90-021

FINAL
STAFF TECHNICAL POSITION
DESIGN OF EROSION PROTECTION COVERS FOR STABILIZATION OF URANIUM MILL TAILINGS SITES

U. S. Nuclear Regulatory Commission

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CONTENTS

EXECUTIVE SUMMARY

In designing erosion protection covers for uranium mill tailings sites, licensees and applicants must meet the requirements of 10 CFR Part 40, Appendix A for Title II (active) sites and 40 CFR Part 192 for Title I (inactive) sites. These criteria establish broad design objectives for long-term protection of uranium mill tailings and specific design objectives that are considered to be applicable to the design of erosion protection covers. These objectives include: (1) preventing radioactive releases due to erosion; (2) providing long-term stability; (3) designing for minimal maintenance; and (4) meeting radon release limits.

In meeting the design objectives established by the regulations and standards, several studies and recent technical assistance efforts performed for the U. S. Nuclear Regulatory Commission (NRC) staff indicate, and the staff agrees, that the design of a cover is significantly affected by several natural phenomena, and that any cover design should take into consideration the following: (1) selection of an appropriate design basis flood or rainfall event; (2) control of gully initiation and gully development; (3) the occurrence of flow concentrations and drainage network development; (4) the effectiveness of vegetation in arid areas; (5) use of permissible velocity and tractive force methods; and (6) long-term durability of rock erosion protection.

It is the position of the NRC staff that cover designs are acceptable if licensees and applicants can demonstrate that the requirements of 10 CFR Part 40, Appendix A and 40 CFR Part 192 are met. This staff technical position (STP) describes technical analyses and design approaches that are acceptable to the NRC staff in demonstrating compliance with these regulations and standards. Acceptable design options include: (1) designing soil covers and soil slopes to be stable; (2) designing combinations of stable soil slopes and rock-protected slopes; (3) designing rock-protected slopes; (4) designing soil slopes that permit controlled gullying or gullying of limited extent; and (5) designing slopes that do not meet long-term stability requirements, but can be exempted in accordance with applicable regulations. There may also be other acceptable design options that are developed by licensees; such designs will be considered by the staff on a case-by-case basis.

Design methods for the aforementioned options have been developed by the NRC staff and are included in this position. Each method is discussed in detail, and a technical basis is provided, including appropriate references. Specific design and calculation procedures for implementing each option are also provided, including illustrative examples. General recommendations are discussed, along with any limitations that are inherent in the calculation methods or in the design assumptions. Erosion protection design guidance for uranium mill tailings applications is summarized in Table 1.

This staff position is intended to provide guidance in designing erosion protection covers; it has not been developed to provide guidance in other areas, such as groundwater protection or radon barrier design. However, the design of an erosion protection cover is intrinsically linked to the performance of a cover in other design areas, such as infiltration and slope stability. In recognition of the fact that an overall systems approach is needed in completing a total reclamation plan, the staff has provided a summary of design criteria and guidance documents that are applicable in other design areas. These criteria and documents are summarized in Table 2.

Appendix A provides guidance on the design of soil covers. Specific methods are discussed for designing stable soil slopes and swales.

Appendix B provides guidance on the design of soil slopes that permit gullying of limited extent. Specific methods are provided for use in designing sacrificial soil outslopes where no tailings are placed directly under the soil cover. This method is to be used when licensees can justify that designing for 1000 years is not reasonably achievable.

Appendix C provides general documentation procedures that should be followed in justifying that designing for 1000 years is not reasonably achievable.

Appendix D provides guidance on the design of rock riprap erosion protection. Specific procedures are discussed for designing riprap for top and side slopes, diversion channels, aprons and channel outlets, and the banks of large streams. Procedures are also provided for evaluating the quality of riprap to be used as erosion protection and for oversizing of marginal-quality rock.

Appendix E discusses and analyzes public comments that were received from various sources on the draft STP. The comments are summarized and grouped into major issue categories, and staff analyses of these issues are provided. In many cases, the analyses provide additional rationale and bases for staff conclusions on important issues identified by commenters.

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1. INTRODUCTION

Criteria and standards for environmental protection may be found in the Uranium Mill Tailings Radiation Control Act (UMTRCA) of 1978 (PL 95-604) (see Ref. 1) and 10 CFR Section 20.106, "Radioactivity in Effluents to Unrestricted Areas." In 1983, the U. S. Environmental Protection Agency (EPA) established standards (40 CFR Part 192) for the final stabilization of uranium mill tailings for inactive (Title I) and active (Title II) sites. In 1980, the United States Nuclear Regulatory Commission (NRC) promulgated regulations (10 CFR Part 40, Appendix A) for active sites and later revised Appendix A to conform to the standards in 40 CFR Part 192. These standards and regulations establish the criteria to be met in providing long-term stabilization.

These regulations also prescribe criteria for control of tailings. For the purpose of this staff technical position (STP), control of tailings is defined as providing an adequate cover to protect against exposure or erosion of the tailings. To help licensees and applicants meet Federal guidelines, this STP describes design practices the NRC staff has found acceptable for providing such protection for 200 to 1000 years and focuses principally on the design of tailings covers to provide that protection.

Presently, very little information exists on designing covers to remain effective for 1000 years. Numerous examples can be cited where covers for protection of tailings embankments and other applications have experienced significant erosion over relatively short periods (less than 50 years). Experience with reclamation of coal-mining projects, for example, indicates that it is usually necessary to provide relatively flat slopes to maintain overall site stability (Wells and Jercinovic, 1983, see Ref. 2).

Because of the basic lack of design experience and technical information in this area, this position attempts to adapt standard hydraulic design methods and empirical data to the design of erosion protection covers. The design methods discussed here are based either on: (1) the use of documented hydraulic procedures that are generally applicable in any area of hydraulic design; or (2) the use of procedures developed by technical assistance contractors specifically for long-term stability applications.

It should be emphasized that a standard industry practice for stabilizing tailings for 1000 years does not currently exist. However, standard practice does exist for providing stable channel sections. This practice is widely used to design drainage channels that do not erode when subjected to design flood flows. Since an embankment slope can be treated as a wide channel, the staff concludes that the hydraulic design principles and practice associated with channel design are generally appropriate for designing stable slopes that will not erode; these principles have been used in the development of the design criteria presented in this STP. Using these principles, a wide range of options and design approaches can be developed.

This STP makes no attempt to limit engineering judgment and practical experience from consideration in developing a reclamation design. Although the STP may not specifically address a particular design option, the use of such options is not precluded by this STP. The STP generally recommends a method or reference for determining a particular design parameter; however, these recommendations are not intended to preclude the use of other methods, if such methods can be reasonably justified. It must be recognized that this STP does not provide design requirements; rather, the STP provides design criteria that are acceptable to the NRC staff. Any design option using different design strategies will be considered by the staff on a case-by-case basis.

This STP supersedes Branch Technical Position WM-8201, "Hydrologic Design Criteria for Tailings Retention Systems" (see Ref. 3), with regard to long-term stabilization and tailings reclamation. However, it should be noted that many portions of that position remain applicable, particularly with regard to operational aspects of tailings dam design. Those operational aspects have not been incorporated into this position.

2. DISCUSSION

2.1 Design Objectives

Several major design objectives for long-term stabilization of uranium mill tailings are established in 40 CFR Part 192 for Title I sites and in 10 CFR Part 40, Appendix A, for Title II sites. These can be summarized as follows: (1) prevent radioactive releases caused by wind and water erosion; (2) provide long-term stability; (3) require minimal maintenance to assure performance; and (4) provide sufficient protection to limit radioactive releases.

2.1.1 Prevention of Radioactive Tailings Releases due to Erosion

Criteria for minimizing dispersion of radioactive tailings, with emphasis placed on isolation of tailings and protection against natural phenomena, are established in 40 CFR Part 192 and 10 CFR Part 40, Appendix A. Specifically, 40 CFR 192.02 and 10 CFR Part 40, Appendix A, Criterion 6, require that control methods be designed to limit radioactive tailings releases to specified levels.

The NRC staff has concluded that prevention of releases due to erosion was an important consideration in the development of both 40 CFR Part 192 and 10 CFR Part 40, Appendix A. Therefore, it becomes very important to assess the forces associated with surface water erosion, to design flood protection measures for appropriately severe flood conditions, and to minimize the potential for erosion and release of radioactive materials.

2.1.2 Long-Term Stability

As required by 40 CFR 192.02 and 10 CFR Part 40, Appendix A, Criterion 6, stabilization designs must provide reasonable assurance of control of radiological hazards for a 1000-year period, to the extent practicable, but in any case, for a minimum 200-year period. The NRC staff has concluded that the risks from tailings could be accommodated by a design standard that requires that there be reasonable assurance that the tailings remain stable for a period of 1000 (or at least 200) years, preferably with reliance placed on passive controls (such as earth and rock covers), rather than routine maintenance.

2.1.3 Design for Minimal Maintenance

Criteria for tailings stabilization, with minimal reliance placed on active maintenance, are established in 40 CFR Part 192 and 10 CFR Part 40, Appendix A, Criteria 1 and 12. Criterion 1 of 10 CFR Part 40, Appendix A specifically states that: "Tailings should be disposed of in a manner [such] that no active maintenance is required to preserve conditions of the site." Criterion 12 states that: "The final disposition of tailings or wastes at milling sites should be such that ongoing active maintenance is not necessary to preserve isolation."

It is evident that remedial action designs are intended to last for a long time, without the need for active maintenance. Therefore, in accordance with regulatory requirements, the NRC staff has concluded that the goal of any design for long-term stabilization to meet applicable design criteria should be to provide overall site stability for very long time periods, with no reliance placed on active maintenance.

For the purposes of this STP, active maintenance is defined as any maintenance that is <u>needed</u> to assure that the design will meet specified longevity requirements. Such maintenance includes even minor maintenance, such as the addition of soil to small rills and gullies. The question that must be answered is whether longevity is dependent on the maintenance. If it is necessary to repair gullies, for example, to prevent their growth and ultimate erosion into tailings, then that maintenance is considered to be active maintenance.

2.1.4 Radon Release Limits

Titles 40 CFR 192.02 and 10 CFR Part 40, Appendix A require that earthen covers be placed over tailings at the end of milling operations to limit releases of radon-222 to not more than an average of 20 picocuries per square meter per second (pCi/m^2s), when averaged over the entire surface of the disposal site and over at least a one-year period, for the control period of 200 to 1000 years. Before placement of the cover, radon release rates are calculated in designing the protective covers and barriers for uranium mill tailings. Additionally, recent regulations promulgated under the Clean Air Act require that release rates be directly measured following placement of the protective barriers.

Depending on the selected design configuration, it could be argued that some gullying and exposure of tailings would be permissible under this portion of the regulations. It should be emphasized, however, that if tailings are exposed and eroded, the extent of exposure, erosion, and spread of contamination would be very difficult to assess, thus making a determination of radiological releases very difficult. This also inevitably would lead to a loss of control, as defined in the aforementioned Section 1. EPA standards and NRC regulations require that the disposal strategy be designed to maintain control for 1000 years. Further, such exposure would not seem to meet other portions of the regulations, which suggest that long-term stability and isolation of tailings are primary goals. Therefore, the NRC staff has concluded that tailings should be controlled for long time periods, and that exposure or erosion of tailings should be prevented to the extent practicable by the design of the protective cover. Additional discussion of interpretations of regulations and requirements may be found in the NRC Management Position (NRC, 1989, see Ref. 4).

2.2 Design Considerations

Several long-term stability investigations (Nelson *et al.*, 1983, see Ref. 5; Young *et al.*, 1982, see Ref. 6; Lindsey *et al.*, 1982, see Ref. 7; and Beedlow, 1984, see Ref. 8) have verified EPA's conclusion that the most disruptive natural phenomena affecting long-term stabilization are likely to be wind and water erosion. These authors also discuss important considerations that must be factored into the overall reclamation plan. The staff has concluded that the considerations which will have the most impact on the design of a protective cover include: (1) selection of a proper design flood or precipitation event; (2) analysis of long-term erosion caused by gullying; (3) effects of flow concentrations and drainage network development, if a stable slope is not provided; (4) the effectiveness of vegetated covers in arid areas; (5) design approaches using the concept of permissible velocity; and (6) rock durability and capability to resist weathering effects.

A reclamation design also needs to address other considerations, such as groundwater protection, geotechnical stability, and radon releases. The cover design should not limit consideration only to wind and surface water erosion. It is possible that the placement of a cover with a gentle slope (for example, 0.005 or less) could result in a high rate of water infiltration through the cover. The decision to use a particular reclamation strategy should consider all the possible failure modes with respect to all applicable EPA and NRC standards. A systematic, integrated analysis may result in the use of some steeper slopes with rock armoring or the use of more than one type of cover system.

To provide guidance on review procedures in other areas of tailings reclamation, the staff developed "Standard Review Plan for UMTRCA Title I Mill

Tailings Remedial Action Sites." These procedures address several design areas and provide guidance on the use of integrated approaches to tailings management. Additionally, several regulatory guides and staff guidance documents are available to assist in the development of acceptable reclamation strategies for specific design areas, including groundwater protection, geotechnical stability, and radon considerations. These documents, including applicable regulations and standards, are summarized in Table 2, page 20.

2.2.1 Selection of Design Flood and Precipitation Event

The design flood or precipitation event on which to base the stabilization plan should be one for which there is reasonable assurance of non-exceedance during the 1000-year design life. An event with an exceedance probability of 0.001 per year (return period of 1000 years or as commonly termed, "the 1000-year flood") would have a 63 percent chance of being equaled or exceeded during the 1000-year design life and clearly would not meet the reasonable assurance test. It is clear that events with much lower exceedance probabilities are needed to provide reasonable assurance. However, there is no reliable way of statistically estimating flood probabilities of 0.001 per year or less (Office of Water Data Collection, 1986, see Ref. 9).

An alternate approach is to choose a design event that is based on site-specific extreme meteorological and hydrological characteristics. The probable maximum flood (PMF), as defined and discussed by the Army Corps of Engineers (USCOE, 1966, see Ref. 10), and the probable maximum precipitation (PMP), as defined and discussed by the American Meteorology Society (AMS, 1959, see Ref. 11), are events of sufficiently low likelihood that the NRC staff concludes that there is reasonable assurance that larger events will not occur during the 1000-year design life. Therefore, the staff accepts the use of these events as design events for a stabilization plan. However, other flood and precipitation events may be used for 1000-year designs, if proper justification is provided showing reasonable assurance of non-exceedance during the 1000-year design period.

If a design period of less than 1000 years (but at least 200 years) is used, events less severe than the PMF and PMP may also be used. In order to justify such lesser events, it must be shown that: (1) designing for the PMF and PMP is impracticable; (2) the design event is the most severe that can be practicably designed for; and (3) the design will be effective for at least 200 years. In addressing the third point, the minimum flood event that the staff will accept is the Standard Project Flood (SPF), as defined and discussed by the USCOE (USCOE, 1964, see Ref. 12), <u>or</u> the maximum regional flood of record (transposed to the site on a discharge per unit drainage area basis), <u>whichever is greater</u>. In general, the SPF will have a magnitude of approximately 40 to 60 percent of the PMF (USCOE, 1964, see Ref. 12). In areas where specific procedures for estimating a SPF have not been derived, the staff will accept 50 percent of a PMF as representing a SPF. Regional floods of record may be determined using references such as Crippen and Bue (1977, see Ref. 13).

NRC staff acceptance of the regional flood of record as a minimum design flow is based on examination of historic flood flow data. Staff reviews have indicated that statistical techniques may grossly underestimate the magnitude of rare floods. This occurs principally because sufficient data records are not available, for many small streams, to reasonably estimate floods of such long recurrence intervals. It therefore becomes necessary to expand the data base, using regional data. Additional discussion of this issue may be found in Appendix E.

2.2.2 Gully Erosion

A serious threat to stability at any given site is likely to be gully erosion resulting from concentration of runoff from local precipitation. To ensure long-term stability, it is important to control localized erosion and the formation of rills and gullies. Research performed for the NRC staff (Nelson, et al., 1983, see Ref. 5) has demonstrated that if localized erosion and gullying occurs, damage to unprotected soil covers may occur rapidly, probably in a time period shorter than 200 years. Additionally, since gully development occurs more rapidly on immature surfaces (reclaimed impoundments are relatively recent additions to the normal landscape), it should be assumed that the reclaimed cover is more vulnerable to gully erosion than in-situ materials (Nelson, et al., 1983, see Ref. 5). Therefore, a proposed cover design should ensure that stable slopes that minimize the potential for gully erosion are provided.

Gully erosion differs somewhat from other design considerations because gully growth and erosion will be cumulative and progressive with succeeding storms. Over a long period of time, the cumulative effects of smaller, more frequent flood events could exceed the effects of larger, less frequent events. All these events combined could erode an unstable slope in a manner that could expose or release tailings to the environment before a stable slope is formed.

The NRC staff, therefore, considers that the best method for providing long-term stability is to provide permanently stable slopes that prevent gully initiation during the occurrence of a single, very large, design event. By designing for such a large single event, it is expected that smaller, continual events will have little or no cumulative impact on stability, due to the overall flat slopes necessitated by designing for the rare event.

2.2.3 Flow Concentrations and Drainage Network Development

It is unlikely that evenly-distributed sheet flow will occur from the top to the bottom of a slope. The flow concentration could be initiated by differential settlement of the cover or waste material, abnormal wind erosion, and/or random flow processes. Recent studies (Oak Ridge National Laboratory, 1987, see Ref. 14) performed for the NRC staff have indicated that areas of flow concentration will develop randomly even on carefully-placed and compacted slopes, due to normal flow processes and flow spreading. Such flow concentrations can result in the formation of rills and gullies, and eventually, a complete drainage network can be expected to form on unstable slopes (Schumm and Mosley, 1973, see Ref. 15; Ritter, 1978, see Ref. 16). Network development and the tendency of rills and streams to widen, deepen, extend their length, and capture other rills and streams are discussed by Ritter (1978, see Ref. 16) and by Shelton (1966, see Ref. 17).

Recognizing that drainage network development will eventually occur on unstable slopes, the NRC staff concludes that it is necessary to provide slopes that are flat enough or sufficiently protected to prevent the formation of extensive rills and gullies. Such slopes should be capable of providing protection against tailings exposure, assuming the development of a complete drainage network and the occurrences of many rainfall events to be expected over the design life of the cover system. Such phenomena are considered and evaluated in the design of sacrificial slopes, discussed in Appendix B.

It is expected that a significant increase in the drainage area could occur on an unstable slope over a long period of time. For that reason, any slopes that are designed to permit controlled gullying should be designed using a larger drainage area that would be initially expected. If a slope is designed to be stable, no significant increases in drainage area should be expected. However, it should be emphasized that only very gentle slopes may be assumed to be stable.

2.2.4 Effectiveness of Vegetative Covers

Vegetative covers reduce the potential for erosion because they protect the surface from raindrop impact, reduce the amount of water available for runoff because of evapo-transpiration, and increase the surface roughness, which, in turn, decreases runoff velocity. Plant roots also help bind the soil and keep it in place. Evapo-transpiration also reduces infiltration of water into the tailings.

Based on the results of several studies (Nelson, et al., 1983, see Ref. 5; Lindsey, et al., 1982, see Ref. 7; and Beedlow, 1982, see Ref. 18), it is unlikely that a vegetative cover for long-term erosion protection can be effective on steep embankment slopes in some arid portions of the western United States, where the natural vegetation cover is less than about 30 to 50 percent. However, self-sustaining vegetation may provide some amount of long-term stabilization in some semiarid to humid climates, provided that the slopes are sufficiently flat.

Based on the results of several other studies (Beedlow, 1982, see Ref. 18; Voorhees, 1983, see Ref. 19; Temple, 1987, see Ref. 20), it appears that significant erosion protection is afforded by vegetation only when the climate is capable of supporting a relatively dense growth of grasses. In general, semi-arid climates where only certain types of shrubs and forbs grow readily do not provide adequate vegetative cover to permit credit to be taken for reduction in shear forces. Many of the uranium mills are located in these semi-arid sections of the western United States, where sustaining a vegetative cover over a long period of time may be questionable. Therefore, if licensees wish to take credit for vegetation, they need to substantiate that a vegetative cover will be sufficiently dense to be effective in minimizing erosion.

Copeland (1963, see Ref. 21) compared the percent of vegetative cover with cumulative overland flow and with eroded soil. The results indicated that no less than 70 percent vegetative cover is required to reduce flow to a point of stability. The cumulative values of overland flow and eroded soil increase sharply as the vegetative cover decreases from 70 to 30 percent. Therefore, in order to take credit for vegetation, licensees need to substantiate that the density of the grass cover will be significantly greater than 30 percent and preferably 70 percent. Since the density of a vegetative cover is unlikely to substantially exceed the density of naturally-occurring vegetation, the staff expects that a licensee will base the 30 to 70 percent density determination on naturally-occurring vegetation in the general site vicinity. This can be done by visually surveying existing vegetation in areas adjacent to the site.

If revegetation is considered at any site, it should be based on past research and current practices in the site area. The vegetation species should be indigenous to the area, and provisions should be made to enhance growth during the initial growing season. Sufficient top soil should be placed over the radon cover, since the radon cover is usually compacted cohesive soil that may not be favorable for plant growth. Studies should be conducted to determine the capability of the vegetation to survive over long periods of time. Local experts should be consulted to determine what vegetation species are appropriate for a particular area and for the local soil type. Other considerations, such as vegetation succession, droughts, and extreme climatic conditions should be evaluated to assess the ability of the vegetation to survive over long periods of time. Based on research performed for the NRC staff (Nelson, et al., 1983, see Ref. 5), it is unlikely that the density of a vegetation cover will substantially exceed the density of naturally-occurring vegetation.

2.2.5 Use of Permissible Velocity and Tractive Force Methods

Two methods are generally used for designing stable channels. These are the permissible velocity method and the tractive force (shear stress) method. Flow in an open channel is extremely complex and is influenced by many variables. Therefore, both of these methods should be considered in designing a stable channel.

The use of the method of permissible velocity has widespread use in the design of stable channels to prevent erosion. Such methods are well-documented by Chow (1959, see Ref. 22) and others in determining the maximum mean velocity that a particular channel section can withstand. Unfortunately, this method is sometimes misused to design a stable slope, because the method was intended to apply principally to irrigation and drainage channels, where flow depths are usually greater.

In an open channel, flow velocities vary vertically along the channel section. Generally, the maximum velocity occurs just below the free surface. The velocity decreases with depth, reaching a minimum value near the channel bottom. Consequently, the permissible velocity along the channel bottom is much less than the maximum mean velocities. In designing stable slopes and in considering flow on tailings pile slopes, the flow will generally be only several inches deep, and the flow velocity along the slope will be essentially equivalent to the velocity occurring along the top surface of flow. Therefore, the maximum permissible design velocity for shallow flows must be less than for flows occurring at greater depths. Chow provides reduction factors for the permissible velocity, based on the flow depth. It can be seen that the permissible velocity decreases noticeably at lower depths of flow. If Chow's data are extrapolated to a flow depth of several inches, the recommended reduction in permissible velocity is about 50 percent.

For the design of unprotected soil slopes with shallow flow depths, the staff recommends the use of the tractive force (or shear stress) method. In this method, the tractive force produced by the flow is compared to the allowable tractive forces of the soil. Since the allowable tractive force is not dependent on the depth of flow, methods exist where this value can be directly determined or computed. Such methods are discussed by Temple, et al. (1987, see Ref. 20) and in more general terms by Chow (1959, see Ref. 22). The calculated tractive force produced by the flow is easily computed, after the depth of flow has been determined.

For the design of vegetated slopes, the staff recommends that the permissible velocity method be used, in addition to the shear stress method, to verify the adequacy of the design. In using this method, the selection of appropriate maximum permissible velocities and Manning's "n" values are of utmost importance. The staff recommends that guidance provided by Chow be used to determine these parameters. Chow provides recommended maximum permissible velocities for various types and densities of grass covers. Chow further recommends that permissible velocities exceeding five feet per second (fps) be used only for uniform stands of grass which will receive proper maintenance. Therefore, the maximum velocity that should be used (where no credit can be taken for maintenance) is five fps. This permissible velocity should also be further reduced for shallow flow depths, as previously discussed. Comparisons should then be made between the results obtained using both the shear stress and permissible velocity methods.

2.2.6 Rock Durability

Because tailings and their covers must remain stable for long periods of time, cover protection such as rock must also survive natural weathering for that length of time. Considerable engineering judgment is necessary to develop rational engineering design alternatives when weathering of rock materials is a major consideration. Any rational design method to determine the size and thickness of cover protection should include the durability and weathering characteristics of the material over time.

The technical basis for using rock for long time periods is well-developed. Jahns (1982, see Ref. 23) points out that many kinds of rocks are relatively resistant to weathering. Most of these more resistant rock types have long been used as construction materials, in monuments, or for decorative purposes, with varying degrees of success. However, it must be recognized that there are limitations associated with procedures that are used to assess rock performance for a 1000-year period.

Determining the quality of riprap needed for long-term protection and stability can therefore be a somewhat difficult and subjective task. Very little design guidance is available to assess the degree of oversizing needed for a particular rock type to survive for long periods, based on its physical properties.

In assessing the long-term durability of erosion protection, the NRC staff has relied on the results of durability tests performed at several uranium mill sites and on information and analyses developed by technical assistance contractors, which provide methods for assessing rock oversizing requirements to meet long-term stability criteria. These procedures have also considered actual field data from several sites and have been modified to provide flexibility to meet construction requirements.

3. REGULATORY POSITION

In accordance with 40 CFR 192, Subparts A, B, and C, and 10 CFR Part 40, Appendix A, the design of protective covers should provide reasonable assurance of long-term stability. The design should provide for control of tailings for 1000 years, if reasonably achievable, but, in any case, for at least 200 years.

