C7

MARTIN MARIETTA ENERGY SYSTEMS LIBRARIES

3 4456 0149134 8

NUREG/CR-4620
(ORNL/TM-10067)

Methodologies for Evaluating Long-Term Stabilization Designs of Uranium Mill Tailings Impoundments

Prepared by J. D. Nelson, S. R. Abt, R. L. Volpe, D. van Zyl/CSU
N. E. Hinkle, W. P. Staub/ORNL

Colorado State University

Oak Ridge National Laboratory

Prepared for
U.S. Nuclear Regulatory Commission

INFORMATION RESOURCES ORGANIZATION
TECHNICAL LIBRARY
Report Collection
Building 9711-1, Y-12
LOAN COPY ONLY

Do NOT transfer this document to any other person. If you want others to see it, attach their names, return the document, and the Library will arrange the loan as requested.

UCN-1624 (3 6-85)

NOTICE

This report was prepared as an account of work sponsored by an agency of the United States Government. Neither the United States Government nor any agency thereof, or any of their employees, makes any warranty, expressed or implied, or assumes any legal liability of responsibility for any third party's use, or the results of such use, of any information, apparatus, product or process disclosed in this report, or represents that its use by such third party would not infringe privately owned rights.

NOTICE

Availability of Reference Materials Cited in NRC Publications

Most documents cited in NRC publications will be available from one of the following sources:

1. The NRC Public Document Room, 1717 H Street, N.W.
   Washington, DC 20555

2. The Superintendent of Documents, U.S. Government Printing Office, Post Office Box 37082, Washington, DC 20013-7082

3. The National Technical Information Service, Springfield, VA 22161

Although the listing that follows represents the majority of documents cited in NRC publications, it is not intended to be exhaustive.

Referenced documents available for inspection and copying for a fee from the NRC Public Document Room include NRC correspondence and internal NRC memoranda; NRC Office of Inspection and Enforcement bulletins, circulars, information notices, inspection and investigation notices; Licensee Event Reports; vendor reports and correspondence; Commission papers; and applicant and licensee documents and correspondence.

The following documents in the NUREG series are available for purchase from the GPO Sales Program: formal NRC staff and contractor reports, NRC-sponsored conference proceedings, and NRC booklets and brochures. Also available are Regulatory Guides, NRC regulations in the *Code of Federal Regulations,* and *Nuclear Regulatory Commission Issuances.*

Documents available from the National Technical Information Service include NUREG series reports and technical reports prepared by other federal agencies and reports prepared by the Atomic Energy Commission, forerunner agency to the Nuclear Regulatory Commission.

Documents available from public and special technical libraries include all open literature items, such as books, journal and periodical articles, and transactions. *Federal Register* notices, federal and state legislation, and congressional reports can usually be obtained from these libraries.

Documents such as theses, dissertations, foreign reports and translations, and non-NRC conference proceedings are available for purchase from the organization sponsoring the publication cited.

Single copies of NRC draft reports are available free, to the extent of supply, upon written request to the Division of Technical Information and Document Control, U.S. Nuclear Regulatory Commission, Washington, DC 20555.

Copies of industry codes and standards used in a substantive manner in the NRC regulatory process are maintained at the NRC Library, 7920 Norfolk Avenue, Bethesda, Maryland, and are available there for reference use by the public. Codes and standards are usually copyrighted and may be purchased from the originating organization or, if they are American National Standards, from the American National Standards Institute, 1430 Broadway, New York, NY 10018.

NUREG/CR-4620
ORNL/TM-10067

Methodologies for Evaluating Long-Term Stabilization Designs of Uranium Mill Tailings Impoundments

Manuscript Completed: May 1986
Date Published: June 1986

Prepared by
J. D. Nelson, S. R. Abt, R. L. Volpe, D. van Zyl, Colorado State University
N. E. Hinkle, W. P. Staub, Oak Ridge National Laboratory

Colorado State University
Fort Collins, CO 80523

Under Contract to:
Oak Ridge National Laboratory
Oak Ridge, TN 37831

Prepared for
Division of Waste Management
Office of Nuclear Material Safety and Safeguards
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555
NRC FIN B0279

MARTIN MARIETTA ENERGY SYSTEMS LIBRARIES

3 4456 0149134 8

ABSTRACT

Uranium mill tailings impoundments require long-term (200-1000 years) stabilization. This report reviews currently available methodologies for evaluating factors that can have a significant influence on tailings stabilization and develops methodologies in technical areas where none presently exist. Mill operators can use these methodologies to assist with (1) the selection of sites for mill tailings impoundments, (2) the design of stable impoundments, and (3) the development of reclamation plans for existing impoundments. These methodologies would also be useful for regulatory agency evaluations of proposals in permit or license applications.

Methodologies were reviewed or developed in the following technical areas: (1) prediction of the Probable Maximum Precipitation (PMP) and an accompanying Probable Maximum Flood (PMF); (2) prediction of the stability of local and regional fluvial systems; (3) design of impoundment surfaces resistant to gully erosion; (4) evaluation of the potential for surface sheet erosion; (5) design of riprap for protecting embankments from channel flood flow and overland flow; (6) selection of riprap with appropriate durability for its intended use; and (7) evaluation of oversizing required for marginal quality riprap.

TABLE OF CONTENTS

Page

LIST OF FIGURES

LIST OF TABLES

1. INTRODUCTION

Unprotected uranium mill tailings impoundments can pose a significant risk to nearby inhabitants and the surrounding environment in the event of a prolonged or catastrophic release of contaminants from the tailings. Therefore, reclamation plans for such facilities must include engineering designs to protect against disruption of the tailings impoundment. The goals of engineering design should be to provide overall site stability for the long-term with no planned ongoing maintenance and to provide a repository for the tailings that will not place an undue burden of responsibility on future generations. Although a maintenance-free system should not require monitoring, a short period of surveillance would be reasonable. This period of surveillance can provide the assurance that the reclamation plan that was implemented is functioning adequately.

The purpose of this investigation is to present a review of the currently available, state-of-the-art engineering techniques and methodologies for the evaluation of reclamation plans designed to provide long-term stability against potential failure modes. Evaluative techniques were developed for long term stabilization where methodologies did not exist previously. It is important for the user to recognize the limitations of each methodology presented and acknowledge that, in some cases, additional research may be warranted to expand applicability.

Design considerations for long-term stabilization of uranium mill tailings impoundments were discussed in detail by Nelson, et al. (1983). In that document, the design parameters were defined and potential failure modes were discussed. The main purpose of that report was to evaluate the importance that the stability period (e.g., 200, 500, or 1000 years) would have on the design criteria. It was shown in that report that regardless of the stability period the appropriate design flood would be the Probable Maximum Flood (PMF). The design of various elements of protection systems for the different failure modes should therefore be based on the PMF.

The factors to be considered in the determination of a Probable Maximum Precipitation (PMP) and the subsequent PMF are discussed in Chapter 2. The various methods commonly used to predict PMF's are discussed and comparisons of predicted PMF's and historical record rainfalls are provided.

A methodology for distinguishing between stable and unstable fluvial systems is presented in Chapter 3. It is important to recognize an unstable river channel because its changing flow pattern and increasing erosion potential may threaten the stability of a tailings impoundment.

The designs of impoundment surfaces to avoid gully erosion are discussed in Chapter 4. A methodology of predicting when gully erosion can initiate is presented. Means of predicting stable slopes and threshold values at which gully erosion may initiate are developed. Also, riprap design procedures are presented.

The potential for surface sheet erosion is evaluated in Chapter 5. A methodology for evaluating erosion potential from a surface which is sufficiently flat that gully erosion would not occur is also developed.

Techniques for the selection of riprap are discussed in Chapter 6. Evaluation of riprap durability and potential oversizing of riprap material is also addressed.

The probabilistic risk analysis of long term stabilization and the nature of the risk or hazard that is imposed due to failure of different elements of the impoundment are discussed in Appendix A. The factors that are important in evaluating risk on the basis of probability of failure are discussed in the context of consequences of failure. Standard procedures for the sampling and testing of riprap are addressed in Appendix B. Sample calculations for durability and oversizing of riprap are presented in Appendix C. It should be noted that these appendices, which include substantive material from a variety of sources, were prepared primarily as source documents to provide additional background information for the use of the authors in developing this report and for the reader's benefit in using some of the methodologies in designing reclamation plans.

Secondary failure mechanisms such as human or animal intrusion that are not controlled by natural forces are not considered in this investigation. Also, root penetration is not considered because disruption of the impoundment by this method would not be expected to result in physical dispersion of the tailings, although there would be a potential for uptake of radionuclides.

Site selection and the physical location of an impoundment are perhaps the most important considerations in the construction of stable reclamation alternatives. Although site selection may be of the most important consideration in either minimizing or eliminating the adverse impacts of potential failure modes as a result of natural disruptive forces, it is not included in this report. The report concentrates on the engineering design considerations and methodologies necessary to mitigate disruption by each potential failure mode.

It should be noted that the engineering design considerations discussed herein are not based on any required radon release standard, nor on cost-benefit evaluation of alternative designs. This report is not intended to set forth federal, state, or local policy regarding reclamation plans related to remedial action on existing sites or new facilities. It is only intended to establish a technical basis for the design of necessary reclamation plans based on sound engineering design methodology. This report is intended for use by designers in identifying those factors that must be taken into consideration in the design of long-term stabilization techniques. By providing adequate discussion of the variables affecting required design considerations, the report should be helpful to policy-makers responsible for evaluating long-term stabilization techniques. Although the report does not include large amounts of quantitative data, some examples are given, where appropriate, for purposes of illustration. In a subsequent report, the application of the methodologies discussed herein to selected impoundments will illustrate their use by designers or agency reviewers.

