synthetic liner is used. It may decrease if the alluvium is very uniform.

ii) Magnitude -

Based on the computations shown in example 2 in the discussion of this failure mode (Chapter IV) the magnitude of release was assigned a minimal value of 1. If large shear type displacements or if a thin, brittle liner is used (asphalt, for example) the magnitude could increase. However, with proper foundation design the value would probably not exceed 3.

d. Alternative 2

i) Likelihood -

Because no liner is placed, any seepage that is possible could escape. The likelihood of seepage is 10, similar to the Base Case.

ii) Magnitude -

The magnitude of release will depend on the amount of water in the tailings that may escape. It is assumed that no free water exists in the fixed tailings. Because the open pit mine intersects the water table it is expected that leaching of radioactive material could occur. Leaching, however, would only occur from that portion of the tailings in which the groundwater comes in contact. It appears reasonable that the groundwater could contact about 50 percent of the tailings and a value of 5 was assigned. This value could increase or decrease with a rise or lowering of the water table and with more access to surface water that may infiltrate.

e. Alternative 3

i) Likelihood -

The weight of tailings and overburden would be equal to or less than the weight of material excavated. Differential settlement in the open pit mine, therefore, would be minimal. A value of 1 was assigned.

ii) Magnitude -

Because the tailings are dry and in view of the calculations presented in example 2, the release of dissolved radioactive material due to failure of the liner would be minimal (1).

f. Alternative 4

i) Likelihood -

The likelihood of failure is the same as for Alternative 3 (1).

ii) Magnitude -

On the basis of the computations shown in example 2, the fact that displacements would cause primarily tension type failures and the fact that the tailings would not contact the groundwater directly, a minimal value of 1 was assigned.

g. Alternative 5

i) Likelihood -

If the shale is impervious, the seepage should be less than for an intact liner of thinner dimensions. The likelihood of seepage being released is very low (1).

ii) Magnitude -

Even if differential settlement did occur in the underlying shale there would be few or no discontinuities or cracks through which seepage would occur. A value of unity was assigned.

h. Alternative 6

i) Likelihood -

The weight of tailings would not exceed the weight of excavated material. The likelihood of differential settlement would be minimal (1).

ii) Magnitude -

Similar to Alternative 4 a minimal value of unity was assigned.

i. Alternative 7

i) Likelihood -

Because no liner is placed and the sandstone is permeable there is a certainty that the maximum seepage possible will be released. A value of 10 was assigned.

ii) Magnitude -

The magnitude of release will depend on the amount of leaching possible and the extent of contact with the groundwater. Similar to Alternative 2, a value of 5 was assigned. This value may increase depending on the position of the mine relative to aquifers.

j. Alternative 8

i) Likelihood -

There is an almost certain likelihood that ground displacements around the mine will occur even within short long-term periods. The brittle nature of the lining material (shotcrete) makes it susceptible to cracking. A value of 9 was assigned.

ii) Magnitude -

The amount of radioactive release will depend on the interaction between the groundwater and the water in the tailings. Over short periods there may not be much leaching or seepage release through cracks in the liner. A value of 1 was assigned. Over medium long-term periods a greater extent of cracking due to creep would occur and more seepage or leaching would have occurred. A value of 5 was therefore assigned. Over long long-term periods the mine ma: evel collapse, and the maximum amount of seepage possible is to be expected, a value of 9 was assigned.

9. Failure of the Liner due to Subsidence (A2b)

a. Base Case

Since no liner exists, the likelihood and magnitude of failure will be ten for all time periods.

b. Alternative 1

i) Likelihood -

The likelihood of sink holes or caverns occurring in alluvium is very small. Since the impoundment is located on a thick deposit of alluvium, the likelihood for subsidence is small and will be assigned a value of one for the short long-term. However, if subsurface mining takes place below or near the impoundment, the likelihood for subsidence could increase. It is assumed, however, that adequate design would minimize the potential for subsidence. The value of unity was used for all three time periods.

ii) Magnitude -

The potential magnitude of release due to large values of localized subsidence is large. If it is assumed that ten percent of the liner were to fail by subsidence, a magnitude of 1 would be appropriate for a short long-term period. However, over medium and long long-term periods the entire impoundment could be drained through a single hole of that size. Therefore, the magnitude of release will be considered to be equal to one for the short long-term period and nine for both the medium and long long-term periods.

c. Alternative 2

Because no liner exists and the subsoil is permeable, the likelihood of failure is 10. However, because the tailings are fixed the magnitude would be the same as for the previous failure mode (A2a), 5.

d. Alternative 3

This alternative is the same as Alternative 1 except the tailings are dry. The magnitude of release would be small even even if subsidence did occur because there would be no seepage. The likelihood could increase if deep mining commenced nearby.

e. Alternative 4

This alternative is similar to Alternative 1. The likelihood of failure could increase if deep mining commenced nearby. If arching occurred in the overburden placed in the bottom of the pit and the upper liner did not fail, the magnitude could decrease.

f. Alternative 5

i) Likelihood -

If the tailings are placed on the cretaceous shale it is possible that some deep mining could have occurred in the sandstone. If the mine is sufficiently shallow to be of concern for short long-term periods its presence would probably be known. The impoundment therefore would be located elsewhere or adequate design to avoid subsidence would be employed. Over medium and long long-term periods subsidence from deeper mines may occur, but this can not be predicted. A value of unity was assigned for all three periods.

ii) Magnitude -

Similar to Alternative 1, a value of unity was assigned for the short long-term period and 9 for the other two time periods.

g. Alternative 6

This alternative is similar to Alternative 1 both in terms of likelihood and magnitude.

h. Alternative 7

i) Likelihood -

There is a relatively higher likelihood that subsidence due to collapse of the deep mine itself could occur. A value of 3 for the short long-term period was assigned. Over the medium and long long-term periods the existence of creep and creep failure in the sandstone would increase the likelihood to about 7.

ii) Magnitude -

Over a short long-term period only a portion of the tailings may be exposed to the groundwater and leaching. A value of 5 was used for that time period. That value would change, however, if larger or smaller aquifers were intercepted. Over medium and long long-term periods, however, more leaching would occur and the magnitude was assigned a value of 7.

i. Alternative 8

i) Likelihood -

This alternative is similar to Alternative 7. The same values of likelihood were assigned.

ii) Magnitude -

If subsidence of the mine occurs water may seep from the tailings into the aquifer. For a short long-term period a value of similar magnitude to that for Alternative 7 was assigned. However, over medium and long long-term periods it would be expected that nearly all of the water in the tailings would be drained. A value of 9 was therefore assigned for those two time periods.

10. Failure of Liner due to Chemical Attack (A2c)

a. Base Case

Since no liner is incorporated in this disposal plan the likelihood and magnitude are set at 10.

b. Alternative 1

i) Likelihood -

The likelihood of some level of chemical attack on the liner is great. The likelihood of failure resulting from this chemical attack is very dependent on the thickness and properties of the soil used for the liner, as well as the chemical properties of the tailings. The likelihood of failure for the short long-term will be set equal to three assuming that compatibility of the liner to the tailings was pretested. Chemical attack will continue over time until equilibrium is reached or reagent quantities dissipated. Therefore in the medium long-term the likelihood of failure will increase but remain stable thereafter.

ii) Magnitude -

The magnitude of release through a liner due to chemical attack will be due to changes in the liners permeability. (This assumes a natural soil liner. If a synthetic liner is used chemical or radiation attack could cause disintegration of the liner, not a change in permeability). The change in the amount of seepage is therefore a function of the rate of chemical change and the thickness of the liner. For the short long-term period the change in seepage is expected to be relatively low and the magnitude is therefore set at 1.0. For the medium long-term chemical changes could have altered a significant thickness of the liner with corresponding increases in permeability magnitude. For this period is estimated at 3.0. For the long long-term it is expected that chemical changes would probably have ceased and seepage stabilized at levels not much higher than for the medium long-term period. The long long-term magnitude is, therefore, set at 5.0.

c. Alternative 2

Because no liner is used the likelihood of seepage is 10. However, because the tailings are fixed, a value of 5 is assigned for the magnitude. This value assumes that the groundwater leaches radionuclides from only half of the tailings.

d. Alternative 3

i) Likelihood

The likelihood of chemical attack on the liner occurring will be reduced by drying the tailings. However, some chemical attack will occur over time and the rate will likely be speeded by infiltrating surface water. The likelihood of failure for the short-long term will be assigned a value equal to one, for the medium-long term equal to three and for the long long-term equal to five.

ii) Magnitude -

The potential magnitude of release will be less since the tailings are initally dry and seepage would not occur. This is assuming the cover is restricting the flow of surface water into the tailings. The magnitude of release will be considered to be small during the short long-term and have a value of one. The possibility that surface water will infiltrate the impoundment over time causes the magnitude to increase. A value of three is assigned for the medium-long term and five for the long-long term.

e. Alternative 4

i) Likelihood

The likelihood of failure of both liners in the short-long term is very dependent on the thickness and soil type of the liners, temperature and the pH of the tailings. Since two liners are used, the likelihood of seepage increases during the short-term is small and will be set equal to one. Since chemical attack will continue to occur, the likelihood of failure will continue to increase with time. Therefore, the medium long-term likelihood of failure will be assigned the value of three while the long long-term will be assigned a value of five.

ii) Magnitude

The magnitude of release to the groundwater would be reduced by using two liners and the intervening overburden layer, and therefore, are set at the same values as for Alternative 3.

f. Alternative 5

i) Likelihood

Since no liner was used likelihood is 10.

ii) Magnitude -

Since the impervious shale acts as a liner the magnitude of release will be small for all time periods. This assumes that the shale thickness and uniformity are sufficient to preclude significant chemical change. Therefore a magnitude of one will be assigned for all time periods.

g. Alternative 6

This alternative is basically the same as Alternative 1. Therefore evaluation of likelihood and magnitude for Alternative 1 applies here.

h. Alternative 7

i) Likelihood -

Since no liner is used likelihood is 10.

ii) Magnitude -

The magnitude of release will depend on the amount of leaching possible and the extent of contact with the groundwater. This is similar to Alternative 2, therefore a value of 5 is assigned. This value may increase depending on the position of the mine relative to aquifers.

i. Alternative 8

i) Likelihood -

The likelihood of failure will be the one level higher than for the other alternatives with liners 2 and 6. This is based on the assumption that mechanical failure of the shotcrete liner will interact with chemical attack to produce failure.

ii) Magnitude -

The magnitude of release will be moderate for the short long-term period. This is based on the assumption that some period of time will be needed for the shotcrete lining to deteriorate. Chemical attack and mechanical failure will be complete by the medium long-term period and release will be high and about the same as if no liner was used. The magnitude will not change for the long long-term.

11. Failure of the Liner due to Physical Penetration (A2d)

a. General Comments

In this particular failure mode it has been assumed that the liner is a synthetic (non self-healing) liner. A clay liner as described for other failure modes would not be susceptible to this type of failure. A shotcrete liner, such as was assumed for the deep mine, would not be susceptible to this failure mode either.

b. Base Case

Because no liner is placed the likelihood and magnitude of failure were both assigned values of 10. This is tantamount to assuming that the maximum amount of seepage possible will occur.

c. Alternatives 1, 3 and 6

i) Likelihood -

During placement of the liner some punctures and small failures of the seams are almost inevitable. A value of 9 therefore was assumed.

ii) Magnitude -

On the basis of the computations presented in example 2 in Chapter IV a minimal value of unity was assigned. The seepage through small holes in the liner would be even less than through shrinkage cracks and the value of 1 is even somewhat conservative.

d) Alternative 2

Because no liner is used the likelihood of seepage is 10. However, because the tailings were fixed a value of 5 was assigned for the magnitude. The value of 5 assumes that the groundwater leaches radionuclides from only half of the tailings.

e. Alternative 4

Because two liners were used the likelihood of puncture was decreased to 7. The magnitude is minimal as was true in Alternative 1.

f. Alternative 5

Although there is no liner the shale is impervious. Therefore, the severity was assigned a value of zero.

g. Alternative 7 and 8

This failure mode is not applicable to these alternatives and a severity of zero was assigned.

12. Failure of the Embankment due to Differential Settlement and Cracking (A3a)

a. General Comments

The embankment is constructed of compacted material and is relatively long. It is not a "cross-valley" embankment and is situated on fairly deep alluvium. It is assumed that when a liner is used the liner extends up the upstream face of the embankment. When no liner is used it is assumed that a compacted clay loam was used for the embankment. For this failure mechanism all modes of release could be important. However, in most cases failure reduces the thickness of the embankment and does not increase seepage or cause transport of tailings. Therefore, radon emanation and gamma-ray emission are considered most important. If flood erosion or slope failure are of sufficient magnitude then dissolved radionuclides and tailings could be removed. In the case of cracking due to differential settlement seepage of dissolved radionuclides is the important mode of release. Radon emanation and gamma-ray emission remain at design levels because of the large relative embankment thicknesses. The specific modes of release are identified in the individual discussions and shown in Table 14.

b. Base Case and Alternative 1

i) Likelihood

Because the foundation soils below the embankment are fairly uniform in contrast to a cross-valley embankment on irregular bedrock, cracking due to differential settlement would not be expected to be high. Furthermore, much of the settlement would occur during construction. Nevertheless, the fact that the borrow is probably dry in its natural state and the uniform mixing of water is difficult some cracking is expected to occur. A moderate value for likelihood (3) was assigned.

ii) Magnitude

If cracking did occur the amount of radioactive seepage would be small. However, cracking could progress and seepage could continue over extended periods of time. A value of 1 was assigned for the short long-term period and a value of 3 was assigned for the medium long-term period. Over long long-term periods it is not expected that water would remain in the tailings and the value of 3 was not changed.

c. Alternatives 2 through 8

Because no embankment exists there is no severity of failure.

13. Failure of the Embankment Due to Slope Failures (A3b)

a. Base Case and Alternative 1

i) Likelihood

Because the embankment was assumed to be constructed of compacted clay soil there does exist some possibility of creep movements of the slopes. If longitudinal cracks form and can fill with water there may be some possibility of slope instability. However, proper design could minimize that. Some embankments and mounds have been observed to remain stable over several thousand years. Slope instability is not considered to be a likely mode of failure. A value of unity was assigned.

ii) Magnitude

If slope failure occurs it would be on the downstream face. It is doubtful that the failure surface would intersect the tailings and the amount and thickness of material cannot be estimated with any accuracy. For purposes of this discussion it is assumed radon emanation and gamma-ray emission could increase to a release magnitude of 3.

b. Alternatives 2 through 8

Because there is no embankment for these alternatives there is no severity of failure.

14. Failure of the Embankment Due to Gullying (A3c)

a. Base Case and Alternative 1

i) Likelihood

The likelihood of failure is moderately high initially and increases with time. In accordance with the discussions presented in Chapter IV values of 5, 7 and 9 were assumed for short, medium and long long-term periods. These values could change with different maintenance and cover plans.

ii) Magnitude

As with gullying of the cap, maintenance, monitoring and remedial operations are significant in estimating the potential release for the short long-term period. Radon emanation and gamma-ray emission are the release modes most likely to occur. A value of unity was assumed for that period. Without maintenance, monitoring and remedial measures, gullying of the embankment lead to total failure of the impoundment. In this case all modes of release are important. Site specific conditions are extremely important in the interpretation of long term potential for failure magnitudes. However, without maintenance, potential release magnitudes are high for the longer periods. For the medium long-term period a value of 3 was assigned and for the long long-term period a value of 9 was assigned.

a. Alternatives 2 through 8

No embankments are specified. Therefore the severity is zero.