Several methods have been developed for designing unprotected soil covers or soil covers with some vegetation, to prevent the development and inhibit the growth of gullies. These methods, illustrated in Appendix A to this STP, are based on staff licensing and review experience and applicable hydraulic engineering principles. The computational procedures outlined in Appendix A were developed based on NRC staff experience with damage to erosion-protection structures during the occurrence of relatively minor storm events. Of necessity, these procedures attempt to account for the limited quantitative data base available to document long-term degradation and the questionable ability of vegetated soil covers to be effective in arid areas. Reasonable and conservative engineering judgment has been used, after evaluating the results of the various methods, to decide on the best estimate of the stable slope.

Methods are also presented for the design of sacrificial soil slopes (Appendix B), for evaluation of feasibility of covers (Appendix C), and for the design of riprap (Appendix D).

The aforementioned design procedures are concerned only with surface water erosion of the cover. The additional soil cover needed to account for wind erosion or sheet erosion needs to be factored into the soil cover design. Procedures discussed by Nelson, et al. (1986, see Ref. 24) may be used to determine the additional cover requirements.

In designing a protective cover, there are many options and design combinations that may be used. There are, in fact, an infinite number and variety of designs, and their selection will depend on site-specific conditions and phenomena. In general, however, cover designs fall into several broad categories. Based on NRC licensing experience with Title I and Title II sites, various options are normally employed to design cover systems:

Option 1 — Soil covers designed to be stable for 1000 years.

Option 2 — Combinations of soil covers on the top slopes and rock-protected soil covers on the side slopes, both designed to be stable for 1000 years.

Option 3 — Soil covers totally protected by a layer of rock riprap on both the top and side slopes.

Option 4 — Sacrificial soil covers designed to permit controlled erosion.

Option 5 — Designs that are not able to meet the minimum long-term stability requirement of 200 years. Such designs may be exempted under Section 84(c) of the Atomic Energy Act (see Ref 25) for Title II sites and under the supplemental standards of 40 CFR Part 192 for Title I sites. Such exemptions may be granted, based on licensee justification of inability to meet primary regulations.

The preferred options to design a cover system are Options 1,2, and 3; such designs will be stable and will be effective for a 1000-year period. Option 4 is not considered to be a preferred design option; this option should be used only when detailed justification can be provided to demonstrate that designing for time periods greater than 200 years is not reasonably achievable.

Option 1 can generally be implemented only for very short slope lengths, or where significant credit can be given for vegetation. Discussion of unprotected stable soil covers may be found in Section 3.2.1, p. 14; design guidance may be found in Appendix A.

Option 2 may be implemented if Option 1 is impractical due to pile height, size, or topography. In these cases, combinations of stable soil covers over flatter areas and rock-protected soil covers over steeper areas should be considered as possibilities in meeting the 1000-year stability requirement. Discussion of combination covers may be found in Section 3.2.2, p. 16. Design guidance may be found in Appendix A (for soil top slopes) and in Appendix D (for rock-protected side slopes).

Option 3 may be implemented in those cases where rock riprap is available. The placement of riprap protected covers is considered by the NRC staff to be the most effective method of assuring long-term stability. Discussion of riprap cover design is provided in Section 3.3, p. 17. Design guidance may be found in Appendix D.

Option 4 may be implemented if providing combined stable soil top slopes and/or rock-protected side slopes is not practicable or is excessively costly. In such cases, sacrificial side slopes that permit controlled erosion may be acceptable, provided that the tailings will not be exposed or eroded. In general, this option should be considered only when tailings are not placed directly under the soil slope. The staff considers that such designs should be adopted only when licensees or the U. S. Department of Energy (DOE) can provide detailed justification that designing for a 1000-year stability period is not reasonably achievable and that designing for a 200-year period is the only reasonably achievable design option. Discussion of sacrificial side slopes, where tailings are not placed under embankment outslopes, may be found in Section 3.2.4, p. 16; design guidance may be found in Appendix B. Discussion of the detailed justification needed to demonstrate that other designs are not reasonably achievable may be found in Appendix C.

Option 5 may be implemented in those cases where designing for a 200-year stability period is not reasonably achievable. Where DOE or licensees can document the clear impracticability of such designs, they will be considered on a case-by-case basis, considering the possibility of alternatives under Section 84(c) of the Atomic Energy Act for commercial processing sites, or under the supplemental standards of 40 CFR Part 192, for inactive sites.

For the convenience of licensees and designers, Table 1, "Summary of Design Guidance," may be used to direct attention to appropriate sections of this STP and to provide guidance in the design of various features, according to the design option selected.

Table 1

Summary of Design Guidance

Option	Item	Discussion		Design Procedures	
		Section	Page	Section	Page
1. Soil Covers 1000 Years No Rock	Top Slopes	3.2.1	14	A.2	A-2
	Side Slopes	No Methods Available for Steep Slopes Using Unprotected Soil			
	Swales/Channels	3.2.2	16	A.3	A-15
2. Combinations of Soil + Rock	Soil Covers	(See Option 1.)			
	Rock Covers	(See Option 3.)			
3. Rock Covers	Top and Side Slopes	3.3.1	17	D.2	D-1
	Diversion Channels	3.3.2	17	D.3	D-7
	Outlets/Aprons	3.3.2	17	D.4	D-16
	Streambanks	3.3.3	17	D.5	D-20
	Rock Durability	3.3.4	18	D.6	D-23
	Rock Placement	3.3.5	18	3.3.5	18
4. Sacrificial Soil Covers 200 Years	Top Slopes	3.2.1	14	A.2	A-2
	Side Slopes	3.2.4	16	B.3	B-2
	Justification	3.2.4	16	App.C	C-1
5. Exemption	Various designs used -- Licensee must justify --NRC Staff will review on case-by-case basis.				

3.1 General Information Submittals

For the cover design selected, the following engineering data, information, and analyses should be provided for NRC staff review:

a. Drainage areas of principal watercourses and drainage features

b. Drainage basin characteristics, including soil types and characteristics, vegetative cover, local topography, flood plains, morphometry, and surficial and bedrock geology

c. Maps and/or aerial photographs showing the site location and the upstream drainage areas

d. Site geomorphological characteristics, including slopes, gradients, and processes

e. Drawings and photographs of site features

f. Location, depth, and dimensions of tailings and proposed soil cover, including results of subsurface explorations

g. Physical and engineering properties of the proposed soil cover and radon suppression cover, tailings, and foundation materials, including results of laboratory and field tests, including dispersivity and permeability data of the radon cover

h. Radiological parameters, including activity and emanating coefficient of contaminated material

i. As applicable, pertinent construction records of the tailings retention system, including as-built drawings, construction control tests, construction problems encountered, any alterations or modifications that were necessary, and the history of needed maintenance and repair

j. Principal design assumptions and analyses for the protective cover, including hydrologic, geotechnical, hydraulic, and stability analyses

3.2 Cover Design Criteria

The following are specific design considerations and criteria for developing cover designs.

3.2.1 Design of Stable Soil Covers for Top Slopes

In general, it is expected that soil covers will be practical only on the flatter top slopes of a reclaimed impoundment. Exceptions may occur to this generalization, where slope lengths are very short, where significant credit can be given for vegetation (such as in the eastern United States, where good grass covers can be established); where rocky soils are available to increase average soil particle size, and thus increase stability; or where some gullying of sacrificial slopes is acceptable. As discussed in Appendix A, in situations where licensees or DOE can substantiate that vegetation will be self-sustaining and sufficiently dense to reduce erosion potential, procedures such as those given by Temple (1987, see Ref. 20) may be used to evaluate the effectiveness of the vegetation. It is unlikely that soil covers alone will be capable of providing long-term stability on slopes steeper than a few percent in the semi-arid western United States. Therefore, it will usually be necessary to provide stable soil slopes on the top and rock-protected (or sacrificial) slopes on the steeper sides of a reclaimed pile.

Soil slopes of a reclaimed tailings impoundment should be designed to be stable and thus inhibit the initiation, development, and growth of gullies. The slopes should be designed for an occurrence of the most severe precipitation event reasonably expected during the design life; because of the problems associated with extrapolating limited data bases using statistical methods, the staff concludes that use of the PMP/PMF will provide an acceptable design basis. The slope design should also consider the effects of flow concentrations and drainage network development, because such phenomena cannot be realistically discounted, even on perfectly-constructed slopes (Schumm and Mosley, 1973, see Ref. 15; Ritter, 1978, see Ref. 16). Specifically, soil covers are acceptable if they are designed to be stable and if the shear stresses and flow velocities produced by concentrated runoff from design-basis flood events are less than the allowable shear stresses and velocities of the soils. See Appendix A for additional discussion and for methods of designing stable soil covers.

In addition to having a slope that is shown by analyses to be stable, the soil cover should be designed to be thick enough so that there is reasonable assurance that tailings will not be exposed and that radiological criteria will be met, considering the combined effects of wind erosion, sheet erosion, and minor rill and gully erosion. Acceptable methods of analysis are provided by Nelson, et al. (1986, see Ref. 24) for computing the additional soil cover needed to protect against wind erosion and sheet erosion; such methods include the Modified Universal Soil Loss Equation (for sheet erosion and minor rill erosion) and the Chepil Equation (for wind erosion).

For any locations on the tailings pile where the required criteria cannot be met using soil covers alone (such as the steeper side slopes), use of rock riprap will provide an acceptable design. Discussion of rock covers is found in Section 3.3, p. 17. Guidance for design of rock covers is provided in Appendix D.

3.2.2 Design of Swales on Unprotected Soil Slopes

In some cases, it may be possible to direct concentrated surface runoff over unprotected soil covers, using very flat ditches or swales. As discussed in Section 2.2.5, p. 8, the NRC staff recommends that both the tractive force and permissible velocity methods be used to determine the size and maximum slope of such swales. The design of swales using these methods is very simple and straightforward, and guidance is presented in Appendix A, Section 2.

3.2.3 Design of Stable Slopes Using Combinations of Soil Covers for Top Slopes and Rock Covers for Side Slopes

In most cases where slope lengths are relatively long and where vegetation cannot be shown to be effective, the stable soil cover required over a large area of tailings may need to be so flat that it is not economically feasible to construct. In those cases, it may be acceptable to use combinations of soil covers and rock covers to provide the necessary protection.

A hypothetical example of such a design may be to provide soil slopes of 0.8 percent on the top of a 300-foot-long pile for the first 250 feet and 20 percent riprap-protected side slopes for the remaining 50 feet. If such a composite design is implemented, the Horton Method discussed in Appendix A may be used to design the stable top slopes; the Stephenson Method discussed in Appendix D may be used to estimate the side slope rock cover requirements.

3.2.4 Design of Sacrificial Slopes

The design of soil slopes that permit gullying of limited extent may also be acceptable if the total soil cover provided will prevent the release of radioactive materials. The basis for such designs is that more stable levels and slopes will eventually be formed during the selected design life, but the amount of cover material provided will prevent gully intrusion into the tailings.

If tailings or waste materials are not placed directly under the soil cover outslopes, the construction of such sacrificial soil outslopes may provide an acceptable design. In such cases, the outslope may erode, but sufficient cover protection will be provided so that tailings will not be exposed or eroded during the design life. Guidance for designing sacrificial outslopes is presented in Appendix B.

In general, the design procedures discussed in Appendix B are intended to apply for only a 200-year period, or less. Due to the lack of an extensive data base associated with gully erosion, sacrificial soil slopes that are expected to erode should be used only when the 1,000-year stability criterion cannot be reasonably met. In using this approach, licensees should clearly justify and document with pertinent analyses that designing for a 1,000-year stability period is not reasonably achievable and that the resulting design will be effective for a minimum of 200 years. A step-by-step procedure for providing such justification may be found in Appendix C.

3.3 Rock Cover Design Criteria

All portions of a reclaimed tailings impoundment should be designed to resist the effects of local intense precipitation. In many cases, where it is not feasible to provide unprotected soil covers or where vegetation is not likely to be effective, a rock riprap layer may be necessary to provide the required protection. The rock may be needed to protect: (1) the top and side slopes; (2) aprons, diversion channels, and channel outlets; and (3) other design features from the effects of offsite flooding. In arid portions of the western United States, where the effectiveness of vegetation may be questionable, the use of a rock cover of acceptable durability is considered by the NRC staff to be the preferred method for satisfying the long-term stability requirements of 40 CFR Part 192 and 10 CFR Part 40, Appendix A.

3.3.1 Top and Side Slopes

The design of rock riprap for the top and side slopes of a tailings pile is simple and relatively straightforward. Acceptable analytical methods for designing a rock cover to resist erosion and prevent gullying on the top and side slopes of a remediated embankment may be found in Appendix D to this STP.

3.3.2 Aprons / Diversion Channels / Ditch Outlets

Erosion protection for those locations where man-made stabilized slopes and channels meet natural slopes and channels should be designed to prevent headcutting and/or lateral erosion into the tailings. Flow velocities and concentrations produced by runoff on man-made slopes could also cause erosion of the natural soils just beyond the toe of the stabilized slope, particularly if the slopes are steep. It is necessary, therefore, to provide a transition section where those conditions exist, which serves to reduce velocities to non-erosive levels. These flatter transition sections, normally called aprons, also need to be designed to prevent upstream headcutting by existing gullies in the area of the pile toe. The apron or transition area may be designed using design procedures similar to these for other engineered slopes. Guidance for designing aprons and toes may be found in Appendix D, Section 4. Acceptable methods for designing erosion protection of diversion ditch outlets may also be found in Appendix D.

3.3.3 Design of Rock Covers to Resist Flooding by Nearby Streams

The slopes of a reclaimed tailings pile or waste disposal facility should be protected from the effects of flooding of nearby watercourses. If the pile is located near a large stream, and if floods impinge on the pile slopes with erosive velocities, rock riprap erosion protection should be provided to resist the stream velocities and shear stresses produced by such flood events.

Regulatory Guide 1.59, "Design Basis Floods for Nuclear Power Plants," (see Ref. 26) provides guidance for the determination of peak flood flows for large streams. HEC-2 (USCOE, 1976, see Ref. 27) may be used to compute water surface profiles and local velocities. Guidance for the design of riprap for river and channel banks is discussed by Walters (1982, see Ref. 28), the USCOE (1970, see Ref. 29), and Nelson, et al. (1986, see Ref. 24).

3.3.4 Rock Durability

Frequently, situations arise where it may be necessary to use marginal-quality rock for erosion protection. These situations may arise in areas of the western United States where many uranium mill sites are located. Where rock riprap is proposed for erosion protection, investigations should be conducted to identify sources of available rock within a reasonable distance of the site. The suitability of these rocks as protective covers should then be assessed by laboratory tests, to determine the physical characteristics of the rocks. Several durability tests, such as those listed in Appendix D, should be performed to determine if the rock is suitable for use as riprap.

Where rock of good quality is reasonably available, the riprap design should incorporate this rock. In those cases where only rock of marginal quality is reasonably available, increases in the average rock size and riprap layer thickness may be necessary. An acceptable procedure for evaluating rock quality and for using marginal-quality rock may be found in Appendix D. If rock does not meet the minimum quality ratings established in the scoring procedure in Appendix D, it will generally not be acceptable. However, the use of such rock will be considered on a case-by-case basis, if no other rock is available, or if no other design options are reasonably feasible.

3.3.5 Rock Placement

It has been the experience of the NRC staff that it may be difficult to achieve proper placement of riprap layers, particularly when the rock sizes are large relative to the layer thickness. It is relatively easy to adequately place a 12-inch layer of 2-inch rocks, for example, but it is much more difficult to place a 12-inch layer of 8-inch rocks.

The proper placement of rock riprap in ditches and on embankment slopes is necessary to dissipate the energy associated with flowing water and thus prevent erosion that could lead to gullying and exposure of contaminated material. In general, such proper placement is created by providing a relatively uniform thickness of rock at the specified gradation.

Following are general guidelines that should be used to achieve adequate placement of rock riprap layers:

1. Riprap should be placed in a layer thickness that is at least 1½-2

times the average rock size (D_{50}). If care is used in placing the riprap layer, such as using specialized equipment or rearranging individual large rocks by hand, a thickness of 1½ times D_{50} is acceptable.

2. Where the D_{50} size is eight inches or more, the placement procedures should include a certain amount of individual rock placement (using specialized equipment or hand labor) to ensure that proper thicknesses and areal coverage are achieved. Where the D_{50} size is less than 8 inches and the layer thickness exceeds two times the average rock size, dumping and spreading by heavy equipment will generally be the only procedures necessary to achieve adequate rock placement.

3. After the start of construction of the riprap layer, a test section of the proper thickness and gradation should be constructed. This test section should be visually examined, and contractor personnel should become familiar with the visual properties of this section; that is, the acceptable section should be used as visual guidance of proper placement and should be used to evaluate future riprap placement. This section should be tested to determine its gradation and rock weight/unit volume that will be achieved in future rock placement activities. Weight and gradation tests may be needed at any locations where the rock placement does not appear to be adequate, based on visual examinations, or if difficulties are experienced during rock production or placement. These visual examinations should be performed by a person experienced in rock placement and inspection.

Table 2

Available Guidance in Other Uranium Mill Tailings Review Areas

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A. RADON ATTENUATION

1. Standard Review Plan - Chapter 3 (see Ref. 30)
2. Regulatory Guide 3.64 (see Ref. 31)
3. NUREG/CR-3533 (see Ref. 32)

B. GROUNDWATER PROTECTION

1. Standard Review Plan - Chapter 4 (see Ref. 30)
2. NRC Staff Technical Position (see Ref. 33)

C. GEOTECHNICAL ENGINEERING

1. Standard Review Plan - Chapter 2 (see Ref. 30)
2. NRC Staff Technical Position (see Ref. 34)

Regulations and standards in each of these review areas are given in 10 CFR Part 40, Appendix A and in 40 CFR Part 192.

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APPENDIX A

DESIGN OF SOIL COVERS

CONTENTS

APPENDIX A

DESIGN OF SOIL COVERS

1. INTRODUCTION

Because regulations require that tailings remain stable for very long time periods, and because of the limited amount of performance data available for soil slopes, it is necessary to exercise caution in their design. Such designs should be based on the premises that: (1) unconcentrated sheet flow is not a realistic assumption, and there will always be some random flow spreading and/or flow concentrations as flow progresses down embankment side slopes; (2) phenomena such as differential settlement and wind erosion can cause uneven surfaces that provide pockets for erosion and preferential flow paths to occur on a slope; and (3) freezing/thawing of the soil cover can cause deterioration and damage (e.g., frost heave) to slopes, thus producing areas prone to the formation of concentrated flow.

The recent management position developed by the U. S. Nuclear Regulatory Commission (NRC) staff (USNRC, 1989, see Ref. A1) provides guidance in the selection of the design flood and the level of conservatism needed in designing tailings covers. In general, the position calls for use of reasonable conservatism in those areas that are not well-understood; however, extremely conservative values of design parameters are not to be used. In those areas where the phenomena are well-understood or where the range of design parameters is relatively narrow, typical or average values may be used in design. For the design of soil covers, there are several design parameters that are not well-understood, such as flow concentrations, effectiveness of vegetation as erosion protection, allowable stresses or velocities, roughness of the cover when flow depths are small, and other miscellaneous problems that could occur over a period of 1000 years.

The NRC staff has therefore concluded that the slope of a soil cover should be one that is stable and will: (1) minimize the potential for development and growth of a gully over a long period of time, assuming that flow concentrations occur; and (2) prevent the erosion of tailings due to gullying.

2. DESIGN OF UNPROTECTED SOIL COVERS

2.1 Technical Basis

2.1.1 Horton/NRC Method

Horton (1945, see Ref. A2) determined that an area immune to erosion existed adjacent to a watershed divide. The distance from the watershed divide to the point down the slope at which erosion will occur was termed the critical distance, xc. At this point the eroding force becomes equal to the soil resistance. The following expression was developed by Horton to determine the critical distance:

$$xc = \frac{65\ R^{5/3}}{qs\ n\ f(S)^{5/3}}$$

where:

xc = critical distance, feet

qs = runoff intensity, in inches/hour, corresponding to the computed time of concentration

n = roughness factor

R = soil resistance, lb/ft^2

f(S) = slope function = $\frac{\sin x}{\tan^{.3} x}$

where:

x = slope angle in degrees.

If the following substitutions are made, the stable slope (S_s) can be determined:

$Ss = \sin x = \tan x$, for small values of slope;

$t = R$ = allowable shear stress (pounds per square foot);

$P = qs$ = design precipitation intensity (inches/hour); and

$L = xc$ = slope length (feet).

A flow concentration factor (F) is used in the equation to account for imperfections in the slope and is multiplied by the rainfall intensity.

Therefore,

$$Ss^{7/6} = \frac{65\ (t)^{5/3}}{P\ L\ F\ n}$$

This equation may also be derived by simultaneous solution of the Manning Equation, the peak shear stress formula, and the Rational Formula.

Use of this equation allows direct solution of the value of the stable slope necessary to prevent the initiation of gullying. The slope thus determined represents the maximum slope that can be provided to minimize the potential for gully initiation due to the occurrence of one single intense rainfall event, and thus should also minimize erosion due to a series of less intense storms to be expected over a period of 200 to 1000 years.

Temple, et al. (1987, see Ref. A3) and Chow (1959, see Ref. A4) discuss methods for determining allowable shear stresses and recommend that the shear stress method be applied to design a stable section. The shear stress method is often used to assess the size and slope of channels needed to maintain stability. Data are available to estimate permissible shear stresses for various types of soils and various ranges of vegetative cover (Temple, et al., 1987, see Ref. A3). (Also, see discussion in Section 2.2.5, p. 8, in the main section of this position.)

It is expected that the use of this method will result in relatively flat slopes for achieving long-term stability. Basic hydraulic design principles indicate that the resulting slopes are likely to be flat enough to achieve subcritical flow, even if small rills and channels are formed on the embankment slope. The staff concludes that the resulting subcritical flow regimes that are formed will generally not result in severe erosion of a tailings cover, even if a gully is formed, based on an examination of standard bed load equations and sediment transport models (Chow, 1959, see Ref. A4; Fullerton, 1983, see Ref. A5).

2.1.2 Permissible Velocity Method

Use of the permissible velocity method is discussed in detail by Chow (1959, see Ref. A4). The method is widely used to design stable channel sections, both for cohesive and non-cohesive soils.

The staff does not favor the use of this method, due to the potential for misuse when applied to design stable slopes or any application other than channel design. However, if properly applied, there is no reason for rejecting its use. The most common misuse is the failure to reduce the permissible velocity if the depth of flow is relatively shallow (less than 3 feet). As stated by Chow:

"When other conditions are the same, a deeper channel will convey water at a higher mean velocity without erosion than a shallower one. This is probably because the scouring is caused primarily by the bottom velocities and, for the same mean velocity, the bottom velocities are greater in the shallower channel..."

It can be seen that reductions to the permissible mean channel velocity are needed to reflect the velocities that are to be used for slope designs, where the depths of flow are shallow. Chow has published correction factors, based on the depth of flow. Abt and Hogan (1990, see Ref. A6) have determined that such corrections are appropriate, based on an examination of the original data and based on hydraulic theory.

Additionally, Chow recommends that the maximum permissible velocity for grassed channels be limited to 5 feet per second. This limit is necessary because no credit may be taken for active maintenance in designing for long-term stability. Further discussion may be found in Section 2.2.5, p. 8, in the main section of this position.

2.2 Procedures

Procedures have been developed to (a) use the allowable shear stress method, as modified and developed in the Horton/NRC Method and to derive input parameters to the aforementioned stable slope equation and (b) use the permissible velocity method. These procedures provide two acceptable methods for designing stable soil covers. It is recognized that in many cases, specific values of parameters may be difficult to justify. In those cases where licensees can justify values of individual parameters, to be used in the equation, that depart from the values given by suggested references, the resulting designs will be considered on a case-by-case basis.

Step-by-step procedures for implementing (a) the allowable shear stress method and (b) the permissible velocity method are presented below:

Step 1. (a) Maximum allowable shear stress may be determined using procedures developed by Temple, et al. (1987, see Ref. A3) or Chow (1959, see

Ref. A4). The staff considers Temple's method to be a more accurate method for determining shear stresses because it is related to the Unified Soil Classification System and can be applied for specific soil types and degrees of cohesiveness. In general, the Temple procedure for determining allowable shear stress (tractive force) for sites where vegetation effectiveness is questionable is based primarily on the soil particle size and the soil cohesiveness. The amount of resistance for granular non-cohesive soils, including rocky soils, is principally a function of the D_{75} grain size, where the allowable tractive force is equal to 0.4 x D_{75} (Temple, et al., 1987, see Ref. A3). For granular soils, the increase in shear resistance due to cohesiveness is minimal. For cohesive soils where the particle size is smaller, the amount of resistance is principally a function of the soil cohesiveness and not the particle size. In those locations where a vegetation cover can be effective, procedures are discussed for determining the increases in allowable shear forces. Additional guidance and need for justification of design parameters for vegetation covers are discussed in Section 2.2.4, p. 7, in the main section of this position.

(b) Maximum permissible velocity may be estimated using data provided by Chow (1959, see Ref. A4), who has published values of permissible velocity for stable channel sections. These velocities need to be reduced, as discussed in Step 6(b).

Step 2. (a) or (b) Determine slope and slope length to be considered, as developed in the preliminary reclamation design.

Step 3. (a) or (b) Determine flow concentration factor (F). Documentation of the occurrence of flow concentrations and the ability of an individual rock or soil particle to resist given flow rates is discussed further by Abt, et al. (1987, see Ref. A7). The actual value of F will depend on several factors, including grading practices during cover construction, cover slope, and potential for differential settlement. The staff recommends a default value of 3, for most soil slopes; other values may be used, if properly justified.

Step 4. (a) or (b) Estimate Manning's 'n' value using general procedures given by Temple, et al. (1987, see Ref. A3); by Nelson, et al. (1986, see Ref. A8); or by Chow (1959, see Ref. A4). If a channel or slope is heavily vegetated, increases in flow resistance can be determined, using quantitative procedures developed by Temple, et al. (1987, see Ref. A3).

Step 5. (a) or (b) Determine the rainfall intensity using the procedures given by Nelson, et al. (1986, see Ref. A8, Section 2.1.2)

(b) Determine the peak runoff rate, using the Rational Formula.

