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Stability of Multilayer Earthen Barriers Used to Isolate Mill Tailings: Geologic and Geotechnological Considerations

J. T. Zellmer

August 1981

**Prepared for the U.S. Department of Energy
under Contract DE-AC06-76RLO 1830**

**Pacific Northwest Laboratory
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TOE SOILS BEARING CAPACITY
CONSIDERATIONS

U. S. LEIMER

August 1967

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SUMMARY

The stability of multilayer earthen barriers used to isolate uranium mill tailings depends on the morphology of the disposal site, the engineering of the barrier, the condition of the tailings, and the possible impacts of earthquakes, erosion, and climactic changes. When designing a cover for or siting a tailings pile, one must take into account both geologic and geotechnological variables.

GEOLOGIC FACTORS

Seismicity and erosion are the two major causes of instability in tailings-pile covers. Earthquakes can lead to ground ruptures and settlement and liquefaction of the tailings, and erosion can remove the cover material and cause the offsite migration of the radioactive tailings.

To alleviate the adverse effects of possible seismic activity, tailings piles should never be located on or near active or capable faults. Existing piles near faults should be moved to safer sites or engineered to withstand possible displacement and shaking. Generally, tailings piles in geomorphically stable areas will require less protection than piles in unstable areas.

Liquefaction generally can be prevented if the tailings and their underlying material are compacted to a relative density of 60% or greater, or if they are kept dry. If the tailings are saturated, dewatering schemes may have to be used.

Erosion may be caused by streams, glaciers, or wind, depending on the geomorphic and atmospheric conditions at the site. Fluvial erosion can be prevented by using dikes (and avoided by initially siting the pile a safe distance away from stream courses). In some cases, fluvial waters or rainfall may have to be rerouted over the pile via armored (i.e., water repellant) ditches. Eolian erosion can be minimized by vegetating the disposal site or covering it with gravel. Glacial erosion probably will not pose a significant danger for at least the next few thousand years.

GEOTECHNOLOGICAL FACTORS

Detailed geotechnological data are not available for most sites, making it almost impossible to predict the response of a tailings pile to the load imposed by the earthen cover or to the effects of time and natural phenomena. The longevity and integrity of a cover system are tied to 1) the stability of a pile's slopes; 2) the ability of the pile to recharge the clay layer with water without saturating the tailings; 3) the protection the pile has from erosion; and 4) the possible effects of differential settlement.

Because of the wide geomorphic and geotechnological variations extant at most sites, a single cover design cannot be cost-effectively and efficiently used at all sites.

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INTRODUCTION

This report briefly discusses how seismic activity, erosion, climatic change, slope stability, differential settlement, and cover design could affect the long-term integrity of multilayer earthen cover systems. In addition, the report suggests ways to design and construct covers so that adverse impacts can be avoided or minimized.

When designing a cover for isolating uranium mill tailings, one must consider both the geomorphic and geotechnological characteristics of the tailings piles and disposal site. If detailed geotechnological data are available, it is possible to determine how a tailings pile will respond to the loads imposed by the cover or to the effects of time and natural phenomena. Tailings piles respond differently to natural and man-induced stresses, depending on any vertical or lateral variations in the underlying structure. Unfortunately, many of the geotechnological data required for analysis are not available.

Until the geotechnological aspects of individual tailings piles are understood, few definitive statements about the long-term stability of covers or piles can be made. Nonetheless, we have reason to believe that the Grand Junction tailings piles will remain relatively stable unless saturation or undercutting or slope steepening occurs. The tailings pile slopes are generally less than about 17% and apparently are well-drained except where zones of slimes are present. An analysis of possible differential settlement at the site suggests that the cover could tolerate greater deflections than are likely to occur. Furthermore, if the moisture content of the cover increases within certain limits, the cover could probably tolerate even greater deflections without failure. The cover designs discussed below are engineered to prevent moisture removal by vegetation and evaporation but still provide for recharge by precipitation.

The preliminary analyses discussed in this report indicate that both geomorphic and geotechnological considerations are important in assessing the long-term stability and integrity of cover systems for uranium mill tailings. A single design should not be used for all locations; rather, cover systems

should be tailored to individual sites. Because of the geomorphic and geotechnological variations expected at the individual sites, a single design could prove to be overly conservative and unnecessarily expensive for some locations.

GEOLOGIC CONSIDERATIONS FOR LONG-TERM STORAGE OF URANIUM MILL TAILINGS

This section briefly discusses several geologic factors that should be considered in developing methods to isolate, store, or dispose of uranium mill tailings. The list of topics is not exhaustive and is intended only to draw attention to the importance of considering these and other geologic factors when assessing the applicability of various sites and designs.

EARTHQUAKE EFFECTS

The United States is divided into four seismic risk zones (Figure 1). The zones, identified as 0, 1, 2, and 3, indicate the severity of the damage that may occur during an earthquake. Areas are assigned a specific risk level based on the following criteria: the distribution of and Modified Mercalli intensity of historic earthquakes; evidence of tectonic strain release; and consideration of geologic structures and provinces believed associated with seismic activity. Seismic-energy releases are generally reported in terms of either Modified Mercalli intensity (Table 1) or earthquake magnitude. The relationship between these values is shown in Figure 2. When comparing earthquakes, the distance from the causative fault must also be considered, because seismic energy attenuates with distance. Figure 3 shows the effect that distance from the epicenter has on the reported intensity of shaking for various earthquake magnitudes.

The uranium mill tailings sites of interest to this study are shown superimposed upon the seismic risk map in Figure 1. Although most of these locations lie in zones of no damage or minor damage (Zones 0 and 1), several are in or adjacent to zones of moderate to major damage (Zones 2 and 3). Because of their locations, some of the sites may be affected by seismic activity. This section discusses the effects that earthquakes may have on proposed radon containment systems and methods of decreasing possible damage to these systems.

Ground Rupture

Ground rupturing associated with an earthquake could cause the containment system to fail, depending on the amount of fault displacement and the type, design, and condition of the containment system, plus other factors.