2. DESIGN FLOOD ESTIMATION

In a recent document on the design considerations for long term stabilization of tailings impoundments, Nelson et al., (1983) showed that the design event for evaluating the long-term stability of a reclaimed tailings impoundment should be the Probable Maximum Flood (PMF). The PMF has been defined (COE, 1975) as "the flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region." The precipitation associated with the PMF is known as the Probable Maximum Precipitation (PMP) which is defined as "the theoretically greatest depth of precipitation for a given duration that is physically possible over a particular drainage basin at a particular time of year" (AMS, 1959).

Nelson et al. (1983) indicated that for a particular impoundment, two different situations related to the PMF must be considered. For an impoundment located in the flood plain of a major stream or wash, the PMF of concern would be that caused by an occurrence of the PMP over appropriate drainage areas upstream of the impoundment. The impact on the toe or face of the impoundment depends on the magnitude of the PMF and the location of the impoundment relative to the main channel. On the other hand, some sites are located on high ground beyond the influence of the PMF of a major stream. For these cases, the PMF of concern is that corresponding to occurrence of the PMP on only the drainage area on and above the impoundment site.

A PMF due to the PMP occurring on the watershed above the impoundment would primarily influence surface erosion of the impoundment cover if flows are not diverted around the impoundment. When these storm flows are diverted around the reclaimed impoundment, the cover design and diversion structure must withstand the PMF caused only by the onsite PMP. Therefore, regional, local and on-site PMF's must be evaluated in the comprehensive long-term stability analysis of tailings impoundment designs.

2.1 DESIGN STORM

2.1.1 PMP Design Storm

The design storms that are traditionally used to estimate the PMF are a set of maximized intensity-duration values formulated for mountainous and non-mountainous regions across the United States. Each region can be evaluated by the influence of the type of storm that characteristically impacts a specific area. The types of storms considered depend upon location, topograhic influences, potential for convergence, moisture potential and meterorologocial transportation. Commonly, the local thunderstorm and general storm are transposed over a region or site for PMF estimates.

A series of generalized precipitation charts have been prepared by the National Weather Service to rapidly determine design storm values for any specific area in the United States. The design storm values represent a conservative upper limit of potential precipitation. Generalized storm values have been compiled in Hydrometeorological Report (HMR) Nos. 43, 49, 51, and 55 for both general type storms and thunderstorms in areas west of the 103° meridian, but only for general type storms in areas east of the 103° meridian (NWS, 1961, 1977, 1979, and 1984). Values of the local-storm PMP, defined by the National Weather Service as one hour duration over a one square mile area (1-hr, 1-mi^2), are presented in Figure 2.1. The charts portraying the PMP thunderstorm values and PMP general-type storm values were originally derived for the U.S. Army Corps of Engineers from Hydrometeorological Report (HMR) No. 33 (NWS, 1956). Hydrometeorological reports with adjusted precipitation estimates have been published for various regions throughout the United States. Figure 2.2 presents the regions in which updated PMP studies have been conducted and the HMRs in which these precipitation estimates are published. It is recommended that values presented in these reports be used to estimate the PMP.

The 1-hr, 1-mi^2 PMP values presented in Figure 2.1 are directly applicable to locations between sea level and 5000 foot elevation. However, the 1-hr, 1-mi^2 rainfall should be decreased by 5 percent per 1000 feet of additional elevation over 5000 feet.

Depending upon the drainage areas, regions east of the 103° meridian in the United States generally use the six-hour general-type storm as the design storm for PMF analysis. The general-type storm is derived from an extensive data base and is commonly extended for periods of 72 hours to 96 hours. The general-type storm yields large volumes of runoff. The general-type storm will usually yield peak runoff and runoff volume values greater than the thunderstorm (1-hr) in the eastern United States, depending upon the size of the drainage area.

Regions west of the 103° meridian in the United States should evaluate the PMF with both the general-type storm and the thunderstorm. Application of the general-type storm in areas west of the 103° meridian will usually yield a PMF peak discharge lower than that estimated by the thunderstorm, yet yield a volume of runoff greater than the thunderstorm. The thunderstorm will generally produce a PMF with peak runoff greater than the general-type storm for small watersheds. However, if the volume of runoff is a consideration, it is recommended that PMF values for both storm types be estimated as the runoff volume for the general-type storm generally exceeds the runoff volume from the thunderstorm.

Based upon the southwestern rainfalls, a depth-area relation was adopted as presented in Figure 2.3. It is apparent that as the drainage basin area increases, the PMP decreases. Therefore, the PMP depths should be adjusted to reflect the drainage basin size.

2.1.2 PMP Rainfall Intensity

The 1-hr, 1-mi^2 local-storm PMP values presented in Figure 2.1 are based on rainfall data collected by the National Weather Service that is adjusted for durational, areal and atmospheric variations. However, the National Weather Service determined that the PMP local storm could last up

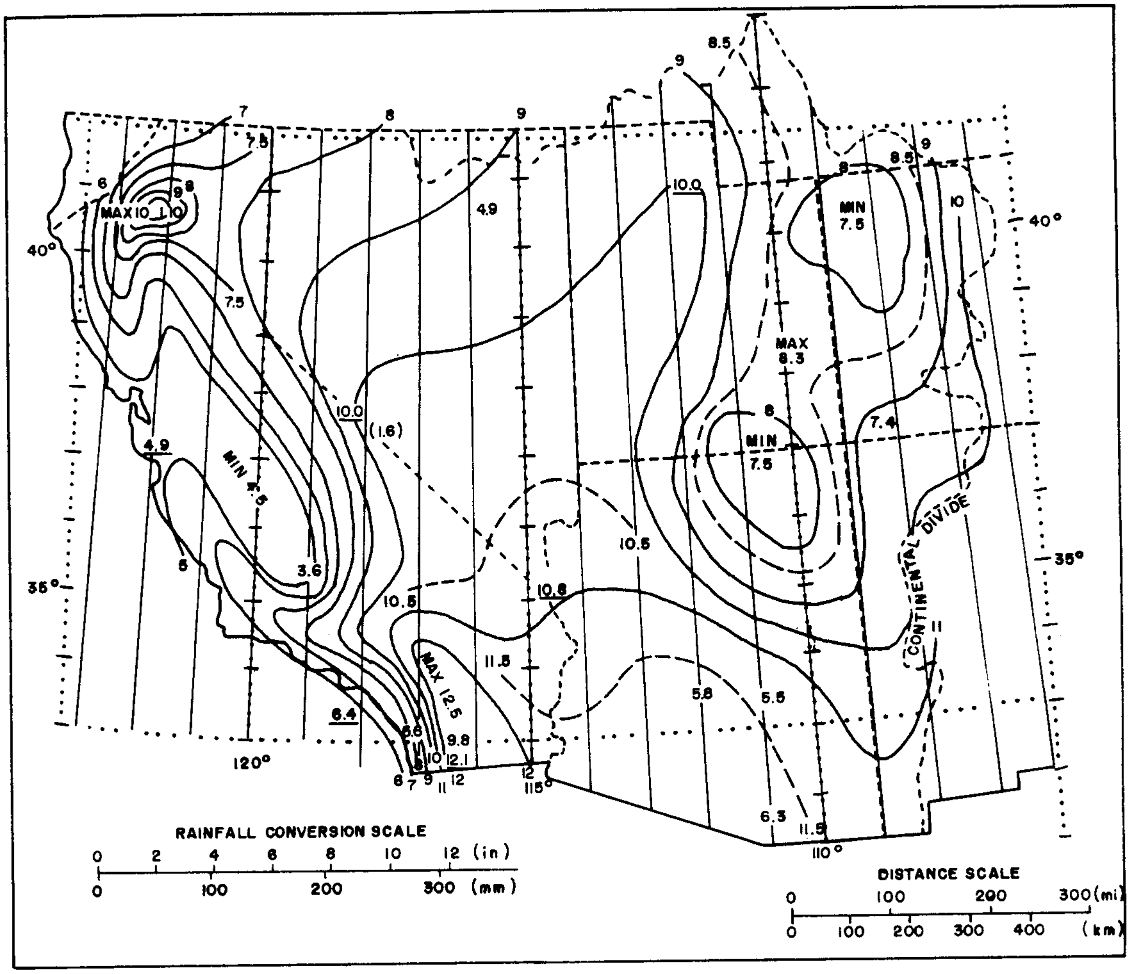


Fig. 2.1. Local-storm PMP for a one-square-mile area and a one-hour duration applicable for locations between sea level and 5000 ft. *Source:* NWS, 1977.