15. Failure of Embankment Due to Water Sheet Erosion (A3d)

a. Estimation of Soil Loss

The Universal Soil Loss Equation (USLE) will be used to estimate the amount of soil loss due to water erosion on the embankment. For purposes of calculation the following assumptions were made:

- The average height of the embankment is 60 ft.
- The length of the embankment is 1500 ft.
- The slope is 3 to 1 or 18 degrees. The embankment slope is therefore approximately 225 feet long.

- Vegetation is used to stabilize slopes.
- Embankment soils are the same as used on the cap.
- Terraces are used to reduce slope length to 40 feet.

Following the same procedure as described for failure mode, A1c, the soil loss calculation using the USLE is as follows:
(Note that the LS factor is the only quantity that changes)

A = RKLSCP

A = 75% (40) (.37) (2.2) (0.25) (1) = 6.11

A = 20% (100) (.37) (2.2) (0.25) (1) = 4.07

A = 5% (150) (.37) (2.2) (0.25) (1) = 1.53

Total soil loss = 11.70 tons/acre/year or 0.0076 inches/year. Over the time periods considered this rate of soil loss would remove 1.5 inches in 200 years, 15.1 inches in 2000 years and 63 feet in 100,000 years.

b. Base case and Alternative 1

i) Likelihood

The likelihood of erosion on the embankment will be high and increase with time as previously discussed

ii) Magnitude

The magnitude of failure will be a function of the amount of soil loss calculated in relation to the thickness of the embankment. The USLE which was used to calculate soil loss assumes a uniform loss. This is not entirely correct for the relatively steep slopes of the embankment. More loss will occur at the head of the slope while accumulation is likely to occur at the toe. Judgment will have to be used to evaluate the quantities of soil loss calculated and their effect on release.

The losses and thickness reduction for the short and medium long-term periods are not significant. Even if 3 or 4 times that amount of material is removed from the embankment no increase in radon emanation or gamma-ray emission is expected. Clearly, the long long-term loss is excessive. Flatter slopes or stabilization with nonerodible material such as rock may reduce the long long-term effect. The entire embankment could fail. Radon emanation and gamma-ray emission would increase significantly, and seepage and tailings release could occur. For purposes of this discussion total failure is assumed. Magnitude 9 is assigned for all release modes.

For the short and medium long-term periods values of 1 were assigned. For the long long-term period a value of 9 was assigned.

c. Alternatives 2 through 8

Embankments are not part of the design plan; therefore the severity is zero.

16. Failure of Embankment Due to Wind Erosion (A3e)

a. Wind soil loss equation

The steep slopes of the embankment adds to the soil loss potential due to wind. The embankment characteristics assumed here are the same as for the water sheet erosion evaluation presented previously. The calculation of soil loss will use the same factors for the wind soil loss equation as were used for wind erosion of the cap (A1d), except for the knoll erodibility factor, I_s, and the wind fetch factor, L. For an 18 degree, windward slope I_s is approximately 1800 percent. The wind fetch factor, L, is equal to the length of the slope, 225 feet.

From Figure 27 the soil loss is calculated to be significantly less than 0.5 tons/acre/year before applying I_s. Taking into account, I_s, in Figure 27 the soil loss rate is about 4000 pounds/acre/year. That loss rate is equal to a thickness reduction of 0.0013 inches/year. For the periods being considered, 0.26 inches would be lost in 200 years, 2.6 inches in 2000 years, and 10.8 feet in 100,000 years.

These values do not include the possiblity of "blowouts" or localized erosion from occuring. It will be assumed that blowouts would appear within a short long-term period and would be stabilized during maintenance throughout that initial period. It should be noted that the use of a coarse rock cover may prevent blowouts and can reduce water sheet erosion as well.

b. Base Case and Alternative 1

i) Likelihood

The likelihood of wind erosion is certain. A value of 10 was assigned.

ii) Magnitude

Based upon the estimated soil loss and thickness reductions computed above the magnitudes of release would be similar to those for water sheet erosion. The magnitude of erosion over both short and medium long-term periods was set at 1. Although the calculated values would indicate that a value of 0 is appropriate this value recognizes that some blowouts may occur that are not repaired.

For the long long-term period the calculated soil loss would remove major portions of the embankment. In connection with other erosion mechanisms after removal of the embankment the entire impoundment could be destroyed. The magnitude for all modes of release is set at 9.

c. General Comments

The above values of magnitude are based upon "best-estimate" values of soil loss. The methods employed, for both wind and water erosion, assume that the soil loss will occur uniformly. They cannot predict or evaluate gullying or blowout potential. Those events could occur at any time and without remedial action significant erosion loss could results.

It is emphasized that the most appropriate use of the wind and water erosion equations may be to compare alternative control practices or design concepts, rather than to calculate finite amounts of soil loss.

d. Alternatives 2 through 8

These alternatives do not include embankments. The severity is zero.

17. Failure of Embankment Due to Flooding (A3f)

a. General comments

It was assumed previously that for the model site regional flooding of the tributary river would not contact the impoundment. Flooding of the impoundment area, therefore, would occur only due to runoff or floods from the ephemeral streams greater than the design magnitude or by runoff overfilling previously failed diversion structures. If periodic maintenance is performed as part of the plan, failure would only result from runoff. Maintenance is assumed only for the short long-term period. For a failure of the embankment due to flooding all modes of release are included. This is based on the assumption that flood failure of the embankment will remove entire portions of the embankment as well as tailings. The magnitude values represent an estimate of the percent of the total embankment and tailings removed.

b. Base Case and Alternative 1

i) Likelihood

The likelihood of failure is set at 1, 7 and 9 for the short, medium and long long-term periods respectively. These values are based on the assumptions that short long-term periods would be less than the recurrence interval of the design flood, the likelihood for floods greater than the design flood will increase time, and that over very long periods the chance for diversion failure increase.

ii) Magnitude

Because of the lower likelihood of a very severe flood occurring in the SLT period, and because maintenance is employed, M is set at 5. This is relatively high and reflects the chance that the event that may possibly occur be of very large magnitude. For medium and long long-term periods a value of 9 was assigned. This value reflects the fact that there is no maintenance and over long periods large events can reoccur.

c. Alternatives 2 thorugh 8

Embankments are not included in the disposal plan; therefore the severity is zero.

18. Failure of Embankment Due to Chemical Weathering (A3g)

a. Base case

i. Likelihood

All materials will be subjected to physical and chemical weathering. The rate of weathering will depend on the embankment materials, climate, and pH of seepage. The likelihood of weathering will increase with time. In view of the large amount of material in the embankment, the likelihood of failure from weathering should be small in the short long-term periods and will be assigned a value of unity. Since no liner or core is assumed for the base case embankment, the chemical composition of the seepage would be that in the tailings and may lead to conditions where active weathering from external factors could take place (e.g. clays may be dissolved and erosion could then be accentuated). Therefore, the likelihood of failure in medium long-term periods would increase and a value of three will be assigned. For the long long-term period a value of five will be assigned. The type of soil used in the embankment and the nature of the seepage could increase.

ii) Magnitude

The potential magnitude of release by a failure of the embankment will be influenced by the nature of the failure caused by weathering. In the short long-term period the effect would probably be just to allow more seepage. If the chemical action changes some of the minerals into clay minerals the seepage could even decrease. The release is expected to be small. A value of unity was assigned. For medium and long long-term periods the effect could be increased erosion and loss of tailings. All modes of release are considered possible and a value of 3 was assigned. This value, however, is subjective and in the absence of any typical long term observations it must be considered to be very approximate.

b. Alternative 1

i. Likelihood

All materials will be subjected to chemical weathering. The use of the liner will reduce seepage rates and reduce the area of the embankment subjected to seepage. The outer shell of the embankment would be less subject to weathering. Therefore, the likelihood of failure by weathering of the embankment will be less than for the base case which has no liner or core. However, the use of the liner will maintain a high water level in the tailings for a longer period of time and subject the embankment to seepage for a longer time period. Therefore, the short long-term likelihood of failure was assigned a value of one, whereas the medium and long long-term periods were values of five and seven, respectively.

ii Magnitude

The magnitudes of failure were considered to be the same as for the Base Case.

c. Alternatives 3 through 8

Because no embankment exists in these alternatives the severity is zero.

19. Failure of Revegetation Due to Fire (A4a)

a. General comments

Fire caused failure of the revegetation plan will have limited consequences just by itself. The most important effects will be as in interaction with the erosional processes. If long term changes in climate do not occur, vegetative reestablishment after fire can be expected to take place within a few years. For these short periods of denudation some erosional soil loss will occur.

The base case is the only above ground disposal plant that does not specify a cover. In that case it was assumed that vegetation would be established, naturally or by planting, but the effect of the vegetation on the magnitude of release is negligible. The cap is considered to be the element that reduces gamma-ray emission and radon emanation, the two modes of release being considered. Consequently, loss of vegetation would influence the magnitude of release only to the extent that the cover's effectiveness is decreased. The base case release would be the 100 percent basis for comparison.

b. General likelihood

The likelihood of fire failure will be low initially but will increase with time. The likelihood would be the same for the Base Case and Alternatives 1 through 6 because the fire danger is based on natural circumstances and is not plan related. It is also assumed that all revegetation or natural vegetation would be the same regardless of alternatives. That may not be exactly true, however, if top soil is placed as part of the cap.

The quantification of likelihood of brush fires in uranium mining areas is somewhat subjective. It would be less than the probability of forest fires. However, brush fires do occur and in the past large brush fires burned the Great Plains. Values of likelihood of 3, 7 and 9 were therefore assigned for the short, medium and long long-term periods respectively.

c. Base Case

For the Base Case, because no cap is placed the likelihood and magnitude of radon emanation and gamma-ray emission is the maximum that could occur.

d. Alternatives 1 through 6

The suitable cover may not include vegetation. The values of likelihood and magnitude shown in Table 14, therefore, only apply to the situations in which the cover dpends on vegetation for stabilization. The likelihood was discussed above.

The magnitudes of release would be those resulting from wind and water erosion during the periods of vegetation failure. On the basis of the computations of soil loss for erosion of the cap (A1c and A1d) it may be assumed that the amount of cover or embankment loss would be minimal until revegetation can be reestablished. Gullying and blowouts would be possible but existing root systems would prevent that to some extent. Minimal values of magnitude (1) were therefore assigned.

e. Alternatives 7 and 8

Because loss of cover on the surface would have no effect on release of radioactive material the severity is zero.

20. Revegetation Failure due to Climate Change (A4b)

a. General Comments

The likelihood of failure of vegetation on the tailings due to a change in climate will depend on the type of vegetation that has been established, the growth medium, and the rate of climatic change. The model site is semiarid and will be revegetated with desert shrub species such as sagebrush, rabbitbrush, and greasewood. When established, these species should be resistant to periods of low rainfall. Increased rainfall will stimulate growth and will not cause failure. However, periods of drought are certain to occur even within the short long-term period. Consequently, the likelihood of short periods (less than about 5 years) of vegetation failure are very likely to occur. General failure of the vegetation to a situation such as the Sahara Desert is considered to be remote.

Values of likelihood were assigned to be 5, 7 and 9 for the short, medium and long long-term periods. These values reflect the fact that short drought periods will almost certainly occur but the may or may not be of sufficient duration to cause failure of the vegetation.

b. Base Case

Because there is no cover the likelihood and magnitude of radon emanation and gamma-ray emission would be the maximum possible. The magnitudes of release was assigned a value of 10.

c. Alternatives 1 through 6

If vegetation failure occurs release would also require erosion of the cap and embankment similarly to failure of vegetation due to fire. Magnitudes of release were, therefore, considered to be unity.

d. Alternatives 7 and 8

Because surface erosion would release no radionuclides from the tailings, the severity is zero.

21. Failure of Water Diversion Structures Due to Slumping (A5a)

a. General comments

For the setting of the tailings impoundment in the alluvial plain, flooding would occur only from small streams flowing into the Tributary river. Diversion structures would therefore be relatively low and would not exist on steep slopes.

b. Base case and Alternatives 1 through 6

i) Likelihood

The general setting for all of these cases is similar. Because the slopes above the diversion ditches and the cut slopes for the ditches themselves are not high or steep, the likelihood of failure is small. The likelihood of slumping of diversion ditches is small for all time periods and was assigned a value of unity.

ii) Magnitude

The magnitude of release due to failure of a diversion ditch would depend on the nature of the impoundment failure that it induced by that. A value of 3 for radon emanation and gamma-ray emission was assigned on the same assumptions used for flood damage to the cap (i.e. that 25 percent of the impoundment would be washed out by a flood). That value could change considerably, depending on the consequences of inadequate flood routing.

c. Alternatives 7 and 8

i) Likelihood

Similarly to the base case the likelihood of failure would be the same as for the base case. It was assigned a value of unity.

ii) Magnitude

Because the tailings would be below ground there would be no consequences of a surface flood.

22. Failure of Water Diversion Structures due to Obstructions (A5b)

a. Base Case and Alternatives 1 through 6

i) Likelihood

Flood diversion ditches have been assumed to intercept surface runoff from above the tailings disposal site to prevent local flooding. The diversion ditches are considered herein to be intact and obstruction would be due to external factors such as sedimentation, snow, ice, or debris.

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The likelihood of failure will be the same for the base case and Alternatives 1 through 6. Obstruction is very likely to occur but with maintenance during the short long-term it may be remedied. Therefore, a value of 3 was assigned. During medium and long long-term periods the likelihood increases considerably because of the absence of maintenance.

ii) Magnitude

The magnitude of release was assigned a value of 3 for the same reasons as for failure due to slumping.

b. Alternatives 7 and 8

Erosion at the surface would have no influence on the tailings in the deep mine. The severity, therefore, is zero.

23. Failure of the Impoundment due to Earthquakes (B1)

a. General Comments

An earthquake of magnitude greater than the design earthquake could lead to failure of the cap, liner, or embankment. If liquefaction occurs the tailings themselves may be released and transported over significant distances. The model site is not, in general, in a zone of great seismisity but earthquakes of magnitude 4 can be expected. It will be assumed that the site was designed to resist an earthquake of magnitude somewhat greater than 4 and failure of the impoundment therefore would involve the occurrence of an earthquake of magnitude greater than that for which it was designed.

The tailings themselves would be most subject to liquefaction when they are saturated. Also, the embankment would be most subject to earthquake damage when water is impounded behind it and seepage is occurring through it. Even if a liner is used the tailings will probably become unsaturated in medium and long long-term periods. Therefore, for those periods the likelihood of liquefaction is zero and failure would only be the result of cracks and discontinuities in the liner, cap or embankment.

In general, all modes of release are likely for an earthquake caused failure of the impoundment. The magnitude of release for each will depend upon the design elements of the disposal plan and the period in which failure occurs. The discussion of the base case and alternatives describes each situation.

To predict the probability of occurrence of an earthquake of a particular magnitude a recurrence chart of the form shown in Figure 13 would be used. Earthquake data is not available for the model site that contains sufficient detail to construct a recurrence chart such as Fig. 13. One earthquake of magnitude 4.1 has been observed at the site in the past 5 years. The data shown in Fig. 13 is for a site in a fairly similar setting and in an area of similar seismicity. For purposes of this application the data shown in Fig. 13 will be used for reference. That data indicates that four earthquakes of magnitude 4 to 4.5 have been observed in the five year observation period. It is believed that the data shown is conservative for the model site.