FIGURE 1. Seismic Risk Map of the United States Showing Locations of Inactive Uranium Mill Tailings Piles (Adapted from Algermissen 1969)

TABLE 1. Modified Mercalli Intensity Scale (after Richter 1958)

Scale	Physical Response
I.	Not felt. Marginal and long-period effects of large earthquakes.
II.	Felt by persons at rest, on upper floors, or favorably placed.
III.	Felt indoors. Hanging objects swing. Vibration like passing of light trucks. Duration estimated. May not be recognized as an earthquake.
IV.	Hanging objects swing. Vibration like passing of heavy trucks, or sensation of a jolt like a heavy ball striking the walls. Standing motor cars rock. Windows, dishes, doors rattle. Glasses clink. Crockery clashes. In the upper range of IV, wooden walls and frame creak.
V.	Felt outdoors, direction estimated. Sleepers wakened. Liquids disturbed, some spilled. Small unstable objects displaced or upset. Doors swing, close, open. Shutters, pictures move. Pendulum clocks stop, start, change rate.
VI.	Felt by all. Many frightened and run outdoors. Persons walk unsteadily. Windows, dishes, glassware broken. Knickknacks and books fall off shelves, pictures fall off walls. Furniture moved or overturned. Weak plaster and masonry D ^(a) cracked. Small bells ring (church, school). Trees, bushes visibly shaken or heard to rustle.
VII.	Difficult to stand. Noticed by drivers of motor cars. Hanging objects quiver. Furniture broken. Damage to Masonry D, including cracks. Weak chimneys broken at roof line. Fall of plaster, loose bricks, stones, tiles, cornices, unbraced parapets, and architectural ornaments. Some cracks in Masonry C. Waves on ponds, water turbid with mud. Small slides and caving in along sand or gravel banks. Large bells ring. Concrete irrigation ditches damaged.
VIII.	Steering of motor cars affected. Damage to Masonry C, partial collapse. Some damage to Masonry B, none to Masonry A. Fall of stucco and some masonry walls. Twisting, fall of chimneys, factory stacks, monuments, towers, elevated tanks. Frame houses moved on foundations if not bolted down, loose panel walls thrown out. Decayed piling broken off. Branches broken from trees. Changes in flow or temperature of springs and wells. Cracks in wet ground and on steep slopes.
IX.	General panic. Masonry D destroyed, Masonary C heavily damaged, sometimes with complete collapse, Masonry B seriously damaged. General damage to foundations. Frame structures, if not bolted, shifted off foundations. Frames cracked. Serious damage to reservoirs. Underground pipes broken. Conspicuous cracks in ground. In alluviated areas, sand and mud ejected, earthquake foundations, sand craters.

TABLE 1. (contd)

Scale	Physical Response
X.	Most masonry and frame structures destroyed with their foundations. Some well-built wooden structures and bridges destroyed. Serious damage to dams, dikes, embankments. Large landslides. Water thrown on banks of canals, rivers, and lakes. Sand and mud shifted horizontally on beaches and flat land. Rails bent slightly.
XI.	Rails bent greatly. Underground pipelines completely out of service.
XII.	Damage nearly total. Large rock masses displaced. Lines of sight and level distorted. Objects thrown into the air.
<hr/>	
(a)	To avoid ambiguity of language, the quality of masonry or brick is specified by the following lettering.
Masonry A.	Good workmanship, mortar, and design; reinforced, especially laterally, and bound together by steel or concrete; designed to resist lateral forces.
Masonry B.	Good workmanship and mortar; reinforced, but not designed in detail to resist lateral forces.
Masonry C.	Ordinary workmanship and mortar; no extreme weaknesses like failing to tie in at corners, but neither reinforced nor designed against horizontal forces.
Masonry D.	Weak materials such as adobe; poor mortar; low standards or workmanship; weak horizontally.

The probable displacement that an earthquake could create at a specific site can be estimated using available information on the site's seismic risk zone (Figure 1), on what we know about the relationship between earthquake magnitude and intensity (Figure 2) and epicentral distance (Figure 3), and on records of maximum fault displacement for historic earthquakes of a given magnitude (Figure 4). For example, earthquakes in Seismic Zone 3 have Modified Mercalli intensities of VII or greater, which correspond to earthquakes with magnitudes of at least 5 occurring at the site. Historically, earthquakes of this magnitude in western North America have had vertical displacements ranging between about 2 and 50 cm (Figure 4). Because the magnitude of future earthquakes cannot be confidently predicted, covers for tailings piles located on or near active faults may have to be designed to withstand the full effects of

FIGURE 2. Relationship Between Earthquake Magnitude and Intensity (Modified Mercalli Scale) in Western United States (Modified from Krinitzsky and Chang 1975)

the maximum expected earthquake for the region. Any decision about the cover design should also be based on a study of faults at each specific site. Indiscriminate or unjustified overdesign should be avoided.

In designing a containment system, both the possibility of a fault rupturing the tailings pile and the effect of a rupture on the integrity of the containment system must be considered. In all cases, these considerations are site specific. In aseismic regions or areas removed from active faulting [see Slemmons and McKinney (1977) for a discussion of criteria for identifying active faults], the probability of direct fault ruptures appears negligible. For tailings piles located on or near active faults or in seismically active areas, the possibility of direct rupture is much greater. If the containment

FIGURE 3. Earthquake Intensity Versus Magnitude and Epicentral Distance for Western United States (Modified from Krinitzsky and Chang 1975)

system is to be 100% effective, it must be able to withstand a maximum displacement without rupturing or seriously weakening. If such a system cannot be technologically or economically designed to withstand immediate or delayed rupturing or weakening, the tailings pile should be moved to a more appropriate location.

Containment systems designed to withstand vertical and horizontal displacements should be sufficiently thick and flexible and preferably have self-healing properties. The multilayer cover system proposed in this study meets these criteria in that its thickness can be adjusted to accommodate the maximum expected displacement for each tailings site. As stated previously, the maximum displacement can be estimated from analyses of earthquakes and fault displacements that have occurred in similar geologic situations or at the site itself.

FIGURE 4. Relationship Between Displacement and Earthquake Magnitude for Historic Faulting in Western North America (Wallace 1970)

Settlement

Loose soils commonly settle during earthquakes as a result of vibratory densification. Dry or unsaturated sandy soils are the most susceptible to compaction and settlement (Union Carbide 1972). In comparison, gravels and most clays are usually sufficiently dense in their natural states that little additional densification occurs. The effect of earthquake loading on saturated soils is discussed in the section on liquefaction.

