DRAFT

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 and 40 CFR Part 192. These criteria establish broad design objectives for long-term protection of uranium mill tailings and specific design objectives which 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 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 describes technical analyses and design approaches which 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 which 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 above 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.

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 which 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 which 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.

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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, control of tailings is defined as providing an adequate cover to protect against exposure or erosion of the tailings. To help operators meet Federal guidelines, this Staff Technical Position 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 200 to 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 which 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.

This Staff Technical Position supercedes Branch Technical Position WM-8201 "Hydraulic 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.

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 to: (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 minimal 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 Section 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 Section 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 which 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. In a recent management position (NRC, 1989, see Ref. 4), it was concluded that proof of the future performance of engineered barrier systems over long time periods is not obtainable in the ordinary sense of the word. Reasonable assurance is required, however, with allowances made for the time period, hazards, and uncertainties involved.

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." As stated in the NRC management position (NRC, 1989, see Ref. 4), "NRC regulations do not permit credit for active maintenance. A design that would require any maintenance during the anticipated life of the project to maintain reasonable assurance that the Part 40 requirements are met would be considered to require active maintenance and would not be acceptable."

It is evident that remedial action designs are intended to last for a long time without the need for active maintenance, if reasonably possible. 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.

2.1.4 Radon Release Limits

The requirements of 40 CFR Section 192.02 and 10 CFR Part 40, Appendix A, Criterion 6 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. 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 200 to 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. NRC management recently stated (NRC, 1989, see Ref. 4) that: "While the radiological performance standard set forth in 40 CFR Part 192 specifies only the radon emission rate or the increasing average radon concentration, implicit in that standard are controls over the dispersion and misuse of tailings and protection of ground water. A narrow interpretation of the radon emission standard could envision portions of the tailings being exposed in time, but with an average emission rate over the pile still meeting the 20 pCi/M^2s limit. Such a narrow interpretation, however, would fail to deal with the implicit controls, since tailings would be accessible for misuse and dispersion. Therefore, a more reasonable interpretation would be to ensure, over the design life, that not only will the radon emanation rate be acceptable, but that the tailings would not be available for human misuse or dispersal by natural forces."

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 which must be factored into the overall reclamation plan. 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 ground water protection. 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 an unacceptable 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.

The "systematic" process to address certain design aspects, other than the surface water erosion considerations for cover designs, is beyond the scope of this Staff Technical Position and is, therefore, not addressed. However, addressing only the concerns and criteria detailed in this position may not be sufficient to address the other features necessary to comply with other applicable regulations and standards.

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-occurrence 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 equalled 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 (OWDC, 1986, see Ref. 9).

An alternate approach is to choose a design event which 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 these 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.

Although other flood and precipitation events may be used for 1000-year designs, if proper justification is provided showing reasonable assurance of non-occurrence during the 1000-year design period, the staff concludes that it may be very difficult to provide such justification. 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 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 Army Corps of Engineers (USCOE, 1964, see Ref. 12), or the maximum regional flood of record transposed to the site on a discharge per drainage area basis, whichever is greater. 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).

With regard to design basis events, the NRC management position (NRC, 1989, see Ref. 4) states that:

> "The conclusion that a design will be effective for 1000 years to the extent reasonably achievable, and, in any case, for at least 200 years and, therefore, will provide reasonable assurance of protection of the public health and safety over these time periods, shall be based, in part, on appropriate design criteria. Design criteria can be of two types: design basis events and smaller, but continual events. For some designs, the former may be the more significant, while for other designs, the latter may be the more significant.
>
> Standard engineering design criteria should be used to limit the probability of failure over the required longevity period to a value consistent with other design situations where public health and safety are important considerations.
>
> The design criteria applied to tailings reclamation design should reflect current standard engineering design practices. Examination of similar

design situations can help in establishing the type and reasonableness of design criteria applied to tailings reclamation.

Given the general demographic and physiographic characteristics of mill tailings sites, the risk of tailings reclamation failure is not life-threatening in the short term and is unlikely to be significantly greater over the long term. Therefore, the engineering criteria should be commensurate with this risk.

In evaluating the magnitude of a design basis event or the acceptability of a particular design criteria [sic], 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 distribution 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 parameter, with consideration given to phenomena which can be reasonably expected to occur during the period for which the design is required to perform."

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 which 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 which will ultimately lead to the formation of a stable slope configuration and could expose or release tailings to the environment.