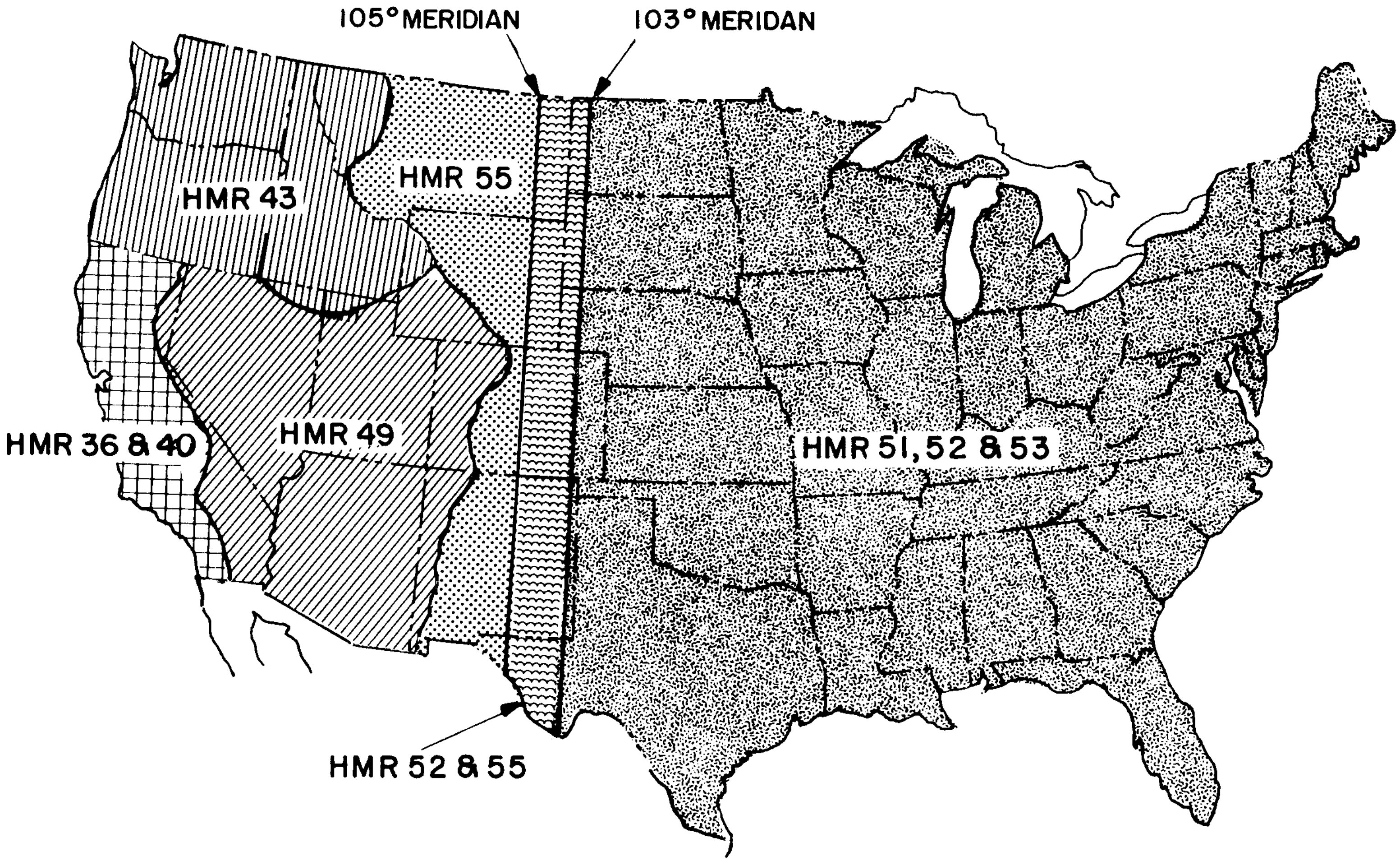


Fig. 2.2. Regions of the conterminous United States for which PMP estimates are provided in indicated Hydrometeorological Reports (HMRs). *Source:* NWS, 1984.

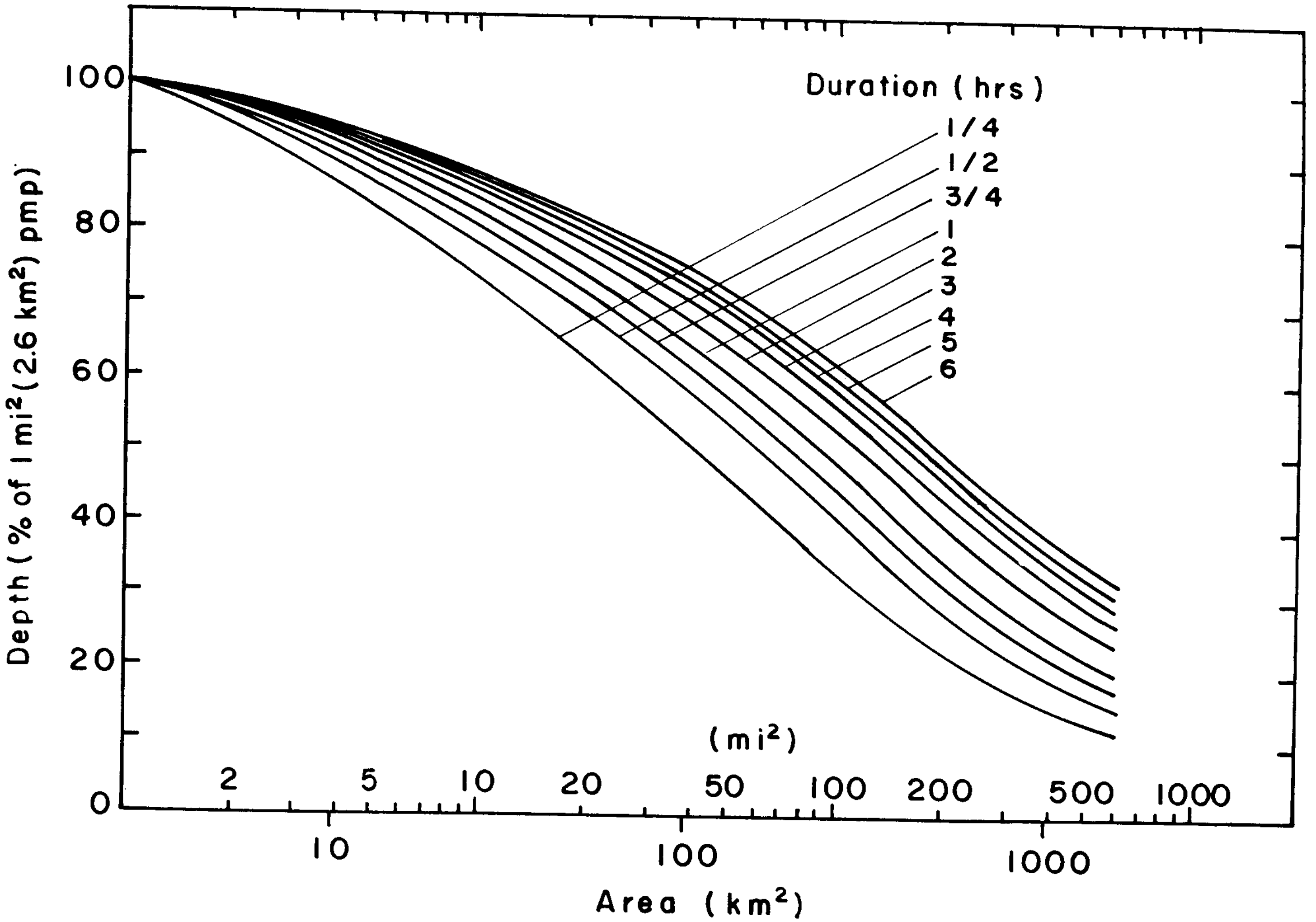


Fig. 2.3. Adopted depth-area relations for local-storm PMP. *Source:* NWS, 1977.

to six hours although the majority of the precipitation falls within the first three hours of the storm. In order to reflect the durational and geographical differences between sites, ratios between the 6-hour general type storm and the 1-hour thunderstorm (6/1-hr ratios) were determined and are presented in Figure 2.4 for the southwest United States. Similar information is available in HMR 51 and HMR 55 (NWS, 1979 and 1984). Figure 2.4 indicates that the 6/1-hr ratios range from 1.1 to 2.0 and that a single depth-duration relation is not regionally applicable. Therefore, the 6/1-hr ratio is site specific and varies with drainage basin area. Furthermore, the lower the 6/1-hr ratio, the greater the local-storm PMP percentage that falls in the initial period of the storm.

For determining the PMF for a watershed or reclaimed site with small drainage areas, the rainfall intensity corresponding to the time of concentration must be determined. In order to determine the PMP rainfall intensity, the incremental PMP rainfall depths for a specific site must first be derived. The PMP rainfall depths can be estimated as a percent of the PMP values for both the 1-hour thunderstorm and the 6-hour general-type storm. Table 2.1 presents the rainfall duration and percent PMP values (thunderstorms ranging in duration from 2.5 to 60 minutes) for determining appropriate rainfall depths in the Colorado River basin (NWS, 1977 and NRC, 1985). Similar rainfall duration and PMP percentage relations can be developed for the northwest states (HMR 43), for the midwestern and eastern states (HMR 51) and for the region between the continental divide and the 103° meridian (HMR 55) as shown in Figure 2.2.

Table 2.1 Percent of Probable Maximum Precipitation for Various Rainfall Durations in the Colorado River Drainage Area.

Rainfall Duration min.	% of 1-hour PMP*
2.5	27.5
5	45
10	62
15	74
20	82
30	89
45	95
60	100

*The 1-hour, 1 square mile local storm is derived using 6/1-hour ratios from 1.2 to 1.3.

Source: NWS, 1977 and NRC, 1985.