The multilayer cover system will be constructed of a layer of compacted bentonite clay and gravel, topped by a layer of gravel forming a vapor barrier, topped again by a thick layer of native soil. Little or no settlement of the

clay/gravel layer or moisture barrier is expected, and any settlement of the overlying native soil would actually increase the effectiveness of the system.

Densification of the silt- and sand-sized mill tailings must also be considered. Depending on the characteristics of the tailings and their distribution within the pile, the surface of the pile may undergo either uniform or, in some places, differential settlement. Uniform settlement should have no detrimental effect on the cover system, but differential settlement could weaken or otherwise damage it if proper care was not taken when designing and building it. Standard soils-engineering techniques can be used to estimate the maximum amount of differential settlement. This information can be used to modify the cover design appropriately. A discussion of differential settlement is given in a later section.

Liquefaction

Liquefaction of soils, defined by Youd (1973) as "the transformation of a granular material from a solid state into a liquefied state as a consequence of increased pore-water pressures," commonly occurs in response to earthquake loading. The result of liquefaction is that the soil has a partial or complete loss of strength and can support little or no load. The soils most susceptible to liquefaction are medium, fine, and silty sands. Coarse gravels and clays generally have low liquefaction potentials. Seismically-induced liquefaction is most likely to occur if the soil is saturated, uniformly graded, and of low density; if the soil is under low confining stress; and if the intensity and duration of the ground shaking exceed certain threshold values. These parameters are discussed below.

As stated earlier, liquefaction occurs as the result of increased pore water pressure within the soil. During an earthquake, loose soils tend to densify. If the soil is saturated and the water is unable to drain, pore-water pressures will temporarily increase. As a result of the increased pore-water pressure, the effective stress (or grain-to-grain contact pressure in the soil mass) decreases. When this occurs, much or all of the bearing capacity of the soil is transferred from the soil particles to the pore water. Because the

pore water cannot resist shear stresses, the soil mass behaves as a fluid. Unless confined, the fluid mass will flow. As a result, slope, embankment, and foundation failures commonly occur.

Field and laboratory data indicate that uniformly graded, medium- to fine-grained sands and silts are the most susceptible to liquefaction (Seed and Idriss 1967; Lee and Fitton 1968; and Kishida 1969b). This susceptibility is shown in Figure 5, where grain size distribution ranges for several observed cases of liquefaction are plotted. Based on analyses of seismically-induced liquefaction in Japan, Kishida (1969a) suggests that the soils most susceptible to liquefaction have a particle diameter of $D_{50} < 2\text{mm}$ and a uniformity coefficient of $U < 10$. Ohsaki (1969) suggests that the soils have $2\text{mm} > D_{60} > 0.2\text{mm}$ with $D_{10} > 0.074\text{mm}$ and $U < 5$. (D_{10} , D_{50} , and D_{60} are the particle diameters for which 10, 50, and 60% of the sample weight is finer. The coefficient of uniformity, U , is defined as the ratio of D_{60} to D_{10} .)

FIGURE 5. Grain-Size Distributions of Soils Most Susceptible to Liquefaction (Modified from Union Carbide 1972)

The effects of void ratios (the ratio of the volume of the voids to the volume of the solids) on liquefaction have been investigated in several studies, e.g., Lee and Seed (1967a), Whitman (1970), Peacock and Seed (1968), Seed and Peacock (1970), and Seed and Idriss (1970). These investigators found that soils with high void ratios tend to densify when subjected to stress, especially vibratory stresses such as earthquake loading. As a result, if the soil is saturated, the pore-water pressures increase and the soil mass may liquefy. Dense soils (i.e., soils having a low void ratio), however, tend to expand slightly when subjected to shear stresses. In so doing, the pore-water pressure decreases, the effective stress increases, and thus, the soil becomes less susceptible to liquefaction.

Relative density (D_r), another common description of soil density, is based on the comparison of the in-situ void ratio with its void ratio in its loosest and densest states. The relative density is defined as:

$$D_r = \frac{e_{\max} - e_{\text{field}}}{e_{\max} - e_{\min}} \times 100$$

where $e_{\max}$ and $e_{\min}$ are the void ratios in the densest and loosest states, respectively, and e_{field} is the in-situ void ratio. The relative density above which liquefaction will not occur is a function of many parameters, e.g., soil properties, stratigraphy, and earthquake characteristics. Experience indicates that minimum relative densities ranging from 50 to 75% may be necessary. Mittal and Morganstern (1977) indicate that $50\% < D_r < 60\%$ may be sufficient in areas of moderate seismicity. Kishida (1969a) reports that $D_r > 75\%$ prevented liquefaction in several Japanese earthquakes. The Canadian government (PSM 1977) recommends a value of at least 60% for mill tailing embankments. When dealing with hydraulically placed mill tailings, relative densities of 60% or greater can generally be obtained only by mechanical compaction. Consequently, relative densities of 60% or less may be common in abandoned uranium mill tailing piles, especially if the piles are poorly drained or contain a high percentage of slimes.

When assessing the liquefaction potential of a soil mass, the confining stress at various depths must also be considered because of its impact on effective stress, and thus on the shear strength of the soil. Because increased confining stress strengthens the soil, there should be some depth beneath the surface below which liquefaction will not occur. Field observations suggest that in saturated sand, liquefaction is not likely to occur at depths greater than about 15 to 18 m (Union Carbide 1972). This finding is in agreement with Kishida's (1969a) conclusion that sandy soil is not liquefiable if the value of effective overburden pressure exceeds 2 kg/cm^2 .

The characteristics of the earthquake must also be considered when assessing liquefaction potential. Of these characteristics, the intensity and duration of shaking are probably the most important. The intensity is determined by such factors as earthquake magnitude, epicentral distance, and attenuation, and is detected at the site as a series of ground-motion accelerations. Seed and Idriss (1970) proposed a method of estimating the liquefaction potential of sand deposits based on the density of the sand and the maximum ground acceleration. For given values of maximum ground acceleration, they identified three ranges of relative density and the relative liquefaction potential for each range. These ranges and associated accelerations are: 1) a range in which liquefaction is likely to occur; 2) a range in which liquefaction may occur, depending on the characteristics of the sand and the number of significant stress cycles; and 3) a range in which liquefaction is not likely to occur. Table 2 illustrates liquefaction potentials for a hypothetical site consisting of sand deposits with the water table about 1.5 m below the surface.