Step 6. (a) Solve for stable slope, using the Horton/NRC equation. If the computed slope is different from that assumed, return to Step 2 with new values of slope and/or slope length.

(b) (1) Determine the flow depth (y) by solving the Manning Equation for normal depth on a one-foot-wide strip. This equation can be solved directly in this case using the following derivation:

$$y^{5/3} = Qn / (1.486\ S^{1/2}).$$

(b) (2) Determine the permissible velocity for the slope, based on the computed depth of flow. Chow has developed correction factors that may be applied to determine the permissible velocity. The permissible velocity is multiplied by the following correction factors, depending on the depth of flow.

Depth of Flow (ft)	Correction Factor
3.0 or greater	1.0
1.9	0.9
1.0	0.8
0.65	0.7
0.4	0.6
0.25 or less	0.5

(b) (3) For the assumed one-foot-wide strip, determine the actual flow velocity (Va) by dividing the discharge by the flow depth:

$Va = Q/y$.

If this velocity is greater than the permissible velocity computed in Step 6(b)(2), return to Step 2 with new values of slope and/or slope length.

2.3 Recommendations

Recommendations are discussed in Section 2.2, p. A-5, for various steps of the design procedure. Particular attention should be given to determining allowable shear stress values and permissible velocities, since these parameters are likely to be the most sensitive parameters in the calculations.

Use of the procedures given by Temple could result in very high values of allowable shear stress, if the vegetation is assumed to be relatively dense. The values could be in the range of 2 to 4 lb/ft^2, approximating the degree of protection provided by 6 to 12-inch riprap. The staff considers it unlikely that an non-maintained vegetated slope will provide the same degree of protection that is provided by riprap. Therefore, the staff recommends that the design procedure be checked using the permissible velocity method, using a maximum permissible velocity of 5 feet per second (or less, if the flow depths are shallow).

Dispersive clay soils should not be used, since they may be susceptible to rapid erosion and slaking.

2.4 Examples of Procedure Application

2.4.1 Stable Slope on Unprotected Soil

For a site located in northwest New Mexico with a slope length of 1000 feet, the stable slope of an unprotected soil cover may be computed using (a) the shear stress method and (b) the permissible velocity method, as follows:

Step 1. (a) The allowable shear stress may be estimated using methods given by Temple, et al. (1987, see Ref. A3). For a clay soil having a void ratio (e) of 0.5 and a plasticity index of 15, the allowable shear stress (t_a) is computed using:

$$t_a = t_{ab}\, C_e^2,$$

where t_a = basic allowable shear stress (pounds per square foot),

C_e = void ratio correction factor,

$$C_e = 1.38 - (0.373)(e) = 1.38 - (0.373)(.5) = 1.19,$$

$$t_{ab} = 0.0966 \text{ (from Table 3.3 of Ref. A3)},$$

$$t_a = (0.0966)(1.19)^2 = 0.14 \text{ lb/ft}^2 .$$

(b) The permissible channel velocity is estimated to be 3.5 feet per second (fps), using data provided by Chow (1959, see Ref. A4, Section 7-9). This corresponds to a clayey soil having an allowable shear stress of about 0.15 pounds per square foot.

Step 2. (a) or (b) The slope length is assumed to be 1000 feet. The slope is assumed to be 0.002.

Step 3. (a) or (b) The flow concentration factor is assumed to be 3. It is also assumed that uniform grading will be done during construction and that differential settlement has been shown to be insignificant.

Step 4. (a) or (b) Manning's "n" value may be estimated using Chow (1959, see Ref. A4). For a uniform weathered earth section (using normal values),

$$n = 0.025 \; .$$

Step 5. (a) or (b) The rainfall intensity may be estimated using the procedures given by Nelson, et al. (1986, see Ref. A8). It is assumed that the intensity has been calculated to be 40 inches/hour, using this reference.

Step 6. (a) The stable slope may be computed using the aforementioned NRC derivation of the Horton Equation:

$$(S_s)^{7/6} = (65)(.14)^{5/3} \; / \; (40)(1000)(3)(0.025)$$

$$S_s \;=\; 0.002 \text{ ft/ft} \; .$$

Since the stable slope is equal to the assumed slope, the design is acceptable.

(b) (1) The peak runoff rate is calculated to be:

$q = c\ i\ A\ F = (1)\ (40)\ (1000 / 43560)\ (3) = 2.75$ cfs/ft .

The depth of flow is computed as follows:

$y^{5/3} = (2.75)\ (0.025) / (1.486 \times (0.002)^{1/2})$

$y = 1.0$ ft.

(b) (2) The reduction in permissible velocity is determined using the data previously provided. For a depth of 1.0 feet, the correction factor is equal to 0.8.

The permissible velocity is computed to be:

$V = (0.8)\ (3.5) = 2.8$ fps.

(b) (3) The actual velocity (Va) is computed to be :

$Va = Q/y = (2.75)/(1) = 2.75$ fps.

Since this velocity is less than the computed velocity, the design is acceptable.

2.4.2 Stable Slope with Vegetative Cover

Step 1. The allowable shear stress is estimated using Chow (1959, see Ref. A4) or using Temple, et al. (1987, see Ref. A3). From these references, a reasonable value of shear stress is 0.25 pounds per square foot.

Step 2. The slope length is assumed to be 1000 feet.

Step 3. F is assumed to be 3.

Step 4. Manning's "n" value is assumed to be 0.025, using typical values from Chow (1959, see Ref. A4).

Step 5. The rainfall intensity is assumed to be 40 inches/hour.

Step 6. Using the NRC derivation of the Horton Equation, the stable slope is calculated to be:

$$S_s = 0.005 .$$

NOTE: This stable slope value should be verified using the permissible velocity method. This is particularly important if relatively large values of allowable shear stress have been used. It is important to determine that the maximum recommended permissible velocity of five feet per second (or less, if flow depths are small) will not be exceeded.

2.4.3 Stable Slope with Rocky Soil

It is proposed that a rocky soil will be provided to closely simulate naturally-occurring desert armor and desert pavement at a site in the semi-arid southwestern United States. Based on grain-size analysis, the rocky soil is found to have a D_{75} particle size of 1.0 inches. The rock in the soil also meets the minimum rock quality criteria given in Appendix D.

Step 1. The allowable shear stress is estimated using the procedures discussed by Temple, et al. (1987, see Ref. A3):

$$t = 0.4 \times D_{75}$$

where D_{75} is the particle size in inches for which 75 percent is finer.

$$t = 0.4\ (1.0) = 0.4\ \text{lb/ft}^2\ .$$

Step 2. The slope length is assumed to be 1000 feet.

Step 3. The flow concentration factor (F) is assumed to be 3.

Step 4. Manning's "n" value is estimated to be 0.03, using typical values from Chow for rocky sections.

Step 5. The rainfall intensity is assumed to be 40 inches/hour.

Step 6. Using the Horton/NRC equation:

$$Ss^{7/6} = \frac{65\ (t)^{5/3}}{P\ L\ F\ n}$$

$$Ss^{7/6} = \frac{(65)\ (0.4)^{5/3}}{(40)(1000)(3)(0.03)}$$

$$Ss = 0.009\ .$$

It should be noted that other procedures may be used to determine slope requirements for rocky soils, including the Safety Factors Method or the Corps of Engineers Method. The selection of the Horton/NRC Method is based on ease of calculation for this illustrative example. If the Safety Factors Method is used, for example, other input parameters can be easily derived and substituted into the equations.

2.5 Limitations

The procedure has been developed to assess the slope requirements for sheet flow on plane slopes, and assumes only minor channelling, gullying, or rilling. Such assumptions, while considered reasonable, may or may not represent actual conditions that are expected to occur. For example, it is possible that more severe flow concentrations could occur or that vegetation would not provide any significant protection in very arid areas. Conversely, it is possible that less severe flow concentrations would occur and that more credit could be given for vegetation. Therefore, the NRC staff concludes that the Horton Method provides a reasonable method for assuring that adequate pro-ection will be provided for earthen covers over tailings, such that applicable criteria and regulations are met. In keeping with the management position on mill tailings (USNRC, 1989, see Ref. A1), absolute protection against erosion is not provided by this method; rather, the slope requirements computed in accordance with this method provide a broadly acceptable generic method for assuring tailings control, as defined above. The staff considers that the design parameters are within reasonable ranges, and that use of this equation will result in relatively flat slopes that will produce subcritical flow where channelling occurs.

The procedures discussed above are not applicable to dispersive soils, since such soils tend to be very unstable. Particular attention should be given to the selection of soil types, and dispersive soils should not be used.

3. DESIGN OF UNPROTECTED SOIL SWALES

In many cases, it may be desirable to limit slope lengths by constructing swales or interceptor ditches directly over tailings. These situations are extremely critical design cases for soil covers, since flow will be concentrated.

3.1 Technical Basis

The design of unprotected soil swales is similar to the design of soil covers, except that the flow is concentrated, rather than sheet flow. The basis for the selection of the slope and shape of a swale is to prevent the occurrence of shear stresses that exceed the allowable shear stresses of the soil.

Swales provide an exceptional opportunity to use any available source of rocky soils. The use of rocky soils is a primary method to increase the allowable shear stress. The rock in the soil, however, should be of good quality and meet the minimum rock quality criteria given in Appendix D.

3.2 Design Procedure

The procedures for the design of an unprotected swale are iterative in nature, but are relatively straightforward. Procedures exist to determine every critical design parameter. Following is a step-by-step procedure:

Step 1. Assume a channel slope (S) and cross-section.

Step 2. Determine the design flow rate (Q) using procedures discussed by Nelson, et al. (1986, see Ref. A8).

Step 3. Determine normal depth (y) in the swale, using Manning's Equation.

Step 4. Determine peak shear stress, equal to WyS, where W = 62.4 pounds per cubic foot.

Step 5. Determine allowable shear stress. (See Example 2.4.1, above.)

Step 6. Compare the values of allowable and computed shear stress. If the computed stress exceeds the allowable, return to Step 1 with flatter values of slope or a larger cross-section, or both. It should be noted that rock-protected swales can also be provided. Procedures for the design of rock protection are discussed in Appendix D.

3.3 Recommendations

The staff suggests that the following recommendations be implemented in the computational procedure, for most cases at typical uranium mill sites in the western United States:

1. Channel slopes should be as flat as practicable. Side slopes of swales should also be as flat as practicable. In fact, if the swale is placed perpendicular to the slope of the cover, critical forces may be produced on the side slopes of the swale, and rock protection may be necessary to prevent erosion of the side slopes.

2. The peak flow rate should be determined similarly to the peak flow rates for any small drainage area. Guidance is given by Nelson, et al. (1986, see Ref. A8).

3. In computing normal depth, Manning's "n" values appropriate for earth channels should be used. Guidance for selection of 'n' values is provided by Chow (1959, see Ref. A4).

4. The shear force should be computed based on the peak shear stress, not the average shear stress, in the channel.

5. The allowable shear stress may be computed using procedures given by Temple, et al. (1987, see Ref. A3) or by Chow (1959, see Ref. A4).

3.4 Examples of Procedure Application

3.4.1 Unprotected Swale

It is proposed that an unprotected trapezoidal earth swale be constructed in the soil cover directly over tailings. The maximum drainage area (A) to the swale is 20 acres.

Step 1. As a first trial, assume the following:

The bottom width of the section is 25 feet, and the side slopes are 1V on 10H.

The bottom slope is 0.001.

Step 2. Using the Rational Formula (Nelson, et al. 1986, see Ref. A8); a peak rainfall intensity of 50 inches/hour, computed using the same reference; and a runoff coefficient of 0.8, the design discharge (Q) is:

$$Q = ciA = (0.8)\ (50)\ (20) = 800 \text{ cfs.}$$

Step 3. Solving the Manning Equation by trial and error with:

$Q = 800$
$n = 0.025$
$S = 0.001$

Normal depth $(y) = 3.81$ feet.

Step 4. The maximum shear force (t) is computed by:

$$t = WyS = (62.4)\ (3.81)\ (0.001) = 0.24\ \text{lb/ft}^2\ .$$

Step 5. The allowable shear force is estimated to be 0.1 lb/ft^2, using procedures similar to those discussed in Section 2.4, p. A-9.

Step 6. Since the shear force produced is larger than the allowable, return to Step 1 with new values of channel slope or channel cross-section, or both.

3.4.2 Swale with Vegetation

It is proposed that a trapezoidal earth swale protected by vegetation will be constructed directly over tailings. The drainage area (A) is 20 acres.

Step 1. It is assumed that the bottom width of the swale is 25 feet, the side slopes are 1V on 10H, and the bottom slope is 0.001.

Step 2. Using the rational formula, with a runoff coefficient of 0.7 and peak intensity calculated to be 50 inches/hour, the design discharge (Q) is:

$$Q = ciA = (0.7)\ (50)\ (20) = 700\ \text{cfs}.$$

Step 3. Solving the Manning Equation by trial and error with:

$Q = 700$
$n = 0.03$
$S = 0.001$

Normal depth $(y) = 3.9$ feet.

Step 4. The maximum shear force is computed to be:

$$t = WyS = (62.4)\ (3.9)\ (0.001) = 0.24\ \text{lb/ft}^2\ .$$

Step 5. The allowable shear force is estimated to be 0.25 pounds per square foot, using procedures and recommended values discussed by Temple, et al. (1987, see Ref. A3) and by Chow (1959, see Ref. A4).

Step 6. Since the allowable shear force is greater than the peak shear stress produced by the flood flow, the design is acceptable.

3.5 Limitations

This procedure assumes that the the channel will be uniform in slope and in cross-section, throughout its entire length. If this is not the case, it may be necessary to perform backwater calculations to compute depths of flow in various portions of the channel. Such calculations can complicate this method of channel design. However, backwater calculations should be used where the slope or the cross-section changes, since normal depth is not likely to occur along the entire length of such a channel.

Care should be exercised in the alignment and layout of the swale to assure that shear forces produced on the side slopes do not exceed the allowable shear forces. For example, if a swale is constructed to intercept flows perpendicularly to the slope, excessive forces may be produced on the side slopes. Separate computations will be needed to determine the values of normal depth and maximum shear stresses on the channel side slopes.

4. REFERENCES

A1. U.S. Nuclear Regulatory Commission (USNRC), "Uranium Mill Tailings Management Position," 1989.

A2. Horton, R. E., "Erosional Development of Streams and Their Drainage Basins: Hydrophysical Approach to Quantitative Morphology," Geol. Soc. America Bull, Vol. 56, pp. 275-370, 1945.

A3. Temple, D. M., et al., U.S. Department of Agriculture (USDA), "Stability Design of Grass-Lined Open Channels," Agricultural Handbook Number 667, 1987.

A4. Chow, V. T., Open-Channel Hydraulics, McGraw-Hill Book Company, Inc., New York, N.Y., 1959.

A5. Fullerton, W. T., "Water and Sediment Routing from Complex Watersheds and Example Application to Surface Mining," (MULTSED), Colorado State University, 1983.

A6. Abt, S. R., and Hogan, S. A., "Corrections of Permissible Velocity for Depth: A Review," 1990.

A7. Abt, S. R., et al., "Development of Riprap Design Criteria by Riprap Testing in Flumes: Phase I," NUREG/CR-4651, 1987.

A8. Nelson, J. D., et al., "Methodologies for Evaluating Long-Term Stabilization Designs of Uranium Mill Tailing Impoundments," NUREG/CR-4620, 1986.

APPENDIX B

METHOD FOR DETERMINING SACRIFICIAL SLOPE REQUIREMENTS

CONTENTS

APPENDIX B

METHOD FOR DETERMINING SACRIFICIAL SLOPE REQUIREMENTS

1. INTRODUCTION

In many cases where tailings extend over a large area, flow lengths may be so long that extremely gentle slopes will be needed to provide long-term stability. Such gentle slopes may necessitate the use of very large amounts of soil, such that some of these slopes (with no tailings directly under them) may extend greatly beyond the edge of the tailings pile.

In such cases, licensees may be able to demonstrate that it is impractical to provide stability for 1,000 years and may choose to show that stability for less than 1,000 years, but for at least 200 years, is a more cost-effective option. Such a design may incorporate tailings embankment "outslopes," where there are no tailings directly under the soil cover. Such slopes, designed for less than the 1000-year stability period, may be acceptable if properly justified by the licensee.

It should be emphasized that the staff considers that a 200-year sacrificial slope design should be used only in a limited number of cases and only when a design life of 1000 years cannot be reasonably achieved. However, it should not be assumed that the design period should immediately jump from 1000 years to 200 years. The staff concludes that the selection of a design period should proceed in a stepwise fashion, with consideration given to intermediate design periods from 200-1000 years. In determining a minimum design, a 200-year sacrificial slope design, as presented below, may be used. However, such a design has a considerable amount of uncertainty associated with its use, due to its development by extrapolation of a relatively limited data base. Therefore, the staff considers that the procedure should be used only after other reclamation designs have been considered. The staff considers that the procedures for justifying a design period of less than 1000 years, as discussed in Appendix C, should be carefully followed to document that a 200-year sacrificial slope design is the best design that can be reasonably provided.

2. TECHNICAL BASIS

A procedure for determining sacrificial slope requirements and the tailings setback distance required from the edge of an embankment crest has been developed by the U. S. Nuclear Regulatory Commission (NRC) staff. The procedure is based on the assumption that a specific depth of gullying (as defined by Nelson, et al. (1986, see Ref. B1) will not be exceeded within a 200-year period. This procedure also assumes no drainage area above the embankment crest (see Figure B1 for clarification).

The NRC staff has modified the procedure to provide other values of stable slope and maximum depth of gullying. These changes were necessary to provide more precise guidance on designing gullied slopes, by allowing consideration of soil cohesiveness, vegetation, and other factors that enter into the calculation of stable slope, as discussed in Appendix A. It is expected that the NRC staff will conduct future studies to further evaluate gully incision and growth on tailings embankment slopes. Until that time, this procedure should provide reasonable assurance of tailings stability for at least 200 years.

This procedure generally conforms to theoretical slope configurations that will be produced over a long time period. At the upstream end of a slope where the drainage area approaches zero, the stable slope approaches infinity. However, the maximum slope is limited by the natural angle of repose of the soil.

3. PROCEDURES

See Figure B1 for clarification of variables.

Step 1. Assume values of slope length (L_1), tailings setback distance (X), and elevation difference (H) from the top of the slope to Point A.

Step 2. Using the methods discussed in Appendix A (guidance for the selection of individual design parameters is also given in Appendix A), determine the stable slope (S_s) for a slope length (L) equal to (L_1 + X). The slope length is based on the assumption that erosion and slumping will occur and that Point B defines the acceptable limit of erosion. The horizontal distance from Point A to Point B is approximately equal to L for relatively flat slopes. (The methods discussed in Appendix A are considered to be more appropriate than those given in Ref. B1 for determining the stable slope.)

Step 3. Using the gully intrusion procedures given by Nelson, et al. (1986, see Ref. B1, Chapter 4), calculate the transitional slope (S_t). During the period of evaluation, the side slope will erode to a level between the initial slope (S_i) and the stable slope (S_s). This interim slope is the transitional slope.

Step 4. Calculate D_{max} and L_D , where D_{max} is the maximum depth of gullying and L_D is the horizontal distance from Point A to the gully bottom. D_{max} may be determined using the equation:

$$D_{max} = L_D/L \, [H - L(S_t)].$$

Step 5. Calculate the elevation of the bottom of the gully (Pt. G).

Step 6. Calculate L_R = Y/tan R where Y = (Elev. Pt. B - Elev. Pt. G) and R = angle of repose of cover material.

Step 7. Compute the total slope length (L_t) required to provide erosion protection for at least 200 years, which is equal to $L_D + L_R$. If L_t is less than (L), then the assumed sacrificial outslopes are acceptable. If L_t is greater than (L), return to Step 1, assuming new values of L_1 or X, or both.

If there is an appreciable drainage area or slope length above Point C (see Fig. B2), the computations are performed similarly, except L_1 is set equal to the total slope length contributing runoff at Point A. The total slope length L_t computed in Step 7 must be less than the distance from Point A to Point C, plus X. See Fig. B2 for clarification.

4. RECOMMENDATIONS

The stable slope should be determined using the procedures presented in Appendix A. Appropriately conservative values of input parameters should be used in the computation.

Additional refinements can be made by determining exact slope lengths directly along the slopes, rather than the horizontal distances between the points. This example was presented for graphic clarity and simplicity.

5. EXAMPLE OF PROCEDURE APPLICATION

As an illustrative example, it is assumed that a licensee has demonstrated that designing for a 1000-year stability period is not reasonably achievable, that the tailings will be designed to remain stable for at least 200 years, and that sacrificial "outslopes" will be employed to provide this protection. It is assumed that a sacrificial slope 200 feet long and 40 feet high (a 20 percent slope) is provided to protect tailings that will be set back 50 feet from the top edge of the embankment (see Figure B1). The soil cover material has a uniformity coefficient (C_u) of ten, based on soil tests for the topsoil cover.

Step 1. From the stated assumptions:

L_1 = 200 feet,
X = 50 feet, and
H = 40 feet, the elevation difference between Points A and C.

Step 2. Using the Horton Method discussed in Appendix A, the stable slope may be determined. For the purposes of this illustration, it is assumed that the procedures in Appendix A have been followed and that the stable slope for a slope length of L = 250 feet is computed to be

$S_s = 0.009$.

Step 3. Using Nelson, et al. (1986, see Ref. B1), the transitional

slope (S_t) is calculated to be:

$S_t = (S_i)\ e^{-GSt}$, where G is a coefficient and t is the time in years (not to exceed 200 years),

$S_t = (.2)/e^{(1.0)(0.009)(200)}$ using Fig 4.3 (Nelson, et al., 1986,

see Ref. B1), where G = 1.0

$S_t = 0.033$.

Step 4. For a value of $(S_t \times C_u) = (0.033)(10) = 0.33$,

where the uniformity coefficient is assumed to be 10,

$L_D / L = 0.78$, using Figure 4.4 (Nelson et al., 1986, see Ref. B1).

Note that if the values of the parameter L_D/L are off the curve to the left in Figure 4.4, the estimated location of the gully would be close to the top of the outslope and would probably intersect tailings, and the design would not be acceptable. In this case, the analysis would have to be reiterated using a flatter outslope and/or a greater setback distance. If the value of L_D/L is off the curve to the right, the design is probably acceptable because the gully would form close to the toe of the outslope.

D_{max} = 0.78 [(40) - (250)(.033)] = 24.8 feet

L_D = .78 (250) = 195 feet.

Step 5. Assuming Point A to be at Elevation 0 feet, the elevation of the bottom of the gully (Elev. G) is calculated to be:

Elev. G = 0.78(40) - 24.8 = 6.4 feet.

Step 6. For an assumed angle of repose of 30°, and an elevation difference of 33.6 feet (40.0 - 6.4) between Point B and the bottom of the gully,

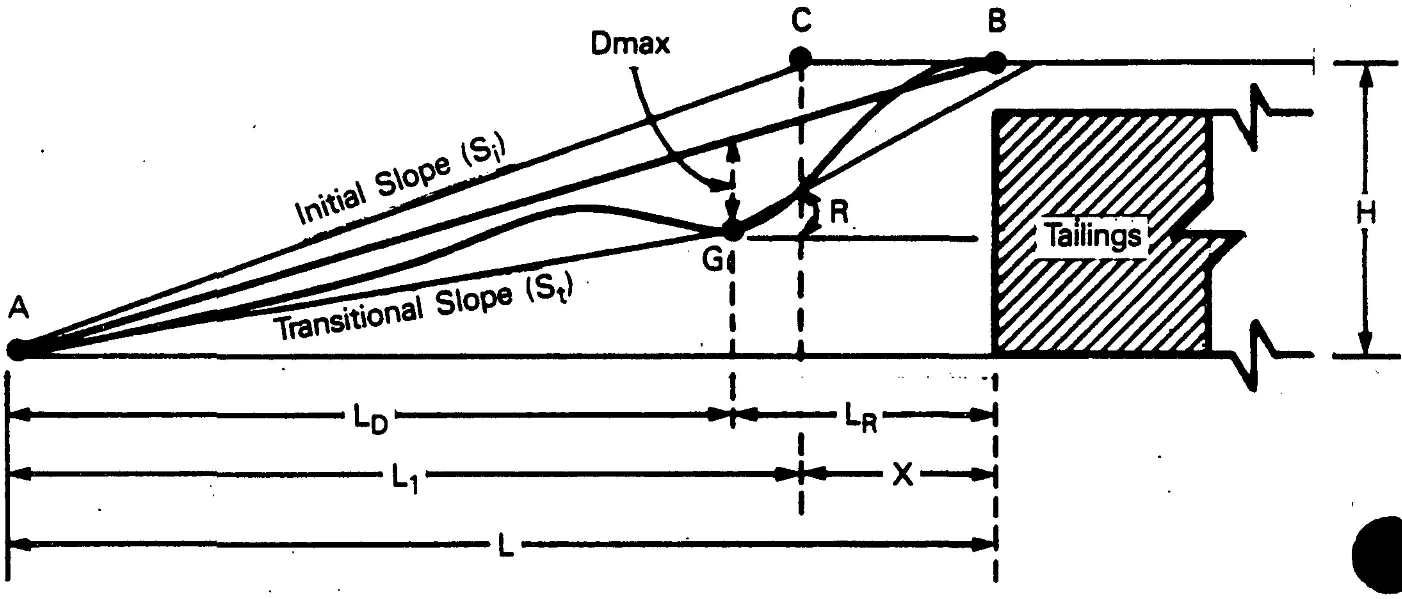
L_R = 33.6 / (tan 30°) = 33.6 / 0.58 = 58 feet.

Step 7. L_t = 195 + 58 = 253 feet.

Since L_t = 253 feet is greater than (L) = 250 feet, the design is not acceptable. Return to Step 1 with new values of slope length or setback distance, or both. Note that in this case, the values are approximately equal; an increase of 3 to 5 feet in the setback distance is the most likely choice.