The NRC staff therefore considers that the best method for providing long-term stability is to provide permanently stable slopes which 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 (ORNL, 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 which 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.

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. In fact, the presence of vegetation may cause the situation to be worse, in that flow concentrations may be produced as the flow randomly spreads around shrubs, and the vegetation may increase the roughness of the flow surface. This would produce an increase in the depth and shear forces without providing a significant increase in erosion resistance.

Many of the uranium mills are located in the 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 ground cover with cumulative overland flow and with eroded soil. The results indicated that no less than 70 percent ground cover is required to reduce flow to a point of stability. The cumulative values of overland flow and eroded soil increase sharply as the ground 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. Full credit will be given where licensees can substantiate that the naturally-occurring cover density will be greater than 70 percent.

Revegetation at each site 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 which 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 channels, where flow depths are greater.

Chow (1959, see Ref. 22) presents values of permissible velocities for well-seasoned channels. Drainage channels in a typical mill reclamation plan are composed of immature soils that may require an extensive period of time before they become well-seasoned. Therefore, when using permissible velocities for designing stable channels, lower velocities than those shown by Chow should be selected.

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. However, Chow's recommended reductions do not extend below a flow depth of one foot. If Chow's data are extrapolated to a flow depth of several inches, the recommended reduction in permissible velocity is about 50 percent.

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 must be less.

For the design of stable 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.

2.2.6 Rock Durability

Because tailings and their covers must remain stable for a 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 200 to 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 Staff Technical Position 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 which are 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, which are both designed to be stable for 1000 years.

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

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

Option 5 — Designs which 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 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. 15. 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 document that designing for a 1000-year stability period is not reasonably achievable and that this design is capable of meeting the 200-year minimum stability requirement. 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 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 Staff Technical Position 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	15	A.3	A-9
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-2
	Diversion Channels	3.3.2	17	D.3	D-6
	Outlets/Aprons	3.3.2	17	D.4	D-14
	Streambanks	3.3.3	17	D.5	D-17
	Rock Durability	3.3.4	18	D.6	D-21
	Rock Placement	3.3.5	18	3.3.5	3.3.5
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	12	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 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), 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 armor may provide an acceptable design. Discussion of rock covers is found in Section 3.3, p. 17.

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. 9, the NRC staff recommends that both the tractive force and permissible velocities 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 position.

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 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 usually 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 U.S. Army Corps of Engineers (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 utilize 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 which could lead to gullying and exposure of contaminated material. In general, such proper placement is created by providing a uniform and adequate thickness of rock. The purpose of this guidance is to develop procedures that can be used to assure that adequate rock placement is achieved.

Following are general guidelines which should be used in the placement of rock riprap layers:

1. Riprap should be placed in a layer thickness which is at least 1½-2 times the average rock size (D_{50}). If extreme care is used in placing the

riprap layer, such as using specialized equipment or hand work, a thickness of 1½ times D_{50} may be 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 assure that proper thicknesses and areal coverage are achieved. Where the D_{50} size is less than eight 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.

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APPENDIX A

DESIGN OF SOIL COVERS

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APPENDIX A

DESIGN OF SOIL COVERS

1. INTRODUCTION

Because regulations require that tailings remain stable for very long periods (200 to 1000 years), 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 which 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 which 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 which 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 which could occur over a period of 200 to 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

Horton (1945, see Ref. A2) determined that an area which was 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, x_c. At this point the eroding force becomes equal to the soil resistance. The following expression was developed by Horton to determine the critical distance:

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

where:

x_c = critical distance, feet

q_s = 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.

Horton derived this equation by simultaneous solution of the Manning Equation, the peak shear stress formula, and the Rational Formula.

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

S_s = sin x = tan x, for small values of slope;

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

q_s = PLF, where

P = design precipitation intensity (inches/hour),
L = x_c = slope length (feet),
F = flow concentration factor.

Therefore,

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

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.9.)

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 (Fullerton, 1983, see Ref. A5; Chow, 1959, see Ref. A4).

2.2 Procedures

Procedures have been developed to derive input parameters to the aforementioned stable slope equation and provide one acceptable method 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.

A step-by-step procedure for implementing this approach is as follows:

Step 1. 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 is principally a function of the D_{75} grain size, where the allowable tractive force is equal to $0.4 \times D_{75}$ (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 allowable shear force. Additional guidance and need for justification of design parameters for vegetation covers are discussed in Section 2.3, above.

Step 2. Determine slope and slope length to be considered, as developed in the preliminary reclamation design.