TABLE 2. Ground Acceleration Versus Liquefaction Potential
(Seed and Idriss 1970)

Maximum Ground-Surface Acceleration	Liquefaction Very Likely	Liquefaction Potential Depends on Soil Type and Earthquake Magnitude	Liquefaction Very Unlikely
0.10 g	$D_r < 33\%$	$33 < D_r < 54\%$	$D_r > 54\%$
0.15 g	$D_r < 48\%$	$48 < D_r < 73\%$	$D_r > 73\%$
0.20 g	$D_r < 60\%$	$60 < D_r < 85\%$	$D_r > 85\%$
0.25 g	$D_r < 70\%$	$70 < D_r < 92\%$	$D_r > 92\%$

Laboratory studies show that liquefaction begins only after a certain number of stress or strain cycles has occurred (Seed and Idriss 1970). These cycles are a function of the duration of the shaking. During the 1964 Alaskan earthquake, for example, landsliding did not begin until almost 90 seconds after the shaking began (Shannon and Wilson 1964). The liquefaction that triggered the landslides would not have occurred if the duration of shaking had been significantly less.

Seismically-induced liquefaction can have a serious impact on the stability of uranium mill tailings piles and the integrity of radionuclide containment systems. These problems result from the mode of deposition and the physical characteristics of the tailings. Tailings impoundment dams are generally hydraulically filled structures designed for minimum safety factors of 1.1 to 1.5 (PSM 1977). The coarse fraction of the tailings used in dam construction is generally composed of relatively uniformly-graded, sand-sized material. The finer-grained materials and slimes are impounded behind the dam.

The tailings dam construction method shown in Figure 6 is the most economical of the popular methods. The principal disadvantage of this method is that the upper levels of the dam are constructed over the fine-grained materials and slimes, which may result in poor foundation conditions and drainage, and thus threaten stability. Seismically-induced liquefaction has been responsible for the failure of numerous tailings dams. Several such failures manifested during the 1965 Chilean earthquake are described by Derby and Alvarez (1967).

Depending on the local site conditions, similar failures could also occur at abandoned uranium mill tailings piles. The failure of a cover system due to liquefaction is influenced by the local and regional seismicity and the physical character of and degree to which the tailings and underlying materials are saturated. If the tailings and underlying deposits are well-drained and unsaturated, liquefaction is highly unlikely. If the tailings are either saturated or contain poorly-drained, saturated, silty or sandy zones, liquefaction is possible. Liquefaction of the tailings would probably cause at least partial failure of the cover system. If liquefaction occurred, the

FIGURE 6. Upstream Method of Tailings Dam Construction (PSM 1977)

tailings could conceivably mobilize and flow downgradient. Youd (1973) cites several cases in which natural and man-made slopes have demonstrated liquefaction-induced failure. In these cases, the mobilized material flowed down slopes as shallow as 0.8%. Even if the tailings did not move, the loss of support and the release of pore water could cause partial collapse of the multilayer cover.

Mitigation of Earthquake Effects

The potential severity of earthquake damage to radionuclide containment systems can be greatly reduced through careful design that takes into consideration the characteristics of each tailings pile and its surrounding region. A site-specific analysis is required of all factors that may influence the seismic safety of each tailings pile. Seismic loading probably would not seriously damage containment systems whose underlying materials are unsaturated, well-drained and likely to remain so. Ground-rupture effects can largely be avoided if the tailings pile does not cross and is not near an active or capable fault. In some cases, if the pile is on or adjacent to a fault, the tailings may have to be moved to a safer location.

Containment systems for saturated tailings or tailings piles underlain by saturated sandy material are the most susceptible to seismically-induced damage. Damage as a result of densification and liquefaction could be prevented or decreased in severity by compacting the tailings to relative densities of perhaps 60% or greater. Liquefaction also could be prevented by dewatering. However, a method must be devised to ensure that the tailings (and possibly the underlying materials) remain in an unsaturated condition. For tailing piles in many portions of the arid western United States, saturation may not be a serious problem.

Tailings Piles in High-Risk Seismic Zones

Tailings piles in high-risk seismic zones present special hazards, especially if they are near population centers. The Vitro pile at Salt Lake City, Utah, is one such pile. Located in Seismic Zone 3, the Vitro pile is surrounded by commercial developments, has characteristics suggesting potential

seismic instability, and is underlain by a high water table and potentially unstable clay, silt, and sand deposited by pluvial Lake Bonneville. Because it is difficult to guarantee that this pile can be isolated, it might be best to move it to a safer area.

Economic considerations may prevent relocating the Vitro pile outside of Seismic Zone 3 and away from saturated Lake Bonneville deposits. The pile may have to be relocated somewhere within Seismic Zone 3, and its foundation and cover will have to be designed and engineered to meet the site-specific criteria for Zone 3 discussed in this report.

EROSIONAL EFFECTS

Rock and soil are eroded, transported, and deposited principally by water, ice, and wind. Depending on local conditions, these erosive agents can have major impacts on the integrity of uranium mill tailings containment systems, especially upon covers. The methods used to prevent erosion of the cover should be based on site-specific analyses that consider all pertinent geologic, geomorphic, and climatic variables. The following sections briefly discuss the erosional effects of water, ice, and wind and suggest methods to protect uranium mill tailings cover systems from erosion.

Fluvial Erosion

Water-related erosion includes a variety of processes such as flooding, stream channel migration, sheetwash, and rill development. This section focuses principally on fluvial (or stream) erosion and briefly discusses the possible impacts of less dramatic processes.

Streams operate at their greatest erosion and transport efficiency during the relatively brief periods of maximum flow. The lower flows that predominate over most of the year have far less erosion and transport capability and commonly are orders of magnitude less than the peak flood flows. The ratios of maximal to minimal flow for several major streams range from 13:1 to 313:1 (Gilluly, Waters, and Woodford 1975). As a rule, increased water volume leads to increased flow velocity. Gilluly, Waters, and Woodford (1975) indicate that

most streams have flow velocities of less than about 2.2 m/s. (A maximum flow velocity of 9.2 m/s for one stream has been reported.) The Columbia River in Washington serves as a good illustration of change-in-flow dynamics. In 1945, for instance, the low-stage flow velocity of the river was about 0.6 m/s. During that same year, at flood stage, the velocity increased to about 3.4 m/s.