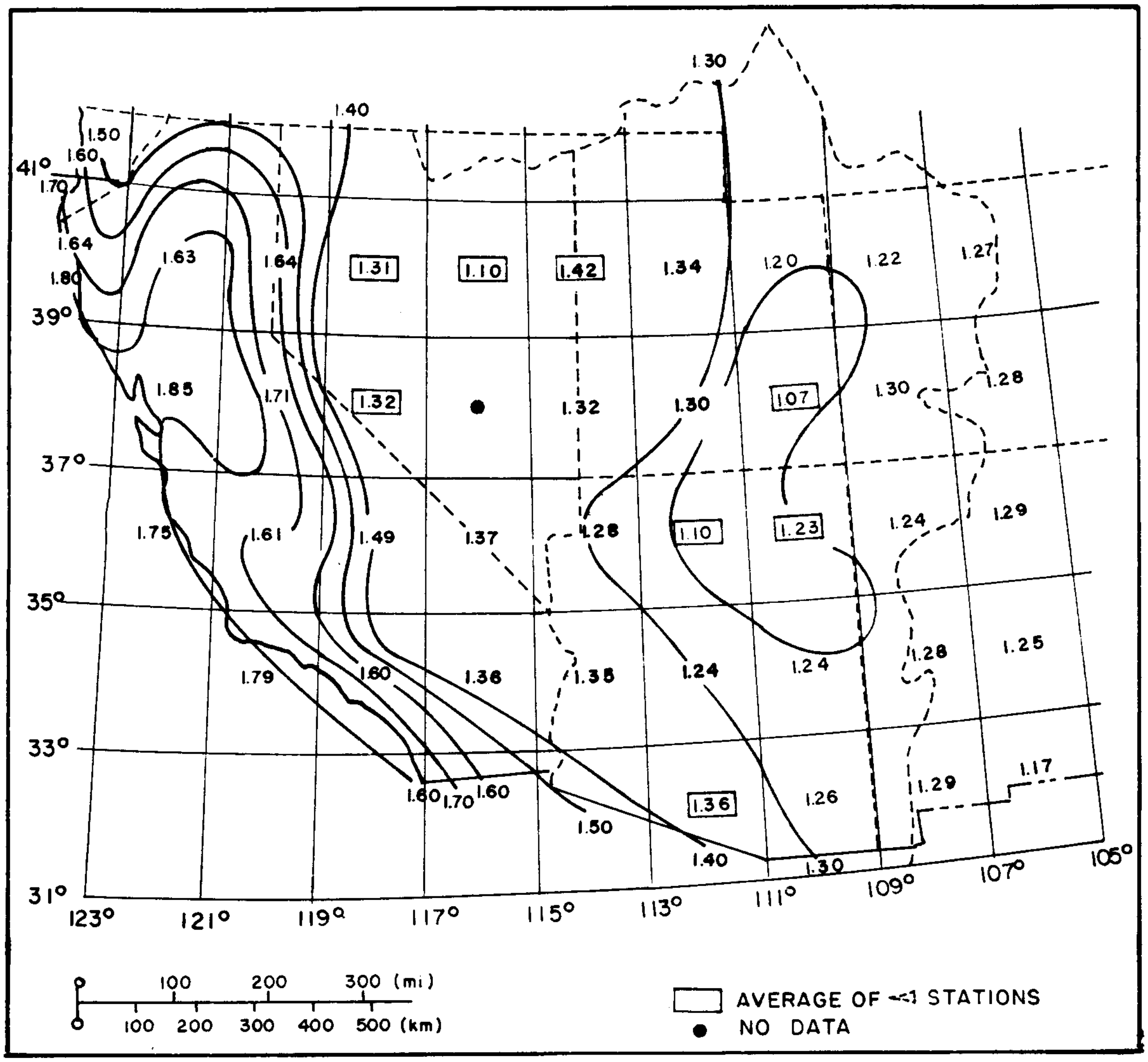


Fig. 2.4. Analysis of the 6/1-hour ratios for averaged maximum station data plotted at midpoints of a 2° latitude-longitude grid. *Source:* NWS, 1977.

The rainfall depth for a specific site is estimated by determining the rainfall duration and/or appropriate time of concentration. The resulting rainfall depth in inches, is

$$\text{PMP rainfall depth} = (\%\ \text{PMP}) \times (\text{PMP}) \tag{2.1}$$

where the percent PMP is obtained from Table 2.1 and the PMP is obtained from the appropriate PMP design storm presented in Section 2.1.1.

The rainfall intensity, i, in inches per hour can be computed as

$$i = \text{rainfall depth (inches)} \times \frac{60}{\text{rainfall duration (minutes)}} \tag{2.2}$$

The rainfall intensity determined from Equation 2.2 is generally a conservative value and represents the peak rainfall intensity of the design storm.

To compute the rainfall intensity for any rainfall duration, it is recommended that a rainfall intensity versus rainfall duration curve be plotted on semilogarithmic paper. Because of the extremely conservative rainfall intensity values obtained for short durations, it is recommended that the minimum rainfall duration be 2.5 minutes. Rainfall depths should be extracted from the appropriate Hydrometeorological Report.

2.2 PMP COMPARISON STORMS

A comparison of estimates of the PMP with greatest observed rainfall and estimates of the 100-year events for areas both east and west of the 105° meridian was prepared (NWS, 1980). Information from 6500 precipitation reporting stations in the eastern U.S. and about 2100 stations in the west was used. Including storm durations of 6 to 72 hours, the study indicated that 177 separate storm events have been recorded in which the rainfall was greater than or equal to 50 percent of the PMP for stations east of the 105° meridian. Only 66 separate storm events were recorded west of the 105° meridian where rainfalls were greater than or equal to 50 percent of the PMP.

The National Weather Service also reported the number of storm events which met or exceeded the 100-year rainfall values and compared them with the regional PMP values (NWS, 1980). Table 2.2 summarizes these rainfall events for 6 and 24-hour storms occurring over a 10 square mile area. It is interesting to note that a storm has not been officially recorded west of the Continental Divide that exceeds 90% of the PMP value. However, it is evident that a number of storms approach the PMP values, thereby substantiating that the prescribed PMP values are not extremely conservative.

Table 2.2 Comparison of Probable Maximum Precipitation (PMP) with 100-Year Rainfalls

	> 50%	> 60%	> 70%	> 80%	> 90%
East of 105th Meridian	59	32	19	7	3
West of Cont. Divide	77	39	13	4	0

Source: NWS, 1980

A comparison of the 6-hour, 10-square mile PMP to the 100-year rainfall depth is presented for areas west of the Continental Divide in Figure 2.5. The map indicates that the mountains and other topographic masses significantly affect the regional variation in rainfall magnitudes. The ratio of the 6-hour, 10-square mile PMPs to the 100-year rainfall range from 3 to 8. Ratios of 3 to 5 prevail in the uranium mining areas.

2.3 PMF ESTIMATION

The Probable Maximum Flood (PMF) is an estimate of the rainfall-runoff relationship for a particular drainage basin with site specific conditions. The determination of the volume and duration of the PMF is determined through an extensive assessment of the watershed parameters and application of the appropriate PMP (Sect. 2.1).

Input parameters commonly used in a PMF determination include, but are not limited to, the watershed area, average slope, elevation differences, length of watercourse, soil type and runoff potential, type and amount of cover, antecedent moisture conditions, soil infiltration rates and soil compaction. The flood hazard should also be determined. It is recommended that a high hazard analysis, as discussed in Design of Small Dams (DOI, 1977), be used for evaluating the long term stability for the reclamation of uranium mill tailing impoundments due to the radioactive nature of the tailings and Environmental Protection Agency regulations which quantify the time period of stability (EPA, 1983).

It is recommended that state-of-the-art procedures be used to estimate the PMF. One of the most commonly accepted procedures is the triangular Hydrograph Procedure developed by the Soil Conservation Service (SCS) as presented in Design of Small Dams (DOI, 1977) for moderate sized watersheds. The SCS procedure is readily available and is incorporated as a design option in HEC-1 (COE, 1974). The American Nuclear Society has also specified procedure ANS 2.8 for estimating the PMF for major stream systems (ANSI, 1985).