Step 3. Determine flow concentration factor (F). The value of F is dependent on whether the slope is completely unprotected or has some protection due to the presence of vegetation. The percentage of the area covered by vegetation (vegetation density) is a very important factor. For completely unprotected soil covers, an F value of 3 is suggested. As discussed in Section 2.2.4, p.7, a vegetation density of at least 70 percent is needed to reduce flow to a point of stability. Therefore, if the density of coverage can be shown to be at least 70 percent, a flow concentration factor of 1 may be used. For a density of less than 30 percent, a F value of 3 would be appropriate. For densities between 30 and 70 percent, the F value should be interpolated between 3 and 1.

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. A6). The actual value of F will depend on several factors, including grading practices during cover construction, cover slope, and potential for differential settlement.

Step 4. Estimate Manning's 'n' value using general procedures given by Temple, et al. (1987, see Ref. A3) or by Nelson, et al. (1986, see Ref. A7). 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. Determine the rainfall intensity using the procedures given by Nelson, et al. (1986, see Ref. A7, Section 2.1.2)

Step 6. Solve for stable slope, using the aforementioned derivation of the Horton equation. If the computed slope is different from that assumed, 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-4 for various steps of the design procedure. Particular attention should be given to determining allowable shear stress values, since this parameter is likely to be the most sensitive parameter in the calculation.

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 as follows:

Step 1. The allowable shear stress may be estimated using methods given by Temple, et al. (1987, see Ref. A3). For an assumed clay soil (CH) 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_ε = void ratio correction factor

$$C_e = 1.38 - (.373)(e) = 1.38 - (.373)(.5) = 1.19$$

t_{ab} = 0.0966 (from Table 3.3 of Ref. A3)

$$t_a = (.0966)(1.19)^2 = 0.14 \text{ lb/ft}^2 .$$

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

Step 3. Since the vegetation cover is expected to be less than 30 percent, 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. Manning's 'n' value may be estimated using Chow (1959, see Ref. A4). For a uniform weathered earth section (using normal values),

n = .025 .

Step 5. The rainfall intensity may be estimated using the procedures given by Nelson, et al. (1986, see Ref. A7). It is assumed that the intensity has been calculated to be 40 inches/hr.

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

$$(S_s)^{7/6} = (65)(.14)^{5/3} / (40)(1000)(3)(.025)$$

$$S_s = .002 \text{ ft/ft} .$$

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 reasonably conservative 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 1, since a dense vegetation cover will be provided.

Step 4. Manning's 'n' value is assumed to be 0.03, 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:

$$S_s = 0.011 .$$

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 protection 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 which will produce subcritical flow where channelling occurs.

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 which exceed the allowable shear stresses of the soil.

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. A7).

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 using procedures given in 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 erosion protection are discussed in Appendix D.

3.3 Recommendations

The staff recommends that the following parameters be used 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. A7).

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 Example 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 ft/ft.

Step 2. Using the rational formula (Nelson, et al. 1986, see Ref. A7); 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:

$Q = ciA = (.8)\ (50)\ (20) = 800$ cfs .

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

$Q = 800$
$n = .025$
$S = .001$

Normal depth $(y) = 3.81$ feet.

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

$t = WyS = (62.4)\ (3.81)\ (.001) = 0.24$ lb/ft^2 .

Step 5. The allowable shear force is estimated to be 0.1 lb/ft2, using procedures similar to those discussed in Section 2.4, p. A-6.

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 ft/ft.

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:

$Q = ciA = (.7)\ (50)\ (20) = 700$ cfs.

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

$Q = 700$
$n = .03$
$S = .001$

Normal depth $(y) = 3.9$ feet.

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

$t = WyS = (62.4)\ (3.9)\ (.001) = 0.24$ pounds per square foot.

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 cross-section 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, these methods 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. Consideration may be given to constructing the upstream side slope at the same slope as the embankment.

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., et al., "Development of Riprap Design Criteria by Riprap Testing in Flumes: Phase I," Colorado State University, 1987.

A7. 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.

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 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 which 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) and tailings setback distance (X).

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).

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 6 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.

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

$$S_s = .009 .$$

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

slope (S_t) is calculated to be:

$S_t = (S_i)\ e^{-GSt}$

$S_t = (.2)/e^{(1.0)(.009)(200)}$ using Fig 4.3 (Nelson, et al., 1986, see Ref. B1), where G = 1.0

$S_t = .033$.