Figure 7 shows the effect that flow velocity may have on stream competence—the maximum particle size that the stream can transport. The width of the band in the figure indicates approximate variations depending on stream depth and the cohesiveness of the material. Fine sand is the most readily eroded; materials that are either finer or coarser require greater velocities. The curves are based on experiments with materials of carefully chosen sizes,

FIGURE 7. Curve Showing Least Velocity of Flow at Which Sedimentary Particles Begin to Erode (After Gilluly, Waters and Woodford 1975)

not upon mixtures of a wide size range. Such mixtures act as though one-third were coarser and two-thirds finer than the actual mean grain size (Gilluly, Waters, and Woodford 1975).

For the Columbia River example cited above, the average stream competence may have increased from about 1.5 mm to about 40 mm. As flow volume and velocity increase, the amount and size of transported material increases. As a result, the stream's erosive capability increases.

When determining the suitability of possible sites for a uranium mill tailings pile, the sites' positions relative to streams, arroyos, or other permanent or ephemeral water courses must be assessed. Canyons, flood plains, and arroyos are highly susceptible to flooding and erosion. Even arid regions that are essentially devoid of permanent streams can be seriously affected by flash-flooding and mud flows. At any tailings pile location in which flooding or stream erosion may jeopardize the integrity of the cover system, some form of protection must be provided because the multilayer cover, as now envisioned, probably cannot completely resist these forces. The pile could be protected with riprap or other coarse material or surrounded with a dike; or the stream channel could be diverted so that it bypasses the pile. As a last resort, the pile could be relocated to a safer location. Several of these possibilities are being investigated. Whichever solution is used, it must be tailored to the specific site and should be maintenance-free.

Even if the site is not susceptible to flooding or stream erosion, it will probably be affected by sheet wash, rain-drop impact, and rill development. During periods of intense rainfall, erosion of unconsolidated material by these methods can be severe, especially in areas without well-developed vegetation or armoring. If the multilayer cover is not protected by vegetation or other methods, and if sufficient slope exists, erosion of the overburden will occur. This erosion could, with sufficient rainfall or time, cut through the overburden, moisture barrier and clay layers, and expose the tailings to erosion, recharge and saturation. The potential effects of moisture on the tailings are discussed in other sections. To prevent erosion of this sort, the use of relatively flat and vegetated or otherwise protected slopes is

advocated. A combination of flat slopes, vegetation, and graded gravel, riprap, or other material would provide the greatest protection.

Episodes of flooding, stream migration, and precipitation can also affect the stability of the cover and tailings by increasing their moisture contents. This increase could be either short- or long-term, depending on the specific site conditions. As discussed in other sections, slope instability and liquefaction become more probable as the soil's moisture content increases or reaches saturation.

Glacial Erosion

During the Pleistocene, glacial ice covered a large portion of the United States (Figure 8). Several abandoned uranium mill tailings piles are located near the limits of this glaciation. Should the ice extend beyond these limits during future glaciations, the integrity of the tailings cover system could be jeopardized. However, the likelihood of such a glaciation occurring during the next several thousand years is extremely remote. Even if a major climatic change were to occur immediately, ice sheet advance to the maximum limits of the Pleistocene glaciers would probably require several thousand years. Consequently, glacial ice erosion will not be considered further.

Eolian Erosion

The erosive effect of wind is generally much less pronounced than that of either water or ice. Where conditions have been conducive, though, eolian erosion (or deflation) has formed depressions up to 100 m or more deep that cover tens of square kilometers (Gilluly, Waters, and Woodford 1975). Such features are rare and develop only in regions where sufficiently thick, dry, unconsolidated, unvegetated, homogeneous deposits of silty or sandy material exist. In most areas of the arid western United States, the surficial materials are mixtures of silt, sand, and gravel. As the wind removes the finer grained materials, the gravels eventually form an armored surface, or desert pavement, that prevents further erosion of the underlying silt and sand. A similar situation sometimes occurs on playas, or dried lakes, where clay acts as a binding material for silt and sand.

FIGURE 8. Glacial Areas of the Conterminous United States (Modified after Gilluly, Waters and Woodford 1975)

Although dry tailings are an ideal material for eolian transport, wind could probably not remove enough of the multilayer cover to gain access to the tailings. Even if an entire unprotected overburden layer were removed, eolian erosion would probably cease at the gravel moisture barrier because of the formation of armor by the gravel. In the event that the gravel moisture barrier were removed by some process, e.g. flooding, the compacted clay would continue to resist eolian erosion. In areas where eolian erosion may be a problem, the entire multilayer barrier can be protected by vegetating the surface or by covering it with gravel or riprap.

Climatic Change

An additional consideration in assessing the suitability of a given tailings site is the effect of climatic change. At present, much of the western United States is arid, but during the next 1000 yr the climate could become either more arid or more humid. The magnitude and direction of such changes are difficult to predict. Methods exist to predict gross, long-term regional or global climatic changes (e.g. Kukla 1978, 1981), but these methods are not directly applicable to the shorter-term and more localized changes that are of interest in this study. Despite this lack of certainty, allowances for climatic changes should be made in the design of uranium mill tailings covers and liners.

To assess the impact of climatic change on cover design, the type of climatic change must be known, i.e., will it become cooler and wetter or warmer and drier? Work by Kukla, discussed in Petrie et al. (1980), indicates that global climate has been gradually shifting from interglacial toward glacial for the past 6000 yr. This trend will probably continue until a full glacial climate is reached in about 2000 yr. However, the glacial maximum is not expected to occur until about 4000 yr from the present.

In changing from interglacial to glacial climate, North America would probably become cooler and wetter. Superimposed upon this general trend may be rapid and dramatic changes or fluctuations in local climates. However, the expected trend suggests that the multilayer cover should be designed to withstand larger and more frequent flooding as well as greater precipitation than

would otherwise be anticipated for the present climatic state. A discussion of flooding and erosion considerations is given in the section on fluvial erosion.

should be taken in the present climatic state. A discussion
of the present and future consequences of the present climatic state
is given in the section on future
erosion.