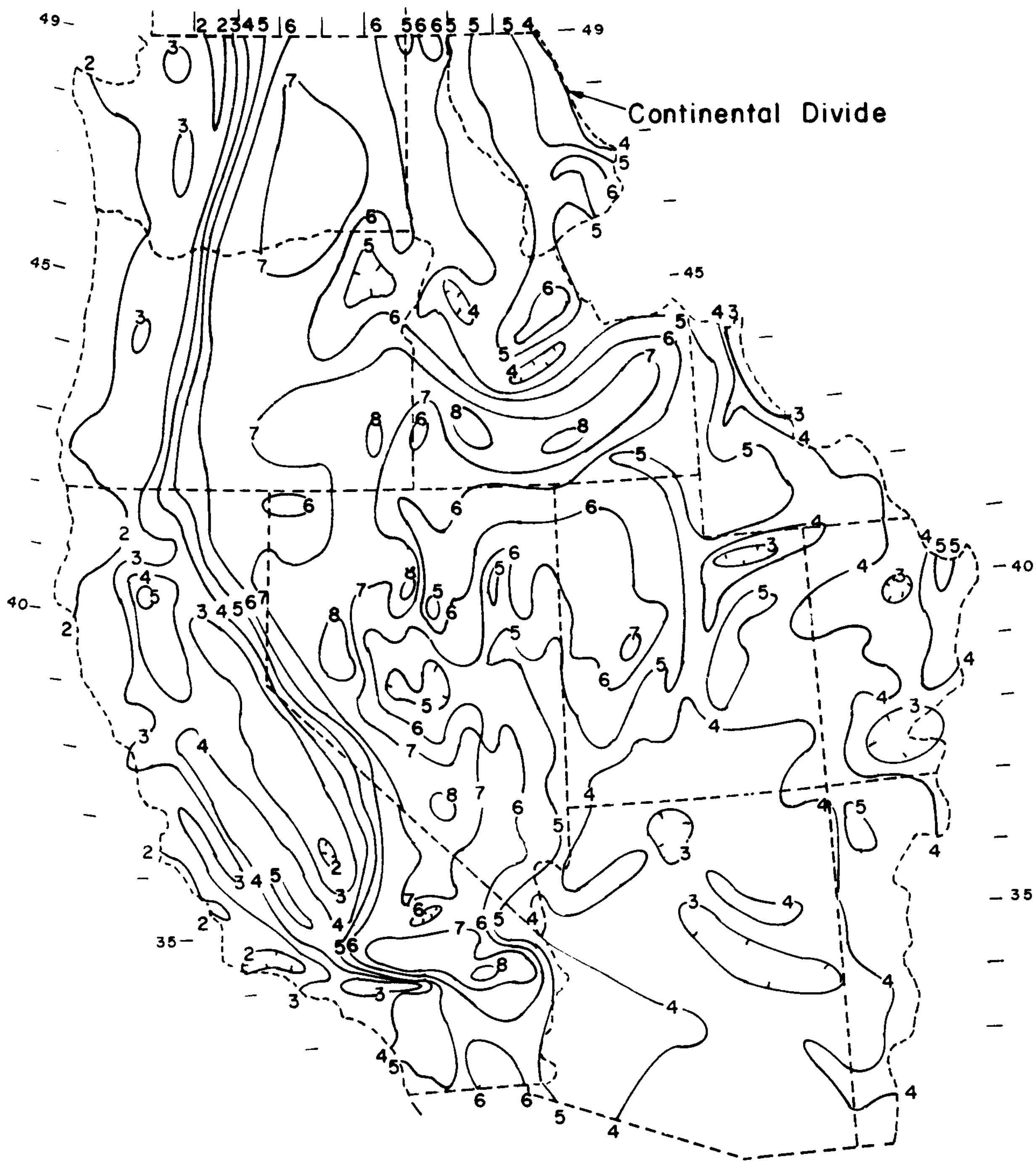


Fig. 2.5. Ratios of PMP for a 10-square-mile-area and 6-hour duration to actual 100-year rainfalls. *Source:* NWS, 1980.

The Rational Method (Chow, 1964) can be applied to determine the PMF peak discharge for drainage basins or covers with areas less than approximately one square mile. However, it is recommended that one of the state-of-the-art procedures be used when possible since the Rational Method may not directly account for many of the basin parameters.

3. FLUVIAL GEOMORPHIC INFLUENCES

Several of the older tailings impoundments such as Grand Junction, Rifle, Gunnison and Durango are located in areas where an existing stream may intrude upon the tailings site if changes in the stream channel should occur. To assess the extent and effect of channel intrusion it is necessary to evaluate the stability of the fluvial system under consideration. A major area of concern is the potential for flood intrusion.

Schumm (1977) considers an ideal fluvial system to consist of three distinct zones [upper (1), middle (2), and lower (3)] as indicated in Figure 3.1. In zone 1, a channel's location is generally stable. Down cutting and erosion of the channel floor are dominant and the flood-plain is poorly developed. In Zone 2, a channel's location is generally unstable. Side cutting dominates over down cutting, and erosion on the outside of meander loops is more or less balanced by deposition on the inside of meander loops. Meander loops are often cut off and abandoned as the main channel migrates across a well developed floodplain. In Zone 3, a channel becomes more or less braided or breaks up to form a series of distributaries in forming a delta. Deposition is dominant and erosion is more localized. Channel location is generally stable except during a flood when the channel may break through a natural levee.

With the emphasis on minimizing the upstream drainage area above uranium mill tailings impoundments, most of the sites developed after 1975 are located in Zone 1. The inactive sites may be in either Zone 1 or Zone 2. With the possible exception of south Texas, there are no uranium mill tailings impoundments in Zone 3.

In Zone 1, the major factor of concern is erosion and instability of the system such that localized erosion or gullying could encroach on the impoundment. These considerations are covered in Chapter 4 where gully formation and geomorphic stability of the site are discussed.

In Zone 2, a major concern would be flood-induced changes in the river channel that results in intrusion of the river on an impoundment. A brief discussion of methods of field investigation to estimate river stability was prepared by S.A. Schumm [Appendix D of Nelson, et al. (1983)] based on a report by S.A. Schumm and R.J. Chorley (1983). Three distinct phases of investigation are noted therein.

The first phase of investigation is reconnaisance both upstream and downstream for several miles away from the site under consideration. The purpose of this reconnaissance is to observe ongoing erosion and deposition and identify potential unstable characteristics along the river channel. These factors can influence a river's stability by changing its slope or altering the sediment load.

The second phase involves detailed site inspections in the immediate area of the impoundment site. This would involve a study of the channel morphology, bank erosion, sediment characteristics, and vegetation type for

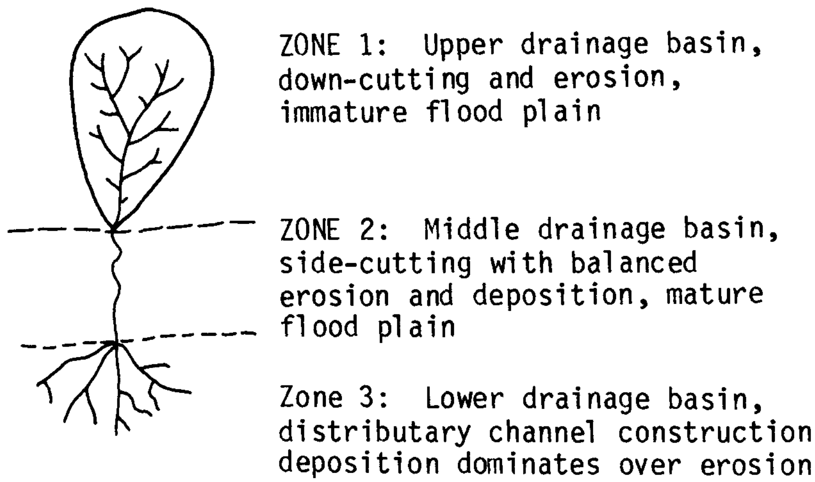


Fig. 3.1. Idealized fluvial system.

a short distance upstream and downstream. Characteristics of the river including channel dimensions, pattern, slope, and stability of the banks should be investigated.

The third phase consists of a historical study which would compare the past and present channel behavior. This phase includes a review of the history of nearby bridges. Channel width changes can be estimated by comparing the present river channel cross-section with those indicated to have been in existence when the old bridge was designed and constructed. Old photographs and recollections of long-time residents of the area provide indications of the past behavior of the river. Newspaper reports, railroad company files, gauging station records and old aerial photographs are also useful resources. Rates of channel shift can also be assessed by determining the age of vegetation and trees on the floodplain.

3.1 IDENTIFICATION OF FLUVIAL INSTABILITY

A quantitative method for assessing fluvial stability can be developed on the basis of equations and charts presented by Schumm (1977). Factors influencing river morphology include bed-material load, mean water discharge, median sediment size, channel slope, and other external geomorphological controls on the overall river system.

Rivers can be classified into three types of channels: straight, meandering, and braided channels (Figure 3.2). Factors influencing the type of channel include slope, mean annual discharge, amount of sediment load, and whether the channel sediment is characterized as bed load, mixed load, or suspended load.

In assessing the potential for the river channel to intrude upon the tailings impoundment, it is necessary to consider factors affecting both horizontal and vertical channel stability. Table 3.1 summarizes the changes that can occur in or along river channels, including changes in channel type.

3.1.1. Horizontal Stability

Horizontal stability refers to the potential for a river to change from one type to another with accompanying change in location. Figure 3.3 shows the general effect of slope on the sinuosity (Schumm defines sinuosity as the ratio of channel length to valley length) for experiments that were conducted in a flume under controlled conditions. In these experiments the sediment load and discharge rate were controlled.

At low slopes the river is just capable of carrying the sediment load. If the slope were to decrease due to development of a meander, the flow rate (velocity) would decrease and the meander channel would begin to fill with sediment. As a result, the stream would probably return to the original straight channel.

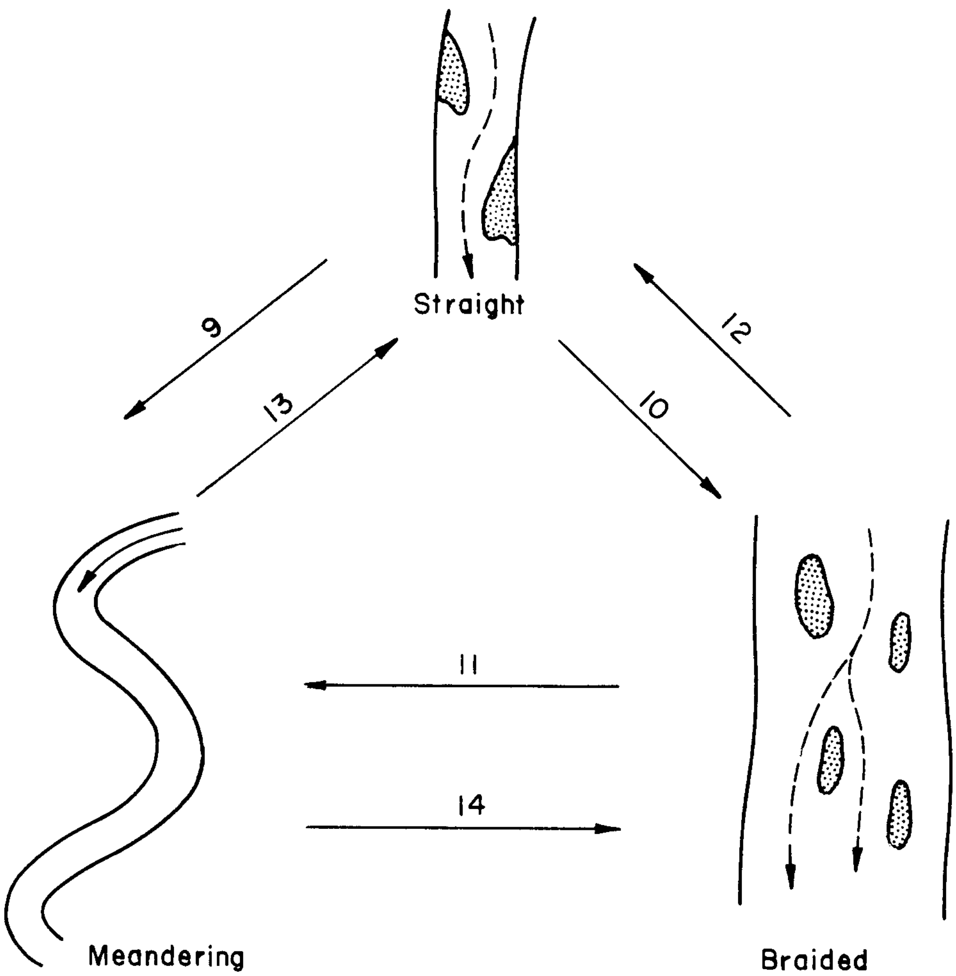


Fig. 3.2. Principal types of river channels. Numbers refer to metamorphic changes in river channels as given in Table 3.1.