Step 4. For a value of $(S_t \times C_u) = (.033)(10) = .33$, 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 not on Fig. 4.4, extrapolations will be necessary.)

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

$L_R = 33.6 / (\tan 30°) = 33.6 / .58 = 58$ 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 three to five 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.

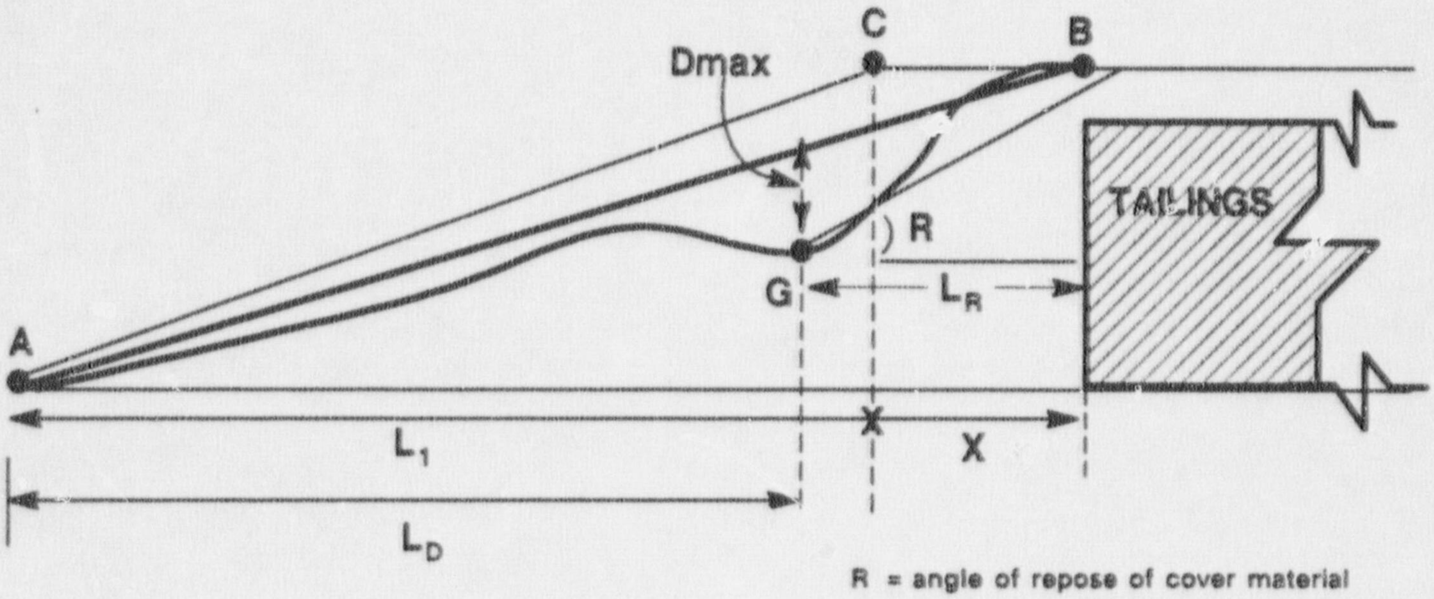

Figure B1

Procedure for Determining Sacrificial

Slope Requirements and Setback Distance,

No Drainage Area Above Point C

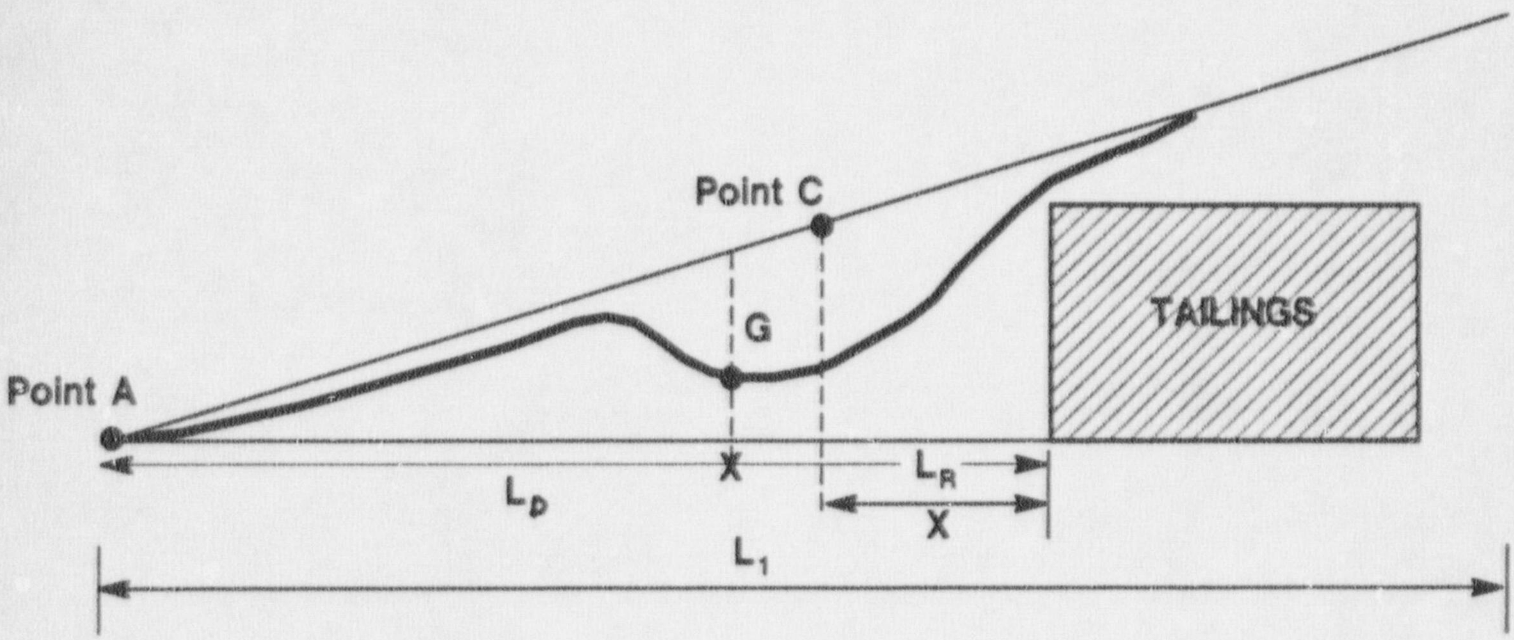

Figure B2

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

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 which 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" <u>may</u> 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 which 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 of this position.

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

(3) supporting hydraulic calculations, and

(4) supporting logic and bases which 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) streambanks of larger rivers and/or areas of the reclaimed side slopes where floods impinge. Procedures for designing riprap erosion protection for each of these situations 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 which 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 basis for design of riprap on the top and side slopes of a reclaimed tailings pile. Abt, et al. (1988, see Ref. D3) developed a family of curves relating unit discharge to median stone diameter (D_{50}). These curves indicated that for slopes of about 10 percent or greater, the required D_{50} sizes would be smaller than comparable sizes determined using the Stephenson Method. Likewise, the curves indicated that the required D_{50} sizes for flatter slopes would be smaller than comparable sizes determined using the Safety Factors Method. However, since Abt's relationships are based on idealized laboratory conditions, the staff has concluded that some conservatisms are appropriately provided by the Stephenson Method and the Safety Factors Method to account for actual field conditions. Therefore, 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.