The maximum credible earthquake for the site considered in Fig. 13 was estimated to be of magnitude 5.5. The impoundment can be designed for an earthquake of that magnitude without undue conservatism and it will, therefore, be assumed that 5.5 was the design magnitude. The likelihood of an earthquake of magnitude greater than 5.5 would be considered to be minimal (1) for the short long-term period. The likelihood would not be much greater for the medium long-term period because 5.5 is the maximum credible earthquake. A value of 3 is considered to be conservative. For the long long-term period the seismicity of the region could change or a major fault movement could occur. A value for likelihood of 7 was assigned. These likelihoods of occurrence would be the same for each alternative. The nature and magnitude of potential release, however, may differ for different alternatives.

b. Base case

For the base case, failure would be in the form of cracking of the embankment or potential slope failure if the earthquake were large enough. Clay embankments are not susceptible to liquefaction and liquefaction of the tailings without embankment failure would not be of much consequence. If slope failure of the embankment did occur a large amount of tailings could be released. For purposes of discussion, it is assumed that 50 percent of the impoundment is released. Therefore, the magnitude of release of undissolved radionuclides will be set at 5. The release magnitude dissolved radionuclides will be almost complete because it is likely that almost all impounded water will be released. A value for this mode of release is set at 9. Because no cap existed in the base case values of 10 are set for radon emanation and gamma-ray emission. Dispersion of tailings due to failure will increase the magnitude of release of both radon emanation and gamma emission above the nonfailed condition. However, an estimate of this is difficult without more complete information.

For the medium and long long-term periods the tailings would be dry and would not be susceptible to liquefaction. Changes in the geometry could accelerate erosion but little other failure would occur. A minimal value of unity was assigned for the dissolved and undissolved radionuclide modes of release. Radon emanation and gamma-ray emission remain at 10.

c. Alternative 1

The presence of the cap and the liner would cause the tailings to remain saturated for a greater period of time thereby increasing the possibility of liquefaction. If liquefaction of the tailings occurs and even if the embankment does not fail, differential settlement may cause the cap to crack thereby increasing the release of radon gas. If settlement or displacement in the alluvium occurs, the liner could also crack thereby increasing the seepage rate. Because of the increased possibility of liquefaction it is assumed that an earthquake caused impoundment failure would release 70 percent of the confined tailings. Therefore, the undissolved radionuclide magnitude is set at 7. As in the base case, the dissolved radionuclide release magnitude remains at 9. Since a cover and liner are specified the increase in radon emanation and gamma-ray emission is considered proportional to the amount of tailings released. A value of 7 is assigned.

For medium and long long-term periods liquefaction is not likely to occur. Cracking of the cap and embankment is likely and would increase radon emanation and gamma-ray emission. However, the magnitudes of release are expected to be minimal. A value of 1 is set for both. Since the tailings are assumed to be dry in these periods seepage will not be a factor. The release of dissolved radionuclides is zero. The magnitude of release of undissolved radionuclides is also set at zero since liquefaction is unlikely.

d. Alternatives 2 and 3

For all three time periods the possibility of liquefaction is zero because the tailings are either fixed or dry. The only release mode would be some gamma-ray emission and radon emanation due to cracking on the cap. Values of unity were therefore assigned for these modes for all three time periods. Furthermore, the nature of the failure is more similar to failure of the cap and the negative utility factor should therefore be 0.5 (the same as for failure of the cap).

e. Alternative 4

Failure in this case may result from settlement due to liquefaction of the tailings which would cause cracking in the cap. Shear stresses induced by the ground shaking could also cause failure of the liner. The conceivable magnitude of cracking, however, would not justify assigning a value greater than unity for increases in gamma-ray emission and radon emanation through the cap, and seepage of dissolved radionuclides through the liner. No release of tailings is expected in any period. For the medium and long long-term tailings are assumed to be dry, therefore increased seepage will not occur.

f. Alternative 5

Failure would only be due to increased radon emanation and gamma-ray emi[illegible]on because of cap cracking. The magnitude was assigned a [illegible] unity and a negative utility factor of 0.5 was used.

[illegible] [illegible]ativ[illegible]

[illegible]n view [illegible] the complex geometry of this site, failure due to cracking of the liner would occur. Some cracking of the cap would also occur. Because of the long thin nature of the impoundment a crack over the surface could cover a greater percentage of the area.

For the short long-term increase gamma-ray emission and radon emanation through the cap, and seepage of dissolved radionuclides through the liner result in a magnitude of 3 for each. For the medium and long long-term seepage is no longer a problem, but gamma-ray emission and radon emanation remain at 3.

h. Alternatives 7 and 8

The earthquake would have a high probability of causing subsidence and caving of the mine. Seepage would be released and leaching of radionuclides by groundwater would occur. Over long long-term periods it is conceivable that leaching of all radionuclides could occur. Cracking of the fixed tailings would also facilitate groundwater intrusion and leaching. A value of 9 was assigned for the long long-term period for release of dissolved radionuclides. This is the only mode of release considered.

Lower values of 3 and 5 were assigned for the short and medium long-term periods respectively. These lower values reflect the fact that leaching is a time dependent process and recurrence of earthquakes would accelerate leaching over longer time periods. The negative utility factor would be 2.0 as for all seepage failures.

24. Failure of the Impoundment due to Floods (B2)

a. Likelihood

In order for flooding of the Tributary River to be a threat to the model site a flood level in excess of 200 ft would be necessary. However, the potential for extreme precipitation events in the local watershed above the site could cause severe problems. The likelihood of this occurring is low in the short long-term period because of diversion structure maintenance and the low probability of occurrence of an event greater than the design magnitude. Over the longer periods, the likelihood of occurrence would increase to near certainty for the long long-term. In fact, within the long long-term period a flood greater than the probable maximum flood may be expected. Values of 1, 5, and 9 were assigned for the three time periods reflecting the higher likelihood of occurrence over longer time periods.

b. Magnitude

i) Base Case

Because no cap is incorporated in this disposal plan the magnitude of release for radon emanation and gamma-ray emission are set at 10. As stated in the discussion on earthquake failures of the impoundment, dispersion due to flooding would undoubtedly increase the release of both. However, it is difficult to estimate this and the value of 10 is assigned. Also, because there is no cap, flood waters could remove significant portions of tailings, releasing dissolved and undissolved radionuclides. For the short long-term, with maintenance applied a value of 5 is assigned for the magnitude of release of undissolved radionuclides and 9 for dissolved. A high value for the magnitude of release for dissolved radionuclides is assigned on the assumption that flood waters would liberate most of this potential. For the medium and long long-term complete release of both the undissolved and dissolved radionuclides is expected. Values of 9 are assigned for both magnitudes.

ii) Alternatives 1 through 6

Because of the presence of the cap increases in radon emanation and gamma-ray emission are assumed proportional to the amount of tailings released due to flooding. For the short long-term, with the cap and maintenance flood release of tailings is set at a magnitude of 3. The magnitude of release for the dissolved radionuclides is assumed slightly higher, 5, based on the previous discussion. For the larger periods the beneficial effects of the cover will diminish with time. The magnitude of release of tailings is set at 70 percent for the medium long-term and 90 percent (essentially total release) for the long long-term. Corresponding magnitudes for the release modes are shown in Table 14.

iii) Alternatives 7 and 8

Flooding on the surface would have no influence in the deep mine. The severity is zero.

iv) Negative Utility

The negative utility for flood failure of the impoundment is high, 1.75. This reflects the almost impossible task of recovering tailings dispersed over wide areas by extensive flooding. It also is based upon the ability to control the failure once it begins, i.e., flooding is a sudden catastrophic event, and the inability to design or maintain control elements against the extreme event that is likely over the long periods considered.

25. Failure of the Impoundment due to Wind Storms (B2)

The application of the Wind Soil Loss Equation previously with regard to wind erosion has indicated that soil loss due to wind would be small for any time period other than the long long-term period. The occurrence of abnormally high velocity wind storms for periods of even several days would have a small influence and the severity is considered to be zero.

26. Failure of the Impoundment due to Tornadoes

a. General Considerations

i) Likelihood

From Figure 14 it can be seen that the probability of tornado winds exceeding about 150 mph is less than 1×10^{-4}. (Although the axis label on the figure indicates that the probability is for a 50 year period the text of the reference uses these values as probabilities for a year). The value is considered for the probability quoted for the Sweetwater Uranium Project (NRC, 1977b). That windspeed is about the minimum speed that needs to be considered for erosion possibilities because other windspeeds would be considered with regard to wind storms and wind erosion. The probability of occurrence over a 1,000 year period would then be $1{,}000 \times 10^{-4}$ or 10%. Furthermore, Fig. 14 is prepared for the area east of the Continental Divide and may be conservative for uranium mining areas. Consequently, a likelihood of occurrence of 1 will be assigned for the short and medium long-term period, but for the long long-term the likelihood will be 10. This will be for the base case and all alternatives.

ii) Magnitude

The magnitude of release of radioactive material due to a tornado on or near the tailings deposit will be a function of the size and speed of movement of the tornado. Structures above ground will tend to suffer more damage by tornadoes than below grade impoundments. Underground disposal plans will suffer no release.

The magnitude of release due to tornadoes will be a function of transported tailings only. Radon emanation and gamma-ray emission, due to the presence or absence of a cover is not included, but adequately addressed in wind and water erosion of the cap. The presence or absence of a cover does, however, dramatically effect tailing transport and is reflected in magnitude of release for the base case and alternatives. Because of the limited duration of contact and reduced surface effects the erosion power of tornadoes is limited.

b. Base Case

Without a cover tailings are exposed to potential transport by tornadoes. However, the erosive effects would be small. The magnitude for all time periods was assigned a value of unity.

c. Alternatives 1 through 6

Covers are specified for all alternatives. Therefore the ability of a tornado to transport tailings is zero as long as the cover is intact. This is assumed to be the case for the short and medium long-term periods. For long long-term periods the cover may no longer exist. Therefore the magnitude is 1.0, to an uncovered impoundment. Fixing of tailings does not reduce the magnitude of release for this period because it is assumed that weathering will have destroyed any cementation that was accomplished. This is especially true at the surface where tornadoes would be effective.

27. Failure of the Impoundment due to Glaciation (B5)

For the model site the likelihood of a glacier occurring in this area is considered to be very remote even in the long long-term. Therefore, the severity of glaciers was zero for all alternatives.

28. Failure of the Impoundment due to Fire and Pestilence

These natural phenomena could cause failure of the vegetation. Therefore, the discussion and values developed in Sections 19 and 20 for fire and drought apply here.

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Table 14. Application of Methodology to Base Case and Alternative (continued)

1. Cap Failure Due to Differential Settlement (Ala)

Case	Likelihood (L)			Magnitude (M)			Negative Utility (U)	Severity (S)		
	Short	Med.	Long	Short	Med.	Long		Short	Med.	Long
Base (1)	10	10	10	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	0.5	$\begin{bmatrix}50\\0\\0\\50\end{bmatrix}$	$\begin{bmatrix}50\\0\\0\\50\end{bmatrix}$	$\begin{bmatrix}50\\0\\0\\50\end{bmatrix}$
Alt 1 (2)	7	7	7	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	0.5	$\begin{bmatrix}4\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}4\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}4\\0\\0\\0\end{bmatrix}$
Alt 2	1	1	1	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	0.5	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$
Alt 3	1	1	1	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	0.5	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$
Alt 4	5	5	5	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	0.5	$\begin{bmatrix}3\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}3\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}3\\0\\0\\0\end{bmatrix}$
Alt 5	3	3	3	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	0.5	$\begin{bmatrix}2\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}2\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}2\\0\\0\\0\end{bmatrix}$
Alt 6	1	1	1	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	0.5	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$
Alt 7 & 8	--	--	--	--	--	--	--	--	--	--

Comments

(1) No cap installed.
(2) May increase if thin, brittle cap used.

Table 14. Application of Methodology to Base Case and Alternative (continued)

2. Cap Failure Due to Gullying (Alb)

Case	Likelihood (L)			Magnitude (M)			Negative Utility (U)	Severity (S)		
	Short	Med.	Long	Short	Med.	Long		Short	Med.	Long
Base (3)	10	10	10	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	0.5	$\begin{bmatrix}50\\0\\0\\50\end{bmatrix}$	$\begin{bmatrix}50\\0\\0\\50\end{bmatrix}$	$\begin{bmatrix}50\\0\\0\\50\end{bmatrix}$
Alt 1	5	7	9	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}5\\0\\0\\5\end{bmatrix}$	$\begin{bmatrix}9\\0\\0\\9\end{bmatrix}$	0.5	$\begin{bmatrix}3\\0\\0\\3\end{bmatrix}$	$\begin{bmatrix}18\\0\\0\\18\end{bmatrix}$	$\begin{bmatrix}42\\0\\0\\42\end{bmatrix}$
Alt 2-6 (4)	3	5	7	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}5\\0\\0\\5\end{bmatrix}$	$\begin{bmatrix}9\\0\\0\\9\end{bmatrix}$	0.5	$\begin{bmatrix}2\\0\\0\\2\end{bmatrix}$	$\begin{bmatrix}8\\0\\0\\8\end{bmatrix}$	$\begin{bmatrix}32\\0\\0\\32\end{bmatrix}$
Alt 7 & 8	-	-	-	-	-	-		0	0	0

Comments

(3) No cap ∴ L & M equate to certain failure and 100% release respectively.

(4) L reduced for Alt 2-6 because of the absences of embankment interaction.

632 176

Table 14. Application of Methodology to Base Case and Alternative (continued)

3. Cap Failure Due to Water Sheet Erosion (Alc)

Case	Likelihood (L)			Magnitude (M)			Negative Utility (U)	Severity (S)		
	Short	Med.	Long	Short	Med.	Long		Short	Med.	Long
Base (3)	10	10	10	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	0.5	$\begin{bmatrix}50\\0\\0\\50\end{bmatrix}$	$\begin{bmatrix}50\\0\\0\\50\end{bmatrix}$	$\begin{bmatrix}50\\0\\0\\50\end{bmatrix}$
Alt 1-6 (5)	10	10	10	$\begin{bmatrix}0\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}3\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	0.5	$\begin{bmatrix}0\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}15\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}50\\0\\0\\50\end{bmatrix}$
Alt 7 & 8	-	-	-	-	-	-		0	0	0

Comments

(5) These levels of magnitude assume only soil loss and no deposition.

632 177

Table 14. Application of Methodology to Base Case and Alternative (continued)

4. Cap Failure Due to Wind Erosion (Ald)

Case	Likelihood (L)			Magnitude (M)			Negative Utility (U)	Severity (S)		
	Short	Med.	Long	Short	Med.	Long		Short	Med.	Long
Base (3)	10	10	10	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	0.5	$\begin{bmatrix}50\\0\\0\\50\end{bmatrix}$	$\begin{bmatrix}50\\0\\0\\50\end{bmatrix}$	$\begin{bmatrix}50\\0\\0\\50\end{bmatrix}$
Alt 1-6 (5)	10	10	10	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	0.5	$\begin{bmatrix}5\\0\\0\\5\end{bmatrix}$	$\begin{bmatrix}5\\0\\0\\5\end{bmatrix}$	$\begin{bmatrix}50\\0\\0\\50\end{bmatrix}$
Alt 7 & 8	-	-	-	-	-	-		0	0	0

Comments

Table 14. Application of Methodology to Base Case and Alternative (continued)

5. Cap Failure Due to Floods (Ale)

Case	Likelihood (L)			Magnitude (M)			Negative Utility (U)	Severity (S)		
	Short	Med.	Long	Short	Med.	Long		Short	Med.	Long
Base (3)	10	10	10	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	0.5	$\begin{bmatrix}50\\0\\0\\50\end{bmatrix}$	$\begin{bmatrix}50\\0\\0\\50\end{bmatrix}$	$\begin{bmatrix}50\\0\\0\\50\end{bmatrix}$
Alt 1-6	1	5	9	$\begin{bmatrix}3\\0\\0\\3\end{bmatrix}$	$\begin{bmatrix}5\\0\\0\\5\end{bmatrix}$	$\begin{bmatrix}9\\0\\0\\9\end{bmatrix}$	0.5	$\begin{bmatrix}2\\0\\0\\2\end{bmatrix}$	$\begin{bmatrix}13\\0\\0\\13\end{bmatrix}$	$\begin{bmatrix}41\\0\\0\\41\end{bmatrix}$
Alt 7 & 8	-	-	-	-	-	-		0	0	0

Comments

Table 14. Application of Methodology to Base Case and Alternative (continued)

6. Cap Failure Due to Chemical Attack (Alf)

Case	Likelihood (L)			Magnitude (M)			Negative Utility (U)	Severity (S)		
	Short	Med.	Long	Short	Med.	Long		Short	Med.	Long
Base (3)	10	10	10	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	0.5	$\begin{bmatrix}50\\0\\0\\50\end{bmatrix}$	$\begin{bmatrix}50\\0\\0\\50\end{bmatrix}$	$\begin{bmatrix}50\\0\\0\\50\end{bmatrix}$
Alt 1-6	1	1	1	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	0.5	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\0\end{bmatrix}$
Alt 7 & 8	-	-	-	-	-	-		0	0	0

Comments

632 180

Table 14. Application of Methodology to Base Case and Alternative (continued)

7. Cap Failure Due to Shrinkage (Alg)

Case	Likelihood (L)			Magnitude (M)			Negative Utility (U)	Severity (S)		
	Short	Med.	Long	Short	Med.	Long		Short	Med.	Long
Base (3)	10	10	10	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	0.5	$\begin{bmatrix}50\\0\\0\\50\end{bmatrix}$	$\begin{bmatrix}50\\0\\0\\50\end{bmatrix}$	$\begin{bmatrix}50\\0\\0\\50\end{bmatrix}$
Alt 1-6[4] (6)	5	9	9	$\begin{bmatrix}9\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}9\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}9\\0\\0\\0\end{bmatrix}$	0.5	$\begin{bmatrix}23\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}42\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}42\\0\\0\\0\end{bmatrix}$
Alt 7 & 8	-	-	-	-	-	-		0	0	0

Comments

(6) Assumes cohesive cap. If noncohesive material is used shrinkage failure will not occur.