GEOTECHNOLOGICAL CONSIDERATIONS FOR CONSTRUCTION OF MULTILAYER BARRIERS AT GRAND JUNCTION, COLORADO, TEST SITE

Geotechnological assessments of inactive tailings sites at Grand Junction, Colorado, have been prepared by Ford, Bacon, and Davis (1977) and Nelson et al. (1980). The following section relies heavily on data extracted from these reports.

SLOPE STABILITY

Slope angles of the Grand Junction uranium mill tailings pile range from about 1 to 17% (Figure 9). Exceptions to this occur along the south and west edges of the pile where berm slopes are about 100%. These steep slopes are apparently supported by locally-obtained, clayey soil rather than by tailings materials. Along the south edge of the pile, the slopes are also protected from being eroded by the Colorado River by brick and concrete riprap.

Insufficient data are available for detailed slope analyses. However, several comments regarding stability can be made. If the tailings pile is not relocated, extensive grading will probably be required to decrease the soil erosion potential and increase slope stability. Even with the present configuration, the high angles of friction reported by Nelson et al. (1980) would probably be sufficient to prevent slope failures in the sand tailings unless there is a marked increase in moisture content, or other extenuating circumstances. Slope failure could occur in the saturated slimes if there is insufficient lateral support. Consequently, draining, compacting, or otherwise increasing the shear strength of any loose, saturated materials may be necessary.

Slope failure probably has the greatest potential of occurrence along the Colorado River bank. Failure would probably be related to erosion of the tailings pile berm either by flooding or normal channel migration. Ford, Bacon, and Davis (1977) reported that the Colorado River has a long history of flooding and that during floods the river could surround the tailings pile. This could lead to slope failure by either saturating the tailings or by removing lateral support along the periphery of the pile. Slope failure could also result from seismic loading, as discussed in the section on earthquake effects.

FIGURE 9. Topographic Map of Grand Junction Tailings Piles (Modified from Ford, Bacon, and Davis 1977)

EROSION PROTECTION AND RECHARGE

If the multilayer cover is to maintain maximal containment efficiency, the moisture content of the clay layer should remain sufficiently high, and the entire barrier should be protected from erosion and denudation. Possible methods of recharging the clay layer and protecting the cover are shown in Figures 10, 11, 12, and 13. These methods allow precipitation falling upon the cover to infiltrate the outer protection, the overburden, and the moisture barrier, and then recharge the clay layer. In each design, the outermost layer can be modified as necessary to suit the present and possible future conditions at each site. For tailings piles located at or near canyons, arroyos, rivers, springs or other permanent or ephemeral sources of water, it may be possible to route or allow water to flow across an armored portion of the cover, thus increasing both the amount and potential for recharge.

DIFFERENTIAL SETTLEMENT

Within the Grand Junction tailings pile, the sands and slimes are neither homogeneous nor uniformly distributed. As a result, there will be a tendency for differential settlement to occur as the materials compact under their own weight and the weight of the multilayer cover. To estimate the amount of differential settlement that may occur, a series of settlement calculations were made along the profile shown in Figure 14. Because many critical data were unavailable, several assumptions were necessary. Consequently, the differential settlement estimates are intended only as gross estimates. Unless otherwise stated, the data for the calculations (Table 3) were based on Nelson et al. (1980).

The idealized tailings profile (Figure 14) was used as the basis for the settlement calculations. The thicknesses of the sands and slimes were measured at 35 evenly spaced locations along the profile. At each location the estimated amount of settlement was calculated and plotted on Figure 15. The calculations assumed that the settlements would occur uniformly across the entire profile in proportion to the thickness of the sands and slimes at each location. The underlying gravels were assumed to be incompressible so that the settlement would occur only in the tailings.

FIGURE 10. Multilayer Cover Recharge--I
(Not to Scale)

FIGURE 11. Multilayer Cover Recharge--II
(Not to Scale)

FIGURE 12. Multilayer Cover Recharge--III
(Not to Scale)

Because the settlement calculations were intended to reflect extreme long-term effects, they assumed that the sands and slimes will eventually reach standard Proctor densities of 100%. The following standard soil mechanics equation (Peck, Hanson and Thornburn 1974) was used to calculate the amount of settlement at each location:

$$\Delta H = H_o \left(\frac{\Delta e}{1+e_o} \right)$$

FIGURE 13. Multilayer Cover Recharge--IV
(Not to Scale)

where

ΔH = the settlement

H_0 = the thickness of the material undergoing settlement

Δe = the change in void ratio due to settlement

e_0 = the void ratio prior to settlement.

For the sands, e_0 and Δe were 0.90 and 0.31, respectively. Values of 1.77 and 0.64 were used for the slimes. Based on these data and deposit thicknesses measured from the profile in Figure 15, the following settlement was calculated for a location near the center of the pile:

$$\begin{aligned}\Delta H_{\text{sand}} &= 4.58 \text{ m} \left(\frac{0.31}{1+0.90} \right) \\ &= 0.75 \text{ m}\end{aligned}$$

FIGURE 14. Idealized East-West Profile Through the Grand Junction Uranium Mill Tailings Pile

TABLE 3. Soil Properties at Grand Junction Tailings Pile. Based on data from Nelson et al. (1980)

Type	Void Ratio e (natural)	Void Ratio e (100% Std. Proctor)	Density γ_d (natural)	Density γ_d (compact)	Δe	Moisture Content wt%
Tailings sand	0.99		1362 kg/m ³			
	0.89		1426			
	0.84		1474			
	0.72		1570			3-9
	1.11		1282			
	1.27		1185			
	0.70		1602			
	0.64		1602			
				1746 kg/m ³		
				1778		
		0.59		1618		10
avg.	0.90	0.59	1442	1714	0.31	10
Tailings slimes	1.65		1041			37
	1.89		929			
		1.13		1282		30
avg.	1.77	1.13	993	1282	0.64	30

FIGURE 15. Calculated Differential Settlement Along Idealized Profile Shown in Figure 14

$$\begin{aligned}\Delta H_{\text{slimes}} &= 2.29 \text{ m} \left(\frac{0.64}{1+1.77} \right) \\ &= 0.53 \text{ m}\end{aligned}$$

$$\begin{aligned}\Delta H_{\text{total}} &= 0.75 \text{ m} + 0.84 \text{ m} \\ &= 1.28 \text{ m}\end{aligned}$$

Because realistic estimates of differential settlement require data that are not yet available, numerous simplifying assumptions were made. Consequently, the settlement estimates shown in Figure 15 are intended to be illustrative only.