Table 3.1 Types of Changes Occurring Along River Channels

Erosion	
1.	Degradation and scour
2.	Nickpoint migration
Deposition	
3.	Aggradation and fill
4.	Down filling and back filling
Pattern-change	
5.	Meander growth and shift
6.	Channel bars and islands
7.	Cutoffs
8.	Avulsion
River-metamorphosis	
9.	Straight to meandering
10.	Straight to braided
11.	Braided to meandering
12.	Braided to straight
13.	Meandering to straight
14.	Meandering to braided

Source: Nelson, et al. 1983 and Shen and Schumm, 1981.

As the slope of the channel increases, the river is capable of transporting more sediment and meanders can develop, thus increasing the sinuosity. However, if the sinuosity increases to a point that is too great, the river may become unstable again. As the slope increases, the stream can become braided and depending upon flow conditions and sediment load changes, the river can fluctuate between braided and meandering. Schumm (1977) notes that "....if one can identify the range of patterns along a river, then within that range the most appropriate channel pattern and sinuosity probably can be identified. If so, their engineer can work with the river to produce its most efficient or most stable channel. Obviously a river can be forced into a straight configuration or it can be made more sinuous, but there is a limit to the changes that can be induced beyond which the channel cannot function without a radical, morphological adjustment ....".

Changes in the sediment load will also influence the type of river channel that forms. Figure 3.4 shows the effect of slope and sediment load on channel type for a given discharge rate. These data have been combined in Figure 3.5 which shows sinuosity as a function of stream power. Stream power is the product of tractive force and velocity, and velocity depends on hydraulic radius, channel slope, and specific weight of the fluid. Thus, stream power is a function of the same hydrologic variables. In Figure 3.5, the portion of the data curve that would represent straight, meandering, and braided streams are the same as indicated in Figure 3.3.

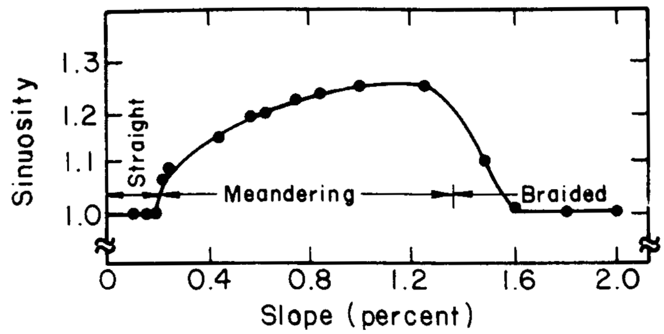


Fig. 3.3 Relation between channel sinuosity and flume slope. *Source:* from Schumm and Khan, 1972.

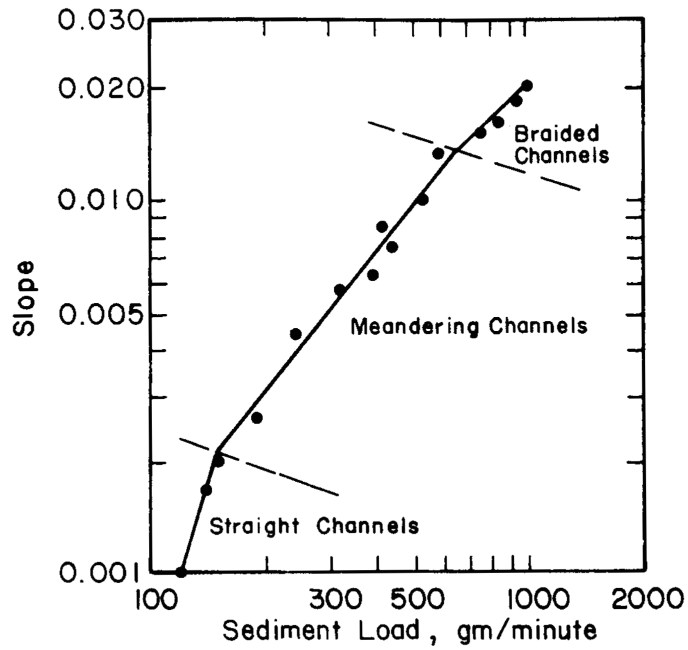


Fig. 3.4. Relation between sediment load and flume slope showing increased rate of sediment transport at pattern changes. *Source:* from Schumm and Khan, 1972.

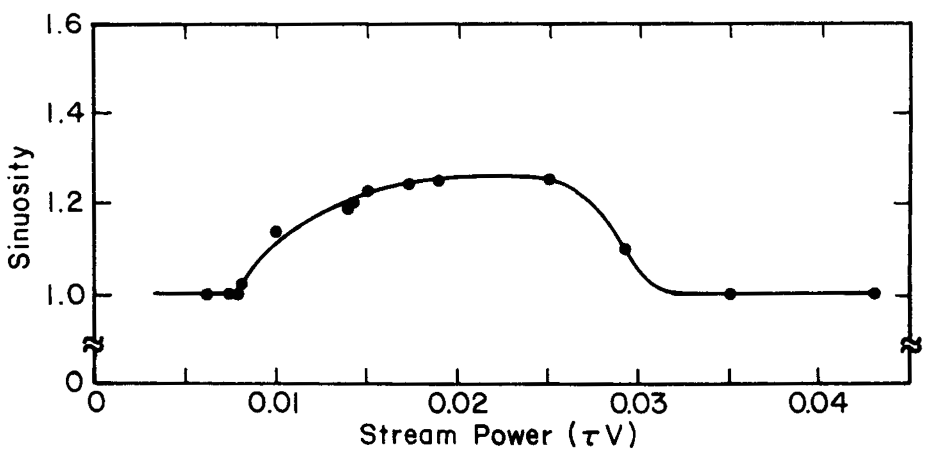


Fig. 3.5. Relation between sinuosity and stream power. *Source:* from Khan, 1971.

Slope vs. mean annual discharge is shown in Figure 3.6. The experimental points suggest that the lower line defining the threshold between braided and meandering channels may be the appropriate line to use. Thus, the horizontal stability of the river can be assessed by plotting the parameters of the river on Figures 3.5 and 3.6 for comparison with threshold values. This will provide an indication of the stability of the river. For actual rivers, different relationships will exist for mean annual discharge, mean annual flood, etc. (Schumm, 1977).

The horizontal stability of a river channel is also controlled by site specific factors that are independent of the channels own flow regime. Main channels are typically diverted away from the confluence of delta-forming tributaries whose energy has been dissipated by the slower moving main channel. The delta is built outward, thus creating a large and steadily growing bend in the river. Geologic structure also plays a significant role in determining the horizontal stability of a stream channel. Streams displaying an angular flow pattern are likely to be controlled by joint or fault systems. Trellis (parallel) drainage patterns suggest that long, parallel trends of easily eroded strata control the positions of stream channels. Mill tailings impoundments located in erosion resistant strata and away from joint or fault systems are less likely to be threatened by structurally controlled streams.

3.1.2 Vertical Stability

Vertical stability relates to the potential for the slope to change which can result in down cutting. Down cutting can lead to erosion at the impoundment site or cause a channel to change from one type to another.

Rivers may be categorized as bedrock-controlled channels or alluvial channels depending upon their freedom to adjust their shape and gradient. Bedrock-controlled channels are those where the slope of the river is controlled by nickpoints (sharp breaks in channel slope, often the result of variation in erosion resistance) and bedrock outcrops. The vertical stability of a bedrock-controlled channel is dependent primarily upon the erosion resistance of the bedrock forming the nickpoints. Generally, bedrock-controlled channels are vertically stable.

The stream bed and banks of alluvial channels are composed of sediment transported by the river under present flow conditions. The vertical stability of alluvial channels varies widely, depending upon a more complex set of parameters, including the percent of silt and clay in the channel sediment and the percentage of total load that is carried as bedload by the stream. Table 3.2 provides an indication of the relative vertical stability of alluvial channels based on channel characteristics and type and amount of sediment load.