6. LIMITATIONS

This method of analysis is considered to represent an approximate method of analyzing setback and sacrificial slope requirements. It should be emphasized that the gully intrusion method has been developed by extrapolating empirical data, which could lead to significant errors in the determination of gully depths and transitional slopes. Because of the possible errors associated with extrapolating such a limited data base, the staff expects that additional monitoring of the slope will be needed following closure of a sacrificial slope design with a design life of only 200 years. Licensees and applicants will be expected to conduct additional monitoring of the slope to assure that the design is performing as expected. If deviations are found, the licensee may be required to redesign and revise the sacrificial slope.



R = Angle of Repose of Cover Material

Figure B1

Procedure for Determining Sacrificial Slope Requirements and Setback Distance, No Drainage Area Above Point C

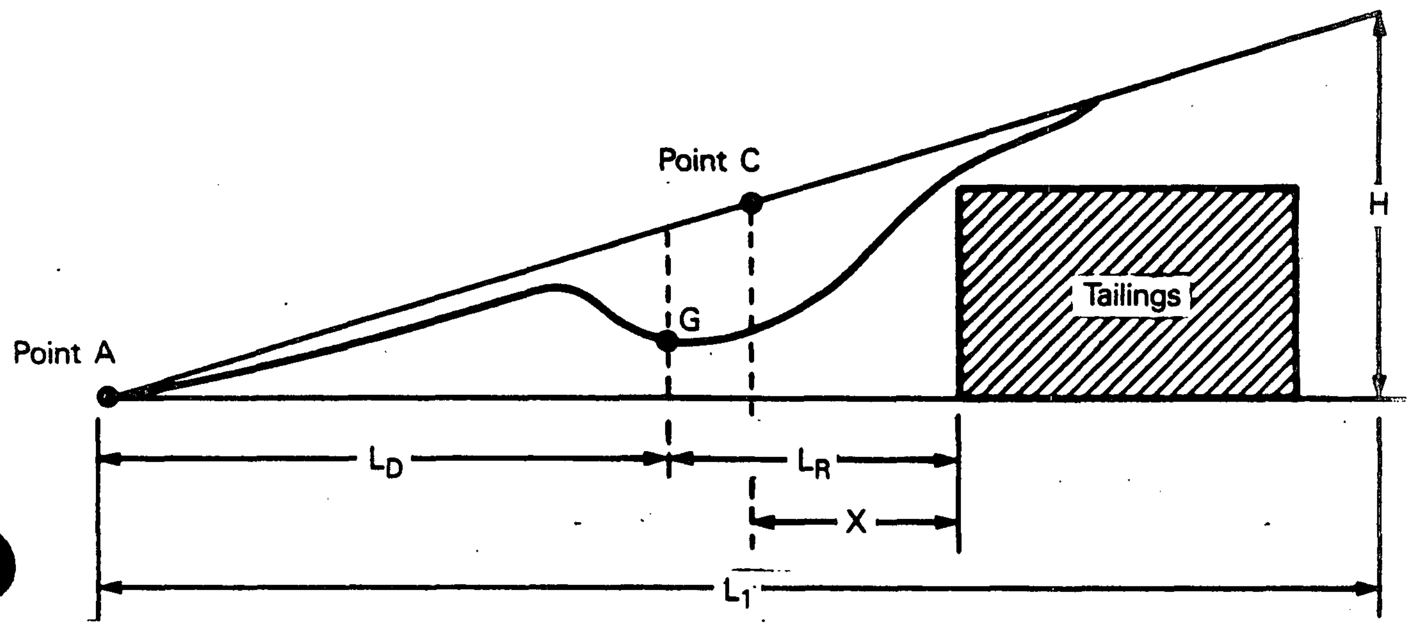


Figure B2

Procedure for Determining Sacrificial Slope Requirements and Setback Distance with Significant Drainage Area Above Point C

7. REFERENCE

B1. Nelson et al., "Methodologies for Evaluating Long-Term Stabilization Designs of Uranium Mill Tailing Impoundments," NUREG/CR-4620, 1986.

APPENDIX C

PROCEDURES FOR DETERMINING IF A 1000-YEAR DESIGN IS NOT REASONABLY ACHIEVABLE

A tailings pile must be designed to remain stable for 1000 years, unless it can be shown that designing for 1000 years is not reasonably achievable. One aspect of demonstrating that a design is not reasonably achievable is total cost. If it can be demonstrated that a 1000-year design is not practicable because of excessive costs, a licensee or applicant can design for a shorter time period, in accordance with applicable regulations and standards. In no case, however, can the stability period be less than 200 years.

In order to justify that providing an erosion protection cover for a 1,000-year period is not reasonably achievable, the following step-by-step procedure is suggested:

Step 1. Identify several designs and design configurations that would meet the 1000-year stability criterion. Such designs should include, as a minimum, soil covers with stable slopes, combinations of soil and rock covers, and rock-protected soil covers. Alternative designs may also include vegetated slopes, if it can be shown that vegetation will be dense and self-sustaining over a long period of time.

Step 2. Identify the least costly of several rock sources that could be used with the designs identified in Step 1. The sources should be evaluated based on cost, rock size availability, and durability.

Step 3. Determine the costs associated with the least costly design that will be capable of meeting the 1000-year stability criterion. Costs, including transportation costs, should be broken down by unit cost and total cost in the following categories:

1. Soil covers and/or rock erosion protection for top of pile

2. Soil covers and/or rock erosion protection for sides of pile

3. Rock erosion protection for aprons/toes, as necessary

4. Rock erosion protection for drainage and diversion channels

5. Rock erosion protection for banks of large adjacent streams

6. Earthwork and miscellaneous features needed specifically for erosion protection (for example, diversion dikes)

Step 4. Compute the total cost of the project for meeting the 1000-year stability criterion, as compared to the cost of designing for stability periods of less than 1000 years. In order to determine if the costs of providing such protection are clearly excessive, the following minimum criteria are suggested:

(1) the total project cost for the 1,000-year design significantly exceeds the average total project cost for other similar sites, assuming that information on other sites is available,

(2) the cost of providing erosion protection (a soil cover, a soil and rock cover, or a total rock cover) for the 1,000-year design, as a percentage of the total project cost, is significantly greater than the average percentage cost for other similar sites, and

(3) a <u>significant</u> savings results from using the less expensive design.

Step 5. As applicable, determine the magnitude of the flood and the percentage of the design flood (Probable Maximum Flood/Probable Maximum Precipitation, for example) that a less expensive design will withstand. The analyses should assume designs and computational methods similar to the designs and computational methods employed in Step 1, and should assume that the less costly erosion protection will be used.

A plot should be developed to graphically show the relationship of costs vs. the percentage of the design flood event that can be withstood. If a well-defined "break point" exists in the graph, where the costs increase dramatically as a result of increasing the flood discharge, this "break point" may provide a reasonable basis for determining an appropriate flood magnitude for design.

Step 6. Demonstrate that applicable standards and regulations are met by the "reduced" design. Information and analyses that should be provided include the following:

(1) drawings, cross-sections, and supporting hydraulic calculations for each design analyzed, including any other general information requirements, as discussed in Section 3.1 (p. 14),

(2) backup calculations that provide the bases for the cost estimates,

(3) supporting hydraulic calculations, and

(4) supporting logic and bases that document that the design selected meets applicable longevity criteria.

APPENDIX D

PROCEDURES FOR DESIGNING RIPRAP EROSION PROTECTION

CONTENTS

APPENDIX D

PROCEDURES FOR DESIGNING RIPRAP EROSION PROTECTION

1. INTRODUCTION

To minimize the potential for initiation of gullying and erosion damage on steep slopes, it is often necessary to provide rock riprap erosion protection. Because vegetation alone is often not effective, and because natural steep slopes are common on small watersheds in the Western United States, riprap is often needed to provide the required protection. At a typical reclaimed tailings site, riprap may be needed to protect: (1) top and side slopes; (2) diversion channels; (3) aprons and diversion channel outlets; and (4) banks of larger rivers and/or areas of the reclaimed side slopes where floods impinge. Procedures for designing riprap erosion protection for each of these areas are given in Sections 2 through 5, following. In addition, procedures are presented in Section 6 for evaluating and oversizing marginal-quality rock to meet longevity requirements.

2. RIPRAP DESIGN FOR TOP AND SIDE SLOPES

The principal objective in determining the riprap requirements for stabilized top slopes and side slopes of embankments is to provide a design that meets long-term stability requirements. Since the most disruptive event for these designs is likely to be gully erosion, it is important to provide a rock layer that will minimize the potential for gully erosion, which, once started, may worsen and continue unchecked.

2.1 Technical Basis

To better understand the phenomena and mechanisms affecting the design of riprap to prevent erosion by overtopping flows, the U. S. Nuclear Regulatory Commission (NRC) staff sponsored technical assistance efforts. As a result of these efforts, it was determined that existing methods can be adapted to design erosion protection for these situations.

The reports and information developed by Nelson, et al. (1983, see Ref. D1; 1986, see Ref. D2) and by Abt, et al. (1988, see Ref. D3) provide the technical bases for concluding that the design of riprap for a reclaimed tailings pile depends upon the slope. Abt, et al. (1988, see Ref. D3) developed a family of curves relating failure flow rates to median stone diameter (D_{50}). These studies have verified that the Safety Factors Method is appropriate for riprap design on flat slopes and that the Stephenson Method is appropriate for riprap design on steep slopes.

The staff has further concluded that some conservatisms are appropriately provided by the Stephenson Method and the Safety Factors Method, to account for actual field conditions. These methods are generally based on flow rates that produce rock movement, and since the failure flow rates are greater than flow rates that produce rock movement, the staff recommends use of the Stephenson Method for slopes of 10 percent or greater and the Safety Factors Method for slopes of less than 10 percent.

Rock mulches (rock layers where the average rock size is relatively small, such as gravel) may provide a practical solution for stabilization of slopes and channels at many sites. The procedures for designing such rock layers are identical to those mentioned in the preceding paragraph. Studies by Abt, et al. (1988, see Ref. D3) have indicated that a rock/soil matrix (riprap layer with the rock voids filled with soil) has similar (or possibly better) stability characteristics as the riprap layer, alone. The staff, therefore, accepts the use of a rock mulch, a rock/soil matrix, or a rocky soil. The design of a rock mulch or a rock/soil matrix should be based on the D_{50} rock size and should follow the procedures given in Appendix D; the designs are similar because there is no clear-cut distinction between a riprap layer and a layer of rock mulch. The design of a rocky soil cover should be based on the D_{75} particle size and should follow the procedures given in Appendix A; the design follows the procedure for a soil cover, because the layer is predominantly soil, rather than rock.

2.2 Design Procedures

A step-by-step procedure for designing riprap for the top and side slopes of a reclaimed pile is presented below:

Step 1. Determine the drainage areas for both the top slope and the side slope. These drainage areas are normally computed on a unit-width basis.

Step 2. Determine time of concentration (tc).

The tc is usually a difficult parameter to estimate in the design of a rock layer. Based on a review of the various methods for calculating tc, the NRC staff concludes that a method such as the Kirpich method, as discussed by Nelson, et al. (1986, see Ref. D2), should be used. The tc may be calculated using the formula:

$$tc = (11.9L^3/H)^{.385}, \quad \text{where } L = \text{drainage length (in miles)}$$

$$H = \text{elevation difference (in feet)}$$

Step 3. Determine Probable Maximum Flood (PMF) and Probable Maximum Precipitation (PMP).

Techniques for PMP determinations have been developed for the entire United States, primarily by the National Oceanographic and Atmospheric Administration, in the form of hydrometeorological reports for specific regions. These techniques are commonly accepted and provide straightforward procedures for assessing rainfall potential, with minimal variability. Acceptable methods for determining the total magnitude of the PMP and various PMP intensities for specific times of concentration are given by Nelson, et al. (1986, see Ref. D2, Section 2.1).

Step 4. Calculate peak flow rate.

The Rational Formula, as discussed by Nelson et al. (1986, see Ref. D2), may be used to calculate peak flow rates for these small drainage areas. Other methods that are more precise are also acceptable; the Rational Formula was chosen for its simplicity and ease of computation.

Step 5. Determine rock size.

Using the peak flow rate calculated in Step 4, the required D_{50} may be determined. Recent studies performed for the NRC staff (Abt, et al., 1988, see Ref. D3) have indicated that the Safety Factors Method is more applicable for designing rock for slopes less than 10 percent and that the Stephenson Method is more applicable for slopes greater than 10 percent. Other methods may also be used, if properly justified.

2.3 Recommendations

Since it is unlikely that clogging of the riprap voids will not occur over a long period of time, it is suggested that no credit be taken for flow through the riprap voids. Even if the voids become clogged, it is unlikely that stability will be affected, as indicated by tests performed for the NRC staff by Abt, et al. (1987, see Ref. D4).

If rounded rather than angular rock is used, some increase in the average rock size may be necessary, since the rock will not be as stable. Computational models, such as the Safety Factors Method, provide stability coefficients for different angles of repose of the material. The need for oversizing of rounded rock is further discussed by Abt, et al. (1987, see Ref. D4).

2.4 Example of Procedure Application

Determine the riprap requirements for a tailings pile top slope with a length of 1000 feet and a slope of 0.02 and for the side slope with an additional length of 250 feet and a slope of 0.2 (20 percent).

Step 1. The drainage areas for the top slope (A1) and the side slope (A2) on a unit-width basis are computed as follows:

$$A1 = (1000)\ (1)\ /\ 43560 = 0.023 \text{ acres}$$

$$A2 = (1000 + 250)\ (1)\ /\ 43560 = 0.029 \text{ acres.}$$

Step 2. The tcs are individually computed for the top and side slopes, using the Kirpich Method, as discussed by Nelson, et al. (1986, see Ref. D2).

$$tc = [(11.9)(L)^3/H]^{.385}$$

For L = 1000 feet and H = 20 feet,

tc = 0.12 hours = 7.2 minutes for the top slope

For L = 250 feet and H = 50 feet,

tc = 1.0 minute for the side slope.

Therefore, the total tc for the side slope is equal to 7.2 + 1.0, or 8.2 minutes.

Step 3. The rainfall intensity is determined using procedures discussed by Nelson, et al. (1986, see Ref. D2), based on a 7.2-minute PMP of 4.2 inches for the top slope and an 8.2-minute PMP of approximately 4.5 inches for the side slope. These incremental PMPs are based on a one-hour PMP of 8.0 inches for northwestern New Mexico and were derived using procedures discussed by Nelson, et al. (1986, see Ref. D2).

Rainfall intensities, for use in the Rational Formula, are computed as follows:

i_1 = (60)(4.2)/7.2 = 35 inches/hr for the top slope

i_2 = (60)(4.5)/8.2 = 33 inches/hr for the side slope.

Step 4. Assuming a runoff coefficient (C) of 0.8, the peak flow rates are calculated using the Rational Formula, as follows:

Q1 = (0.8) (35) (0.023) = 0.64 cfs/ft, for the top slope, and

Q2 = (0.8) (33) (0.029) = 0.77 cfs/ft, for the side slope.

Step 5. Using the Safety Factors Method, the required rock size for the pile top slope is calculated to be:

D_{50} = 0.6 inches.

Using the Stephenson Method, the required rock size for the side slopes is calculated to be:

D_{50} = 3.1 inches.

2.5 Limitations

The use of the aforementioned procedures is widely applicable. The Stephenson Method is an empirical approach and is not applicable to gentle slopes. The Safety Factors Method is conservative for steep slopes. Other methods may also be used, if properly justified.

3. RIPRAP DESIGN FOR DIVERSION CHANNELS

3.1 Technical Basis

The Safety Factors Method or other shear stress methods are generally accepted as reliable methods for determining riprap requirements for channels. These methods are based on a comparison of the stresses exerted by the flood flows with the allowable stress permitted by the rock. Documented methods are readily available for determining flow depths and Manning "n" values.

3.2 Design Procedures

3.2.1 Normal Channel Designs

In designing the riprap for a diversion channel where there are no particularly difficult erosion considerations, the design of the erosion protection is relatively straightforward.

1. The Safety Factors Method or other shear stress methods may be used to determine the riprap requirements.

2. The peak shear stress should be used for design purposes and can be determined by substituting the value of the depth of flow (y) in the shear

stress equations, instead of the hydraulic radius (R). The resulting shear stress equation, t = WyS, provides a very simple analytical method for determining shear stress produced.

3. Flow through the riprap voids should be ignored. Over a long period of time, it is unlikely that the rock voids will not be filled with sediments, debris, and organic material.

4. The Manning's "n" value may be determined using a variety of methods, depending on the slope of the ditch and the depth of flow. For relatively flat ditches where the depth of flow exceeds the average size of the riprap, the U.S. Army Corps of Engineers (USCOE) relationships may be used (USCOE, 1970, see Ref. D5). For relatively steep slopes or for those instances where the depth of flow is not large relative to the rock size, the "n" value should be computed in accordance with the recommendations of Abt, et al. (1987, see Ref. D4). Abt found the "n" value was directly related to the slope and the riprap size, when the relative depth of flow was small.

5. Rainfall and rainfall intensities may be derived using procedures discussed by Nelson, et al. (1986, see Ref. D2).

6. Times of concentration may be computed using the Kirpich Method, as discussed by Nelson, et al. (1986, see Ref. D2).

7. The depth of flow in the channel may be calculated by solving the Manning Equation for the normal depth (Chow, 1959, see Ref. D6), if the channel is relatively uniform in cross-section and there are no changes in the bottom slope. If there are cross-section changes or changes in bottom slope, models such as those developed by the Hydrologic Engineering Center (USCOE, see Ref. D7), or other gradually-varied flow models, should be used to determine the depth of flow and slope of the energy grade line.

3.2.2 Design for Inflow from Natural Gullies

There have been several cases where proposed diversion ditches have been provided to divert flood flows around a reclaimed tailings pile, and in several locations, the ditches receive direct inflow from several existing gullies. Particular care must be taken in such instances to avoid damage to the ditches in the general area where the natural gullies discharge into the diversion ditches. This occurs in many cases where diversion channels are constructed generally perpendicular to the natural slope. The diversion channel may be constructed on a flatter slope than the slope of the natural gullies that will discharge into it, and the velocities in the natural gully are higher than the riprapped diversion channel can withstand.

1. The riprap in the immediate area where the natural gully discharges into the diversion channel should be designed for the peak velocities and shear forces that occur in the natural gully. This may be very important if the gully is significantly steeper than the proposed diversion channel. Assuming that the flow in the gully will spread and/or dissipate upon contact with the diversion ditch riprap may not be a valid assumption. The peak shear stress (t) for design purposes can be determined by calculating the normal depth in the gully and calculating the peak shear stress using $t = WyS$, where $W = 62.4$ pounds per cubic foot, y is the normal depth of flow in the natural gully (in feet), and S is the slope of the natural gully. The Safety Factors Method, for example, may also be used. The rock size in the diversion channel should be checked to ensure that it is sufficient to resist the flow velocities down the channel side slope.

2. To determine normal depth in the natural gully, the assumed gully cross-section should be one that currently exists, unless there is a potential for a more critical configuration to develop over a period of time. An example of this development would be the gradual vertical erosion of a gully that could narrow the cross-section or steepen the side slopes of an existing cross-section.

3. It may be necessary to provide riprap to the computed depth of scour in the natural gully at the point where the natural gully meets the top of the slope of the diversion channel. This scour depth may be estimated using procedures of the U. S. Department of Transportation (USDOT) (USDOT, 1983, see Ref. D8) or using geomorphic analyses. It appears that the thickness of the rock layer at this location should not be less than the depth of any natural gullies in the area, taking into consideration the drainage area to the gullies in the site area.

4. In addition to the larger natural gullies that discharge into the diversion channel, consideration should be given to possible areas of flow concentration at other points along the diversion channel. It is possible, particularly if the inflow slopes are steep, that smaller gullies could be formed at other locations and may generate more erosive force than the rock in the diversion channel is capable of withstanding. Geomorphic or geologic evidence provides a reasonable guide regarding the possible formation of other smaller gullies.

5. The larger rock, as determined using the considerations previously discussed, may also need to be placed on the bottom and opposite bank of the diversion channel. This is necessitated by turbulence caused by energy dissipation in the channel and on the banks of the channel.

3.2.3 Specific Design Procedure

The design of riprap for diversion channels is relatively straightforward. The following step-by-step procedure is suggested:

Step 1. Determine Time of Concentration

The time of concentration should be determined using a velocity-based method. For steep drainage areas, it is likely that overland flows will channelize relatively quickly; thus, the velocities that occur in these gullies and channels should be used to estimate the total time of concentration for the basin. The channel hydraulics method of the U. S. Bureau of Reclamation (USBR) (USBR, 1977, see Ref. D9) is suggested for use in such cases. The Kirpich Method, as previously discussed, or other methods, may also be used.

Step 2. Determine rainfall intensities of the design storm.

Determine total PMP and various PMP intensities (corresponding to the time of concentration) using procedures such as those discussed by Nelson, et al. (1986, see Ref. D2)

Step 3. Determine Design Flow Rate

Depending on the complexity, size, and shape of the drainage basin, several methods may be used to calculate the peak flow rate to be used for designing the riprap protection. The Rational Formula may be used for small basins with very little shape irregularity. The triangular unit hydrograph method (USBR, 1977, see Ref. D9) may be used for somewhat larger basins with no significant shape irregularities. HEC-1 (USCOE, see Ref. D10) should be used if the basins are large or if it is necessary to route inflows from irregularly-shaped basins.

Regardless of the method selected, it is important to select appropriate values of infiltration and runoff in determining the peak flow rate. This will necessitate the use of reasonably conservative values of C, if the Rational Formula is used. It will also necessitate the use of reasonably high antecedent moisture conditions and critical placement of peak rainfall values in the storm sequence.

The NRC staff considers that reasonably conservative values of design parameters are necessary to account for flood events that have actually occurred in various areas where tailings sites are located. Although it is not possible to exactly predict the moisture conditions of the drainage basin soils or the distribution of rainfall within a given storm event, the magnitude of historic flood events can provide some guidance in the selection of design parameters. For example, a flood with a magnitude of 2630 cfs occurred on a 200-acre drainage basin in southwestern Utah (Crippen and Bue, 1977, see Ref. D11). It can be seen that very high values of rainfall intensity and very low values of infiltration were necessary to produce such a flood.

Step 4. Calculate Riprap Size Required

a. Assume a trial rock size D_{50}.

b. Calculate Manning's "n" value using either (1) the method discussed by Abt, et al. (1987, see Ref. D4), if the channel slope is steep and the depth of flow is small relative to the assumed D_{50} or (2) using the USCOE method (USCOE, 1970, Plate 4, see Ref. D5), if the slope is mild and the depth of flow is large, relative to the assumed D_{50}.

c. Calculate normal depth using Manning's equation (Chow, 1959, see Ref. D6) if the channel cross-section and slope are uniform. Otherwise, a standard-step backwater model, such as HEC-2 (USCOE, see Ref. D7) should be used to determine flow depths and velocities.

d. Compute the peak shear stress produced in the channel. The peak shear stress for a typical V-shaped or trapezoidal channel will be produced at the point where the depth of flow is the greatest. This depth should be used for design and should be used to compute shear stress.

e. Compute the rock size necessary to resist the computed shear stress. Return to (a) if the computed D_{50} is significantly different from the assumed D_{50}.

3.3 Recommendations

Recommendations for each design area are discussed in the design procedures. As stated, the rock in the channels should be designed for the peak shear stress (rather than the average shear stress) produced. Manning's "n" values should be determined based on the relative depth of flow in the channel.

In many cases where natural gullies discharge into diversion ditches, it may be necessary to assess the potential for possible clogging of the ditch due to sediment and debris. Particularly where the inflow slopes are greater than the ditch slopes, it is possible that the natural gully will be capable of moving material that the diversion ditch cannot flush out. If the larger material cannot be flushed by the ditch flows, the capacity of the ditch may be compromised, resulting in possible overtopping of the ditch. The following recommendations should be followed in such cases.

1. Diversion ditches should be designed to be self-cleaning.

2. If a ditch cannot be designed to be self-cleaning, it should be designed to contain the sediment/debris that will be deposited in the ditch during the design life. Justification may also be provided to show that there is little or no debris/sediment to be transported. It may also be possible to show that the configuration of the deposits in the ditch will have no adverse effects on either the flow capacity or the stability of the ditch.

3.4 Example of Procedure Application

A 15-foot wide trapezoidal channel with 1V on 5H side slopes will be constructed on a 5 percent slope and will carry a discharge of 1000 cfs. Determine the riprap requirements.

Step 1. Assume a trial D_{50} equal to 2.0 feet (24 inches).

Step 2. Compute Manning's "n" value.

Since the slope is relatively steep, the flow depth is likely to be small relative to the riprap size. Therefore, the "n" value should be computed in accordance with the recommendations of Abt, et al. (1987, see Ref. D4).

Using the equation from Ref. D4:

$$n = 0.0456\ (24 \times 0.05)^{.159}$$

$$n = 0.047\ .$$

Step 3. Determine normal depth (y).

By trial and error for the trapezoidal channel, with

$n = .047$; $Q = 1000$ cfs; and $S = 0.05$,

$y = 3.0$ feet.

Step 4. Compute the actual shear stress produced.

Using the Safety Factors Method or the simple equation, $t = WyS$, which closely approximates the Safety Factors Equation for computing shear stress,

$$t = (62.4)\ (3.0)\ (0.05) = 9.36\ \text{lb/ft}^2\ .$$

Step 5. Compute the required rock size.

Using an equation of the USCOE (USCOE, 1970, see Ref. D5),

$t = a(W_s - W_w)\ (D_{50})$ where:

$a = 0.04$
W_s = unit weight of rock, in lb/ft^3, and
W_w = unit weight of water = $62.4\ \text{lb/ft}^3$.