Table 14. Application of Methodology to Base Case and Alternative (continued)

8. Liner Failure Due to Differential Settlement (A2a)

Case	Likelihood (L)			Magnitude (M)			Negative Utility (U)	Severity (S)		
	Short	Med.	Long	Short	Med.	Long		Short	Med.	Long
Base (7)	10	10	10	$\begin{bmatrix}0\\10\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\10\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\10\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\200\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\200\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\200\\0\\0\end{bmatrix}$
Alt 1	5	5	5	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\10\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\10\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\10\\0\\0\end{bmatrix}$
Alt 2	10	10	10	$\begin{bmatrix}0\\5\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\5\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\5\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\100\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\100\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\100\\0\\0\end{bmatrix}$
Alt 3-6	1	1	1	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\2\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\2\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\2\\0\\0\end{bmatrix}$
Alt 7	10	10	10	$\begin{bmatrix}0\\5\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\5\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\5\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\100\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\100\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\100\\0\\0\end{bmatrix}$
Alt 8	9	9	9	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\5\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\9\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\18\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\90\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\162\\0\\0\end{bmatrix}$

Comments

(7) No liner specified.

Table 14. Application of Methodology to Base Case and Alternative (continued)

9. Liner Failure Due to Subsidence (A2b)

Case	Likelihood (L)			Magnitude (M)			Negative Utility (U)	Severity (S)		
	Short	Med.	Long	Short	Med.	Long		Short	Med.	Long
Base (7)	10	10	10	$\begin{bmatrix}0\\10\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\10\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\10\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\200\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\200\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\200\\0\\0\end{bmatrix}$
Alt 1	1	1	1	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\9\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\9\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\2\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\18\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\18\\0\\0\end{bmatrix}$
Alt 2	10	10	10	$\begin{bmatrix}0\\5\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\5\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\5\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\100\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\100\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\100\\0\\0\end{bmatrix}$
Alt 3	1	1	1	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\2\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\2\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\2\\0\\0\end{bmatrix}$
Alt 4-6	1	1	1	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\9\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\9\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\2\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\18\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\18\\0\\0\end{bmatrix}$
Alt 7	3	7	7	$\begin{bmatrix}0\\5\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\7\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\7\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\30\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\98\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\98\\0\\0\end{bmatrix}$
Alt 8	3	7	7	$\begin{bmatrix}0\\5\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\9\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\9\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\30\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\126\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\126\\0\\0\end{bmatrix}$

Comments

Table 14. Application of Methodology to Base Case and Alternative (continued)

10. Liner Failure Due to Chemical Attack (A2c)

Case	Likelihood (L)			Magnitude (M)			Negative Utility (U)	Severity (S)		
	Short	Med.	Long	Short	Med.	Long		Short	Med.	Long
Base (7)	10	10	10	$\begin{bmatrix}0\\10\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\10\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\10\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\200\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\200\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\200\\0\\0\end{bmatrix}$
Alt 1	3	5	5	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\3\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\5\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\6\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\30\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\60\\0\\0\end{bmatrix}$
Alt 2	10	10	10	$\begin{bmatrix}0\\5\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\5\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\5\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\100\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\100\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\100\\0\\0\end{bmatrix}$
Alt 3-4	1	3	5	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\3\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\5\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\2\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\18\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\50\\0\\0\end{bmatrix}$
Alt 5	10	10	10	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\20\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\20\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\20\\0\\0\end{bmatrix}$
Alt 6	3	5	5	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\3\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\5\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\6\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\30\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\50\\0\\0\end{bmatrix}$
Alt 7	10	10	10	$\begin{bmatrix}0\\5\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\5\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\5\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\100\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\100\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\100\\0\\0\end{bmatrix}$
Alt 8	5	7	7	$\begin{bmatrix}0\\5\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\9\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\9\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\50\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\126\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\126\\0\\0\end{bmatrix}$

Comments

Table 14. Application of Methodology to Base Case and Alternative (continued)

11. Liner Failure Due to Physical Penetration (A2d)

Case	Likelihood (L)			Magnitude (M)			Negative Utility (U)	Severity (S)		
	Short	Med.	Long	Short	Med.	Long		Short	Med.	Long
Base (7)	10	10	10	$\begin{bmatrix}0\\10\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\10\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\10\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\200\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\200\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\200\\0\\0\end{bmatrix}$
Alt 1	9	9	9	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\18\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\18\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\18\\0\\0\end{bmatrix}$
Alt 2	10	10	10	$\begin{bmatrix}0\\5\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\5\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\5\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\100\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\100\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\100\\0\\0\end{bmatrix}$
Alt 3	9	9	9	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\18\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\18\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\18\\0\\0\end{bmatrix}$
Alt 4	7	7	7	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\14\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\14\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\14\\0\\0\end{bmatrix}$
Alt 5	-	-	-	-	-	-		0	0	0
Alt 6	9	9	9	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\18\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\18\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\18\\0\\0\end{bmatrix}$
Alt 7 & 8	-	-	-	-	-	-		0	0	0

Comments

Table 14. Application of Methodology to Base Case and Alternative (continued)

12. Embankment Failure Due to Differential Settlement (A3a)

Case	Likelihood (L)			Magnitude (M)			Negative Utility (U)	Severity (S)		
	Short	Med.	Long	Short	Med.	Long		Short	Med.	Long
Base & Alt 1	3	3	3	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\3\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\3\\0\\0\end{bmatrix}$	1.0	$\begin{bmatrix}0\\3\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\9\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\9\\0\\0\end{bmatrix}$
Alt 2-8	-	-	-	-	-	-		0	0	0

Comments

Table 14. Application of Methodology to Base Case and Alternative (continued)

13. Embankment Failure Due to Slope Instability (A3b)

Case	Likelihood (L)			Magnitude (M)			Negative Utility (U)	Severity (S)		
	Short	Med.	Long	Short	Med.	Long		Short	Med.	Long
Base & Alt 1	1	1	1	$\begin{bmatrix}3\\0\\0\\3\end{bmatrix}$	$\begin{bmatrix}3\\0\\0\\3\end{bmatrix}$	$\begin{bmatrix}3\\0\\0\\3\end{bmatrix}$	1.0	$\begin{bmatrix}3\\0\\0\\3\end{bmatrix}$	$\begin{bmatrix}3\\0\\0\\3\end{bmatrix}$	$\begin{bmatrix}3\\0\\0\\3\end{bmatrix}$
Alt 2-8	-	-	-	-	-	-		0	0	0

Comments

Table 14. Application of Methodology to Base Case and Alternative (continued)

14. Embankment Failure Due to Gullying (A3c)

Case	Likelihood (L)			Magnitude (M)			Negative Utility (U)	Severity (S)		
	Short	Med.	Long	Short	Med.	Long		Short	Med.	Long
Base & Alt 1	5	7	9	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}3\\3\\3\\3\end{bmatrix}$	$\begin{bmatrix}9\\9\\9\\9\end{bmatrix}$	1.0	$\begin{bmatrix}5\\0\\0\\5\end{bmatrix}$	$\begin{bmatrix}21\\21\\21\\21\end{bmatrix}$	$\begin{bmatrix}81\\81\\81\\81\end{bmatrix}$
Alt 2-8	-	-	-	-	-	-		0	0	0

Comments

Table 14. Application of Methodology to Base Case and Alternative (continued)

15. Embankment Failure Due to Water Sheet Erosion (A3d)

Case	Likelihood (L)			Magnitude (M)			Negative Utility (U)	Severity (S)		
	Short	Med.	Long	Short	Med.	Long		Short	Med.	Long
Base & Alt 1	5	7	9	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}9\\9\\9\\9\end{bmatrix}$	1.0	$\begin{bmatrix}5\\0\\0\\5\end{bmatrix}$	$\begin{bmatrix}7\\0\\0\\7\end{bmatrix}$	$\begin{bmatrix}81\\81\\81\\81\end{bmatrix}$
Alt 2-8	-	-	-	-	-	-		0	0	0

Comments

Table 14. Application of Methodology to Base Case and Alternative (continued)

16. Embankment Failure Due to Wind Erosion (A3e)

Case	Likelihood (L)			Magnitude (M)			Negative Utility (U)	Severity (S)		
	Short	Med.	Long	Short	Med.	Long		Short	Med.	Long
Base & Alt 1	10	10	10	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}9\\9\\9\\9\end{bmatrix}$	1.0	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	$\begin{bmatrix}90\\90\\90\\90\end{bmatrix}$
Alt 2-8	-	-	-	-	-	-		0	0	0

Comments

632 190

Table 14. Application of Methodology to Base Case and Alternative (continued)

17. Embankment Failure Due to Flooding (A3f)

Case	Likelihood (L) Short	Likelihood (L) Med.	Likelihood (L) Long	Magnitude (M) Short	Magnitude (M) Med.	Magnitude (M) Long	Negative Utility (U)	Severity (S) Short	Severity (S) Med.	Severity (S) Long
Base & Alt 1	1	7	9	$\begin{bmatrix}5\\5\\5\\5\end{bmatrix}$	$\begin{bmatrix}9\\9\\9\\9\end{bmatrix}$	$\begin{bmatrix}9\\9\\9\\9\end{bmatrix}$	1.0	$\begin{bmatrix}5\\5\\5\\5\end{bmatrix}$	$\begin{bmatrix}63\\63\\63\\63\end{bmatrix}$	$\begin{bmatrix}81\\81\\81\\81\end{bmatrix}$
Alt 2-8	-	-	-	-	-	-		0	0	0

Comments

632 191

Table 14. Application of Methodology to Base Case and Alternative (continued)

18. Embankment Failure Due to Chemical Weathering (A3g)

Case	Likelihood (L)			Magnitude (M)			Negative Utility (U)	Severity (S)		
	Short	Med.	Long	Short	Med.	Long		Short	Med.	Long
Base	1	3	5	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	$\begin{bmatrix}3\\3\\3\\3\end{bmatrix}$	$\begin{bmatrix}3\\3\\3\\3\end{bmatrix}$	1.0	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	$\begin{bmatrix}9\\9\\9\\9\end{bmatrix}$	$\begin{bmatrix}15\\15\\15\\15\end{bmatrix}$
Alt 1	1	5	7	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	$\begin{bmatrix}3\\3\\3\\3\end{bmatrix}$	$\begin{bmatrix}3\\3\\3\\3\end{bmatrix}$	1.0	$\begin{bmatrix}0\\1\\0\\0\end{bmatrix}$	$\begin{bmatrix}15\\15\\15\\15\end{bmatrix}$	$\begin{bmatrix}21\\21\\21\\21\end{bmatrix}$
Alt 2-8	-	-	-	-	-	-		0	0	0

Comments

Table 14. Application of Methodology to Base Case and Alternative (continued)

19. Revegetation Failure Due to Fire (A4a)

Case	Likelihood (L)			Magnitude (M)			Negative Utility (U)	Severity (S)		
	Short	Med.	Long	Short	Med.	Long		Short	Med.	Long
Base (3)	3	7	9	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	0.25	$\begin{bmatrix}8\\0\\0\\8\end{bmatrix}$	$\begin{bmatrix}18\\0\\0\\18\end{bmatrix}$	$\begin{bmatrix}23\\0\\0\\23\end{bmatrix}$
Alt 1-6	3	7	9	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	0.25	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}2\\0\\0\\2\end{bmatrix}$	$\begin{bmatrix}2\\0\\0\\2\end{bmatrix}$
Alt 7 & 8	-	-	-	-	-	-		0	0	0

Comments

Table 14. Application of Methodology to Base Case and Alternative (continued)

20. Revegetation Failure Due to Climate Change (A4b)

Case	Likelihood (L)			Magnitude (M)			Negative Utility (U)	Severity (S)		
	Short	Med.	Long	Short	Med.	Long		Short	Med.	Long
Base	5	7	9	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	0.25	$\begin{bmatrix}13\\0\\0\\13\end{bmatrix}$	$\begin{bmatrix}18\\0\\0\\18\end{bmatrix}$	$\begin{bmatrix}25\\0\\0\\25\end{bmatrix}$
Alt 1-6	5	7	9	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	0.25	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}2\\0\\0\\2\end{bmatrix}$	$\begin{bmatrix}2\\0\\0\\2\end{bmatrix}$
Alt 7 & 8	-	-	-	-	-	-		0	0	0

Comments

Table 14. Application of Methodology to Base Case and Alternative (continued)

21. Water Diversion Structure Failure Due to Slumping (A5a)

Case	Likelihood (L)			Magnitude (M)			Negative Utility (U)	Severity (S)		
	Short	Med.	Long	Short	Med.	Long		Short	Med.	Long
Base & Alt 1-6	1	1	1	$\begin{bmatrix}3\\0\\0\\3\end{bmatrix}$	$\begin{bmatrix}3\\0\\0\\3\end{bmatrix}$	$\begin{bmatrix}3\\0\\0\\3\end{bmatrix}$	0.75	$\begin{bmatrix}2\\0\\0\\2\end{bmatrix}$	$\begin{bmatrix}2\\0\\0\\2\end{bmatrix}$	$\begin{bmatrix}2\\0\\0\\2\end{bmatrix}$
Alt 7 & 8	-	-	-	-	-	-		0	0	0

Comments

Table 14. Application of Methodology to Base Case and Alternative (continued)

22. Water Diversion Structure Failure Due to Obstructions (A5b)

Case	Likelihood (L)			Magnitude (M)			Negative Utility (U)	Severity (S)		
	Short	Med.	Long	Short	Med.	Long		Short	Med.	Long
Base & Alt 1-6	3	9	9	$\begin{bmatrix}3\\0\\0\\3\end{bmatrix}$	$\begin{bmatrix}3\\0\\0\\3\end{bmatrix}$	$\begin{bmatrix}3\\0\\0\\3\end{bmatrix}$	0.75	$\begin{bmatrix}7\\0\\0\\7\end{bmatrix}$	$\begin{bmatrix}20\\0\\0\\20\end{bmatrix}$	$\begin{bmatrix}20\\0\\0\\20\end{bmatrix}$
Alt 7 & 8	-	-	-	-	-	-		0	0	0