3.2 IMPACT OF FLOOD INTRUSION

The impact of flood intrusion on a tailings impoundment will depend to a large extent upon the velocity of the river flow, and the extent to which

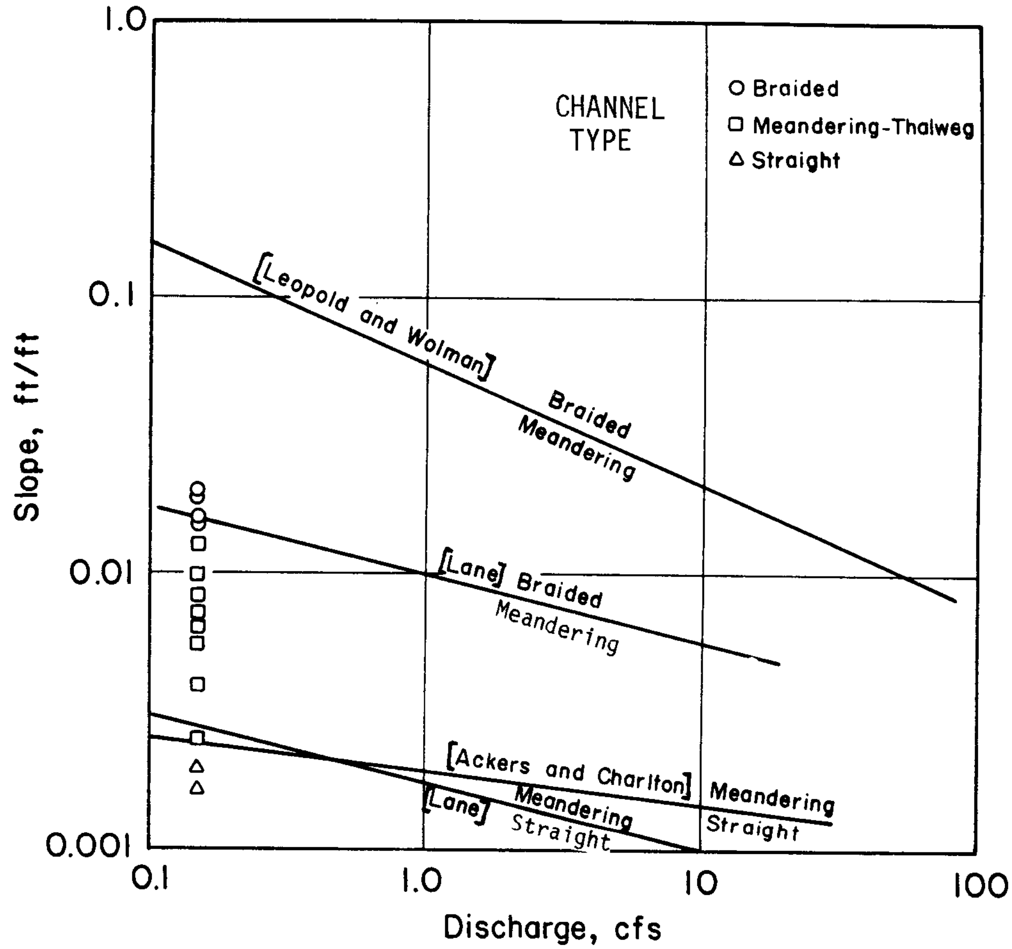


Fig. 3.6. Relation between slope and discharge and threshold slopes at each discharge, as defined by Lane (1957), Leopold and Wolman (1957), and Ackers and Charlton (1971). Symbols show position of experimental channels. *Source:* from Schumm and Khan, 1972.

Table 3.2 Relative vertical stability of alluvial channels based on channel characteristics and sediment load.

Mode of sediment transport and type of channel	Suspended load (percent of total load)	Bedload (percent of total load)	Channel vertical stability: Stable (graded stream)	Channel vertical stability: Depositing (excess load)	Channel vertical stability: Eroding (deficiency of load)
Suspended load	>20	<3	Stable suspended load channel. Width/depth ratio <10; sinuosity usually >2.0; gradient relatively gentle	Depositing suspended load channel. Major deposition on banks cause narrowing of channel; initial streambed deposition minor	Eroding suspended-load channel. Streambed erosion predominant initial channel widening minor
Mixed load	5-20	3-11	Stable mixed-load channel. Width/depth ratio >10, >40; sinuosity usually <2.0; >1.3; gradient moderate	Depositing mixed-load channel. Initial major deposition on banks followed by streambed deposition	Eroding mixed-load channel. Initial streambed erosion followed by channel widening
Bed load	>5	<11	Stable bed-load channel. Width/depth ratio >40; sinuosity usually <1.3; gradient, relatively steep	Depositing bed-load channel. Streambed deposition and island formation	Eroding bed-load channel. Little streambed erosion; channel widening predominant

down cutting can occur. In general, it is assumed that if the river channel comes into contact with the impoundment, localized erosion will occur and release of tailings is possible. Therefore, if it can be shown that the main channel may encroach upon an impoundment, the site should be considered unacceptable. If flood waters with relatively low velocities encroach upon the impoundment, the site may be acceptable with appropriate protection of the impoundment to minimize erosion. This will be dicussed further in Chapter 4.

3.3 MITIGATIVE PROCEDURES

If vertical stability is questionable, stable base levels (theoretically, the lowest level of erosion of a portion of the earth's surface) can be manufactured or created artificially. A base level must resist large flows that may occur under PMF conditions. Creation of base levels must also take into consideration the potential for horizontal instability to occur which would cause the river channel to bypass the artificially created stable base level.

If horizontal instability is a concern, structures may be constructed to divert the river around the impoundment, even under PMF conditions. Stable diversion channels can only be developed where the PMF flow and velocities are of reasonably small size. For large flows, on major rivers in the western United States, a stable diversion channel would be virtually impossible to achieve. Diversion channel stability is determined in the same manner as that for natural channels as previously presented.

On the other hand, if the river is not large, the river channel might be rerouted. In so doing, however, the engineer must be cognizant of the variables previously discussed so as to create a river channel which will be stable. In addition, the effect of these changes on potential variations in such variables must be considered so as to avoid the occurrence of channel instability at a later time.

3.4 SUMMARY

Application of the methodology discussed above would consist of initial gathering of data in accordance with recommendations of S.A. Schumm (Appendix D, Nelson et al., 1983). After this has been accomplished, the horizontal stability of the site can be determined by plotting the appropriate parameters (slope, sinuosity, and discharge) in Figures 3.3, 3.5, or 3.6. If the nature of the river channel disagrees with that shown by the regions on which it plots in these figures, the river may be considered to be unstable. If the data points indicate an unstable condition, the nature of the instability should be assessed and the potential for river intrusion into the tailings impoundment must be determined. In addition, the influence of tributaries on channel morphology should be considered.

An important parameter that will be utilized in plotting the above data will be the slope of the river. This will probably be controlled to a large extent by nickpoints and stable base levels at locations both above and below the impoundment. The stability of these nickpoints and the ability of the river to migrate laterally and bypass the nickpoints must be determined by a competent geologist.

Those parameters which define the stability of a fluvial system have been defined and outlined by Schumm (1977). However, the interpretation of the data and application of the methodology will require considerable engineering and geological judgement. The concepts presented above are based upon threshold considerations and some judgement must be exercised in defining those thresholds. It must also be recognized that meandering streams may experience radical shifts in channel location without transition to straight or braided courses. Channel shifting is likely to be gradual during normal flow but may be catastrophic during extreme flood flows.

Finally, the influence of geologic structure upon stream channels should be considered. Mill tailings impoundments located away from channel controlling geologic features are less susceptible to flood intrusion and are unlikely to be affected by either a temporary or a permanent channel shift under non-flood conditions.

4. GULLY EROSION ESTIMATION AND PROTECTION

Gully erosion is the development of deep gullies by the dislodging and transporting of soil particles by concentrated flow. Nelson, et al. (1983) extensively discussed gully erosion and emphasized the high potential for the gully to intrude upon the impoundment. This form of gully erosion is caused by concentrated runoff and floods resulting from precipitation events occurring on the major watersheds near the impoundment area. In addition to the potential for gully intrusion from offsite activity, gully erosion can also occur directly on the impoundment surface and, as such, is a potential failure mode because it can cut through the embankment and/or the cover material and disperse tailings downstream. Erosion on the impoundment is caused by runoff from tributary catchment areas immediately adjacent to, and at higher elevations than, the impoundment area.

The development of gullies on the impoundment is associated with erosional forces on immature surfaces. Since reclaimed impoundment covers are composed of locally derived materials that were stockpiled or removed from an adjacent site, the cover is immature and may require extensive periods of time to mature. It is generally assumed that the reclaimed cover will be more vulnerable to gully intrusion than an in situ material.

A gully is a relatively deep, recently formed, eroding channel that forms on valley sides and on valley floors where no well-defined channel previously existed. Two major gully types have been recognized: (1) the valley-side gully, which is an extension of the valley network and which is incising into soil colluvium and weak bedrock and (2) the valley-floor gully which may be discontinuous or continuous and which is incising into alluvium.

The development of incised channels of all types, including gullies, can be considered an aspect of drainage-network adjustment. The drainage patterns that develop are assumed to reflect modern climatic and hydrologic conditions. In many areas the channel network does not completely fill the valley network, and it is capable of expansion by gullying if erosional conditions change.

Major site-specific parameters that influence gully development are topographical features such as slope angle and slope length, the existence of stable base levels on or near the site, erodibility of the soil, and the flood flow velocity. Stable base levels, or stable slope, are levels below which no further erosion would be expected. Specific geomorphic and hydrologic conditions that increase the potential for gullying include steep slopes, narrow flow width and large runoff volume as related to the drainage basin area. Site-specific information concerning these parameters is needed in order to determine the potential for gullying on the valley sides and valley floor areas near the impoundment.