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 cover 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:

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

H = 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.

Step 5. Determine rock size requirements.

Recent studies performed for the NRC staff (Abt, et al., 1988, see Ref. D3) have indicated that the Safety Factors Method is most applicable for designing rock for slopes less than 10 percent and that the Stephenson Method is most applicable for slopes greater than 10 percent. Therefore, using the peak flow rate calculated in Step 4, the required D_{50} may be determined using the appropriate method.

2.3 Recommendations

The use of the Safety Factors Method is recommended for slopes of less than 10 percent. The Stephensen Method is recommended for slopes of greater than 10 percent. Since it is likely that clogging of the riprap voids will 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. The need for oversizing of rounded rock is 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 ft/ft and for the side slope with an additional length of 250 feet and a slope of 0.2 ft/ft (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:

$$A1 = (1000)\ (1)\ /\ 43560 = .023 \text{ acres.}$$

$$A2 = (1000 + 250)\ (1)\ /\ 43560 = .029 \text{ acres.}$$

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).

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

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

tc = 7.2 + 1.0 = 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).

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

i = (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 rate is calculated using the rational formula to be:

Q1 = (.8) (35) (.023) = 0.64 cfs/ft, for the top slope, and

Q2 = (.8) (33) (.029) = 0.77 cfs/ft, for the side slope.

Step 5. Using the Safety Factors Method, the required rock size for the pile top 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 Stephensen Method is an empirical approach and is not applicable to gentle slopes. The Safety Factors Method is conservative for steep slopes.