Comments

Table 14. Application of Methodology to Base Case and Alternative (continued)

23. Impoundment Failure Due to Earthquakes (B1)

Case	Likelihood (L)			Magnitude (M)			Negative Utility (U)	Severity (S)		
	Short	Med.	Long	Short	Med.	Long		Short	Med.	Long
Base	1	3	7	$\begin{bmatrix}10\\9\\5\\10\end{bmatrix}$	$\begin{bmatrix}10\\1\\1\\10\end{bmatrix}$	$\begin{bmatrix}10\\1\\1\\10\end{bmatrix}$	2.0	$\begin{bmatrix}20\\18\\10\\20\end{bmatrix}$	$\begin{bmatrix}60\\6\\6\\60\end{bmatrix}$	$\begin{bmatrix}140\\14\\14\\140\end{bmatrix}$
Alt 1	1	3	7	$\begin{bmatrix}7\\9\\7\\7\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	2.0	$\begin{bmatrix}14\\18\\14\\14\end{bmatrix}$	$\begin{bmatrix}6\\0\\0\\6\end{bmatrix}$	$\begin{bmatrix}6\\0\\0\\6\end{bmatrix}$
Alt 2	1	3	7	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	0.5	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}2\\0\\0\\2\end{bmatrix}$	$\begin{bmatrix}4\\0\\0\\4\end{bmatrix}$
Alt 3	1	3	7	$\begin{bmatrix}1\\1\\0\\1\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	2.0	$\begin{bmatrix}2\\2\\0\\2\end{bmatrix}$	$\begin{bmatrix}6\\6\\0\\6\end{bmatrix}$	$\begin{bmatrix}14\\14\\0\\14\end{bmatrix}$
Alt 4	1	3	7	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	0.5	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}2\\0\\0\\2\end{bmatrix}$	$\begin{bmatrix}4\\0\\0\\4\end{bmatrix}$
Alt 5	1	3	7	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	2.0	$\begin{bmatrix}2\\0\\0\\2\end{bmatrix}$	$\begin{bmatrix}6\\0\\0\\6\end{bmatrix}$	$\begin{bmatrix}14\\0\\0\\14\end{bmatrix}$
Alt 6	1	3	7	$\begin{bmatrix}3\\3\\0\\3\end{bmatrix}$	$\begin{bmatrix}3\\0\\0\\3\end{bmatrix}$	$\begin{bmatrix}3\\0\\0\\3\end{bmatrix}$	0.5	$\begin{bmatrix}2\\2\\0\\2\end{bmatrix}$	$\begin{bmatrix}5\\0\\0\\5\end{bmatrix}$	$\begin{bmatrix}11\\0\\0\\11\end{bmatrix}$
Alt 7 & 8	1	3	7	$\begin{bmatrix}0\\3\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\5\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\9\\0\\0\end{bmatrix}$	2.0	$\begin{bmatrix}0\\6\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\10\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\18\\0\\0\end{bmatrix}$

Comments

Table 14. Application of Methodology to Base Case and Alternative (continued)

24. Impoundment Failure Due to Floods (B2)

Case	Likelihood (L)			Magnitude (M)			Negative Utility (U)	Severity (S)		
	Short	Med.	Long	Short	Med.	Long		Short	Med.	Long
Base	1	5	9	$\begin{bmatrix}10\\9\\5\\10\end{bmatrix}$	$\begin{bmatrix}10\\9\\9\\10\end{bmatrix}$	$\begin{bmatrix}10\\9\\9\\10\end{bmatrix}$	1.75	$\begin{bmatrix}18\\16\\9\\18\end{bmatrix}$	$\begin{bmatrix}88\\79\\79\\88\end{bmatrix}$	$\begin{bmatrix}158\\142\\142\\158\end{bmatrix}$
Alt 1-6	1	5	9	$\begin{bmatrix}3\\5\\3\\3\end{bmatrix}$	$\begin{bmatrix}7\\9\\7\\7\end{bmatrix}$	$\begin{bmatrix}9\\9\\9\\9\end{bmatrix}$	1.75	$\begin{bmatrix}5\\9\\5\\5\end{bmatrix}$	$\begin{bmatrix}61\\79\\61\\61\end{bmatrix}$	$\begin{bmatrix}142\\142\\142\\142\end{bmatrix}$
Alt 7 & 8	-	-	-	-	-	-		0	0	0

Comments

Table 14. Application of Methodology to Base Case and Alternative (continued)

25. Impoundment Failure Due to Wind Storms (B3)

Case	Likelihood (L)			Magnitude (M)			Negative Utility (U)	Severity (S)		
	Short	Med.	Long	Short	Med.	Long		Short	Med.	Long
Base & all Alts	-	-	-	-	-	-		0	0	0

Comments

Table 14. Application of Methodology to Base Case and Alternative (continued)

26. Impoundment Failure Due to Tornadoes (B4)

Case	Likelihood (L)			Magnitude (M)			Negative Utility (U)	Severity (S)		
	Short	Med.	Long	Short	Med.	Long		Short	Med.	Long
Base	1	1	10	$\begin{bmatrix}0\\0\\1\\0\end{bmatrix}$	$\begin{bmatrix}0\\0\\1\\0\end{bmatrix}$	$\begin{bmatrix}0\\0\\1\\0\end{bmatrix}$	1.5	$\begin{bmatrix}0\\0\\2\\0\end{bmatrix}$	$\begin{bmatrix}0\\0\\2\\0\end{bmatrix}$	$\begin{bmatrix}0\\0\\15\\0\end{bmatrix}$
Alt 1-6	1	1	10	$\begin{bmatrix}0\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\0\\1\\0\end{bmatrix}$	1.5	$\begin{bmatrix}0\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\0\\0\\0\end{bmatrix}$	$\begin{bmatrix}0\\0\\15\\0\end{bmatrix}$
Alt 7 & 8	-	-	-	-	-	-		0	0	0

Comments

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Table 14. Application of Methodology to Base Case and Alternative (continued)

27. Impoundment Failure Due to Glaciation (B5)

Case	Likelihood (L)			Magnitude (M)			Negative Utility (U)	Severity (S)		
	Short	Med.	Long	Short	Med.	Long		Short	Med.	Long
Base & all Alts	-	-	-	-	-	-		0	0	0

Comments

Table 14. Application of Methodology to Base Case and Alternative (continued)

28. Impoundment Failure Due to Fire and Pestilence (B6)

Case	Likelihood (L)			Magnitude (M)			Negative Utility (U)	Severity (S)		
	Short	Med.	Long	Short	Med.	Long		Short	Med.	Long
Base	3	7	9	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	$\begin{bmatrix}10\\0\\0\\10\end{bmatrix}$	0.25	$\begin{bmatrix}9\\0\\0\\9\end{bmatrix}$	$\begin{bmatrix}18\\0\\0\\18\end{bmatrix}$	$\begin{bmatrix}25\\0\\0\\25\end{bmatrix}$
Alt 1-6	3	7	9	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	0.25	$\begin{bmatrix}1\\0\\0\\1\end{bmatrix}$	$\begin{bmatrix}2\\0\\0\\2\end{bmatrix}$	$\begin{bmatrix}2\\0\\0\\2\end{bmatrix}$
Alt 7 & 8	-	-	-	-	-	-		0	0	0

Comments

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VII. PHYSIOGRAPHIC REGIONS: THEIR RELATION TO LONG TERM FAILURE MECHANISMS

A. Introduction

Six physiographic regions have been identified in which uranium mining and milling is taking place or could take place. This section identifies the general climatic, and seismic characteristics of these regions and relates these characteristics to long-term stability. The failure mechanisms described in section IV and the model site will serve as the basis for discussion. Particular emphasis is placed on natural phenomena based failure mechanisms. The purpose is to highlight those specific regional charactieristics that would most effect tailings disposal design requirements and long long-term stability. The regions cover large geographic areas, therefore this discussion can only be a very generalized description. It does however provide a basis for broad comparison. Hunt (1974) and Thornbury (1965) serve as basic references.

The six physiographic regions being considered are:

- West Gulf Coastal Plain
- Great Plains
- Southern Rocky Mountains
- Wyoming Basin
- Colorado Plateau
- Columbia-Snake River Plateau

B. General Description of the Regions

1. West Gulf Coastal Plain

This region shown in Figure 29, extends along the Gulf coast from the Mexico border of Texas to the Mississippi River, and inland in Texas to the Balcones Escarpment west of San Antonio and Austin on a line up

Fig. 29. West gulf coastal plains (from Hunt, 1974).

to Waco and Fort Worth. Across the West Gulf Coast Plain annual average precipitation increases from 20 inches along the Rio Grande to 60 inches in southern Louisiana. Precipitation is highly variable in the southern part of the region. The area is also subject to extreme precipitation due primarily to late summer and fall hurricanes spawned storms. Figure 30, which shows 100-year, 6-hour maximum rainfall in inches based on existing records, demonstrates this. (This figure is used to evaluate rainfall intensity in conjunction with Hunt (1974) and Thornbury (1965) for all the other physiographic regions discussed.) The southern and western parts are semi-arid due, in part, to high evaporation rates. The principal vegetation is mesquite, thornbushes, cacti, curly grass, buffalo grass, and various shrubs. The soils are alkaline in the semi-arid Texas area, and contain much swelling clay. The seismic risk potential, shown in Figure 12, Chapter IV section B1, are in the lowest category. The tornado frequency, Figure 15 is moderate, but relatively high compared to the other physiographic regions considered here. Figure 31 shows the annual extreme winds for a 100 year recurrence interval for the U.S. The West Gulf Coastal Plain has a general extreme wind of 80 miles per hour.

2. The Great Plains

The Great Plains, shown in Figure 32, is a belt running from Canada to Central Texas which is generally about 400 miles wide with its western edge along the foot of the Rocky Mountains. The climate is semi-arid and continental. Temperature differences are extreme. Average annual precipitation is about 15 inches. The Great Plains, like other semi-arid areas, experiences occasionally hard rain storms which lead to flash flooding. Drought is common. Extreme winds, Figure 31,

Fig. 30. 100-year, 6-hour rainfall (inches) (from Hershfield, 1961).

Fig. 31. Isotach 0.01 quantities, in miles per hour: annual extreme mile 30 feet above ground, 100-year mean recurrence interval from Thom, 1968).

Fig. 32. Great plains (from Hunt, 1974).

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are in the 90 mph ranges. The tornado frequency decreases northward and westward (Figure 15). and is low to moderate depending upon geographic location. For the areas of uranium milling, frequencies are low. Another climatic hazard is hail. Native vegetation type is typically short, tall and mixed grass communities (Oosting, 1956). Much of this has been disturbed or replaced by agricultural and grazing. Soils are variable reflecting parent material and climatic zones. The seismic potential, Figure 12, is generally low. Glaciation may be a factor but only in the northern portion, Figure 33.

3. Southern Rocky Mountains

The Southern Rocky Mountains extend northward from north-central New Mexico to Casper, Wyoming as shown in Figure 34. The climate is extremely variable depending upon elevation. In this region 50 percent of the runoff is due to thunderstorms. As with climate, vegetation is variable going from Alpine (above timberline) to pine and aspen forests at lower elevations. Glaciation was important in developing the mountain terrains and some still exist at high elevation. Tectonic and volcanic activity is responsible for the mountain development and faulting is widespread. Seismic risk is moderate to moderately high, Figure 12.

4. Wyoming Basin

The Wyoming Basin region, Figure 35, is in reality a group of basins, each with its own character but having a regional resemblance. Precipitation is variable within the basin but large areas are arid to semi-arid with 10 to 15 inches annually. Like the Great Plains the weather is extreme and winds can be strong. Figure 31 shows 90 to 100 mph winds. Tornadoes represent a low hazard in the region (Figure 15). The seismic risk is moderate (Figure 12).

Fig. 33. Three stages in the retreat of the last continental ice sheet (from Hunt, 1974).

Fig. 34. Rocky mountains (from Hunt, 1974)

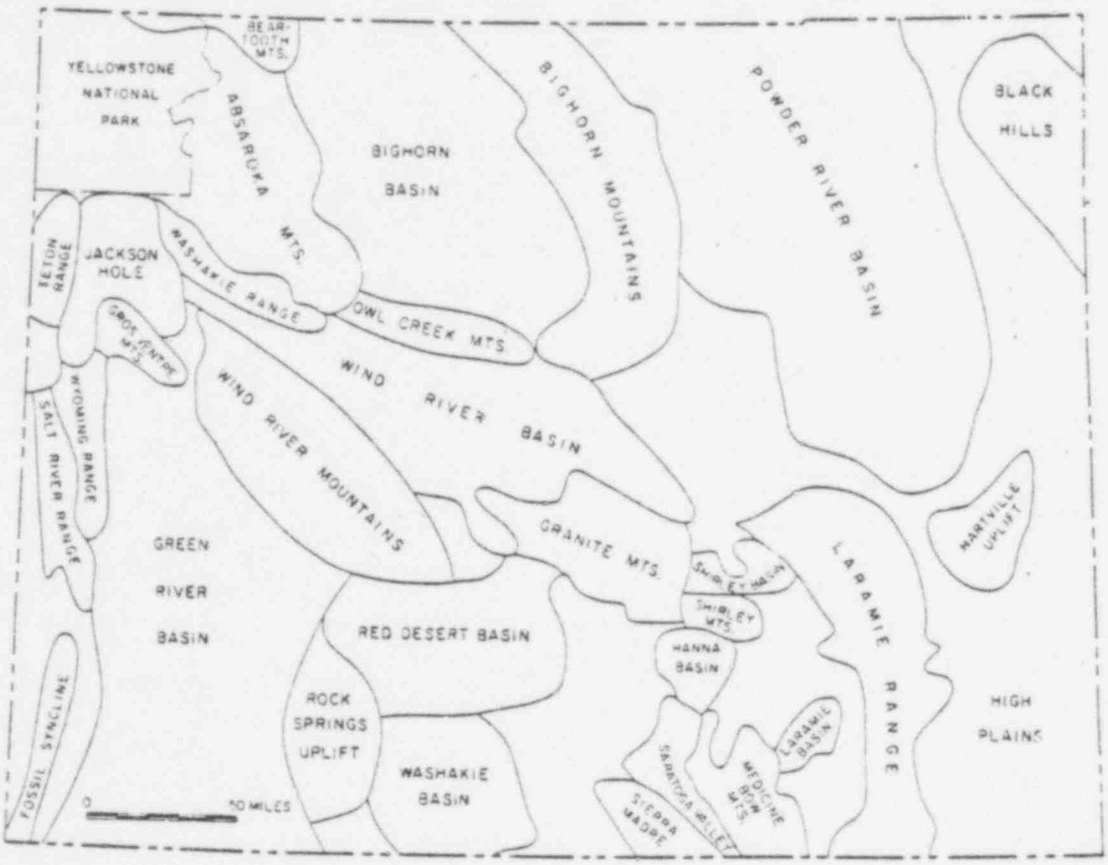

Fig. 35. Wyoming basin (from Thornbury, 1976).

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5. Colorado Plateau

The Colorado Plateau, which includes Grand Junction and the Uravan Mineral Belt, is characterized by deeply incised drainages, sparse vegetation and an arid to semi-arid climate. Precipitation is sporatic causing maximum runoff and erosion. The physiographic province is shown in Figure 36. Glaciation did occur at elevations of about 11,000 feet. Volcanic activity was important among the margins of the region. Seismic activity has been predominant along the western edge of the plateau. The seismic risk (Figure 12) is moderate to moderately high. Winds are generally moderate. The general extreme from Figure 31 is 80 mph. The tornado potential is very low (Figure 15).