Based on an assumed stone weight of 165 pounds per cubic foot,

$$t = 4.1\ D_{50}\ .$$

The required size is calculated to be:

$$D_{50} = t\ /\ 4.1$$

$$D_{50} = 9.36\ /\ 4.1 = 2.3 \text{ feet.}$$

Since the required rock size (2.3 feet) is greater than the rock size assumed (2.0 feet), another iteration with a larger D_{50} will be necessary.

3.5 Limitations

The procedures just discussed may require several iterations before an agreement can be reached between the assumed and computed rock size. In some cases where the slope is very steep and discharges are very large, a balance may never be able to be reached, indicating that the slope or discharge is so great that riprap protection cannot be feasibly provided. For very steep slopes, use of the Stephenson Method, discussed previously, may be considered in sizing riprap.

4. RIPRAP DESIGN FOR APRONS AND DIVERSION CHANNEL OUTLETS

It is usually necessary to direct the flow from a man-made diversion channel into a naturally-occurring gully or stream channel or to discharge the flow onto natural ground at a point where the channel intersects the natural ground surface. In such cases, it is necessary to assure that the flood flows are safely conveyed into the natural environment, without causing erosion and eventual damage to the reclaimed tailings or tailings cover.

4.1 Technical Basis

Several methods exist to design riprap erosion protection to prevent erosion of natural soils and soil channels. These methods can be adapted to predict erosive forces that will exist at the outlets of man-made channels and to properly design aprons, toes, and energy dissipation areas. The USCOE, for example, has wide experience in designing spillways and reservoir outlet works. Additional rock protection at outlets is almost always recommended to prevent erosion and damage to structures.

4.2 Design Procedures

The use of any particular procedure depends on the type of erosion problem to be prevented. In general, the cases most often encountered will be: (1) normal daylight designs where the diversion channel intersects a relatively flat natural slope; or (2) designs where severe gullying has occurred, or will occur, if adequate precautions are not taken.

4.2.1 Normal Daylight Designs

The typical design case requires that a rock-protected outlet section be provided to reduce flow velocities to a level that can be accommodated by the natural earth section that will receive the flows. In addition, a rock toe is normally provided to protect the ditch outlet against possible future headcutting of any potential gully that could be randomly formed downstream of the outlet. In general, two principal options are available:

(1) The outlet section should be sized (widened) such that the shear force produced in the earth section immediately downstream of the rock section is less than the maximum permissible shear force that the earth can withstand, or

(2) the rock toe to be provided at the outlet should be keyed into competent bedrock, whenever reasonably possible. Alternately, the toe should be placed to a depth corresponding to the maximum gully depth to be expected. Geomorphic/geologic factors should be considered in the estimation of the maximum depth of gullying to be expected, or scour depths in the natural channel may be computed using other procedures (USDOT, 1983, see Ref. D8). Typical toe treatment details are provided in EM 1110-2-1601 (USCOE, 1970, see Ref. D5) and are recommended for determining toe configurations.

4.2.2 Design for Severe Gullying

In many cases, where the natural slopes are steep and gully depths extend to more than several feet, it may be necessary to construct special ditch outlets. In such cases, it may be necessary to provide extensive and elaborate ditch outlets to prevent gully erosion from impacting the stabilized tailings. In general, the following criteria should be followed.

1. The toe should be keyed into competent bedrock.

2. If bedrock exists at a substantial depth, and it is not reasonably feasible to extend the toe depth to this elevation, the toe should be designed to collapse and be sufficiently stable to prevent additional headward gully erosion. The depth of the rock toe should be at least equal to the maximum expected depth of gully erosion in the natural gully; this maximum depth of scour may be computed using procedures such as those developed by USDOT (1983, see Ref. D8) or, in some cases, may be estimated by observing natural gullies in the area.

3. The ditch outlet may be placed a sufficient distance away from the stabilized tailings so that the tailings will not be affected during the design lifetime, even if some erosion occurs.

4.3 Recommendations

In general, the bottom elevation of the rock toe at the outlet of a channel or the downstream of an apron should always be placed at an elevation equivalent to the maximum expected depth of scour. Otherwise, the rock toe will be subject to undermining, and damage to the ditch or apron could occur.

4.4 Example of Procedure Application

A licensee proposes to construct a steep rock-lined channel to discharge a peak flood discharge of 880 cfs from the top of a remediated tailings pile. The channel will have a slope of 10 percent and will discharge into a naturally-occurring gully consisting of uniform sand. The channel will be lined with riprap having a D_{50} of 30 inches. Determine the toe requirements, assuming the channel is to discharge into the natural gully.

Step 1. Determine depth of scour and dimensions of the scour hole in the natural gully.

For the assumed channel section, flow rate, and flow area, it is assumed that the procedures of the USDOT (1983, see Ref. D8) have been followed and that the depth of scour (D), the width of scour (W), and the length of scour (L) are computed.

D = 7.8 feet

W = 35 feet

L = 50 feet.

Step 2. Determine toe configuration.

The toe configuration is also evaluated using USCOE EM 1110-2-1601 (1970, see Ref. D5). Using the figures given in Plates 37 and 38, the minimum thicknesses and general configuration of the toe area are determined, using the dimensions derived in Step 1.

4.5 Limitations

The scour depths, slopes, and designs developed using the aforementioned procedures should always be verified by careful analysis of site-specific geomorphic variables. Adjustments may need to be made to the design, based on geomorphic considerations.

5. RIPRAP DESIGN FOR PROTECTION FROM FLOODING FROM NEARBY STREAMS

5.1 Technical Basis

Design of riprap for the stream banks of channels is well-established and is relatively simple. The USCOE and other Federal agencies have developed procedures for designing such protection.

5.2 Design Procedure

The following procedure may be used for the determination of riprap requirements for the banks of major streams or the side slopes of reclaimed tailings piles, where floods impinge.

Step 1. Determine peak flow rate.

Depending on the size of the stream, various methods may be used to determine peak flow rates. For large streams, the procedures discussed in Regulatory Guide 1.59 (USNRC, 1979, see Ref. D12) may be used.

Step 2. Determine depth (y) and velocity (V) of flow and the slope of the energy grade line (S) at the location where riprap will be provided.

In general, HEC-2 (USCOE, see Ref. D7) provides an acceptable computational model for estimating these design parameters.

Step 3. Determine peak shear stress.

Step 4. Determine the riprap size needed to resist the computed shear stress, with corrections made for the side slope

5.3 Recommendations

Because of the possibility of variability of depth and slope between adjacent cross-sections in a flow profile, the use of average values of these parameters should also be considered. Several adjacent sections should be examined, and engineering judgment should be used to estimate these design parameters.

5.4 Example of Procedure Application

It is proposed that riprap will be placed on the 1V on 2H side slope of a natural stream. Determine the riprap size required, given the parameters discussed in the following steps:

Step 1. The peak flow in a stream with a drainage area of 200 square miles is calculated using HEC-1 (USCOE, see Ref. D10) to be 200,000 cfs.

Step 2. Using HEC-2 (USCOE, see Ref. D7), the following design variables are computed at the location in question:

y = depth of flow = 10.2 feet

S = slope of energy grade line = 0.008

V = velocity of flow = 15 ft/sec.

Step 3. Using the simple relationship, $t = WyS$, the peak shear stress is calculated to be:

$t = (62.4)\ (10.2)\ (0.008)$

$t = 5.09$ pounds per square foot.

Step 4. The riprap size is calculated to be:

$$D_{50} = t / 4.1 = 5.09 / 4.1 = 1.24 \text{ feet.}$$

For a 1V on 2H side slope, a correction factor of 0.72 is found using USCOE procedures (1970, see Ref. D5, Plate 36). The corrected riprap size is found to be:

$$D_{50} = 1.24 / 0.72$$

$$D_{50} = 1.7 \text{ feet.}$$

The Safety Factors Method or USCOE procedures may also be used. This method was selected for simplicity and is probably conservative, in most cases.

The toe of the riprap slope should be designed in accordance with procedures of the USCOE (1970, see Ref. D5), with regard to toe width, thickness, length, and general configuration.

5.5 Limitations

Use of this procedure relies heavily on the computational model used to calculate flow depth and slope. Calculation of depth and slope are usually sensitive to small changes in "n" values, expansion or contraction coefficients, and length between sections.

6. OVERSIZING OF MARGINAL-QUALITY EROSION PROTECTION

6.1 Technical Basis

The ability of some rock to survive without significant degradation for long time periods is well-documented by archaeological and historic evidence (Lindsey, et al., 1982, see Ref. D13). However, very little information is available to quantitatively assess the quality of rock needed to survive for long periods, based on its physical properties.

In assessing the long-term durability of erosion protection materials, the NRC staff has relied principally on the results of durability tests at several sites and on information, analyses, and methodology presented in NUREG/CR-4620 (Nelson, et al., see Ref. D2). This document provides a quantitative method for determining the oversizing requirements for a particular rock type to be placed at specific locations on or near a remediated uranium mill tailings pile.

Staff review of actual field data from several tailings sites has indicated that the methodology may not be sufficiently flexible to allow the use of "borderline" quality rock, where a particular type of rock fails to meet minimum qualifications for placement in a specific zone, but fails to qualify by only a small amount. This may be very important, since the selection of a particular rock type and rock size depends on its quality and where it will be placed on the embankment.

Based on NRC staff review of the actual field data, the methodology previously derived has been modified to incorporate additional flexibility. These revisions include modifications to the quality ratings required for use in a particular placement zone, re-classification of the placement zones, reassessment of weighting factors based on the rock type, and more detailed procedures for computing rock quality and the amount of oversizing required.

Based on an examination of the actual field performance of various types and quality of rock (Esmiol, 1967, see Ref. D14), the NRC staff considers it important to determine rock properties with a petrographic examination. The case history data indicated that the singlemost important factor in rock deterioration was the presence of smectites and expanding lattice clay minerals. Therefore, if a petrographic examination indicates the presence of such minerals, the rock will not be suitable for long-term applications.

6.2 Design Procedures

Design procedures and criteria have been developed by the NRC staff for use in selecting and evaluating rock for use as riprap to survive long time periods. The methods are considered to be flexible enough to accommodate a wide range of rock types and a wide range of rock quality for use in various long-term stability applications.

The first step in the design process is to determine the quality of the rock, based on its physical properties. The second step is to determine the amount of oversizing needed, if the rock is not of good quality. Various combinations of good-quality rock and oversized marginal-quality rock may also be considered in the design, if necessary.

6.2.1 Procedures for Assessing Rock Quality

The suitability of rock to be used as a protective cover should be assessed by laboratory tests to determine the physical characteristics of the rocks. Several durability tests should be performed to classify the rock as being of poor, fair (intermediate), or good quality. For each rock source under consideration, the quality ratings should be based on the results of about three to four different durability test methods for initial screening and about six test methods for final sizing of the rock(s) selected for inclusion in the design. Procedures for determining the rock quality and determining a rock quality "score" are developed in Table D1.

6.2.2 Oversizing Criteria

Oversizing criteria vary, depending on the location where the rock will be placed. Areas that are frequently saturated are generally more vulnerable to weathering than occasionally-saturated areas where freeze/thaw and wet/dry cycles occur less frequently. The amount of oversizing to be applied will also depend on where the rock will be placed and its importance to the overall performance of the reclamation design. For the purposes of rock oversizing, the following criteria have been developed:

A. Critical Areas. These areas include, as a minimum, frequently-saturated areas, all channels, poorly-drained toes and aprons, control structures, and energy dissipation areas.

<u>Rating</u>

80-100 - No Oversizing Needed

65-80 - Oversize using factor of (80-Rating), expressed as the percent increase in rock diameter. For example, a rock with a rating of 70 will require oversizing of 10 percent. (See example of procedure application, given in Section 6.4, p. D-28)

Less than 65 - Reject

B. Non-Critical Areas. These areas include occasionally-saturated areas, top slopes, side slopes, and well-drained toes and aprons.

Rating

80-100 - No Oversizing Needed

50-80 - Oversize using factor of (80-Rating), expressed as the percent increase in rock diameter

Less than 50 - Reject

TABLE D1

Scoring Criteria for Determining Rock Quality

Laboratory Test	Weighting Factor			Score										
	Limestone	Sandstone	Igneous	10	9	8	7	6	5	4	3	2	1	0
					Good			Fair			Poor			
Sp. Gravity	12	6	9	2.75	2.70	2.65	2.60	2.55	2.50	2.45	2.40	2.35	2.40	2.25
Absorption, %	13	5	2	.1	.3	.5	.67	.83	1.0	1.5	2.0	2.5	3.0	3.5
Sodium Sulfate, %	4	3	11	1.0	3.0	5.0	6.7	8.3	10.0	12.5	15.0	20.0	25.0	30.0
L/A Abrasion (100 revs), %	1	8	1	1.0	3.0	5.0	6.7	8.3	10.0	12.5	15.0	20.0	25.0	30.0
Schmidt Hammer	11	13	3	70.0	65.0	60.0	54.0	47.0	40.0	32.0	24.0	16.0	8.0	0.0
Tensile Strength, psi	6	4	10	1400	1200	1000	833	666	500	400	300	200	100	0

1. Scores were derived from Tables 6.2, 6.5, and 6.7 of NUREG/CR-2642 - "Long-Term Survivability of Riprap for Armoring Uranium Mill Tailings and Covers: A Literature Review," 1982 (see Ref. D13).

2. Weighting Factors are derived from Table 7 of "Petrographic Investigations of Rock Durability and Comparisons of Various Test Procedures," by G. W. DuPuy, Engineering Geology, July, 1965 (see Ref. D15). Weighting factors are based on inverse of ranking of test methods for each rock type. Other tests may be used; weighting factors for these tests may be derived using Table 7, by counting upward from the bottom of the table.

3. Test methods should be standardized, if a standard test is available and should be those used in NUREG/CR-2642 (see Ref. D13), so that proper correlations can be made. This is particularly important for the tensile strength test, where several methods may be used; the method discussed by Nilsson (1962, see Ref. D16) for tensile strength was used in the scoring procedure.

6.3 Recommendations

Based on the performance histories of various rock types and the overall intent of achieving long-term stability, the following recommendations should be considered in assessing rock quality and determining riprap requirements for a particular design.

1. The rock that is to be used should first be qualitatively rated at least "fair" in a petrographic examination conducted by a geologist or engineer experienced in petrographic analysis. See NUREG/CR-4620, Table 6.4 (see Ref. D2), for general guidance on qualitative petrographic ratings. In addition, if a rock contains smectites or expanding lattice clay minerals, it will not be acceptable.

2. An occasionally-saturated area is defined as an area with underlying filter blankets and slopes that provide good drainage and are steep enough to preclude ponding, considering differential settlement, and are located well above normal groundwater levels; otherwise, the area is classified as frequently-saturated. Natural channels and relatively flat man-made diversion channels should be classified as frequently-saturated. Generally, any toe or apron located below grade should be classified as frequently-saturated; such toes and aprons are considered to be poorly-drained in most cases.

3. Using the scoring criteria given in Table D1, the results of a durability test determines the score; this score is then multiplied by the weighting factor for the particular rock type. The final rating should be calculated as the percentage of the maximum possible score for all durability tests that were performed. See example of procedure application for additional guidance on determining final rating.

4. For final selection and oversizing, the rating may be based on the durability tests indicated in the scoring criteria. Other tests may also

be substituted or added, as appropriate, depending on rock type and site-specific factors. The durability tests given in Table D1 are not intended to be all-inclusive. They represent some of the more commonly-used tests or tests where data may be published or readily-available. Designers may wish to use other tests than those presented; such an approach is acceptable. Scoring criteria may be developed for other tests, using procedures and references recommended in Table D1. Further, if a rock type barely fails to meet minimum criteria for placement in a particular area, with proper justification and documentation, it may be feasible to throw out the results of a test that may not be particularly applicable and substitute one or more tests with higher weighting factors, depending on the rock type or site location. In such cases, consideration should be given to performing <u>several</u> additional tests. The additional tests should be those that are among the most applicable tests for a specific rock type, as indicated by the highest weighting factors given in the scoring criteria for that rock type.

5. The percentage increase of oversizing should be applied to the <u>diameter</u> of the rock.

6. The oversizing calculations represent minimum increases. Rock sizes as large as practicable should be provided. (It is assumed, for example, that a 12-inch layer of 4-inch rock costs the same as a 12-inch layer of 6-inch rock.) The thickness of the rock layer should be based on the constructability of the layer, but should be at least 1.5 x D_{50}. Thicknesses of less than 6 inches may be difficult to construct, unless the rock size is relatively small.

6.4 Example of Procedure Application

It is proposed that a sandstone rock source will be used. The rock has been rated "fair" in a petrographic examination. Representative test results are given. Compute the amount of oversizing necessary.

Using the scoring criteria in Table D1, the following ratings are computed:

Lab Test	Result	Score	Weight	Score x Weight	Max. Score
Sp. Gr.	2.61	7	6	42	60
Absorp., %	1.22	4	5	20	50
Sod. Sulf., %	6.90	6	3	18	30
L.A. Abr., %	8.70	5	8	40	80
Sch. Ham.	51	6	13	78	130
Tens. Str., psi	670	6	4	24	40
Totals				222	390

The final rating is computed to be 222/390 or 57 percent. As discussed in Section 6.2, the rock is not suitable for use in frequently-saturated areas, but is suitable for use in occasionally-saturated areas, if oversized. The oversizing needed is equal to (80 - 57), or a 23 percent increase in rock diameter.

6.5 Limitations

The procedure previously presented is intended to provide an approximate quantitative method of assessing rock quality and rock durability. Although the procedure should provide rock of reasonable quality, additional data and studies are needed to establish performance histories of rock types that have a score of a specific magnitude. It should be emphasized that the procedure is only a more quantitative estimate of rock quality, based on USBR classification standards.

It should also be recognized that durability tests are not generally intended to determine if rock will actually deteriorate enough to adversely affect the stability of a reclaimed tailings pile for a design life of 200 to 1000 years. These tests are primarily intended to determine acceptability of rock for various construction purposes for design lifetimes much shorter than 1000 years. Therefore, although higher scores give a higher degree of confidence that significant deterioration will not occur, there is not complete assurance that deterioration will not occur. Further, typical construction projects rely on planned maintenance to correct deficiencies. It follows, then, that there is also less assurance that the oversizing methodology will actually result in rock that will only deteriorate a given amount in a specified time period. The amount of oversizing resulting from these calculations is based on the engineering judgment of the NRC staff, with the assistance of contractors. However, in keeping with the Management Position (USNRC, 1989, see Ref. D17), the staff considers that this methodology will provide reasonable assurance of the effectiveness of the rock over the design lifetime of the project.

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D14. Esmiol, Elbert E., "Rock as Upstream Slope Protection for Earth Dams - 149 Case Histories," U. S. Bureau of Reclamation, Report No. DD-3, September, 1967.

D15. Dupuy, G. W., "Petrographic Investigations of Rock Durability and Comparisons of Various Test Procedures, Engineering Geology," July, 1965.

D16. Nilsson, S., "Tensile Strength and Compressive Strength of Concrete Determined on the Same Cube," R.I.L.E.M. Bulletin 17, December, 1962.

D17. U. S. Nuclear Regulatory Commission (USNRC), "Uranium Mill Tailings Management Position," 1989.

APPENDIX E

RESPONSES TO PUBLIC COMMENTS

CONTENTS

APPENDIX E

RESPONSES TO PUBLIC COMMENTS

1. INTRODUCTION

The availability of the draft staff technical position (STP) was noticed in the Federal Register on August 13, 1989. Public comments were requested by October 13, 1989. Additionally, copies of the STP were sent directly to licensees, consultants, and other interested parties. Approximately 30 direct requests for copies of the STP were received after publication of the Federal Register notice.

2. SUMMARY OF MAJOR PUBLIC COMMENTS

Comments were provided by fifteen separate parties. The general thrust of comments provided by the uranium mill tailings industry was that use of the STP imposes unreasonable requirements; results in designs that are too conservative, relative to the risks imposed by tailings reclamation failure; and does not provide realistic and flexible approaches that use practical experience. Several commenters felt that use of the probable maximum precipitation (PMP) and the probable maximum flood (PMF) was too conservative. Several specific comments were duplicated word-for-word by other industry comments. In some cases, commenters indicated that the criteria and design methods in the STP may not be conservative enough, because a sufficient data base does not exist to document the acceptability of some design approaches. Comments were also provided that identified specific areas in the STP where additional clarification and explanation were needed to document the staff's rationale for the use of specific approaches and design methods. Comments received from non-industry factions generally indicated satisfaction with the STP.

3. ANALYSIS OF MAJOR PUBLIC COMMENTS

3.1 Requirements Imposed by STP

One of the most significant conclusions reached by the staff in analyzing public comments is that many commenters apparently do not understand that this STP only discusses several of many possible design methods and criteria that are acceptable to the staff in meeting long-term stability requirements. Commenters need to understand that the criteria and guidance presented in the STP are not requirements. Any licensee or designer has the option of using other design approaches. Simply because the staff has not developed or recommended other design appproaches for long-term stability does not mean that such approaches do not exist. The staff encourages designers to use their technical resources to develop and discover alternate design approaches to tailings stabilization. The staff welcomes the opportunity to meet with designers and to discuss these alternate approaches.

Comments received in this area resulted in several changes to the STP. Additional emphasis has been placed on the fact that the STP does not impose any design requirements.

3.2 Conservatisms in STP

The STP attempts to provide a balanced approach to tailings management, considering that designing erosion protection covers to be stable for 1000 years is a problem that has not been previously addressed in the engineering community. There will always be some question regarding the appropriateness of any procedure to provide adequate engineering designs for long time periods. Very little data are available in the technical literature to provide guidance in designing covers for long-term stability. Much of the information that is available has been developed by technical assistance contractors, where the efforts were funded by the Nuclear Regulatory Commission (NRC) staff. The STP attempts to use this information wherever possible, and to adapt standard engineering practice in similar areas, such as channel design, to the design of protective covers. Channel design procedures were selected because, in reality, a slope can be treated as nothing more than a wide channel. These procedures are widely used and based on the concept of providing channel sections that will not erode significantly when subjected to design flows. These channel design methods typically produce relatively flat slopes, which would appear to produce conservative designs. However, it should be recognized that channel design relies on maintenance to assure that the channel section remains in its design configuration. The regulations and standards do not permit maintenance to be performed to achieve the required stability period. However, channel design procedures have been verified by the actual field performance of agricultural drainage channels, and there is reasonable assurance that providing a stable slope (designed in accordance with stable channel procedures) will serve its intended function without maintenance. Hence, a balancing of design verification vs. the lack of maintenance results in the use of a balanced approach in designing erosion protection covers to last for 1000 years without reliance on routine maintenance.

The staff considers that some commenters have inappropriately concluded that the STP design methods do not incorporate the proper level of conservatism and further concludes that commenters should examine in detail the sensitivity and importance of various parameters that are used in the design procedures. In developing criteria for the design of soil covers, for example, the staff evaluated no less than ten different methods and found that most of these methods produce very similar results to the methods recommended in the STP.

Several changes have been made to the STP in this area. Additional discussion of conservatisms is provided in Section 4.1 (p. E-4) of this Appendix.

3.3 Risks of Tailings Reclamation Failure

Commenters indicated that, because of the minimal risks associated with tailings reclamation failure, it is unreasonable to design for 1000 years. Commenters emphasized that a 200-year design period is more appropriate in such cases. Based on examination of the applicable regulations and standards, the staff concludes that the design period should be 1000 years. However, the regulations and standards are flexible enough to permit a reduction in the design period, if it can be shown that the meeting the 1000-year stability criterion is impracticable. The STP provides the necessary guidance to permit licensees and designers to reduce the stability period and also provides qualitative guidance on how that reduction can be justified. The staff concludes that the regulations and standards, as currently written, must be met; that adequate criteria and guidance are provided in the STP to meet these regulations and standards; and that no significant modifications are needed to the STP to reflect these particular comments.

No changes were made to the STP in this area. Additional discussion of risks may be found in Section 4.3 (p. E-12) of this Appendix.

3.4 Flexibility and Use of Practical Experience

Commenters focused on the use of practical experience and other factors such as geomorphic evidence, rather than the use of empirical design procedures. The staff believes that commenters need to examine the causative mechanisms that would lead one to use geomorphic procedures to design protective covers. For example, if stable slopes are observed in a given area, one needs to examine why these observed slopes are stable; one probable reason for such stability could be that the soil is a rocky soil with a large average particle size. The staff concludes that if this rocky soil is proposed for use and the large soil particle size is used to calculate the allowable shear stress, the calculated stable slope will not be significantly different than the observed slope. However, it is not the intent of this STP to preclude the use of practical experience and field observations in designing stable covers.

The staff concludes that the STP offers designers sufficient latitude to design a wide variety of erosion protection cover options and is not overly conservative. If any recommended design method or imput parameter appears to be too conservative, the staff considers that commenters need to examine in detail the hydraulic design principles and rationale associated with its development or suggested usage. Commenters should also examine the close agreement between the recommended methods and other documented methods.

No changes were made to the STP to reflect these comments. Additional discussion may be found in Section 4.1 (p. E-4) of this Appendix.

3.5 Use of PMP/PMF

Commenters indicated that the use of the PMP/PMF is too conservative and that use of the SPF or historic floods of record was more appropriate. Commenters need to realize that the flood of record is often a substantial percentage (60 to 90 percent) of the PMF in many areas of the United States. If this flow rate is applied in the design procedures, the resulting differences in soil cover slope or rock size are usually insignificant, because the calculation procedures are more sensitive to the magnitude of the slope than to the magnitude of the flood event.

Several changes were made to the STP based on the comments received. Additional discussion of use of the PMP/PMF may be found in Section 4.2 (p. E-8) of this Appendix.

3.6 Clarifications and Additional Explanations

The staff agrees with many of the comments that suggested additional explanation and clarification. Changes have been made to the STP to reflect these comments.

4. ANALYSIS OF SPECIFIC COMMENTS AND ISSUES

Public comments were received in several specific areas. These comments have been grouped into 19 specific issue categories. Analyses of each comment received in these issue categories are presented below.