The data used to make the settlement calculations (Table 3) were adjusted as necessary to fit the assumptions discussed earlier. The calculated settlements given in Figure 15 are for evenly spaced intervals of about 23 m along the profile. Settlement was assumed to be linear between adjacent locations. The maximal differential settlement between any two locations is about 0.5 m.

EFFECT OF DIFFERENTIAL SETTLEMENT ON MULTILAYER COVER

To determine the effect that differential settlement would have on the integrity of the multilayer cover, calculations were made to estimate the amount of deformation that the cover could tolerate without failing. The calculations assumed that the cover could be thought of as an elastic beam in which failure would occur if the stress at the outer edge of the beam exceeded the tensile strength of the clay layer. This method provides only a gross estimate of the amount of deflection that the clay layer could tolerate without failure. An accurate estimate of allowable deflection would require a much more sophisticated approach and data that are not now available.

The multilayer cover consists of 0.15 m of a compacted bentonite-sand-gravel mix overlain by a 0.3-m thick gravel moisture barrier and 1 m of locally obtained soil. Because many of the necessary geotechnological data for the analysis were not available, generalized data from several sources were used. The compacted clay-sand-gravel layer was assumed to have a shear strength of 112.5 kN/m^2 , which corresponds to that of "undrained, stiff clay soils" as given by Attewell and Farmer (1976). Because the angle of internal friction is assumed to equal zero in undrained shear strength testing, the cohesion of the clay is also assumed to be 112.5 kN/m^2 . The value for the modulus of elasticity is from Lamb and Whitman (1969) for a "typical clay." In the calculations, the cohesion is assumed to be equal to the tensile strength of the cover.

Provided that the above assumptions are reasonable and the soil properties used approximate those of the cover, the calculations suggest that the cover could tolerate a maximum deflection of about 1.2 m over a 46-m span. This corresponds to a midpoint deflection over three consecutive profile locations. In Figure 14 the maximum calculated deflection over this distance is about 0.2 m.

The following equation was used to determine the maximum deflection of a uniformly loaded beam:

$$Y_c = \frac{5 ML^2}{48 EI}$$

where

Y_c = deflection (m)

M = maximum allowable bending moment (0.63 KN m)

L = span length (46 m)

E = modulus of elasticity (400,000 Kg/m²)

I = moment of inertia (2.81×10^4 m⁴)

The maximum allowable bending moment, M , was calculated from:

$$M = \frac{bd^2}{4} f_y$$

where

M = maximum allowable bending moment

b = width (assumed to be 1 m)

d = depth of beam (0.15 m)

f_y = maximum allowable stress (assumed to be cohesive strength of clay, 112.5 KN m)

These equations were taken from Merritt (1976) for an elastic beam undergoing uniform loading.

The calculations shown above assume that failure occurs if the cohesive strength of the clay is exceeded. Note, however, that the multilayer cover will probably respond plastically to deformation and thus could accept far greater strain without failure than the calculations suggest. If the cover maintains its initial water content, or if the water content increases, the calculated differential settlement should have no detrimental effect on the multilayer cover.

CONCLUSIONS

This preliminary analysis indicates that geomorphic and geotechnological considerations of the specific tailings pile and site are required to evaluate the long-term stability of cover designs. In evaluating the suitability of the proposed cover for a given site, one must consider the potentially disruptive effects of seismic activity at the site, the beneficial or disruptive effects of geomorphic processes, and the long-term engineering stability of the tailings pile and its foundation and cover system.

Currently, the information required for these evaluations is incomplete. The available information needs to be supplemented by general and site-specific investigations of the long-term stability of existing tailings piles and their environment, and the environments of alternate disposal sites. Also, proposed cover systems should be evaluated in terms of their stability and suitability for the conditions expected at the specific sites. Geotechnological characterization of the piles and their underlying materials will be necessary to determine how the tailings will respond to the cover system and to environmental effects. These studies will also determine if compaction, grading, dewatering, or other stabilization procedures will be necessary. The geotechnological characterization will help determine the most efficient and suitable methods of handling, transporting, and placing the tailings that have to be relocated.

Evaluations of the proposed cover system must focus on its long-term stability in the environment in which it will be used. The cover must be able to resist all natural events and processes that may adversely affect the specific disposal site, as well as any changes within the pile and its foundation. Thus, all potentially disruptive phenomena at each site must be identified and characterized. Given this information and the application of suitable safety factors, adequate, cost-efficient cover systems can be designed.

CONCLUSIONS

This preliminary analysis indicates that geologic and geotechnical considerations of the selected tailings pile and site are required to evaluate the long-term stability of cover designs. In evaluating the suitability of the proposed cover for a given site, one must consider the potential for long-term effects of seismic activity at the site, the potential for structural effects of geomorphic processes, and the long-term engineering stability of the tailings pile and its foundation and cover system.

Currently, the information required for these evaluations is incomplete. The available information needs to be supplemented by general and site-specific investigations of the long-term stability of existing tailings piles and their environments, and the requirements of alternate disposal sites. Also, proposed cover systems should be evaluated in terms of their stability and suitability for the conditions expected at the selected sites. Geotechnical observations of the position of the pile and their underlying materials will be necessary to better understand the behavior of the cover system and to evaluate its performance. These studies will also determine if conditions, including covering, or other stabilization procedures will be necessary. The geotechnical observations will help determine the most efficient and suitable methods of handling, transporting, and placing the tailings that have to be relocated.

Evaluations of the proposed cover system must focus on its long-term stability in the environment in which it will be placed. The cover must be able to resist all natural events and processes that may adversely affect the geologic disposal site, as well as any changes within the pile and its foundation. These will include any disruptive phenomena at each site that are identified and characterized. Given this information and the availability of suitable factors, appropriate, cost-effective cover systems can be designed.