Precipitation ranges from less than 10 inches per year in the interior sections to as much as 20 inches plus on the southwest rim. Even in the higher precipitation areas effective moisture is low because of high losses through evaporation, transpiration and infiltration. Temperature extremes are common. Geologically recent climates appear to have been much more hospitable based upon Indian settlements and tree ring studies. Vegetation is generally desert shrubs and grasslands. Most of the plateau soils are alkaline. Sand dunes cover extensive upland areas.

6. Columbia-Snake River Plateaus

The Columbia-Snake River plateau shown in Figure 37 is a great lava plain. The climate ranges from arid to semi-arid and the vegetation is chiefly shrubs (sagebrush) and grasses. The seismic risk potential is moderately high, Figure 12. Winds are moderate, 80 mph extremes in Figure 31, and tornadoes are rare. Temperatures are cool with average annual of about 50°F. Precipitation is less than 10 inches in the west

Fig. 36. Colorado plateau (from Hunt, 1974).

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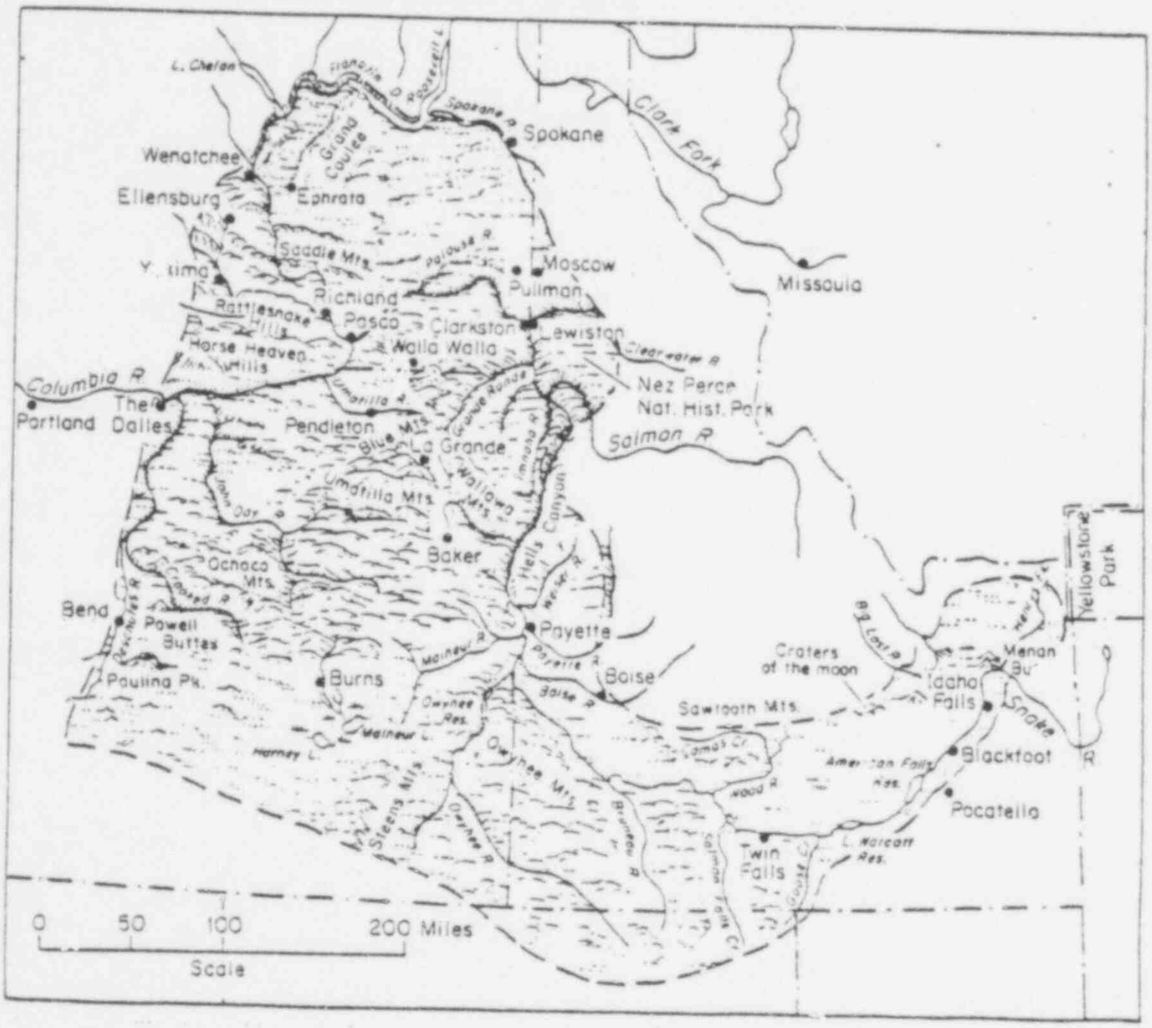

Fig. 37. Columbia-Snake River plateau (from Hunt, 1974).

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due to the rain shadow of the Cascade Mountain Range. Eastward the precipitation increases to as much as 20 inches or more. Over geologic time climates have fluctuated greatly. Soils are generally deep and mostly loessial. Glaciation did not occur in this region but the effects of glacial melt waters are extremely important, e.g. Grand Coulee.

C. Relation of Model Site Conditions to the Six Physiographic Regions

Table 15 summarizes the general character of each physiographic region and the model site in terms of the natural hazard or condition. Analysis of the information presented suggests that in general the model site and the six regions are quite similar. Differences do exist and are discussed below. Table 16 summarizes the comparison with respect to potential failure modes using the model site as a base.

1. West Gulf Coastal Plain

The apparent difference between this region and the model site is the likelihood of receiving extreme precipitation caused by hurricane related storms. Such storms pose the threat of severe erosion or flood damage. Tornadoes also have a higher potential for occurrence and intensities may be higher because of altitude and humidity. Higher annual precipitation potentials may have a beneficial effect in vegetation response. However, the likelihood of drought is reasonably high therefore vegetation performance may be cyclical. The likelihood of earthquake failure is lower than at the model site.

2. Great Plains

Generally conditions are very similar between the model site and the generalized description of the Great Plains.

Table 15

Comparison of Physiographic Regions and Model Site

	Climate	Seismic Risk	Tornado Frequency	Extreme Winds 100 yr mph	Glaciation Potential	Precipitation Intensity
Model Site	semi-arid	moderate	low to moderate	80	low	high
West Gulf Coastal Plain	semi-arid	low	low to moderate	80	low	very high
Great Plains	semi-arid	low to moderate	low to moderate	90	low	high
Southern Rocky Mountains	variable	moderate to moderately high	--	--	high	high
Wyoming Basin	arid to semi-arid	low to moderate	low	90 to 100	low	high
Colorado Plateau	arid to semi-arid	moderate to moderately high	very low	80	low	high
Columbia-Snake River Plateau	semi-arid (cooler)	moderate to moderately high	very low	90	low	moderate

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Table 16

Comparison of Potential Failure Modes to Physiographic Region
(Model Site is Base)

	Earthquakes	Tornadoes	Flash Flooding	Wind Erosion	Water Erosion	Glaciation
West Gulf Coastal Plain	lower	higher	higher	same	higher	same
Great Plains	lower	higher	same	slightly higher	same	same to pot. higher in northeast
Southern Rocky Mountains	higher	lower	higher	same	higher	much higher
Wyoming Basin	lower	lower	same	higher	same	same
Colorado Plateau	slightly higher	lower	same	same	same	same
Columbia-Snake River Plateau	higher	lower	lower	lower	lower	same

3. Southern Rocky Mountains

The great variability in topography and elevation within this province make comparison difficult. Elevation within the region will control climate and precipitation. Common features are the high intensity precipitation and climatic extremes. Seismic and glaciation potential will be higher. Arid climates are not expected and therefore vegetation will be different.

4. Wyoming Basin

The conditions in this region appear to be very similar to the model site. Precipitation can be lower depending on the specific location. Winds appear to be generally higher.

5. Colorado Plateau

The biggest difference is in the potential range of precipitation. Very arid conditions could be encountered in areas of the Colorado Plateau. In these areas wind and water erosion and flash flood failure potentials would be greater than for the model site. Seismic potential appears to be higher. Volcanic activity is a potential along the margins of the region. This is not mentioned for the model site. Tornado potential is very low.

6. Columbia-Snake River Plateau

The similarity between this region and the model site is again great. Dep..ding upon specific location climates could become more arid. Generally the climate and weather is more even and moderate. The potential for flash flooding appears to be reduced. The seismic potential is higher. The volcanism that produced the recent landscape must be consi..red a potential for the future. The tornadao potential is very low.

VIII. SITE VISITS

The site visits described herein were made on January 23-25, 1978. The project team consisted of J. D. Nelson and T. A. Shepherd. The sites included both inactive and active uranium tailings piles located in southwestern Colorado, southeastern Utah, and northwestern New Mexico.

A. New Rifle Site, Rifle, Colorado
Union Carbide Corporation, January 23, 1978

The visit was made with Hubert Miller, NRC, and George Montet, Argonne. The project team was escorted on the visit by Harold Piper, mill manager for the Union Carbide operation, which now processes vanadium concentrate liquor transported from Uravan. The mill produces no tailings at this time.

The impoundment has been inactive since December, 1973, and covers approximately 32 acres. There are two levels, the highest being about 65 feet high and the lower about 35 feet high (Figure S-1). Slopes are steep, at about 1.5:1 to 1:1.

Stabilization has been done with vegetation cover only (Figure S-2). No topsoil has been added. During operations, slopes and any inactive pond surfaces were seeded and watered allowing vegetation to become established before abandonment. Wind erosion and blowing dust have been the biggest problems. The vegetation cover seems to control this very well, but it requires constant maintenance and watering. Water is applied daily in the summer months. The vegetation cover appears to be complete and very healthy.

Mr. Piper said that the most important maintenance required is the immediate repair of any areas of severe wind erosion that may occur. The remedial measures commonly employed are the application of mulch and reseeding. Without constant watering, vegetation would deteriorate quickly. Mr. Piper estimated that if watering were stopped, extensive wind erosion would occur in one season.

Wind is a big problem because of the location. The impoundment is in the middle of an east-west trending portion of the Colorado River valley. Prevailing westerly upslope winds are strong and constant. These winds accelerate as they climb up and over the steep windward slopes of the tailings pile. Mr. Piper said that lower slope angles would reduce the erosive power of the wind. How much this could reduce the problem is a factor that needs to be considered as well as other stabilization measures.

The pile appeared dry and drilling by Ford, Bacon and Davis confirms this (Ford, Bacon and Davis, 1977a). At the present time, vehicles can be driven on the surface with no problem. Some slumping of the slopes was observed, but it did not appear serious. Several areas of severe wind erosion were observed, but revegetation and stabilization appear to have been effectively applied. It was emphasized that timely and thorough maintenance was important in these cases.

Snow fences were used for additional wind erosion control and initial stabilization to allow vegetation to become established. The most important element of stabilization was constant irrigation and maintenance.

- Likely long-term effects--If maintenance and watering is not continued the pile would most likely blow away. There is also the likelihood that because of the proximity to the river, flooding or course changes could erode the pile. This is especially important if long term changes in climate to cooler and wetter conditions should occur.

B. Old Rifle Site, Rifle,Colorado
Union Carbide Corporation, January 23, 1978

The site visit was made with Hubert Miller, NRC, and George Montet, Argonne.

The site became inactive in 1958, and covers about 11 acres. Figure S-3 shows the site, looking east from above. Stabilization was accomplished by covering with 6 inches of topsoil and seeding. It appeared that regrading had been done to slope the surface towards the river. Irrigation is done with sprinklers only during very dry periods. Wind erosion is not a big problem because of the location. The pile is low, approximately 20 feet high, and located on the southeast side of a high bluff. It is, therefore, protected from the prevailing westerly winds. Vegetation appeared to be well established with no evidence of serious erosion. The slopes are shallow and appeared to be stable.

The assay of this pile is sufficiently high that remining is a possibility.

Stability would be threatened by a very large flood of the Colorado River. However, neither a 100-year flood nor a standard project flood would top a railroad embankment which separates the pile from the river (Ford, Bacon and Davis, 1977a).

- Likely long-term effects--Either large flooding or changes in the course of the river could wash the impoundment downstream. If slopes above the pile erode or fail, the pile could be covered with natural materials that are moved downslope.

C. Grand Junction Site, Grand Junction, Colorado
Climax Uranium, January 23, 1978

The site visit was made with Hubert Miller, NRC, and George Montet, Argonne.

The impoundment covers approximately 65 acres (Ford, Bacon and Davis, 1977b) and is about 30 to 40 feet high. It is located directly adjacent to the Colorado River on the north bank. An earth debris dike has been placed along the riverside for protection and stabilization. Slopes of this embankment are steep at about 1.5 to 1.

Approximately 6 inches of silty clay topsoil has been placed and seeded. Success of the vegetation is marginal. The vegetation present is located in a circular area close to existing sprinkler heads. Figure S-4 shows sprinkler system and vegetation. It appeared that only a moderate to low level of maintenance has been provided. Breaks in the water lines exist and local erosion from these was evident. However, no serious erosion or slumping of the slope was observed.

Without a concerted effort of maintenance, watering, and seeding, it is doubtful that the vegetation will ever be very extensive on the pile. Wind erosion was not observed, and it may be that the watering is sufficient to control this.

- Likely long-term effects--Wind erosion could occur or it could be washed away by floods or changes in the river course.

D. Uravan Uranium Mill, Uravan, Colorado
Union Carbide Corporation, January 24, 1978

This is an active uranium and vanadium operation. Ores are received from various mines throughout the Uravan mineral belt. Personnel from Union Carbide Corporation accompanied the project team on the visit.

Two active tailings impoundments are situated about 400 feet above the San Miguel River valley floor directly above the mill and town. The first impoundment is about 120 feet high at the crest and was built in two stages. The first stage was constructed on a slope of 1.5 to 1. The second stage was flattened to a 2:1 slope. Mine rock was added to the second bench (2 to 1) of the first pond to increase stability.

Upstream spigotting from a single outlet is used to distribute tailings. Embankments are dozed up for each successive lift. Decant water is pumped to a holding pond and reused in the wash cycle. The water pool in the tailings pond is kept small to control phreatic surfaces. No apparent major slope stability problems were visible. Some erosion and gullying was apparent on the lower 1:1 slope. Seepage at the toe is captured in a return pond and returned to the plant cycle. Some seepage is directed along the top of a stratum of Mancos shale and is collected at the point where the shale outcrops. The Mancos shale stratum apparently provides an effective impervious barrier for seepage.

The second impoundment is about 75 feet high at the crest. Pond water is decanted back to th[illegible] cycle through an underlying decant line. Seepage is collected in a return pond and returned to the plant cycle. Some erosion on the face due to precipitation is evident.

- Likely long-term effects--Without maintenance, wind and water erosion are likely. Long term slope stability could be a potential problem, and because of the topographic location, a major failure could result in movement into the valley. Instability of the formations forming the valley walls could move the pile along with them into the valley.

E. Slick Rock Site, Slick Rock, Colorado
Union Carbide Corporation, January 24, 1978

The site visit was made with Roger Jones, UCC. The tailings cover about 19 acres (Ford, Bacon and Davis, 1977c).

Stabilization has been effected by placing 6 inches to 1 foot of top soil and seeding. No irrigation is used. Vegetation was well established until unauthorized overgrazing by cattle denuded the area. Erosion resulted. The impoundment has been regraded and reseeding will be done this spring.

The impoundment appears stable. Regrading would have obscured any problems. Vegetation that did exist appears reasonably well established. The relief is low, and water erosion could be expected to be low with maintenance. Drainage around the pile is provided. Figure S-5 shows the features described.