4.1 STP Conservatisms and Conformance with NRC Management Position

4.1.1 Commenters:

Pathfinder Mines Corporation
Exxon Coal and Minerals Co.
Hydro-Engineering
AK GeoConsult, Inc.
Steven R. Abt, Colorado State University
American Mining Congress

4.1.2 Summary of Comments

Commenters indicated that the STP is, in general, too conservative; does not reflect the philosophy expressed in the NRC Management Position (USNRC, 1989, see Ref. E1) on uranium mill tailings; does not represent standard industry practice; and does not set realistic goals for tailings reclamation. Commenters also indicated that the STP does not provide sufficient design flexibility and does not rely enough on engineering judgment and practical experience.

4.1.3 Analysis of Comments

In determining the level of protection needed to stabilize uranium mill tailings, the NRC staff has not arbitrarily established conservative design procedures. Contrary to the statements of various commenters, the staff has attempted to use the NRC Management Position as a broad philosophical framework for development of procedures that conform to standard engineering practice and that provide for a great deal of flexibility and engineering judgment. Most importantly, the design procedures are developed so that the requirements of 40 CFR Part 192 and 10 CFR Part 40, Appendix A, are met.

First, it should be emphasized that the STP does not provide design requirements; rather, the STP provides design criteria that are acceptable to the NRC staff. Any design options using different design strategies will be reviewed by the staff on a case-by-case basis.

Second, it should be emphasized that standard industry practice for stabilizing tailings sites for up to 1000 years does not currently exist. Typically, reclamation success is judged on successful stabilization for tens of years. Various strategies are employed to recontour, revegetate, and restructure mined areas so that minimal impacts are produced and so that applicable State and Federal regulations are met. These design standards do not require the degree of protection envisioned by tailings stabilization regulations. In the case of uranium mill tailings, different regulations apply, resulting in a need to use different reclamation strategies to comply with the long-term stability periods mandated by these regulations.

Third, standard practice <u>does</u> exist for providing stable soil sections. This practice is widely used to design drainage channels that do not erode when subjected to design flood flows. Since an embankment slope can be treated and designed as a wide channel, the staff concludes that the hydraulic design principles associated with this practice are generally appropriate for designing stable slopes that will not erode, and these principles have been used in the development of the design criteria presented in the STP. However, channel design practice normally assumes that some maintenance will be performed as damage and degradation occur.

Fourth, the staff concludes that the lack of flexibility perceived by various commenters is based on an incorrect perception that the criteria in the STP are requirements. The staff believes that a wide range of options and design approaches can be developed using the design criteria provided in the STP. For example, the STP provides criteria for a wide range of soil types, soil grain sizes, and soil cohesion. An infinite number of combinations of cover strategies can be developed using the criteria and methods in the STP. Only a few of these options were developed in detail in the STP.

Fifth, the STP makes no attempt to limit engineering judgment and practical experience from consideration in developing a design. Although the STP may not specifically address a particular design option mentioned by the commenters, the use of such an option is not precluded by the STP. The selection of input parameters to various models <u>is</u> largely a matter of engineering judgment. The

STP generally recommends a method or a reference for determining a particular design parameter; however, these recommendations are not intended to preclude the use of other methods, if such methods can be reasonably justified. Where there are large ranges in the value of a particular parameter, or where a parameter cannot be well-defined, or is not well-known, the guidance provided in the NRC Management Position (NRC, 1989, see Ref. E1) is clear:

> "In evaluating the magnitude of a design basis event or the acceptability of particular design criteria, reasonable ranges and distributions of parameters should be used. For well-known or accepted parameters with narrow empirical distributions or very narrow ranges, expected values should be used as appropriate. For less-well-known parameters, such as those estimated based on little empirical data or with broad distributions, conservative values should be chosen from within the observed distributions or estimated range. Extreme values should not be used. In any case, there should be a reasonable and defensible technical basis for the choice of a design basis event or design criteria...."

Sixth, the design procedures discussed in the STP were developed and recommended only after careful consideration and evaluation of many other procedures and methods that could have been selected. For example, several procedures were considered in selecting a method for designing stable soil covers. To illustrate the degree to which the staff evaluated other methods, following are brief discussions of some of the various procedures that were examined:

1. Horton/NRC Method. This method was developed by the NRC staff and was derived by using the original Horton Equation (1945, see Ref. E2) and substituting various input parameters. This method is considered to be applicable to a wide range of design options.

2. Lacey Method. Henderson (1966, see Ref. E3) provides information on the use of the Lacey Method, developed to design stable soil channel sections. The stable slope is dependent on the soil characteristics and the flow rate.

3. Simons-Albertson Method. Henderson (1966, see Ref. E3) discusses this method, developed for design of stable channels. The slope is dependent on the soil characteristics and the flow rate.

4. Blench Method. Henderson (1966, see Ref. E3) discusses this method, developed to determine stable channel slopes.

5. Critical Slope Method. For a given discharge, channel slope, and channel configuration, the critical slope can be calculated using the Manning's Equation (Chow, 1959, see Ref. E4). The unproven (but qualitatively reasonable) basis for this method is that if a slope is designed to produce subcritical or critical flow during the occurrence of the design discharge, it is likely that the flows will not be erosive, since erosion is more often associated with supercritical flow than with subcritical flow.

6. Stable Gully Method. This method assumes that regardless of attempts to produce and maintain an evenly-graded soil slope, some phenomena will occur to disrupt design conditions and produce a gully, rill, or swale with a depth of flow of at least one foot. (This procedure also considers that maintenance cannot be relied on to repair the damaged areas.) These assumptions of flow depth have no technical basis, and the depth could be more or less, depending on the phenomena involved. Concentrated flows will occur in these areas, even for events much less severe than the probable maximum design events. The assumed gully could be a small triangular-shaped gully or could be large and relatively wide. Regardless of the assumed cross-section, the stable slope for a flow depth of 1 foot can be calculated, as discussed in Appendix A, Section 3.2, p. A-15.

7. Empirical Observation Method. Nelson, et al. (1986, see Ref. E5) developed a method for determining 200-year and stable slopes, based on use of empirical data and field observations. This method is also discussed in Appendix B of the STP for the design of slopes that permit controlled erosion.

8. Geomorphic Method. Nelson, et al. (1983, see Ref. E6) discuss methods for determining stable slopes, based on examination of stable slopes in a specific area or region. This method uses field data, rather than computational procedures.

9. Permissible Velocity Method. The permissible velocity method is discussed in detail by Chow (1959, see Ref. E4). The method is widely used in many other applications to design stable channels. The staff has found that the method needs to be slightly modified to design stable sections where the flow depth is relatively shallow, since the method was originally intended to apply to channels where the flow depth is relatively deep.

10. U.S. Department of Agriculture (USDA)/Temple Method. This method for designing stable channels is discussed in detail by Temple, et al. (1987, see Ref. E7). The method can be used to design stable channels for any soil type, and where grass or vegetation is used.

Each of these methods was considered in detail, in arriving at a conclusion to use the methods recommended in the STP. As an exercise, the staff evaluated a hypothetical soil slope of a postulated length and assumed soil characteristics. The resulting stable slopes determined using the various methods are given below. The slope values given for the Empirical Observation Method and the Geomorphic Method were not actually calculated and are not specifically applicable to the hypothetical slope. The values are given for illustrative purposes only.

Method	Slope (ft/ft)
1. Horton/NRC	0.002

2.	Lacey	0.001
3.	Simons-Albertson	0.001
4.	Blench	0.003
5.	Critical Slope	0.005
6.	Stable Gully	0.002
7.	Empirical Observation	0.010
8.	Geomorphic	0.005
9.	Permissible Velocity	0.002
10.	USDA/Temple	0.002

Because the Horton/NRC Method is capable of accounting for all important variables in slope design, it was selected for use. In keeping with the philosophy of the NRC Management Position, it can be seen that this method is not the most or least conservative, and it was therefore considered to be acceptable for use in designing stable slopes.

Similarly, in designing rock covers, the staff considered various methods, including (but not necessarily limited to): (1) the Safety Factors Method; (2) the Stephenson Method; (3) the Corps of Engineers Method (USCOE, 1970, see Ref. E8); (4) methods discussed by Nelson, et al. (1986, see Ref. E5); (5) the Modified Isbash Method; (6) the U. S. Bureau of Reclamation Method; (7) the California Division of Highways Method; (8) the U. S. Bureau of Public Roads Method; (9) the Lane Method; (10) the Meyer-Peter Method; and (11) the Froude Number Method. The staff determined that the appropriate method depends on the slope and suggested that different methods should be used.

Seventh, and last, staff experience with reclamation designs in the Title I program provided the basis for many of the recommendations that were made in the STP. The staff also met with several authors of various publications, including Temple, Abt, and Nelson, to solicit opinions and recommendations.

In conclusion, it can be seen that the selection of a particular design procedure is based on its appropriateness, not its conservatism. Only after careful consideration of other methods was a procedure recommended for use in the STP. The same degree of consideration went into recommendations for determining input parameters to the other design procedures presented in the STP.

4.1.4 Changes Made

Additional explanation and clarification has been added to the STP regarding staff implementation of the NRC Management Position. Also, additional discussion regarding the conservatisms of the STP has been added.

4.2 Use of PMP and PMF for Designing Covers

4.2.1 Commenters: Pathfinder Mines Corp.
Exxon Coal and Minerals Co.
Hydro-Engineering
AK GeoConsult, Inc.
American Mining Congress

4.2.2 Summary of Comments

Commenters indicated that use of the probable maximum precipitation (PMP) and the probable maximum flood (PMF) for design of reclamation covers is overly conservative. Commenters indicated that some apparent contradictions exist in the STP regarding occurrence of the PMP/PMF.

4.2.3 Analysis of Comments

As indicated in the STP, the most disruptive natural phenomena affecting long-term tailings stabilization are likely to be wind and water erosion. These studies have also indicated that wind and water erosion can be mitigated by a cover of reasonable thickness and that the design of the protective cover will normally be controlled by the precipitation or flood event. Therefore, the selection of the design flood event assumes major importance in the overall reclamation plan.

In general engineering practice, selection of a design flood event must take into consideration the level of risk associated with that event. However, level of risk is difficult to quantify and is very site-specific. In setting the standards contained in 40 CFR Part 192, the Environmental Protection Agency (EPA) has attempted to quantify the risks associated with uranium mill tailings by requiring that control and stabilization will ensure, to the extent reasonably achievable, an effective life of 1,000 years, and in any case, at least 200 years. EPA has stated (EPA, 1983, see Ref. E9)

> "... tailings are vulnerable to human misuse and to dispersal by natural forces for an essentially indefinite period. In the long run, this threat of expanded, indefinite contamination overshadows the present dangers to public health."

It is apparent to the NRC staff that the standards were established because there are substantial long-term risks that must be considered from the standpoint of public health and safety. EPA further concluded that the risks from tailings could be accommodated by a design standard that requires that there be reasonable assurance that the tailings remain stable for a period of 1000 years, with reliance placed on passive controls (such as earth and rock covers), rather than routine maintenance.

Based on several reviews of remedial action plans, the NRC staff has found that the regulations and standards may be subject to different interpretations, especially with regard to computation and development of the design flood. The NRC staff has reviewed design flood computations using both statistical data and deterministic data. In general, use of statistical data to produce flood estimates has been found to be less appropriate than deterministic computations.

4.2.3.1 Use of Statistically-Derived Floods

Design floods are sometimes identified in terms of a recurrence interval, based on the probability of occurrence. The probability of occurrence is the probability that the flood will be equalled or exceeded in a given year and is equal to the inverse of the recurrence interval. For example, a 100-year flood has a probability of occurrence of 1/100, or 0.01 for any particular year.

One misconception that often occurs is equating the design period with the recurrence period of the design flood. It is often assumed that a 1000-year flood would be the appropriate flood to assure stability for a 1000-year design period. However, a 1000-year flood has a probability of occurrence of 63.2 percent during a 1000-year period. The probability of such a flood not occurring would thus be only 36.8 percent. The return period of a flood for a given probability of non-occurrence can also be calculated. For example, for a 1000-year design and a 90 percent probability of non-occurrence, the design flood would have a recurrence interval of approximately 10,000 years. Without discussing what probability constitutes reasonable assurance, it is clear that a design flood for a 1000-year design life must have a return period of many thousands of years.

Furthermore, extrapolation of limited site-specific data bases of 100 years or less (which is the case for most, if not all, uranium mill sites) is not likely to produce meaningful estimates of floods with recurrence intervals of 1000 years or more. The accuracy of flood data deteriorates for probabilities more rare than those defined by the period of record. This is due to the sampling error of the statistics from the data and the fact that the basic underlying distribution of flood data is not known exactly, especially for rare flood events.

Other procedures for estimating floods on a watershed can sometimes be used for evaluating rare flood flows. A comparison between flood and storm records at nearby hydrologically-similar watersheds will often expand the data base and aid in evaluating flood frequencies for a given watershed. The shorter the flood record and the more unusual a given flood event, the greater will be the need for such comparisons.

In determining the reliability of various flood estimates, the NRC staff has used data on historic maximum flood flows in the United States (Crippen and Bue, 1977, see Ref. E10). In that publication, selected historic maximum flood discharges are plotted vs. drainage area, and then enveloped, based on regionalization of the flood data.

Examination of these data for several tailings sites has indicated that 1,000-year and 10,000-year discharges (calculated by extrapolation of historic data using standard statistical techniques) may be underestimated when additional data from nearby or adjoining watersheds are added to the data base. If one makes the assumption that the regionalized data are representative of the stream in question, some of the flood discharges may be grossly underestimated. Based on historical data, flood flows significantly greater than the extrapolated 1,000-year or 10,000-year flood flows, for example, have occurred many times in nearby drainage basins, sometimes on basins much smaller than the one in question. Based on qualitative comparisons of the data, it is very unlikely that flood flows of such recurrence intervals would be exceeded many times. One may conclude that the flood flow data bases available for many streams are not truly representative and that statistically-derived floods are, thus, inappropriate for designing for very long time periods.

4.2.3.2 Use of PMF

An event that is commonly used for design purposes is the PMF, which is based on the occurrence of the PMP over appropriate parts of a watershed. The PMF is defined (U.S. Army Corps of Engineers, 1966, see Ref. E11) as the hypothetical flood (peak discharge, volume, and hydrograph shape) that is considered to be the most severe reasonably possible, based on comprehensive hydrometeorological application of the PMP and other hydrologic factors favorable for maximum flood runoff, such as sequential storms and snowmelt. The PMP is the estimated depth of rainfall for a given duration, drainage area, and time of year for which there is virtually no risk of exceedance. The PMP approaches and approximates the maximum that is physically possible within the limits of contemporary hydrometeorological knowledge and techniques. The estimation of a PMP is based on the concept that there is a limit to the amount of water that an atmospheric column can hold. Because the PMF is based on limitations imposed in part by site-specific meteorological capacities, the PMF represents a limiting value and removes uncertainties associated with extrapolation of limited data bases to extremely long time periods. In view of these uncertainties, the NRC staff concludes that it is reasonable and prudent to use the PMF as the design flood, where reasonable assurance of non-exceedance for a time period of 1000 years is desired.

However, it should be understood that estimating the PMF requires considerable judgment and experience. The degree of conservatism to be applied in estimating a PMF is often subject to considerable differences in interpretation, especially with regard to reasonable or appropriate values of model input parameters. The NRC staff has concluded that the intent of the regulations and standards will be met if certain conservatisms are considered in accounting for the limited quantitative data base currently available to document long-term degradation. Additional guidance and information are provided in the NRC Management Position (NRC, 1989, see Ref. E1) regarding selection of appropriate and reasonable design parameters where there is a limited data base, or where design parameters are not well-known.

4.2.4 Changes Made

Additional clarification has been added to the STP regarding the staff's rationale for selection of design floods. Several changes were made to remove any confusing language and to further clarify implementation of the NRC Management Position.

4.3 Risks of Tailings Reclamation Failure

4.3.1 Commenters:

Pathfinder Mines Corp.
Exxon Coal and Minerals, Co.
Hydro-Engineering
AK GeoConsult
American Mining Congress

4.3.2 Summary of Comments

Several commenters argued that, based on the location and remoteness of tailings sites, the risks associated with the failure of a tailings reclamation system are not great. Commenters indicated that the comparative risks associated with reclamation failure are much less than other societal risks and that design criteria, such as those developed for dams, should be applied to long-term stabilization.

4.3.3 Analysis of Comments

During the development of tailings reclamation standards and regulations, the argument concerning the relative risks of tailings reclamation failure was addressed many times by the NRC staff and other agencies, and the conclusions reached differ from the commenters' statements. EPA (EPA, 1983, see Ref. E9), for example, has stated:

> "The radiation hazard from tailings lasts for many hundreds of thousands of years, and some nonradioactive toxic chemicals persist indefinitely. The hazard from uranium tailings therefore must be viewed in two ways. In themselves, the tailings pose a present hazard to human health. Beyond this immediate, but generally limited, health threat, the tailings are vulnerable to human misuse and to dispersal by natural forces for an essentially indefinite period. In the long run, this threat of expanded, indefinite contamination overshadows the present dangers to public health. The Congressional report accompanying the Act expressed the view that the methods used for remedial actions should not be effective for only a short time. It stated: 'The committee believes that uranium mill tailings should be treated...in accordance with the substantial hazard they will

present until long after existing institutions can be expected to last in their present forms,' and that 'The Committee does not want to visit this problem again with additional aid. The remedial action must be done right the first time.' (H. R. Rep. No. 1480, 95th Cong., 2nd Sess., Pt. I, p. 17, and Pt. II, p. 40 (1978).)

We consider the single most important goal of control to be effective isolation and stabilization of tailings for as long a period of time as is reasonably feasible, because tailings will remain hazardous for hundreds of thousands of years...."

Based on the documented conclusions reached by Congress, EPA, and the NRC staff, it is apparent that the risks associated with tailings are not insignificant. Rather, tailings present a threat to public health and safety, particularly when viewed as a long-term problem. Standards and regulations have been promulgated to control this significant long-term problem. It has been concluded and documented that the risks are significant enough to warrant a design that will last for a period of 1000 years, without the need for maintenance. This STP merely documents acceptable ways to meet the stated requirements. It is not the intention of the staff to revisit the basis for these requirements in this STP.

The staff is, however, aware of other technical viewpoints. Various distinguished technical experts have presented arguments that indicate that the risks from tailings are not great, when compared to dangerous activities in life. A basis may exist for modifying the standards and regulations, but until that is done, the staff is not in a position to do anything other than ensure that these requirements are met.

4.3.4 Changes Made

No changes have been made to the STP to accommodate these comments.

4.4 <u>Use of 200-year and 1000-year Sacrificial Slope Designs</u>

4.4.1 Commenters: Pathfinder Mines Corp.
AK GeoConsult, Inc.
Hydro-Engineering
U. S. Dept. of Energy (USDOE)
Steven R. Abt, Colorado State University
American Mining Congress

4.4.2 Summary of Comments

a. Commenters indicated that sacrificial outslopes, designed to be stable for 1000 years, should be allowed in the STP, and that a 200-year design life of such slopes should not be imposed. One commenter noted that such designs had already been approved by NRC, and that to now revise the design criteria is not appropriate.

b. One commenter indicated that the procedure lacked a sufficient data base and should not be used at all.

c. One commenter suggested that the language in the STP related to cost-effectiveness is confusing and should be clarified.

d. One commenter requested a definition of parameters in the equations, including "H" and transitional slope.

e. One commenter indicated that tailings dam slopes that do not receive runoff from covered tailings can be designed to allow for some erosion during the 1000-year design life. In some cases, significant erosion can be acceptable. The commenter goes on to state that experience with slopes of various ages indicates that by limiting drainage areas, gullying can be significantly reduced for relatively steep 2.5:1 to 5:1 slopes.

f. One commenter indicated that the corrections provided for fine-grained soils are inadequate.

g. Two commenters discouraged the use of extrapolations in Figure 4.4 when using the procedures discussed by Nelson, et al., (1986, see Ref. E5).

h. One commenter indicated that sheet erosion and rill erosion need to be included in the sacrificial slope analyses.

4.4.3 Analysis of Comments

a. The procedure for determining sacrificial slope requirements is limited to a 200-year design. Recently-approved reclamation plans feature outslopes designed for a stability period of 200 years, and not 1000 years, as was asserted by one of the commenters. A 200-year period is suggested for use in design because the technical assistance contractors who developed the procedure recommended that the procedure be limited to time periods of 200 years or less (Nelson, et al., 1986, see Ref. E5). This limitation is necessary because the procedure is based on a very limited data base, and the equation used to calculate the transitional slope may not be valid for time periods exceeding 200 years. Thus, at the present time, the staff is not aware of any procedure for predicting gullying for more than a 200-year period.

   It should be emphasized that if licensees and designers are aware of any procedures to design sacrificial slopes for more than 200 years or for as long as 1000 years, such procedures should be presented for review by the staff on a case-by-case basis.

b. The staff is well aware of the limited data base used to develop the procedure, and because of this, has placed several limitations on its use. These limitations include restricting the procedure to a design life of 200 years and using the procedure only at sites where there are no tailings or waste materials under the soil outslopes where gullying will occur. In addition, a portion of the procedure has been modified to provide more accurate and flexible methods for determining the stable slope. The staff considers that the modified procedure and limitations on its use compensate for the lack of an extensive data base and provide reasonable methods for estimating gullying over a period of 200 years. The staff further considers that licensees will use this procedure in a limited number of cases and will provide extensive justification to demonstrate that designing for longer time periods is not reasonably acheivable.

c. The staff agrees that cost effectiveness will be a consideration throughout the design process. It is expected that applicants and licensees will consider the costs of various design alternatives and will propose the alternative that meets regulatory requirements, at the lowest cost. It was not the intent of the staff to infer that costs should not be considered in any design.

d. The staff agrees that all parameters in the equations should be defined and has made appropriate changes to the STP.

e. The staff's procedure for estimating a stable slope is based on the Horton equation. As discussed in Appendix A of the STP, Horton determined that erosion would not occur on a slope unless the eroding force exceeded the resistance of the soil to erosion. The distance from the watershed divide to the point at which the eroding force became equal to the resistance of the soil, was referred to by Horton as the critical distance xc (see page A-2 of the STP). The staff, in adopting the Horton equation, assumed that the critical distance xc would be equal to the distance from the top of the slope. Therefore, the staff's stable slope equation already assumes no runoff from the covered pile.

   The problem with comparing existing engineered slopes with slopes designed to remain stable for 1000 years without active maintenance is that there are very few examples of slopes that have knowingly survived for periods of 1000 years. Archaeologists have documented examples of prehistoric mounds that have survived for periods in excess of 1000 years. However, in most cases, these mounds were located in relatively humid climates that supported vegetation and generally received some maintenance. In many cases, the mounds were covered with rock or had other erosion protection features (Lindsey, <u>et al</u>., 1983, see Ref. E12). Presently, very little information exists on how to design slopes to remain stable for 1000 years without maintenance. Because of this basic lack of design experience and technical information, the staff's stable slope procedure necessarily has to be conservative in order to provide reasonable assurance that tailings will not be exposed or released to the environment over a 1000-year period.

f. The staff agrees that some fine-grained, cohesive soils may provide greater erosion resistance than sandy, non-cohesive soils. The erosion of soils by water is a complex engineering problem that involves various types of erosive processes. These processes are fairly well-understood in the case of granular soils. However, they are not as well-understood in the case of cohesive soils, and as a result, the amount of information available for use in designing earth channels in cohesive soils is very limited. Most of the research performed has included Plasticity Index (PI) as one important indicator of soil resistance to flowing water. Generally, as PI increases, so does the shear strength of the soil. However, other factors such as dispersion ratio, compaction, slope and mineralogical composition are also important factors that affect the erosion resistance of cohesive soils.

   Because of the complexity and lack of information regarding the erosion resistance of cohesive soils, the staff recommends a method developed by Temple, et al. (1987, see Ref. E7) for determining the allowable effective stress of soil. This method considers the cohesive properties of soil, and the method is easy to apply. However, it is not intended to be a design requirement, and other methods may be used, if properly justified.

g. The sacrificial slope procedure discussed by Nelson, et al. (1986, see Ref. E5) requires the use of Figure 4.4 in that reference. Two commenters discouraged extrapolating the data in this figure. The staff agrees with this comment and believes that extrapolations may not be necessary. An examination of Figure 4.4 shows that if the value of St x Cu is small, such that the curve has to be extrapolated to the left, the estimated location of the gully would be close to the top of the outslope and would probably intersect tailings, and the design would, therefore, not be acceptable. In this case, the designer would, by necessity, have to reiterate the analyses, assuming a flatter outslope and/or a larger setback distance. Large values of St x Cu at the other end of the curve indicate that the design is acceptable because the gully would form close to the toe of the outslope. The staff therefore concludes that the curve on Figure 4.4 does not have to be extrapolated.

h. The inclusion of sheet and rill erosion in the sacrificial slope analysis is recognized by the staff, in the STP, where it is stated:

> "The additional soil cover needed to account for wind and sheet erosion needs to be factored into the soil cover design."

4.4.4 Changes Made

In response to comments received concerning sacrificial slope designs, appropriate changes have been made to the STP.

4.5 Vegetation

4.5.1 Commenters: Pathfinder Mines Corp.
Exxon Coal and Minerals Co.
Colorado Dept. of Health
USDOE
Steven R. Abt, Colorado State University

4.5.2 Summary of Comments

a. Commenters indicated that the technical basis for design of vegetation covers should be further expanded and that recent studies should be considered. Commenters indicated that the discussion of vegetation is superficial and that more discussion should be provided related to climate, species, soil types, soil densities, and surface treatments.

b. Several commenters indicated that the terms "vegetated cover" and "ground cover" are not interchangeable.

c. Several commenters disagreed with the conclusion that vegetation may increase erosion. One commenter suggested that the STP should acknowledge that vegetation reduces infiltration.

d. One commenter indicated that the STP should address root penetration due to vegetation.

e. One commenter expressed the opinion that vegetation is equally as effective as rock on slopes of 5 percent or less.

f. One commenter questioned the long-term survivability of plant species in arid and semi-arid climates.

g. Several commenters questioned the basis for selection of cover density as a means of determining flow concentrations and receiving credit for vegetation.

h. One commenter suggested that if vegetation is considered in a design, the applicant should construct a test plot and document that the selected plant species will be self-sustaining and sufficiently dense over the long term. Another commenter stated that field trials for determining the capability of vegetation to survive over long time periods is not practical.