REFERENCES

- Algermissen, S. T. 1969. "Seismic Risk Studies in the United States." Proceedings of Fourth World Conference of Earthquake Engineers. Asociacion Chilena de Sismologia e Ingenieria Antisismica, Santiago, Chile V 1.
- Attewell, P. B., and I. W. Farmer. 1976. Principles of Engineering, Geology. Chapman and Hall.
- Bull, C. 1980. Glaciological Parameters of Disruptive Event Analysis. PNL-2863, Pacific Northwest Laboratory, Richland, Washington.
- Derby, R., and L. Alvarez. 1967. "Seismic Failure of Chilean Tailing Dams." Jour. Soil Mechanics and Foundations Division, ASCE. 93:(SM6):237-260.
- Ford, Bacon and Davis. 1977. Phase II - Title I Engineering Assessment of Inactive Uranium Mill Tailings, Grand Junction Site, Grand Junction, Colorado. Prepared for U.S. Dept. Energy, Grand Junction, Colorado, Contract No. E(05-1) - 1658.
- Gilluly, J., A. C. Waters, and A. O. Woodford. 1975. Principles of Geology. Fourth ed., W. H. Freeman and Co.
- Kishida, H. 1969a. "Characteristics of Liquefied Sands During Mino-Owari, Tohankai and Fukui Earthquakes." Soils and Foundations (Japan). 9(1):75-92.
- Kishida, H. 1969b. "A Note on Liquefaction of Hydraulic Fill During the Tokachi-Oki Earthquake." Paper submitted to Second Seminar on Soil Behavior and Ground Response During Earthquakes, University of California, Berkeley, California.
- Krinitzsky and Chang. 1975. State of the Art of Assessing Earthquake Hazards in the United States; Earthquake Intensity and the Selection of Ground Motions for Seismic Design. Misc. Paper S-73-1, Report 4, Sept. 1975, U.S. Army Engineer Waterways Experiment Station, C.E., Vicksburg, Mississippi.
- Kukla, G. 1979. "Probability of Expected Climate Stresses in North America in the Next One Million Years." A Summary of FY-78 Consultant Input for Scenario Methodology Development. PNL-2851, Pacific Northwest Laboratory, Richland, Washington.
- Kukla, G., A. Berger, R. Lotti and J. Brown. 1981. "Orbital Signature of Interglacials." Nature. Paper 5442/1.
- Lambe, T. W., and R. V. Whitman. 1969. Soil Mechanics. John Wiley and Sons.
- Lee, K. L., and J. A. Fitton. 1968. "Factors Affecting Cyclic Loading Strength of Soil." Special Technical Pub. No. 450, Symposium on Vibration Effects of Earthquakes on Soils and Foundations, ASTM.

- Lee, K. L., and H. B. Seed. 1967. "Cyclic Stress Conditions Causing Liquefaction of Sand." Jour. Soil Mechanics and Foundations Division, ASCE. 93(SM1):47-70.
- Merritt, F. (ed.). 1976. Standard Handbook for Civil Engineers. 2nd Edition, McGraw-Hill.
- Mittal, H. K., and H. R. Morganstern. 1977. "Design and Performance of Tailings Dams." Proc. Conf. on Geotechnical Practice for Disposal of Solid Waste Materials. Ann Arbor, Michigan.
- Nelson, J., and T. Shephard. 1980. Inactive Uranium Mill Tailings Sites, Analysis of Stability, Quarterly Report July-Sept. 1980. Prepared for Environmental Trace Substances Research Program, University of Colorado by Geotechnical Engineering Program, Civil Engineering Dept., University of Colorado.
- Peacock and Seed. 1968. "Sand Liquefaction Under Cyclic Loading Simple Shear Conditions." Jour. of Soil Mechanics and Foundation Division, ASCE. 94(SM3):689-708.
- Peck, R. B., W. E. Hanson and T. H. Thornburn. 1974. Foundation Engineering, 2nd Edition. Wiley and Sons Publishers.
- Petrie, G. M., J. T. Zellmer, J. W. Lindberg and M. G. Foley. 1980. Simulation Model for Release Scenario Analysis: A Hypothetical Site in Basalts of the Columbia Plateau. PNL-3542, Pacific Northwest Laboratory, Richland, Washington.
- Press, F., and Sevier, R. 1978. Earth. 2nd edition, Freeman and Co.
- PSM. 1977. "Waste Embankments, Energy, Mines and Resources Canada." Pit Slope Manual. Chapter 9. Canada Centre for Mineral and Energy Technology.
- Seed, H. B., and I. M. Idriss. 1970. A Simplified Procedure for Evaluating Soil Liquefaction Potential. Rept No. EERC 70-9, Nov. 1970, Earthquake Engineering Research Center, College of Engineering, University of California, Berkeley, California.
- Seed, H. B., and W. H. Peacock. 1970. Application of Laboratory Test Procedures for Measuring Soil Liquefaction Characteristics Under Cyclic Loading. Report No. EERC 70-8, Univ. Calif., Earthquake Eng. Res. Center, Berkeley, California.
- Shannon and Wilson. 1964. Report on Anchorage Area Soil Studies, Alaska to U.S. Army Engineer District, Anchorage, Alaska. Seattle, Washington.
- Slemmons, D. B. and R. McKinney. 1977. "Definition of Active Fault." Misc. Paper S-77-8, Final Report, May 1977, U.S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Mississippi.

Union Carbide. 1972. Soil Behavior Under Earthquake Loading Conditions; State-of-the-Art Evaluation of Soil Characteristics for Seismic Response Analysis. TID-25953. Prepared for U.S. Atomic Energy Commission.

Wallace, R. 1970. "Earthquake Recurrence Intervals on the San Andreas Fault." Bull. Geol. Soc. America. 81:2875-2890.

Whitman R. V. 1970. Summary of Results from Shaking Table Tests at Univ. of Chile Using a Medium Sand, Progress Report No. 9, Effects of Local Soil Conditions Upon Earthquake Damage. Research Report R70-25, Soils Pub. No. 258, Mass. Inst. of Technol., Dept. of Civil Eng.

Youd, T. L. 1973. Liquefaction, Flow, and Associated Ground Failure. U.S. Geological Survey Circular 688.

United States - 1955. 2nd Edition. New York: McGraw-Hill.
Bureau of the Census. Division of Statistics. Statistical Records
Series. 110-2500. Washington: U.S. Government Printing Office.

Waller, R. 1950. "Population and Urban Growth in the United States"
Bull. Amer. Stat. Ass. 45: 250-255.

Waller, R. 1950. Summary of Data on Urban and Suburban Growth in
the United States. Statistical Record, Series 110-2500, No. 250.
Bureau of the Census, Washington, D.C. 20540.

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