- Likely long-term effects--Wind and water erosion appear to be the only potential problem areas.

F. North Continent Site, Slick Rock, Colorado
Union Carbide Corporation, January 24, 1978

The site visit was made with Roger Jones, UCC. The site is a fan-shaped area immediately adjacent to the Dolores River, about 6 acres in size, approximately 1/2 mile from the Slick Rock Site. This site, shown in Figure S-6 was purchased by the Union Carbide Corporation for purposes of potential remining.

The impoundment was covered with 6 inches to 2 feet of top soil in 1960 and seeded. The vegetation appears fairly well established. No serious erosion of the surface is apparent, and the margin between the river and toe is well vegetated. Very little maintenance is necessary. The impoundment appears to be stable. It blends in well with the surrounding geological features and would not be identifiable as a tailings impoundment without knowledge of its existance.

- Likely long-term effects--Water erosion of surface or removal, either gradually or suddenly, in the event of a major flood on the river.

G. Monticello Site, Monticello, Utah
January 24, 1978

Observations were limited due to two feet of new snow that covered the site. The topographic location and relief of the piles suggested that water erosion would be the only major problem. Some vegetation was visible through the snow and appeared to be about two feet high. Discussions with Department of Energy personnel indicate that unauthorized grazing could be problematical.

H. Shiprock Site, Shiprock, New Mexico
January 25, 1978

Kerr McGee is the previous owner. The property is now maintained by the Navajo Engineering and Construction Agency, a training school for Navajo equipment operators. Personnel from NECA escorted the project team on a tour of the site and surrounding area.

The entire site has been disturbed, decontaminated, and stabilized by NECA as part of their training operation as shown in Figure S-7. Blowing of dust was the major problem in the past. The pile itself has been stabilized against wind erosion by covering with local rock and soil and by watering. Slight erosion or gullying can be seen on covered slopes, Figure S-8. The tailings material was initially watered and allowed to dry to form a fairly strong, resistant crust. Rock and soil were then placed on top. This has effectively controlled blowing.

All contaminated neighboring areas have been excavated. The excavated material was piled, covered, and stabilized in the same manner as the tailings. Around the mill site and raffinate ponds, up to 30 feet of material had to be removed. Contamination was usually found down to the Mancos shale bedrock. Contamination of the shale usually extended to a depth of only about one foot.

Wind blown contamination was detected to a distance of 200 to 300 yards east of the operation boundaries. Topsoil to a depth of only a foot or two was removed to effect decontamination.

Seepage from the pile, Figure S-9, was evident along a shale stratum (probably the Mancos shale). This seepage is radioactive and was captured. During summer months the total seepage evaporates and does not collect.

No vegetation of the surface has been attempted. Some erosion of slopes was evident but was not severe in any case.

- Likely long-term effects--Wind erosion appears to be the only potential problem area.

I. Durango Site, Durango, Colorado
January 25, 1978

The impoundment at Durango consists of a tailings pile about 230 feet high and another lower pile nearby. Some attempt at revegetation has been made. However, at the time of the site visit the site was covered with snow and these efforts were not visible. On the lower pile some material had been removed previously for reprocessing and erosional problems may exist in that area.

On the higher pile the windward side appears to be curved, indicative of wind erosion. If in fact that contour is the result of wind erosion it would appear that whereas local erosion and gullying is not severe, overall erosion of the entire impoundment is appreciable.

Both impoundments are located immediately adjacent to the river and in the town of Durango. A flood of major proportions could cause severe erosion of the toe and potential instability.

- Likely long-term effects--Wind erosion of the entire impoundment is likely. Major flooding may cause potential instability.

J. General Observations

Wind erosion and transport of tailings off the sites by wind appear to be a major problem. Vegetation, sprinkling, rock or soil overburden cover all seem to be effective control measures, employed either separately or together. However, continued maintenance seems to be required in all cases.

In order to be maintenance-free the cover would have to be carefully designed to take into account topographic location, relief, climate and characteristics of the pile.

Slope stability did not appear to be a problem on the sites visited. It is likely that steep slopes will degrade over time. This may not be a severe stability problem in terms of mass movement, but it could remove cover materials and expose tailings. The topographic location or relief with respect to wind erosion potential would play a role in determining the severity of slope stability problems. Long-term slope stability will also be a function of the stability of the underlying geologic materials and geomorphic processes.

Fig. S-1. New Rifle Site - Looking west, shows general slope configuration of upper and lower tailings impoundment.

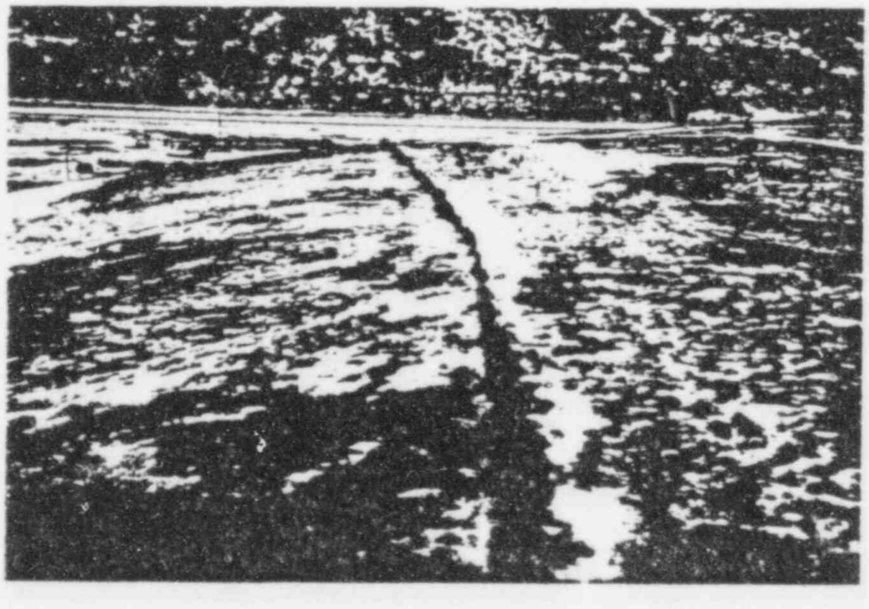

Fig. S-2. New Rifle Site - Looking down on lower tailings impoundment, shows vegetation cover, snow fence for wind control, and small gully on the embankment (left center).

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Fig. S-3. Old Rifle Site - Looking east from above.

Fig. S-4. Grand Junction Site - Surface, shows sprinkler system and sparse vegetation.

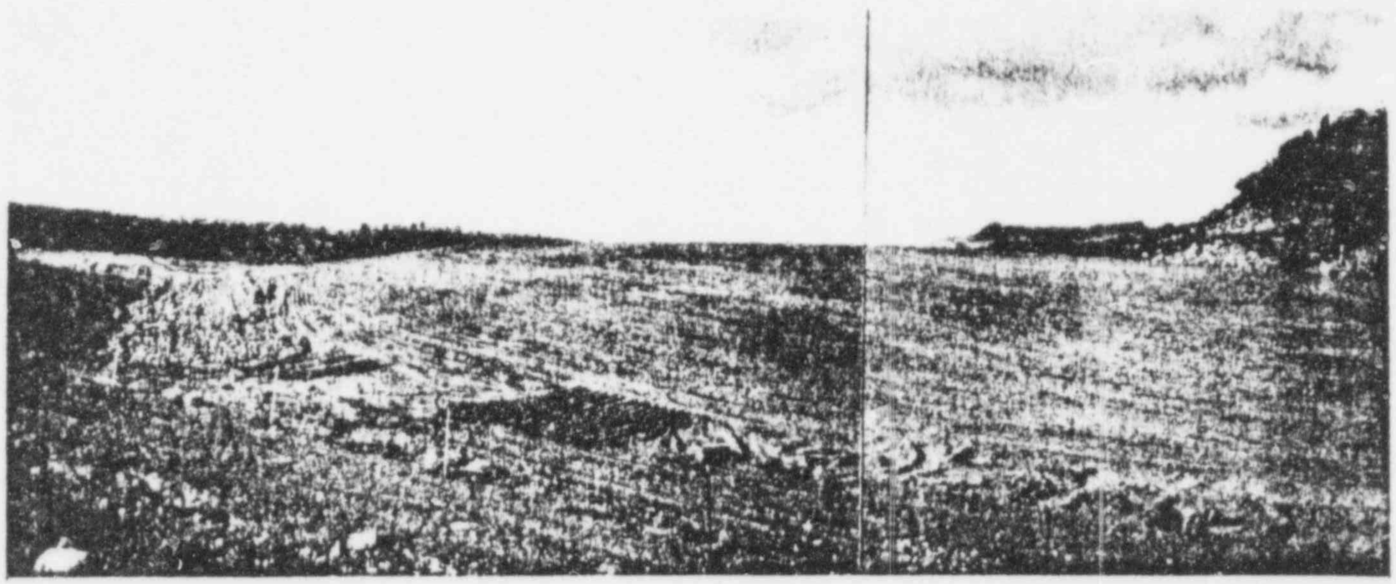

Fig. S-5. Slide Rock Site - Looking north, shows regraded surface and rock protected drainage at the margin (left center).

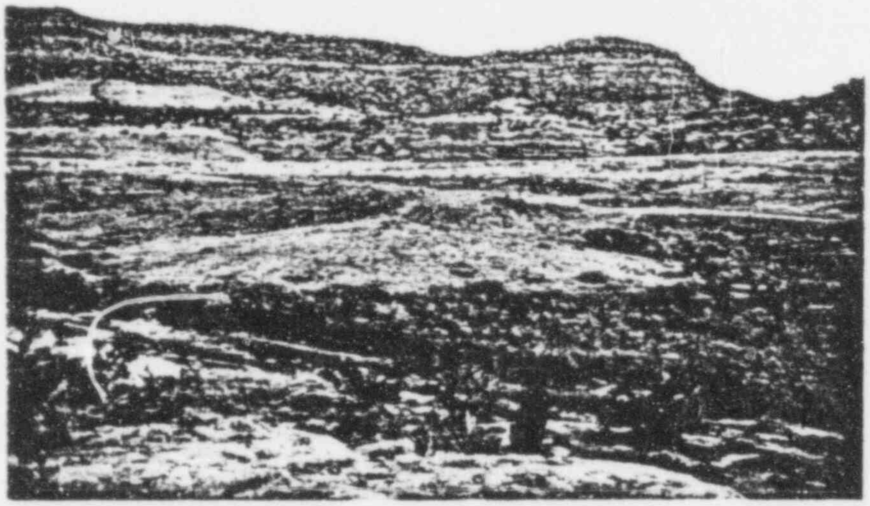

Fig. S-6. North Continents Site - Looking south across Dolores River, the fan shaped delta just above the river is the tailings pile (center).

Fig. S-7. Shiprock Site - Looking east across covered and graded surface of plant and tailings site.

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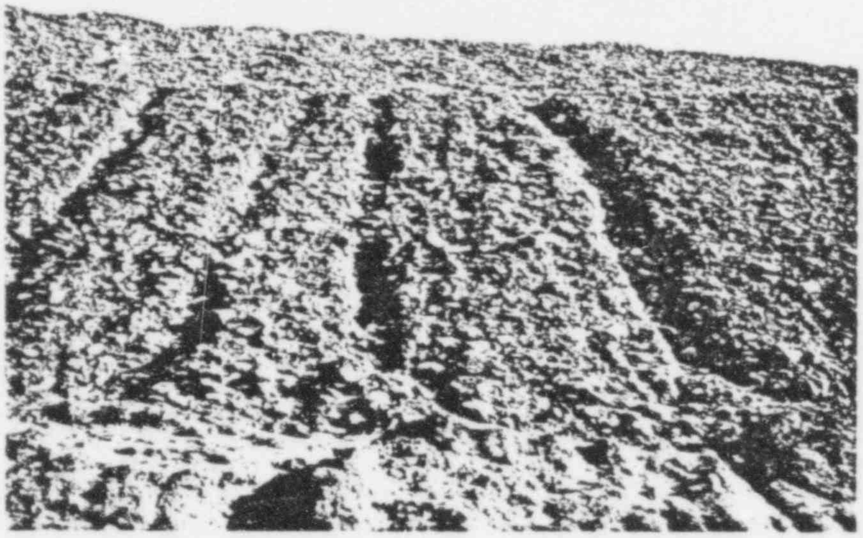

Fig. S-8. Shiprock Site - Small gully formation in cover on slope of tailings impoundment.

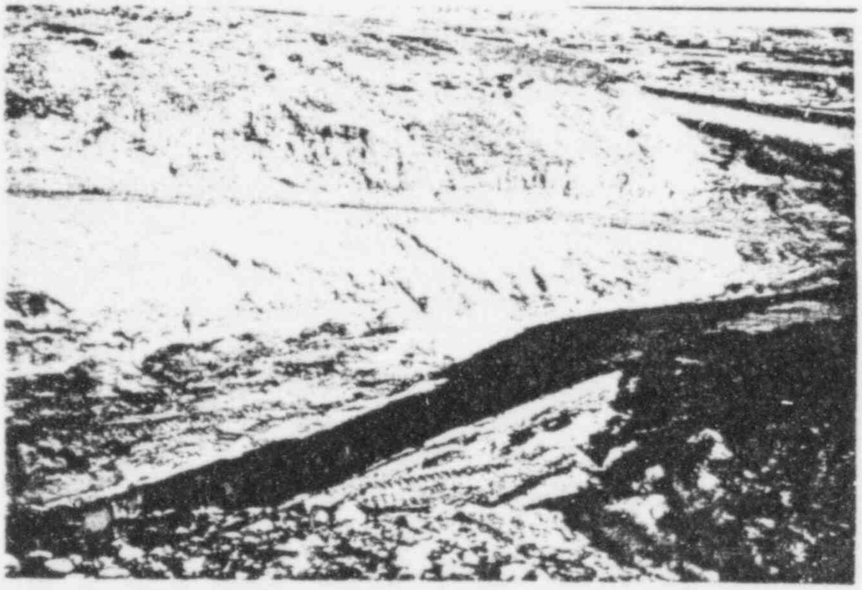

Fig. S-9. Shiprock Site - Seepage from beneath the tailings impoundment channelled on top of shale layer and collected in cut.

IX. SUMMARY

The severity values presented in Table 14 reflect a percentage of likely radiation release based on the nature of release caused by the particular failure mode. Whereas they are useful in comparing effects of different time periods or disposal alternatives, caution must be exercised in using those values to compare failure mechanisms. For example, failure of the cap results in a release of radon whereas failure of the liner results in a release of dissolved radionuclides. No attempt has been made to compare relative magnitudes of the different modes of release.

A. Natural Phenomena

On the basis of the likelihood of failure, the nature of the damage that may occur, interactions with other failure modes and the likelihood that the failure mechanism would be accentuated without maintenance, some generalizations regarding the long-term severity of various failure mechanisms can be made.

Of the natural phenomena that may cause failures, flooding appears to be the most severe. Over extremely long time periods, floods in excess of even the probable maximum flood are possible. The interaction of damage from several floods, even of varying magnitudes, can accentuate the possibility of distress. These views are substantiated to a large degree by the pronounced influence that floods and water erosion have on geomorphological processes.

Erosion by surface water runoff and wind may also have pronounced effects. The Universal Soil Loss Equation and the Wind Soil Loss Equation did not predict large amounts of soil loss in the cases considered. However, gullying and blowouts can result in large amounts of localized erosion. Unfortunately, there are no predictive models that apply to gullying and blowouts.