4.5.3 Analysis of Comments

a. The staff agrees that the discussion on vegetation should be expanded; however, at the present time, very little data exist on designing vegetative covers that will remain effective for 200 to 1000 years. Generally, the more vegetative cover present, the more erosion-resistant a soil will be and the less soil will be eroded. However, for design purposes, the relationship between vegetative cover and soil loss must be quantified. The quantitative relationship between the amount of vegetation and the rate of erosion is very difficult to determine because of the many interacting factors involved. Some of these factors are climate, soil properties, topography, spatial and temporal distribution of rainstorms, and the ability of individual plant species to survive and reproduce. In addition, vegetation in an area can change due to environmental stresses such as droughts, fires, and succession of plant species.

Because of the lack of information on designing vegetative covers, the staff considered not including the subject in the STP. However, since there are many areas of the United States where there is ample precipitation to sustain an adequate native vegetative cover, the staff thought there was a need to address the subject, using available information. The staff believes that the most complete design methods are discussed by Temple, et al. (1987, see Ref.E7) and recommends that reference for use.

Admittedly, any design relationship will be an oversimplification of all the interacting factors that affect vegetation. However, the staff considers that at the present time, the Temple Method is a very good procedure, based on available information. Designers are urged to consult this reference for additional information and data. The staff also recognizes that there may be more technically adequate methods for designing vegetative covers and thus encourages licensees and consultants to perform additional studies necessary to form a firm technical basis for designing vegetative covers.

As suggested by the commenter, the staff reviewed a recent report on a field-scale demonstration study conducted to evaluate the effects of natural precipitation on low-level waste covers. The surface of the covers consisted of either bare soil or gravel mulch with grass and shrubs. Preliminary results of this study showed a significant difference in sediment load transport between mulched and unmulched covers. There was no significant difference in sediment loads between vegetation types.

In another report suggested for staff review by the commenter, it was concluded that vegetative covers are appropriate for use on topslopes at certain sites. It was also concluded that rock mulches and rock biointrusion barriers can enhance durability and performance of pile covers.

Staff review of the reports indicates that rock mulches and vegetation can be useful in stabilizing tailings piles. It should be noted that the STP has been modified to include discussion and criteria for the design of rock mulches.

b. The staff agrees with commenters that the terms "vegetated cover" and "ground cover" may not be interchangeable. In general, vegetative covers include only live transpiring plant tissue. Ground cover may include dead vegetation and rocks. The design of a vegetative cover should be based only on the amount of live vegetation that is present. Appropriate changes have been made to the STP.

c. The staff concurs with those commenters who disagreed with the statement that vegetation may increase erosion. This statement is generally not true, and appropriate changes to the STP have been made. There is no doubt that vegetation reduces infiltration when compared to bare soil or riprap, and as suggested by one of the commenters, the STP has been revised to acknowledge this. However, there may be some cases where the presence of trees and/or woody plants could cause disruptions in surface flow over a reclaimed slope. These disruptions could produce localized turbulence or undesirable flow patterns that could cause additional erosion. The STP has been reworded and clarified in several areas.

d. One of the commenters suggested that root penetration should be addressed in the STP. The staff recognizes that there are grasses, shrubs, and trees that normally have root systems that may extend several feet downward. Roots from these grasses may penetrate radon barriers, thus creating pathways that may increase radon flux and infiltration. The potential for roots to penetrate the entire thickness of a radon cover is dependent on the characteristics of the radon barrier materials. Soils having unfavorable chemical or physical properties will reduce root penetration, as will the highly compacted clayey soils that are typically used for radon covers.

Root penetration as a potential problem is not limited to vegetated covers. DOE reports that even piles having substantial thicknesses of rock riprap have been invaded by deep-rooted forbs and trees. DOE has considered a buried layer of cobbles as a means of limiting root penetration.

At the present time, it is believed that roots that penetrate a radon barrier will not have a significant adverse impact on the stability of a cover. The staff, therefore, is not suggesting the use of any design features for preventing root penetration. This position, however, may change as more knowledge is gained from actual field studies.

e. The staff has no basis for agreeing or disagreeing with the comment that vegetation is equally as effective as rock on slopes of five percent or less. In areas where a dense vegetative cover can be sustained, this comment may be true. In arid areas, this statement may not be true. However, there are a number of other factors that affect the design, other than the magnitude of the slope.

f. The staff agrees with the commenter who questioned the long-term survivability of plant species in arid and semi-arid climates. As stated in the STP, the survivability of the cover will need to be documented in sufficient detail to justify its use.

g. The staff agrees that cover density should not be used to determine the flow concentration factor (FCF). The STP has been revised to provide a recommended FCF value of 3, unless other values can be justified. The credit to be obtained from using a vegetated cover is more appropriately derived from using the design procedures discussed by Temple, where erosion resistance is a function of vegetation density.

h. The staff believes that the vegetation density should be based on the amount of naturally-occurring vegetation in the site area and should be based on the actual density and occurrence of native species. The construction of test plots to document the viability of new species will be interesting, but probably will not be beneficial in documenting the credit that should be taken for plant species that do not naturally occur.

4.5.4 Changes Made

Several changes have been made to the STP, as discussed above.

4.6 Rock Mulches

4.6.1 Commenters:

USDOE
Colorado Dept. of Health
Homestake Mining Co.

4.6.2 Summary of Comments

Several commenters indicated that rock mulches should be permitted and that design guidance should be presented in the STP. One commenter suggested that guidance be provided for rock mulches combined with vegetation. Another suggested that designing for various other considerations be discussed.

4.6.3 Analysis of Comments

The NRC staff never intended to discourage the use of rock mulches as erosion protection. In spite of the fact that they were not specifically addressed in the draft STP, the staff intended that rock mulches could be designed using the rock cover criteria given in Appendix D. It was intended that the rock mulch layer would be treated as riprap with a small average rock size. Based on examination of field data, it appears that the inclusion of a gravel layer or a rocky soil may provide the precise solution needed at many sites.

4.6.4 Changes Made

The STP has been modified to include discussion of rock mulches. Appendix D now indicates that rock design criteria are applicable to design of rock mulches, where the average rock size is relatively small.

4.7 Use of Standard Project Flood and Other Reductions in Design

4.7.1 Commenters:

AK GeoConsult, Inc.
Advisory Committee on Nuclear Waste (ACNW)
U. S. Dept. of the Interior
Hydro-Engineering

4.7.2 Summary of Comments

Commenters requested that additional justification and explanation be provided regarding the rationale for departing from the 1000-year design period. Several commenters questioned the basis for using the Standard Project Flood (SPF) and for reducing the design longevity period when designing for 1000 years is not reasonably achievable. One commenter indicated that the SPF may offer less flood protection than the 1000-year flood and that the design flood should be the 1000-year flood or the flood of record, whichever is greater.

4.7.3 Analysis of Comments

10 CFR Part 40, Appendix A, and 40 CFR Part 192 require that stabilization designs provide assurance that tailings will remain stable for a period of 1000 years to the extent reasonably achievable and in any case, for at least 200 years. It can be seen that these regulations permit licensees and applicants to design for a shorter period (with a 200-year minimum), when designing for a 1000-year period is not reasonably achievable. In most cases, the overriding issue is determining if a longer design period is not reasonably achievable. Detailed procedures for addressing this determination are given in Appendix C of the STP.

As stated in Section 2.2.1 (p. 5) of the STP, the staff concludes that PMP/PMF events meet the 1000-year stability criterion. The basis for this acceptance is also discussed in Section 4.2 (p. E-8) of this Appendix. If a licensee or applicant can justify that a 1000-year design is not reasonably achievable, a smaller design storm event may be used for design. However, the storm event must result in a design that will provide stability for a period of at least 200 years. The staff has determined that the SPF or the maximum regional flood of record, whichever is greater, will meet this stability requirement.

The SPF, by definition, is a flood that may be expected from the most severe combination of meteorological and hydrologic conditions that are considered reasonably characteristic of the geographical region involved, excluding extremely rare combinations (USCOE, 1966, see Ref. E11). Therefore, by definition, the SPF is a large flood event. The SPF is used by the USCOE to design flood control projects in areas where protection of human life or unusually valuable property is involved.

In Section 2.2.1 (p. 5) of the STP, the staff states that in general, the SPF will have a magnitude of approximately 40 to 60 percent of the PMF. In areas where specific procedures for estimating a SPF have not been derived, the staff will accept 50 percent of a PMF as representing a SPF.

The PMF is derived from PMP values published in various hydrometeorological reports. In the western United States, where most uranium mills are located, the PMP is usually at least five times greater than the 100-year rainfall (Nelson, et al., 1986, see Ref. E5). Fifty percent of the PMP is thus about 2.5 times greater than the 100-year rainfall. The staff has concluded that since one-half of the PMP is 2.5 times greater than the 100-year rainfall, the SPF is likely to have a recurrence interval of 200 years or greater in the western United States.

The STP suggests that the reclamation design should be based on the greater of the SPF or the regional flood of record. In many cases, the regional flood of record is greater than 75 percent of the PMF, on a discharge-per-square-mile basis. Based on staff examination of historic flood data (Crippen and Bue, 1977, see Ref. E10), it can be seen that the regional flood of record will be the controlling design event in almost all cases. It is not expected that the SPF will often be the design basis flood event. The SPF was suggested for use in the unlikely event that an adequate regional flood data base does not exist; it would then be a relatively simple matter to calculate the SPF as 40 to 60 percent of the PMF.

4.7.4 Changes Made

No changes have been made to the STP in these areas.

4.8 Miscellaneous Comments and Clarification Requests

4.8.1 Commenters:

AK GeoConsult, Inc.
Exxon Coal and Minerals Co.
New Mexico Health and Environmental Dept.
USDOE
Illinois Dept. of Nuclear Safety
American Mining Congress
MK-Environmental Services

4.8.2 Summary of Comments

Various commenters suggested the following clarifications, editorial changes, and modifications:

a. Clarify the meaning of the statement that a significant increase in drainage area could occur on an unstable slope over a long period of time.

b. Clarify the meaning of "sufficiently flat."

c. Clarify the statement that "lower velocities than those used by Chow" should be selected.

d. Clarify the meaning of "The maximum permissible velocity must be less."

e. The information contained in Appendix D should be summarized in Section 2.2.6 or in the body of the STP.

f. Cost considerations and conclusions should be summarized in the Executive Summary or the Introduction.

g. Aerial photographs are not always easily obtained and should not always be necessary.

h. The PI of the soil in the example problem is not correct.

i. The STP should include the use of more current sources of Manning's "n" values than those presented in Chow.

j. Clarify that the 8.2-minute PMP is for the top and side slopes combined.

k. Reclamation designs and design criteria should be developed by qualified professionals.

l. The equation on p. A-3 is incorrect regarding flood discharge, and the Horton Equation cannot be derived using this relationship.

m. One commenter cited several typographical errors and provided suggestions for corrections.

n. Two commenters provided several suggestions for rewording and clarification of confusing statements.

o. One commenter objected to the statement that "only very gentle slopes may be assumed to be stable."

4.8.3 Analysis of Comments

a. As rills and gullies form and expand on unstable slopes, a drainage network forms. Network development and the tendency of rills to widen, deepen, extend their length and capture other rills and streams, are discussed by Shelton (1966, see Ref. E13). The process of drainage network development is discussed by Ritter (1978, see Ref. E14) who states:

> "It is now generally recognized that rills initiated on a slope cannot long remain as parallel unconnected channels. Deeper and wider rills develop where the length over which erosion can occur is the greatest. These master rills carry more water and, because of the greater depth, they undergo downcutting until all the flow is contained within the channel and the rill becomes a tiny stream. Because they become slightly entrenched, master rills capture adjacent rills when bank caving or overtopping during high flow destroys the narrow divides between them. The repeated diversion of rills, a process called micropiracy, tends to obliterate the original rill distribution, and gradually, the initial slope parallel to the master channel is replaced by slopes on each side that slant toward the main drainage line.
>
> The development of new slope direction in accordance with the master channel is called cross-grading.... In the final stage, only one stream, confined in the master rill channel, crosses the slope. The side slopes presumably develop a new rill system graded to the position of the initial stream, and the process repeats itself, culminating in a secondary master rill serving as an incipient tributary. Each similar tributary evolves in a similar way until the network of streams takes form.
>
> The network pattern develops by repeated division of single channel segments into two branches, a process known as bifurcation...the angle between the limbs of a bifurcated channel probably evolves in one of three possible ways: (1) both limbs grow headward while preserving the original at their juncture; (2) one branch straightens its course and becomes dominant; or (3) the angle on steep slopes progressively decreases until the branches reunite into a single channel. Any or all of these procedures might be found in the evolution of a network, constrained only by the fundamental erosive controls and the geologic framework."

Recognizing that a drainage network will develop over a long period of time, it is necessary to assume larger drainage areas than those that would be expected initially. This is accomplished by using the flow concentration factor recommended in the STP.

b. A slope is considered by the staff to be "sufficiently flat" if it can be shown by analysis to be stable, i.e., to inhibit the initiation, development, and growth of gullies. It is not possible to define a typical "sufficiently flat" slope because it is dependent on many variables such as length of slope, soil shear stress, and roughness. In general, however, a "sufficiently flat" slope would be considered by the staff to produce subcritical flow, rather than supercritical flow, over its entire length, and to be non-erosive even if minor gullying occurred.

c. Section 2.2.5 (p. 8) of the STP discusses the staff meaning of the phrase "lower velocities than those shown by Chow should be selected." The fourth paragraph reads as follows:

> "In an open channel, flow velocities vary vertically along the channel section. Generally, the maximum velocity occurs just below the free surface. The velocity decreases with depth, reaching a minimum value near the channel bottom. Consequently, the permissible velocity along the channel bottom is much less than the maximum mean velocities... Chow provides reduction factors for the permissible velocity, based on the flow depth. It can be seen that the permissible velocity decreases noticeably at lower depths of flow. If Chow's data are extrapolated to a flow depth of several inches, the recommended reduction in permissible velocity is about 50 percent."

d. As stated above, in an open channel, flow velocities vary vertically and reach a maximum near the top and a minimum along the bottom of the channel. Consequently, the actual permissible velocity along the bottom is less than the average values given by Chow. For flows on tailings pile slopes where the flow will be only several inches deep, the permissible average velocity values given by Chow should be reduced by 50 percent. This will probably result in maximum permissible velocities of less than 2 feet per second (fps) for many soil types. In the past, NRC had informally stated that 3 fps was an acceptable design velocity for most channel design applications. This value, while probably acceptable for design of most earth channels where the flow is relatively deep, should not be used for flow on a pile top slope, unless justification can be provided for its use.

e. The Executive Summary already summarizes all the appendices, including Appendix D. In addition, Section 3 references and discusses Appendix D. Therefore, the staff concludes that it is not necessary to include another summary of Appendix D in Section 2.2.6.

f. The staff expects that a tailings pile will be designed to remain stable for 1000 years, unless it can be shown that designing for a 1000-year period is not reasonably achievable. One way of demonstrating that a design is not reasonably achievable is total project cost. If it can be clearly shown that a 1000-year design is not practicable because of excessive costs, then a licensee or applicant can design for a shorter time period. In no case, however, can that time period be less than 200 years. In fact, it should be as close to 1000 years as is practicable.

   Appendix C has been slightly expanded to further discuss costs.

g. The staff agrees that it is not always practicable to obtain aerial photos of an entire upstream drainage area. However, in cases where available topography is not adequate to describe drainage areas, it may be necessary to obtain aerial coverage or additional maps.

h. The staff agrees that a CH soil cannot have a PI as low as 15. A CH soil has a PI of about 22 or greater. The example problem has been revised.

i. Channel roughness coefficients can be obtained from sources other than those cited in the STP. The sources in the STP are only recommended, and are not required.

j. As suggested, the computation of the 8.2-minute PMP duration has been clarified in the example problem.

k. In accepting a reclamation plan, the staff is not concerned about who prepares the plan. The staff is only concerned with the technical merit of the plan. The staff does, however, agree that reclamation plans should be developed and reviewed by competent individuals.

l. The equation shown on page A-3 for a stable slope (Ss) was derived from the Horton equation, using certain assumptions. These assumptions are given in the STP.

   The equation can also be derived by simultaneous solution and substitution of the Manning's Equation, the Rational Formula, and the peak shear stress equation.

   Additional clarification has been added to the STP to better show how these derivations were developed.

m.,n. Typographical and other inconsistencies have been corrected, as suggested by the commenters.

o. One commenter did not agree with the statement that, "...only very gentle slopes may be assumed to be stable," because around the world, there are steep ancient slopes that are erosionally stable. The stability of these ancient slopes results from a combination of soil properties, vegetation, desert pavement, and stable base levels. The staff agrees that steeper slopes can be stable, if the soil is stabilized with vegetation, gravel, and/or other features that characterize typical stable slopes. However, a soil slope devoid of rock and vegetation is generally only stable if it is very flat. Licensees are encouraged to duplicate the features of naturally-occurring stable slopes using gravel, vegetation, or rock mulches.

4.8.4 Changes Made

Several changes have been made to the STP, as discussed above.

4.9 Scope of Position

4.9.1 Commenters: Advisory Committee on Nuclear Waste
Colorado Dept. of Health
Homestake Mining Co.
Exxon Coal and Minerals Co.

4.9.2 Summary of Comments

a. Several commenters suggested that the STP should be expanded to deal with other technical issues such as infiltration, seismic stability, and groundwater considerations. They indicated that the position needs to be expanded to guide a balancing of infiltration versus erosion protection and should consider a systems approach in designing reclamation plans.

b. One commenter indicated that the STP should include procedures for scheduling and tracking of NRC reviews of licensee reclamation plans.

4.9.3 Analysis of Comments

a. The NRC staff agrees with the necessity of using a systems approach in stabilizing uranium mill tailings. Uranium mill tailings regulations and associated guidance documents, including this STP, were developed with a systems approach. Each regulation or guidance document was promulgated with full recognition of the importance of its integration into an overall regulatory framework. This integration was, and is, achieved through the use of technical staff who are knowledgeable of the total program, peer review within NRC, comments from interested members of the public, comments from the Advisory Committee on Nuclear Waste (ACNW), and effective management direction and oversight. The staff believes that the STP, when placed in the context of other regulations and guidance, is appropriately integrated into a systems approach and into a regulatory program that has placed all important technical issues in their proper relationship. Any text that suggested or implied that this is not the case has been revised.

The criteria in the STP on erosion protection covers were provided only as guidance in dealing with one of the applicable regulations for uranium mill tailings reclamation. The design of an erosion protection cover is only a small part of the total reclamation design. For example, 10 CFR Part 40, Appendix A provides several other criteria that must be met, and each of these criteria must be considered in developing a complete design.

Guidance is currently available in many other areas to determine compliance with other portions of the regulations. The staff has revised the STP to indicate that guidance is available and to identify any long-term stability design considerations that must be addressed in meeting other portions of the regulations.

b. The STP is a generic technical position addressing technical approaches for long-term stability. It is not appropriate to provide generic guidance on the scheduling of specific reviews.

4.9.4 Changes Made

Several changes have been made to remove any implication that an integrated systems approach is not used by the NRC staff in dealing with long-term stability and tailings reclamation. Changes have been made to indicate that guidance is available in many other design areas.

4.10 Use of Permissible Velocity Approach

4.10.1 Commenters:

AK GeoConsult, Inc.
USDOE
Illinois Dept. of Nuclear Safety
Steven R. Abt, Colorado State University
American Mining Congress
MK-Environmental Services

4.10.2 Summary of Comments

a. Several commenters requested additional clarification regarding use of the permissible velocity approach for designing stable soil slopes. Several commenters suggested that Chow's data should not be extrapolated and that clarification is needed on the selection of permissible velocities. Several commenters suggested that permissible velocity methods be used where flow depths are appropriate. One commenter expressed support for the use of a shear stress approach, rather than the permissible velocity approach.

b. Several commenters suggested the use of other references for determining permissible velocities.

4.10.3 Analysis of Comments

a. The staff does not generally support the use of the permissible velocity method, primarily due to the potential for its misuse and misapplication. Staff experience indicates that there is usually some misunderstanding regarding proper application of permissible velocities.

   The method was intended to be applied to drainage channels where the depths of flow are relatively large. As explained in the STP, reductions are needed in typical values of permissible velocity to account for the shallow flow depths encountered on broad soil slopes. However, the staff agrees that use of this approach may be acceptable, if properly applied. Therefore, additional guidance has been provided in the STP for the design of stable slopes and channels using the permissible velocity approach.

b. The STP discusses only a few of the the many references that could be used to determine permissible velocities. The use of other references may be acceptable, and the staff encourages comparison of published values in arriving at reasonable and technically-defensible estimates of permissible velocities.

4.10.4 Changes Made

Appendix A has been modified to provide design criteria for slopes and channels, using the permissible velocity method. Also, the permissible velocity method has been further explained and clarified in the text.

4.11 <u>Rock Placement</u>

4.11.1 Commenters: AK GeoConsult, Inc.
Colorado Dept. of Health
Hydro-Engineering
Homestake Mining Co.
Exxon Coal and Minerals Co.
American Mining Congress

4.11.2 Summary of Comments

a. Several commenters indicated that the rock placement techniques suggested in the STP were unnecessary and disagreed with the suggested placement techniques. They questioned the need for hand placement and/or special techniques to achieve a uniform rock layer. Several commenters suggested that it was not important to always achieve the design rock thickness. Commenters also noted that uniformity is less important than meeting the minimum thickness requirements. One commenter suggested that rock placement be evaluated based on performance and not be dependent on procedure. One commenter agreed with the NRC staff suggestions regarding rock placement.

b. One commenter indicated that rock layers less than 6 inches thick are routinely placed by experienced equipment operators.

c. Several commenters indicated that NRC studies show that rock thicknesses of 1½ times the average rock size, or not less than the maximum rock size in the layer, are adequate.

4.11.3 Analysis of Comments

a. The NRC staff has extensive experience in the placement of rock riprap layers. This experience, reflected in the STP, has generally indicated that many designers do not understand the importance of placing riprap in a satisfactory manner. The staff disagrees with commenters' objections to achieving uniform and proper rock placement. Commenters have not considered that the design of a riprap channel can be significantly affected by rock placement, even if the amount of poor placement is not great. Commenters fail to consider that the presence of void areas in a channel can cause the formation of erosion pockets where erosive forces can exceed the normal stresses created by flood flows. If voids and gaps are present in the rock layer, the rock protection can be exposed to a phenomenon known as "plucking," which is described in detail by Longwell and Flint (1956, see Ref. E15). In general, this phenomenon is caused by the formation of eddies and vortices in the gap created by lack of erosion protection. These eddies and vortices produce shear stresses which the riprap cannot withstand, and rock and soil are then "plucked" from the channel. For steep streams, Longwell and Flint discuss the formation of potholes more than 25 feet deep. If such types of erosion were to occur, the stability of tailings could be compromised.

The USCOE (1958, see Ref. E16) have developed criteria for rock placement and recommend that:

> "The larger stones shall be well distributed and the entire mass of stones in their final position shall be roughly graded to conform to the gradation specified....The finished riprap shall be free from objectionable pockets of small stones and clusters of large stones....The desired distribution of the various sizes of stones

throughout the mass shall be obtained by selective loading of the material at the quarry or other source, by controlled dumping of successive loads during final placement, or by other methods of placement which will produce the desired results. Rearranging of individual stones by mechanical equipment or by hand will be required to the extent necessary to obtain a reasonably well graded distribution of stone sizes...any material displaced by any cause shall be replaced...."

It should be emphasized that it is not possible to place a perfectly-uniform layer of rock, and the guidance presented in the STP is not intended to ensure such perfect results (by hand placement, for example). However, as a minimum, licensees and designers should follow this qualitative criteria and should provide riprap to its required thickness and gradation. It should be noted that the placement problem is much more complicated when the rock size is large, relative to the rock layer thickness. For example, a 12-inch layer with an average rock size of 3 inches would not require the same degree of careful placement that would be required in placing a 12-inch layer of 8-inch rocks.

b. The staff agrees that it may be possible to construct a layer of small rock (or gravel) less than 6 inches thick. However, the intent of the guidance was to direct attention to the fact that it may be difficult to do so and that careful placement techniques need to be used in such cases.

c. Rock layer thicknesses of 1½ times the average rock size are generally adequate. However, for average rock sizes of 2 inches or less, thicknesses of 3 inches or less may be difficult to construct or may not provide the needed protection.

4.11.4 Changes Made

The STP has been modified in several areas to clarify staff concerns regarding rock placement.

4.12 Procedures for Justifying Reductions in Designs (Appendix C)

4.12.1 Commenters:

AK GeoConsult, Inc.
Illinois Dept. of Nuclear Safety
Hydro-Engineering

4.12.2 Summary of Comments

a. Commenters requested clarification of how the regulations and standards can be met by a reduced design if a PMP/PMF is not used. Commenters requested clarification of vague and subjective terms such as "significant" and "impracticability," as used in the justification procedures.

b. One commenter indicated that long-term care costs need to be factored into the analysis, because such costs could be important if they are greater than the costs for designing for a 1000-year stability period.

4.12.3 Analysis of Comments

a. Additional discussions of the use of the PMF and SPF are provided in Sections 4.2 and 4.7 of this Appendix.

The procedures given in Appendix C are intended to provide broad guidelines for development of information and analyses needed to justify that the design selected is the best design reasonably achievable. The procedures are provided principally to indicate that some flexibility exists in the regulations and standards and that there are no requirements for designing for a PMP/PMF. If, in fact, the designs that are developed using the STP are much too conservative, as claimed by commenters, it should be relatively easy for licensees to justify departures; it should be easy to show that significant cost savings can be effected by relaxing the design standards.