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 to be concerned about, 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).

3. Flow through the riprap voids should be ignored. Over a long period of time, it is likely that the rock voids will 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 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 values of the slope and the riprap size.

5. Rainfall and rainfall intensities should 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 by trial-and-error 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 HEC-2 (USCOE, see Ref. D7) should be used to determine the depth of flow.

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. The NRC staff concludes that 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 which will discharge into it, resulting in higher velocities 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 shear forces which occur in the natural gully. This may be very important if the gully is significantly steeper than the proposed ditch. Assuming that the gully flow 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 $t = 62.4yS$, where S is the slope of the natural gully, and y is the normal depth of flow in the natural gully (in feet). The rock size in the diversion channel should be checked to assure 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 which currently exists, unless there is a potential for a more critical configuration to develop over a period of time. An example of this would be the gradual vertical erosion of a gully which could narrow the cross-section or steepen the side slopes of an existing cross-section.

3. It may be necessary to provide riprap to a greater depth 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, 1983, see Ref. D8) or using geomorphic analyses. It appears that the thickness of the rock should in no case 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 channel. It is possible, particularly if the inflow slopes are steep, that these smaller rills and gullies will also generate more erosive force than the rock in the diversion channel is capable of withstanding.

5. The larger rock, as determined using the considerations previously discussed, may need to be placed on the opposite bank of the diversion channel, also. 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 small, steep drainage areas, it is likely that overland flows will channelize relatively quickly; thus, the velocities that occur in gullies and channels should be used to estimate the total time of concentration for the basin. The channel hydraulics method (USBR, 1977, see Ref. D9) is suggested for use in such cases. The Kirpich Method, as previously discussed, may also be used.

Step 2. Determine rainfall intensities of 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, <u>et al</u>. (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 U. S. Army Corps of Engineers 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:

$$n = 0.0456\ (D_{50} \times S)^{.159} \ ,$$

$$n = 0.0456\ (24 \times .05)^{.159} \ ,$$

$$n = .047 \ .$$

Step 3. Determine normal depth (y).

By trial and error for the trapezoidal channel, with

$n = .047$; $Q = 1000$ cfs; and $S = .05$,

$y = 3.0$ 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,

$t = (62.4)\ (3.0)\ (.05) = 9.36\ lb/ft^2$.

Step 5. Compute the required rock size.

Using an equation of the U. S. Army Corps of Engineers (USCOE, 1970, see Ref. D5),

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

$a = .04$,
W_s = unit weight of rock,
W_w = unit weight of water = $62.4\ lb/ft^3$.

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

$t = 4.1\ D_{50}$.

The required size is calculated to be:

$D_{50} = t / 4.1$

$D_{50} = 9.36 / 4.1 = 2.3$ 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, 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 which will exist at the outlets of man-made channels and to properly design aprons, toes, and energy dissipation areas. The U. S. Army Corps of Engineers, 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 which 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 minimum toe requirements.

4.2.2 Design for Severe Gullying

In many cases, it may be necessary to construct special ditch outlets where the natural slopes are steep and gully depths extend to more than several feet. 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 developed by USDOT (1983, see Ref. D8).

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 U. S. Army Corps of Engineers 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 procedures discussed may not address certain geomorphic considerations that are beyond the scope of this position. 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 U. S. Army Corps of Engineers 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 using t = WyS.

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

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 determined to be appropriate at the location in question:

y = 10.2 feet,

S = .008 ft/ft,

V = 15 ft/sec.

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

t = (62.4) (10.2) (.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:

$$D_{50} = 1.24/.72 .$$

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

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. It may, therefore, be necessary to use reasonably conservative values of input parameters in estimating the design parameters which determine rock requirements.

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 and analyses 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 presented in NUREG/CR-4620 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. 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 presented in NUREG/CR-4620 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.

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.
L/A Abrasion (100 rev's) %	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.

6.2.2 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 following criteria have been developed based on the general recommendations contained in NUREG/CR-4620 (see Ref. D2), with several modifications.

A. Frequently-Saturated Areas --- Channels, Poorly-Drained Toes and Aprons

<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)

Less than 65 - Reject

B. Occasionally-Saturated Areas --- Top Slopes, Side Slopes, and Well-Drained Toes and Aprons

<u>Rating</u>

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

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. 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 several 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 diameter 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}. For smaller rock, it is generally considered that thicknesses of less than six inches are not practicable to construct.

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 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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