Surface erosion can be controlled by vegetation or non-erodible cover. In arid and semiarid climates vegetation is sparse and subject to periods of failure (e.g. droughts). However, the placement of coarse material on the surface appears to be effective in reducing erosion and requires little maintenance. This has been used successfully at Shiprock, New Mexico. Also the formation of "pavements" or armoring on desert terraces over several thousand years indicates the effectiveness of those methods of stabilization.

Earthquakes can cause damage during short long-term periods due to liquefaction. However, the use of clay materials in the embankment or alternatives that do not employ an embankment reduce the possibility of liquefaction. From this standpoint as well as from the standpoint of controlling seepage it would be desirable to dry the tailings or drain the impoundment as soon after abandonment as is feasible. Drainage may be possible by means of underdrains that lead to points where the seepage can be evaporated.

Over medium and long long-term periods the tailings will become unsaturated and liquefaction would not be of much concern. In those time periods, failure due to earthquakes would consist primarily of cracking of caps and liners and potential instability of the embankment. Magnitudes of radiation release would not be great.

Windstorms and tornadoes have the ability to cause minor erosion amounts for short periods of time. They did not appear to be of serious concern with regard to the long term stability of tailing impoundments.

The occurrence of glaciers in areas where uranium is currently being mined is remote. However, if future uranium mining is conducted in northern or mountainous areas, glaciation could affect a tailing impoundment over long long-term periods. The effect of a glacier would be to completely remove any portion of the impoundment that it contacts. However, release of the radioactive material may not occur until the glacier recedes and considerable dispersion of the radioactivity would occur.

Climate changes are to be expected over medium and long long-term periods. The general opinion at present is that the climate will get warmer for the next several thousand years after which a cooling period is expected.

Changes in climate may influence vegetation schemes that are employed to provide stability. Because the present climates in the uranium mining areas are generally arid or semiarid, species of vegetation that do not require much water would be introduced. Climatic changes to cooler and wetter climates would increase the vegetative cover. Nevertheless, vegetative cover requires fairly extensive maintenance during early periods and is susceptible to even short term (a few years) climate changes. For that reason it is considered to be less reliable than such schemes as rock or riprap.

B. Impoundment Elements

Some impoundment alternatives involved the use of a cover or cap on the tailings. The purpose of that cap is primarily to reduce radon emanation to acceptable values. The previous discussion and applications of predictive models indicate that whereas stabilization schemes may provide for a cap that is stable through the medium long-term period (except if major flooding occurs), it is doubtful that any cap would be effective for long long-term periods (100,000 years). Over short long-term periods maintenance and remedial measures on the cap are not difficult.

Liners were employed in some alternatives to control seepage. In general, it must be assumed that some seepage will occur through liners. Liners constructed of soils tend to be self-healing and less susceptible to differential movements. Synthetic liners will contain some imperfections and potential punctures. The susceptibility of liners (soil or synthetic) to chemical attack is not known for even short long-term periods. The extent to which radionuclides are "fixed" in soil and their mobility in clays is an area in which considerable research is needed.

The concept of "zero seepage" must be thought of in terms of "minimal seepage." It appears that from a seepage standpoint, placement on "impervious" shale is most desirable. On the other hand, drainage of tailings (which is desirable from a strength and liquefaction standpoint) would minimize the dependence on a liner for seepage control.

The embankment of an impoundment is perhaps the most predictable element. Past experience of the engineering community with a large number of water retention dams has provided a good background of knowledge. With good design and appropriate factors of safety, stability and earthquake resistance can be optimized. The main disadvantage of impoundments that are placed above ground, and hence, utilize embankments is that they are more exposed to erosive elements such as wind.

C. General Considerations of Alternatives

1. Base Case

The base case represents a conventional tailing disposal impoundment with no control measures over radon emanation or seepage. It is expected that within the short long-term period most of the water in the tailings will have seeped into the underlying soil. Precipitation will continue to leach radioactive material from the tailings over long time periods. Blowing of tailings will continue to be a problem for long periods. Vegetative cover will require long time periods to be established. Over long long-term periods it is expected that the entire impoundment will have been dispered.

2. Alternative 1

The cap will be effective in reducing the blowing of tailings or radon emanation over medium long-term periods. Similarly, the liner will control seepage. Over medium long-term periods it is expected that all the free water in the tailings will have seeped out. However, if the cap remains intact it will provide protection against continued leaching by precipitation. Over long long-term periods it is expected that the entire impoundment may be dispersed.

3. Alternative 2

Placement of the tailings in the open pit has the advantage of protection from wind and water erosion. It has the disadvantage of potential interaction with the groundwater. Differential settlement is of little concern.

The method of fixing the tailings, however, should be well designed. Portland cement will react with sulfates or may be unstable in a highly alkaline environment. It may, therefore, be effective for only a few years after placement. Fixing may also hinder future remining efforts. Over long long-term periods the impoundment could still remain.

4. Alternative 3

The dry tailings have the advantage of reducing the seepage problem. Drainage should be provided to carry water away from the impoundment so that groundwater recharge will not drain through the tailings. Interaction with the groundwater is to be expected because the liner will not be totally impervious. However, distress to the liner by differential settlement or earthquakes will be smaller than for an above ground impoundment. The impoundment may remain over long long-term periods.

5. Alternative 4

The considerations in this alternative are much the same as for Alternative 3. However, seepage from the untreated tailings into the groundwater will occur. The degree to which radioactive material may be fixed in the overburden placed below the tailings is not known.

6. Alternative 5

This alternative has the advantage of good seepage control for very long periods of time. The potential for chemical weathering of the natural shale must be considered. Extensive subsurface investigation should be conducted to ensure the continuity of the shale. This alternative is also expected to remain in place for even long long-term periods.

7. Alternative 6

This alternative has the advantage of being able to closely control the placement of different fractions of the tailings because of the small lateral dimension of the impoundment. However, the complex geometry makes it more susceptible to differential movement due to earthquakes or settlement. The large area of the liner increases the potential for failure. The general considerations of other factors would be similar to Alternative 4.

8. Alternative 7

Similar to deposition of fixed tailings in the open pit (Alternative 2) the method of fixing the tailings may not be stable for long periods. Over long-term periods collapse of the mine is very likely and interaction with groundwater aquifers is almost certain. If problems develop with regard to the impoundment the tailings would be inaccessible and remedial measures would be difficult. Remining is also complicated.

9. Alternative 8

Because large differential movement around the mine cavity is to be expected the liner would be very ineffective against seepage into or out of the tailings. This alternative therefore has a high likelihood of contaminating the groundwater. Problems caused by the inaccessibility of the tailings are similar to those for Alternative 7.

D. Monitoring Schemes

Monitoring schemes exist that can be utilized to observe radiation and groundwater quality. Also instrumentation is available to monitor slope movements and settlement. These schemes, however require that readings be taken and the results be analyzed periodically. Consequently, maintenance and involvement of personnel would be required for long term periods.

For most areas of distress, the beginning of failure or damage can be observed directly by ground personnel or aerial photography. Photogrammetry can provide some measurements.

A particularly powerful tool appears to be the use of infrared aerial photography. The effects of radiation release on the vegetation and the surrounding environment of a nature that could be detected by infrared photography needs to be investigated. It is believed that such remote sensing techniques could provide a means of rapid assessment of radiation effects from a variety of potential failure modes. Further research in that area is needed.

X. CONCLUSIONS

An investigation was conducted of the long-term stability of uranium mill tailings disposal alternatives. Three long-term periods were considered. These were the short long-term, medium long-term and long long-term periods. Within short long-term periods natural processes will have a small effect and failure that might occur would be design related. Thus, for short long-term periods the engineering design of the site will govern its performance. For long long-term periods natural geomorphological processes would predominate. The medium long-term period represents a transition from the engineering dominance to the geomorphological dominance.

A. FAILURE MECHANISMS

The hazard imposed by the presence of a tailings impoundment depends on the nature of a particular failure that may occur. The worst situation would be a failure that results in dispersion of radioactive tailings over a wide area. Failure of impoundment elements that result in a slow or localized release of radioactive material are less severe.

From that standpoint, floods appear to be the greatest potential cause of severe failure. A flood larger than the design flood is almost certain to occur over long periods and, in fact, one could occur at anytime. The maximum "possible" flood is almost impossible to predict. Also, design measures to prevent flood damage will be unreliable over long-time periods. Furthermore, if a major flood washes out an impoundment it will disperse tailings over a very large area.

Wind and water erosion of embankments and caps can also disperse tailings over relatively large areas. Stabilization of surfaces can utilize either vegetation or the placement of cobbles and rocks. Vegetation may be susceptible to failure due to drought, fire or pestilence, and it may require considerable maintenance. Stabilization of the surface by armoring with rocks may be stable for several thousand years and requires almost no maintenance.

Earthquakes may cause dispersion of tailings if liquefaction of the tailings can occur. However, seepage through any liner is certain to occur. Consequently, it may be assumed that within a medium long-term period all free water in the tailings will have seeped out unless recharge is occurring. Failure due to earthquakes would probably be less severe after short long-term periods have passed.

Thus, failure, to some degree, of all impoundment elements (i.e., cap, liner and embankment) is highly probable. Failure due to natural phenomena would be severe if caused by floods, earthquakes within short-term periods, or dispersion by wind or water erosion. Failures due to other natural phenomena are less severe.

B. DISPOSAL ALTERNATIVES

The tailing disposal alternatives that were considered can be grouped into above grade disposal, open pit disposal (below grade) and deep mine disposal. The above grade disposal alternatives were the most susceptible to failure due to floods, earthquakes or erosion and offered the greatest opportunity for tailing dispersion if failure did occur.

Disposal in open pits below grade provided a greater opportunity for protection from floods or erosion and much smaller susceptibility to dispersion of tailings due to earthquakes. Placement of tailings in deep mines allowed very small chance of dispersion due to floods or earthquakes. However, differential rock movement is almost certain to occur making linings very ineffective in deep mines. Consequently, full release of soluble radionuclides into the aquifers is almost certain. In addition, remining of tailings placed back in deep mines would be very difficult.

In view of the above considerations it was concluded that placement of tailings in open pits is the most desirable alternative. Placement of tailings in deep mines offers less susceptibility to tailings dispersion but does allow dispersion of radionuclides through groundwater. Consequently, disposal of tailings above grade with proper siting and stabilization techniques would be preferable to placement of tailings in deep mines.

The above conclusions do not take into account economic considerations or ramifications of a failure. Such considerations, along with specific site conditions, could change the order of preference at a particular site.

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Appendix A

SUMMARY OF THE METHODOLOGY EMPLOYED TO EVALUATE DISPOSAL PLAN ALTERNATIVES

Appendix A

SUMMARY OF THE METHODOLOGY EMPLOYED TO EVALUATE DISPOSAL PLAN ALTERNATIVES

The methodology was developed to allow comparative evaluation of alternative uranium tailings disposal schemes on the basis of a weighted score of all potential modes of failure. The weighted score was quantified as the product of the likelihood that a particular failure mode would occur, L_i, the relative magnitude of release caused by the failure, M_i, and a negative utility factor, U_i, that described the degree to which such a failure would be undesirable.

Thus, the severity, S_i, of a failure mode was defined as

$$S_i = L_i M_i U_i$$

These individual elements and the rationale behind the method of quantification are discussed in detail in the preceding report. These elements are presented briefly below along with examples of the application of this methodology.

<u>Likelihood of Failure</u> (L_i)

The likelihood of failure is an estimate of the probability that failure will occur. Likelihood is chosen rather than probability because in most cases the value will not be based upon a sufficiently large statistical data base. The value of likelihood could range from 0 to 10. A value of 10 represented certainty of occurrence and 0 represents no chance of occurrence. Values of 1, 3, 5, 7 or 9 were assigned between those extreme values. An interval of 2 was maintained between numbers so as not to imply any closer confidence limit on the determination (i.e., a likelihood of 3 <u>implies</u> a probability of occurrence somewhere between 0.20 and 0.40). In many cases the determination of likelihood was subjective and the level of confidence implied in the spacing between values that were used may not be accurate.

Magnitude of Failure (M_i)

This value describes the expected amount of radioactive material that could released if a particular mode of failure should occur. A value for magnitude of failure was assigned for each potential mode of failure.

Four discrete modes of release were described for purposes of this evaluation. It was considered that radioactive material or gamma-ray emission could be released in the form of radon emanation, generally through the exposed or covered surface of the impoundment, dissolved radionuclides escaping in seepage from the impoundment, undissolved radionuclides escaping by physical transport, (i.e., mass movement, wind-blown dust or flood transport) and a reduction in gamma-ray attenuation. Although these four modes of release are not independent of each other they were considered separately for quantification of release.

For each mode, the magnitude was determined as a percentage of the maximum amount of radioactivity (or reduced gamma-ray attenuation) that could be released via that mode. So as not to imply a confidence level of greater accuracy than exists, only values of 0, 1, 3, 5, 7, 9 and 10 were assigned. The magnitude was then presented as a matrix as shown in Fig. A-1.

$$M_i = \begin{bmatrix} w \\ x \\ y \\ z \end{bmatrix}$$

w = radon emanation
x = dissolved radionuclide release
y = undissolved radionuclide release
z = reduction in gamma-ray emission attenuation

Fig. A-1. Magnitude of Release

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Negative Utility Factor (U_i)

The negative utility is a weighting factor which considers the extent of the problem posed by a particular failure mode. This is introduced because the manner in which radioactive material is released the nature of the hazard imposed and control problems differ for each potential mode of failure. The negative utility factor varies from 0 to 2 and describes the degree of undesirability of occurrence of a failure and the difficulty in mitigating the hazard. The negative utility factors assigned for each failure mode are listed in Table 13. The values so assigned are subjective.

Example of Application

Two examples of the application of the methodology are presented below. The first is a wind erosion failure of the cap for the medium long-term period for alternative disposal plans 1 through 6. The second is an earthquake failure of the entire impoundment for the short long-term period for alternative 1.

Wind Erosion of Cap (A1d) (Medium Long-Term Period, Alternatives 1-6)

Because wind erosion is certain to occur the likelihood of wind erosion was assigned a value of 10. For the assumed cap characteristics and wind speeds it was computed that within the medium long-term period less than five inches of cap would be removed. As discussed in the text of the report this would result in a negligible increase of radon emanation and gamma-ray emission. Therefore, a minimal magnitude of release of 1 was set for both. This failure mode would not influence the release of dissolved or undissolved radionuclides. Therefore, the potential release of these modes is zero. The magnitude of release, therefore, is $\begin{bmatrix} 1 \\ 0 \\ 0 \\ 1 \end{bmatrix}$.

Because dispersion of material is small and remedial measures are possible the negative utility factor was assigned a value of 0.5.

The severity of failure is therefore

$$S_{(A1d)} = 10 \times \begin{bmatrix} 1 \\ 0 \\ 0 \\ 1 \end{bmatrix} \times (0.5) = \begin{bmatrix} 5 \\ 0 \\ 0 \\ 5 \end{bmatrix}$$

Earthquake Failure (B1) (Short Long-Term Period, Alternative 1)

Even if an earthquake larger than the design magnitude should occur, it is not likely that failure of an extent to release tailings would occur. The likelihood of failure of the embankment and liquefaction of tailings was considered to be 1. If that does occur, it was assumed that 70 percent of the tailings would liquefy and be dispersed. The release would be in the form of all four modes of release. The dispersion of 70 percent of the tailings would provide a release of each element in the amount $\begin{bmatrix} 7 \\ 9 \\ 7 \\ 7 \end{bmatrix}$.

Because clean up and remedial measures would be extremely difficult a negative utility of 2.0 was assigned. The severity was therefore computed to be

$$S_{(B1)} = 1 \times \begin{bmatrix} 7 \\ 9 \\ 7 \\ 7 \end{bmatrix} \times 2.0 = \begin{bmatrix} 14 \\ 18 \\ 14 \\ 14 \end{bmatrix}.$$

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