The staff used subjective terms in order to provide licensees some flexibility for achieving an optimum reclamation plan on a site-specific basis. Quantifying the terms would reduce this flexibility. Also, it is not possible, nor is it desirable, to use definitive terms, because what may be reasonable to achieve at one site may be cost-prohibitive at another site. The NRC staff does not intend to provide specific guidance on what constitutes "significant" or "impracticable." The burden of proof is on licensees and designers to demonstrate that their site-specific problems warrant a reduction in the longevity period, based on the concepts of reasonable assurance of stability and significant cost savings.

b. The regulations and standards do not permit consideration of maintenance in achieving the required stability. Therefore, no consideration can be given to the costs of maintenance in determining costs. However, it should be noted that licensees have the option of demonstrating alternate methods of meeting the regulations, as discussed in Section 84(c) of the Atomic Energy Act. Such alternate approaches could possibly include maintenance as one alternative in meeting the actual regulations and standards.

4.12.4 Changes Made

No changes have been made to the STP in these areas.

4.13 Riprap Design Procedures (Appendix D)

4.13.1 Commenters:

AK GeoConsult, Inc.
Hydro-Engineering
Colorado Dept. of Health
Homestake Mining Co.
Steven R. Abt, Colorado State University
USDOE
Illinois Dept. of Nuclear Safety
MK-Environmental Services
Exxon Coal and Minerals Co.
American Mining Congress

4.13.2 Summary of Comments

a. Several commenters questioned the technical basis for design of riprap and suggested that the Abt studies are limited and are neither definitive nor conclusive over a wide range of rock sizes. Several commenters questioned the Abt studies as a basis for determining Manning's "n" values, stating that Abt did not test for shallow flows over large rocks.

b. Several commenters questioned the NRC staff basis for concluding that rock voids will become clogged over a long period of time and questioned how interstitial flows should be accounted for.

c. Several commenters questioned the use of HEC-2, when hand calculations or simpler methods of calculation will sometimes suffice in determining depths and velocities of flow.

d. Several commenters suggested criteria for considering rock designs at gully intersections, stating that the criteria in the STP are important only under certain conditions. Several commenters requested additional clarification and justification for determining depth of rock placement, scour depths, and gully depths. Several commenters questioned the procedures for placing riprap on the opposite bank of a channel receiving gully inflows.

e. One commenter indicated that the STP should include additional discussion and criteria for designing a rock/soil matrix.

f. One commenter suggested that the design criteria are not applicable to top slopes and side slopes, since large areas of the slopes may not be subjected to submerged flows.

g. One commenter indicated that the criteria presented in the STP may necessitate the use of large riprap sizes and a filter layer, and it was suggested that the rock cover gradation be expanded. It was stated that methods are available to construct a rock layer, so that finer materials migrate to the bottom of the layer, and in many cases, a filter layer can be avoided.

h. One commenter indicated that selection of a riprap design method, based solely on its conservatism, is inappropriate. It was stated that the Safety Factors Method is not applicable to ephemeral channels and for overland flow, and it was suggested that the Stephenson Method be used in such cases. The commenter pointed out errors in the Abt references and in the modification to the Stephenson Method. The commenter indicated that use of any method must be based on sound engineering principles and that the degree of conservatism should not be the primary consideration. The commenter stated that the use of shear stress methods are inappropriate for riprap design in many cases, and that engineering judgment should be applied to the selection of various design parameters. It was indicated that oversizing of rounded rock by 40 percent is higher than accepted and is based on too little data and improper reasoning.

i. One commenter indicated that diversion channels are an acceptable alternative to protecting an embankment from flood flows.

j. One commenter suggested that credit be given for interstitial flows in seldom-saturated areas.

k. Several commenters indicated that the Kirpich Method for determining the time of concentration should be replaced by other methods such as the Brant and Oberman Equation or the using of the Manning's equation to determine flow velocities and subsequent times of concentration.

l. Several commenters indicated that the larger riprap on the steeper side slope of an embankment be extended at least 10 feet up onto the flatter top slope.

m. Several commenters stated that the runoff coefficient of 0.8 used in the example problem is not appropriate and should be increased to a value of 1.0.

n. One commenter questioned the subjective language in the design procedure.

4.13.3 Analysis of Comments

a. Based on staff reviews of several reclamation designs, it was determined that use of the Stephenson Method resulted in much smaller rock sizes than the Safety Factors Method for overtopping flows. The difference was especially significant on steeper slopes. In order to determine the appropriateness of these methods, particularly in light of the fact that the Safety Factors Method has widespread use in the engineering community, the NRC staff funded technical assistance efforts. These technical efforts verified that both methods were actually applicable: the Safety Factors Method was appropriate for relatively flat slopes; and the Stephenson Method was appropriate for relatively steep slopes.

   Commenters were apparently confused by the wording provided in the draft STP, which indicated that the Abt studies provided the technical basis for riprap design. Actually, the Abt studies provided the techncial basis for concluding that both methods are applicable, depending on the slope.

   The NRC staff considers that the Abt method for determining Manning's "n" values is appropriate, since the rock sizes that were tested were relatively large compared to the flow depth; therefore, the relative depth is rather small. The staff concludes that this method is more appropriate than any other method for this purpose; there are some comparisons provided by Abt that indicate the applicability of other methods for small values of relative depth.

b. The NRC staff basis for concluding that riprap voids will become clogged results from field observations and experience. Numerous cases can be cited where rock voids have been filled with water-borne and wind-blown deposits. Actually, it may be more appropriate to state that there is no basis for concluding that the voids will not be eventually clogged.

   Fortunately, the clogging of rock voids has little effect on stability against erosion. Abt (1987, see Ref. E17) concluded that a rock/soil matrix is as stable as a rock layer, alone.

   A design problem could occur, however, in the case of a steep channel lined with large rock. In such a case, flow through the rocks could be significant if the voids are not clogged; several cases have occurred where applicants have attempted to inappropriately subtract this interstitial flow from the design flow, since these channels are likely to receive large amounts of sediment to clog the voids. The staff concludes that such reductions in the design are, therefore, not reasonable for large rock layers typically found in diversion channels.

   The staff concludes that it is appropriate to assume that the voids are filled and to assume that the entire design flow passes over, rather than through, the rock layer. Licensees may choose to justify that site-specific circumstances warrant a departure from these assumptions.

c. HEC-2 is a nationally-recognized computational model and, because of its extensive documentation and use, should be used where the channel shape or channel slope changes from one section to another in the computation process. Other models that integrate the equation of gradually-varied flow could also be used. The most important consideration is that merely solving the Manning Equation for normal depth or velocity is often not appropriate when channel geometry varies along the stream length. Normal depth computations are usually appropriate only for constant channel sections and channel slope.

d. The staff believes that the criteria provided in the STP are adequate for designing riprap where natural gullies discharge into rock-protected channels. In order to provide further guidance on determination of gully depths and scour depths (beyond what is already provided in the STP), it would be necessary to expand the scope of the STP to include geomorphic and other considerations. Because this design situation is expected to arise in a limited number of cases, designers are encouraged to consult the references provided in the STP.

e. Additional discussions of a rock/soil matrix and rock mulches are provided in Section 4.6 (p. E-20) of this Appendix. Several changes have been made to the STP, in this area.

f. The most critical case for designing rock riprap exists when overtopping of the rock layer occurs. When the rock layer is totally submerged, the most critical stresses occur. If the rock layer is not submerged, less critical stresses are created because lower-velocity flow exists in the rock voids. Therefore, the design criteria provided in the STP address the most critical design situation.

g. The staff is not aware of any construction practices where finer materials can be forced to reach the bottom of the rock layer. However, such practices may actually exist, and applicants/licensees may be able to demonstrate that such a practice can be successfully accomplished.

It should be emphasized that a filter layer for slope protection does not serve the same classical function as a filter layer for a dam embankment, for example. In the latter case, the main purpose of the filter layer is to prevent piping of fines through the filter and rock. In the former case, the main purpose of the filter layer is to minimize flow velocities at the rock/soil interface and to prevent large rock from penetrating into the radon barrier. Therefore, licensees/applicants may be able to demonstrate, particularly for smaller riprap sizes, that adequate stability and velocity protection is provided without the placement of a separate filter layer. Such designs will be considered on a case-by-case basis.

h. The use of the Safety Factors Method and the Stephenson Method have been shown to be appropriate for mild slopes and steep slopes, respectively.

This verification was based on work done by Nelson, et al. (1986, see Ref. E5). The STP suggests the use of these methods based on their appropriateness, not their conservatism. Additionally, these methods were tested specifically for overland flows and shown to be applicable.

The staff believes that commenters are incorrect in asserting that shear stress methods are not applicable to riprap design. Such methods are widely used in many types of design applications.

The oversizing of rounded rock by 40 percent was based on limited data, as provided by Abt, et al. (1986, see Ref. E17). Based on further examination of the data, the staff now concludes that the oversizing factor for rounded rock could be as low as approximately 10 to 20 percent. The staff, therefore, suggests that a 20 percent oversizing factor be considered. As applicable, designers may be able to justify other reductions based on the angle of repose (Safety Factors Method) or the stability coefficient (Stephenson Method).

i. The staff agrees. The STP provides criteria for design of diversion channels; these criteria are presented in Appendix D.

j. On a case-by-case basis, credit may be taken for interstitial flow in many areas, except for flows through large rock in diversion channels.

   The STP has been revised to provide different nomenclature for seldom-saturated areas and frequently-saturated areas. These areas will now be termed as "critical" or "non-critical" areas, depending, to some extent, on the frequency of saturation.

k. There was never any intent to discourage the use of any particular method for determining the time of concentration. Although the Kirpich Method provides one acceptable method, other methods may also be used.

l. The use of the larger side slope riprap is needed for several feet above the slope break. This is necessitated by the flow regime change that occurs at some point upstream of the slope break. The STP recommendation of 10 feet was provided only as an approximate estimate. For ease of construction, the width of a dump truck may be a more reasonable estimate. Also, the actual width can be calculated.

m. The runoff coefficient of 0.8 was used for illustrative purposes, only. In general, a coefficient of 1.0 is appropriate when computing PMF estimates, since the occurrence of essentially 100 percent runoff is associated with a PMF. It should be recognized that reclamation covers are designed to minimize infiltration, to comply with groundwater standards. However, site-specific circumstances may warrant a reduction in the runoff coefficient.

n. Subjective language was provided to ensure design flexibility. See Section 4.12 (p. E-31) of this Appendix for additional explanation and clarification.

4.13.4 Changes Made

a. Wording has been changed to indicate that the Abt studies verify the applicability of the Safety Factors Method and the Stephenson Method, depending on the slope. No changes were made to staff recommendations for using the Abt Method for determining Manning's "n" values.

b. A slight change was made to the STP, regarding clogging of rock voids, to indicate that it is unlikely that clogging will not occur.

c. Minor modifications have been made to the STP to further clarify that use of HEC-2 is not a requirement, and that other methods may be used, as appropriate.

d. No changes were made.

e. As noted in Section 4.6 (p. E-20) of this Appendix, additional discussion of designs for a rock/soil matrix has been provided.

f. No changes were made.

g. No changes were made.

h. The STP has been revised to suggest 20 percent oversizing and the use of angle of repose or stability coefficients.

i. No changes were made in this area.

j. Terminology has been changed from seldom-saturated and frequently-saturated to non-critical and critical areas.

k. The STP has been revised to suggest that other methods of determining time of concentration are also acceptable.

l. The STP has been revised to suggest an extension of approximately 10 feet, or the width of a dump truck, for ease of construction.

m. No changes have been made to the STP. The runoff coefficient in the example problem is provided for illustrative purposes, only, and assumes that the runoff coefficent of 0.8 has been previously justified.

n. No changes have been made to the STP.

4.14 Rock Durability

4.14.1 Commenters:

AK GeoConsult, Inc.
Colorado Dept. of Health
Hydro-Engineering
Homestake Mining Co.
American Mining Congress

4.14.2 Summary of Comments

a. Several commenters suggested that the rock quality evaluation procedures be expanded and modified. Several commenters suggested the use of geomorphic evidence and field experience in evaluating rock durability. Commenters suggested the use of physical tests, rather than petrographic examination, for determining the presence of smectites and clay minerals. They also questioned the use of the tensile strength test, stating that the test is costly and duplicates other tests.

b. Several commenters suggested that many channels in the southwest United States or channels draining small basins should not be classified as frequently saturated.

c. One commenter requested that NRC provide a list of minerals known to cause problems with rock durability.

4.14.3 Analysis of Comments

a. The staff considers that a petrographic examination by a qualified person, is a useful tool for identifying rock types by mineral composition. Salient features of the rock such as clay veins, cracks, and fractures can also be determined. A petrographic examination is a qualitative analysis that can be used to quickly identify potential rock borrow sources.

In suggesting that a petrographic examination be used to initially screen rock sources, the staff did not intend to exclude the use of geomorphic evidence and field experience for evaluating rock durability. Actually, conventional geologic and geomorphologic office and field techniques can be used to screen rock types for samples to test in the laboratory. However, these techniques, by themselves, cannot be used to determine whether erosion protection will remain effective for 1000 years.

The ability of some types of rock to survive for long periods of time is well-documented by archaeological and historical evidence. However, very little information is available to quantitatively assess the quality of rock needed to survive for periods of up to 1000 years. A rock formation may be thousands or even millions of years old, but once the rock is quarried and crushed to obtain the small sizes needed for erosion protection, the potential for weathering accelerates very rapidly. In addition, quarrying practices can also accelerate weathering of a rock by introducing internal stresses, resulting from blasting during production.

In researching the literature to develop a method for quantitatively assessing rock quality, the staff concluded that some physical tests are better indicators of durability for certain type rocks than for others. For example, the Absorption test is a good indicator for limestone, but not as good for igneous rock. The Sodium Sulfate Soundness test is a good indicator of durability for igneous rock, but not very good for sandstone or limestone. Based on this, the staff developed a scoring method for assessing the acceptability of rock. The method provides maximum flexibility by allowing licensees to select those tests that are the best indicators of durability for the intended rock. For example, if the rock source is limestone, a licensee or applicant probably would not want to use the L.A. Abrasion test because the weighting factor is only 1 (see Table D1 [p. D-27] in the STP). On the other hand, a licensee would probably want to select the Specific Gravity, Absorption, and Schmidt Hammer tests, because these tests have high weighting factors for limestone (12, 13, and 11, respectively).

One of the commenters questioned the use of the Tensile Strength test. It should be emphasized that it is not necessary that all of the durability tests shown in Table D1 of the STP be performed. A licensee can select the tests that are the best indicators of durability for the type of rock that is being tested. If the rock being tested is basalt, for example, a licensee may want to consider the Tensile Strength test, because it is a good indicator of durability for igneous rocks.

b. In section 6.2.2, the staff suggested different rock-sizing criteria for frequently-saturated areas and for occasionally-saturated areas. In the STP, frequently-saturated areas are defined as channels and poorly-drained toes and aprons. Occasionally-saturated areas are defined as top slopes, side slopes, and well-drained toes and aprons. Even in arid areas in the southwest, flat channels may be subjected to freezing and thawing conditions in the spring. For this reason, the staff considers these channels to be critical areas, since channels receive concentrated flow, and frost damage to rocks is one of the primary causes of disintegration.

c. The staff hesitates to provide a list of minerals that can cause problems with rock durability, because the percentage of a deleterious mineral in a rock is more important than the mineral itself. Experienced geologists and mineralogists are capable of determining if the percentage of a deleterious mineral is sufficient to rule out a potential rock source.

4.14.4 Changes Made

Several changes have been made to the Rock Durability section of the STP, regarding classification of frequently-saturated and occasionally-saturated areas. These changes were made to better reflect critical areas of placement.

4.15 Verification of Compliance with NRC Regulations

4.15.1 Commenters: ACNW

4.15.2 Summary of Comments

The commenter suggested that the criteria used for determining compliance with NRC regulations should be discussed in the STP.

4.15.3 Analysis of Comments

To ensure that uranium mill tailings sites are constructed as designed, the staff previously prepared a position paper titled, "Staff Technical Position on Testing and Inspection Plans during Construction of DOE's Remedial Action at Inactive Uranium Mill Tailings Sites, Revision 2, January 1989." That position paper describes the engineering practices, testing, inspection, recordkeeping, nonconformance corrective action, and other controls considered satisfactory for implementing remedial action programs. Although the title of the paper addresses only the Title I (inactive sites), the staff applies the same methods for assuring compliance with regulations at Title II (active) sites.

Once a tailings pile is reclaimed, its performance is verified by long-term surveillance, which is required by Criterion 12 of 10 CFR Part 40, Appendix A. This criterion requires, as a minimum, annual inspections by the government agency retaining ultimate custody of the site. Results of these inspections must be reported to NRC within 60 days after each inspection.

The commenter, as an example, mentioned the radon emission limit of 20 picocuries per square meter per second (pCi/m^2s), and suggested that the criteria for determining the numbers and frequency of the required measurements be specified in the STP. The standard of 20 pCi/m^2s applies only to design, so that monitoring of an appropriately designed cover is not required to demonstrate compliance. However, field and laboratory measurements have been performed to test the validity of the methods used for calculating radon-flux attenuation. The results of these measurements have verified that the methods used to estimate required thicknesses of soil covers provided conservative estimates of the soil depths necessary to limit radon flux to the design standard of 20 pCi/m^2s.

4.15.4 Changes Made

No changes have been made to the STP in this area.

4.16 Rainfall Distributions

4.16.1 Commenters: EarthFax Engineering, Inc.
Hydro-Engineering

4.16.2 Summary of Comments

One commenter provided extensive comments on the use of particular rainfall distributions in determining the magnitude of the PMP or the PMF. It was stated that available data indicate that the suggested criteria in the STP may be overly conservative in placing the peak rainfall period near the middle of a storm, rather than at the beginning. The commenter stated that in the Colorado River and Great Basin drainage areas, as well as elsewhere in the southwestern United States, large thunderstorms are typified by a major percentage of the total rainfall occurring at the beginning of the storm. Thus, the PMP should be arranged in a similar manner. The commenter added that a critical distribution whereby the largest rainfall increment occurs near the temporal center of the storm, as recommended by the National Weather Service and by NRC, is only valid for areas of the United States east of the 105th Meridian.

Commenters thought that since the regulations permit a design reduction from 1000 years to 200 years, there should be some latitude in selecting a less conservative rainfall distribution.

One commenter indicated that the selection of a rainfall distribution should not be based on its conservatism. It was also stated that many different rainfall distributions may be necessary, depending on basin size and hydrograph combinations.

4.16.3 Analysis of Comments

The staff is aware that storms generally have a major portion of the rainfall occurring during the early part of the storm. This rainfall distribution however, is not only typical of the southwestern United States. Storms in areas east of the 105th meridian also exhibit this type of temporal distribution.

Arranging incremental rainfall values in critical order so that the largest increment occurs near the center of the storm is not based on how storms generally occur. The arrangement is a judicial one that gives a computed flood peak discharge that is greater than one based on the assumption that the greatest rainfall increment occurs at the beginning of the storm. By definition, the hypothetical flood that results from this critical distribution of PMP rainfall is the PMF. Federal agencies such as the Soil Conservation

Service, USCOE, and the Bureau of Reclamation, all arrange incremental values of PMP in critical order, when estimating PMF's.

The distribution of rainfall is not as important as the reasonableness of the estimated PMF discharge. If a PMF is compared with a maximum observed flow from a stream in the same meteorological and physiographic region, and the PMF is significantly smaller, obviously, the temporal distribution of the PMP or some other parameter is incorrect. For very small drainage areas, the reasonableness of an estimated PMF can also be checked by using the Rational Formula and back-calculating the value of the runoff coefficient (C). If C is less than about 0.7, then the calculated PMF is probably too small, and the distribution of PMP may be suspect.

If a designer desires to use a temporal distribution different than one where the largest PMP increment occurs near the center of the storm, adequate justification must be provided to show that the magnitude of the estimated flood peak is reasonably indicative of the PMF and of historic flood events that have actually occurred in the region.

4.16.4 Changes Made

No changes have been made to the STP in the area of rainfall distribution.

4.17 <u>Siting on Floodplains and Alluvial Fans</u>

4.17.1 Commenters: U. S. Dept. of the Interior

4.17.2 Summary of Comments

The commenter indicated that siting on floodplains is considered to be implicitly acceptable in the STP and suggested that the criteria be revised to ban disposal of tailings within the floodplain. It was also suggested that the criteria ban disposal on alluvial fans, due to the inherent instability of alluvial fans that could threaten waste disposal.

4.17.3 Analysis of Comments

The criteria provided in the STP address designs primarily for existing sites, and do not provide siting criteria. 10 CFR Part 40, Appendix A provides criteria that must be met in tailings reclamation, and these criteria need to be carefully considered when siting a facility. The intent of this STP is not to address siting concerns.

The staff, however, fully agrees that alluvial fans, due to their inherent instability, are poor choices for waste disposal. The staff agrees that siting should not take place on alluvial fans, if there are reasonable alternatives.

4.17.4 Changes Made

Changes have been made to the STP to discourage waste emplacement on alluvial fans or in areas of potential instability.

4.18 <u>Horton Stable Slope Method</u>

4.18.1 Commenters:
Hydro-Engineering
Steven R. Abt, Colorado State University
USDOE
Illinois Dept. of Nuclear Safety

4.18.2 Summary of Comments

a. Several commenters indicated that the Horton Stable Slope Equation presented in Appendix A yields overly conservative slope designs. They indicated that this was caused by the following factors:

   1. The equation should include a rainfall abstraction factor or runoff coefficient to reduce peak flow rates, particularly since the same factors that cause flow concentration would tend to provide channel storage, which would tend to reduce peak flow rates.

   2. The use of the rational formula in deriving the equation is too conservative.

   3. The allowable shear stress is greater than the stress that the slope will actually experience.

   4. The allowable shear stress values obtained using Temple are excessively low.

   5. One commenter suggested the use of a coefficient in the equation to reduce the conservatism. The commenter stated that the method needs to give credit for cohesive soils and the resultant increase in shear stress values.

b. One commenter agreed with the use of the shear stress approach and recommended that several methods, rather than one, be used to determine allowable stresses. The commenter suggested that criteria be provided for consideration of more than one method.

c. Several commenters indicated that published values for allowable tractive force may not be applicable to dispersive soils and suggested that candidate cover soils first be tested to identify if significant slaking could occur.

d. Several commenters indicated that typical earthwork construction tolerances may be greater than the slopes computed using the Horton Method, and questioned if local variations need to be considered in design. Such considerations would tend to restrict the use of soil covers.

e. One commenter indicated that the procedure should be expanded to cover the period of vegetation establishment, where a lesser rainfall event than the PMP could cause failure.

f. One commenter indicated that the use of a variable flow concentration, as a function of the density of the vegetation cover, may not be appropriate, and stated that no basis for such a correlation exists.

g. One commenter stated that Abt's work did not justify the conclusion that flow concentrations could occur on flatter slopes, since that work was based on slopes of 20 percent.

4.18.3 Analysis of Comments

a. For determining runoff from a relatively impervious cover, there is little basis for using a runoff coefficient of less than 1.0 in the Horton Method. (See Section 4.13 of this Appendix for additional analysis of this topic.) However, it should be emphasized that the Horton Method presented in Appendix A provides only one acceptable method for meeting applicable regulations and standards. Commenters who believe that any portion of the method or any input parameter is too conservative, such as the procedure for determining runoff, may use other methods or parameters to determine runoff, if such methods can be shown to be appropriate. The staff believes that the procedures and recommended input parameters provide a broadly acceptable method, as presented in the STP.

b. The staff agrees that more than one method could be used for determining shear stress values. However, the staff recommends the Temple procedures, since the methods are more detailed and more recent than others and provide values for a wide range of soil types, including cohesive soils. This method also allows credit to be taken for vegetation. Designers who wish to use other methods may do so if the method can be properly justified.

c. The staff agrees. Dispersive soils should not be used.

d. Typical earthwork construction practices call for placement of materials within certain tolerances. Licensees are encouraged to always construct slopes in a manner that provides slopes as uniform as practicable. Before final completion, the uniformity of the slopes should be checked after a rainfall event produces runoff from the slope. Based on observations, licensees will be expected to reconstruct and regrade slopes, as necessary, to preclude obvious areas where ponding or flow concentration has occurred. The final finished grade should be within 0.05 feet of the specified elevations; such values are typical of tolerances for placement of roadbeds or other fairly precise applications.

e. Licensees/applicants are expected to have the vegetation fully established and effective before leaving the site. Exceptions will be considered on a site-specific basis.

f. The staff agrees and has eliminated the variable flow concentration factor. A factor of three is now recommended for use at most sites, unless justification can be provided for a smaller value.

g. Abt's work included a considerable amount of data on slopes less than 20 percent. Commenters should consult Reference E17 for further information.

4.18.4 Changes Made

a. No changes have been made in this area.

b. No changes have been made in this area.

c. A recommendation has been added to the STP to discourage the use of dispersive soils in reclamation covers.

d. No changes have been made in this area.

e. No changes have been made in this area.

f. The variable flow concentration factor (F) has been eliminated. The staff recommends an F value of 3, unless lesser values can be reasonably justified.

g. No changes have been made in this area.

4.19 <u>Active Maintenance</u>

4.19.1 Commenters

Illinois Dept. of Nuclear Safety

4.19.2 Summary of Comments

The commenter requested that the STP address active maintenance, provide a definition of active maintenance, and discuss how NRC staff will determine when a design will or will not require active maintenance.

4.19.3 Analysis of Comments

The staff agrees that active maintenance should be defined in the STP. However, the regulations and standards allow no credit to be taken for active maintenance in meeting longevity requirements.

4.19.4 Changes Made

A definition of active maintenance has been added to the STP.

5. REFERENCES

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E17. Abt, S. R., et al., "Development of Riprap Design Criteria by Riprap Testing in Flumes: Phase I," NUREG/CR-4651, 1987.