Water flowing over a surface will tend to dislodge and transport soil particles from the preferred flow paths on the surface, which ultimately causes formation of a gully. Because gully erosion is usually rapid and progressive, it is essential to prevent gully initiation to assure long-term stability of an area. Nelson, et al. (1983) indicated that protective measures based on runoff from a Probable Maximum Precipitation (PMP) event should prevent gully formation for periods of up to 1000 years and should provide adequate protection for the cumulative effects associated with the mean annual flows. Since the PMP is based on physical constraints and is not time-dependent, protection against gully formation for 200 years is, therefore, the same as providing stability for 500 and 1000 years. This is true as long as a stable base level is maintained at some point to prevent gully formation as a result of downstream influences.

It is evident, therefore, that protection against gully formation for 200, 500, or 1000 years entails the same mechanisms and design procedures for each period. The time frame over which stability can be assured will be governed by the durability of the materials used to provide erosion protection and establish the base levels.

4.1 GULLY INTRUSION PREDICTION PROCEDURE

In order to determine the gully intrusion potential of an impoundment cover, an extensive field investigation was conducted by Falk, et al (1985). A series of reclaimed tailings sites were surveyed in which gullys developed into and in some instances through the cover material. Data collection included cover soil samples, gully dimensions, pile dimensions, precipitation records, age and maintenance records for each site.

Based upon the extensive analysis performed by Falk, et al. (1985), a gully intrusion prediction procedure was developed to estimate the maximum depth of gully incision, the location of the maximum intrusion from the toe of the slope, and the approximate gully top width at the point of maximum intrusion. This procedure is based on the assumptions that:

(a) the toe of the slope is relatively stable over time,

(b) the rate of gully erosion and slope degradation decreases with time,

(c) the material used to reclaim the site is homogeneous and remains relatively homogeneous over time,

(d) differential settlement will be completed within short-term periods and does not pose a long-term concern,

(e) the effects of vegetation are considered negligible for vegetative covers of 30% or less.

The gully intrusion prediction procedure is discussed in the following subsections.

4.1.1 Site Specific Information

It is necessary to determine the site specific characteristics of the reclaimed slope as shown in Figure 4.1. These characteristics include as a minimum:

(a) Cover or cap soil parameters of median particle diameter, d_{50}, in mm, and uniformity coefficient, C_u.

(b) Pile dimensions including the side slope length, L, slope height, H, and the gradient of the initial or proposed slope, S_i prior to gullying.

(c) Precipitation records of the site or nearest gaging station. It is recommended that at least 30 years of precipitation records be obtained where possible.

(d) Estimate the time, in years, over which the potential gully incision is to be evaluated.

4.1.2 Tributary Drainage Area

It is necessary to determine the area tributary or potentially tributary to any point where flows may concentrate. Integration of a cover into the natural terrain often requires contouring. Although sheet flows are desirable, flow concentrations often occur as described in Section 4.9 of this report. Because incipient gully initiation and subsequent gully development are a function of the drainage area, an estimate of the area is required. The tributary area may be determined by one of the following procedures:

(a) Estimate the largest drainage area tributary to any point along the side slope which might serve as a drainage outlet derived from the reclamation plan contour map.

(b) Define the longest potential watercourse that traverses across the cover to the slope toe. Compute the approximate tributary area from the Drainage Density equation presented by Mosley (1971) as

$$D = 0.909 + 22.418\ (S_i) \qquad (4.1)$$

where D is the density of drainage area per unit length (ft^2/ft) of channel and S_i is the design or initial slope of the cover before gullying. Equation 4.1 should be applied to each segment of the watercourse with similar slope. The total tributary area to the outlet at the toe of the slope is estimated by

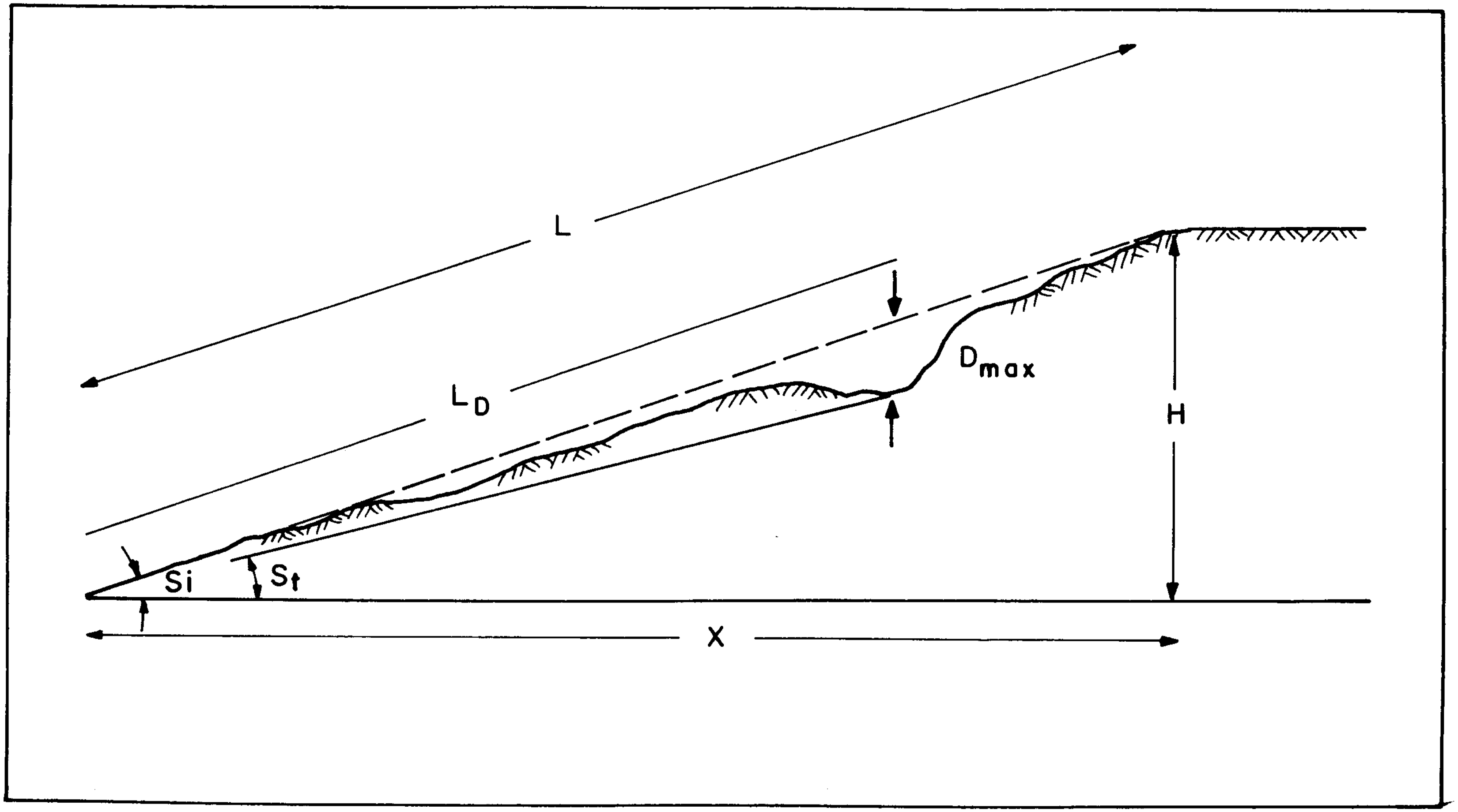


Fig. 4.1. Cross section of a typical reclaimed slope illustrating gully and associated headcut.

$$A_{total} = D_j L_j + D_{j+1} L_{j+1} + \dots \quad (4.2)$$

where L_j is the potential watercourse length at a slope of S_i for drainage density D_j.

4.1.3 Maximum Depth of Incision

Once the site specific characteristics and drainage area are determined, it is possible to estimate the maximum depth of gully incision, the location on the slope of the maximum intrusion referenced to the slope toe, and the top width of the gully at the point of maximum incision. The slope limits are the initial slope gradient, S_i, and the stable slope (S_s) gradient which can be predicted as

$$S_s = \frac{(41.2)\ (1 + D_{50})}{(A)\ (P)} \quad (4.3)$$

where A is the tributary drainage area in square feet, P is the average annual number of precipitation events greater than or equal to 0.5 inches and D_{50} is the median particle size of the reclamation slope in mm. Equation 4.3 is derived from Figure 4.2 (Falk, et al. 1985). It is observed in Figure 4.2 that the x-intercept is the point at which erosion degradation ceases. The gradient where erosion ceases is defined as the stable slope. The estimated stable slopes generally agree with the slope-drainage area relationships derived by Schumm as pesented by Nelson et al. (1983).

During the period of evaluation, the side slope of the pile will most likely be between the initial slope, S_i, and the stable slope, S_s. The interim or transitional slope, S_t, shall be defined as the slope of the tangent extending from the toe-of-the-slope to the deepest point in the gully as shown in Figure 4.1. The transitional slope can be determined at a desired point in time and can be calculated as

$$S_t = (\ S_i\ )\ e^{-(G\ S_s\ t)} \quad (4.4)$$

where G is a coefficient and t is the estimated time in years. The coefficient, G, was determined to be a function of the stable slope, S_s, as presented in Figure 4.3. The exponential is a decaying function that accounts for the decreasing rate of erosion that is assumed to occur as the gully slope decreases over time. It is recommended that a maximum time period of 200 years be used for this analysis. Knowing the stable slope, it is possible to estimate the location of the maximum depth of incision measured from the toe-of-the-slope. The maximum depth of gullying was

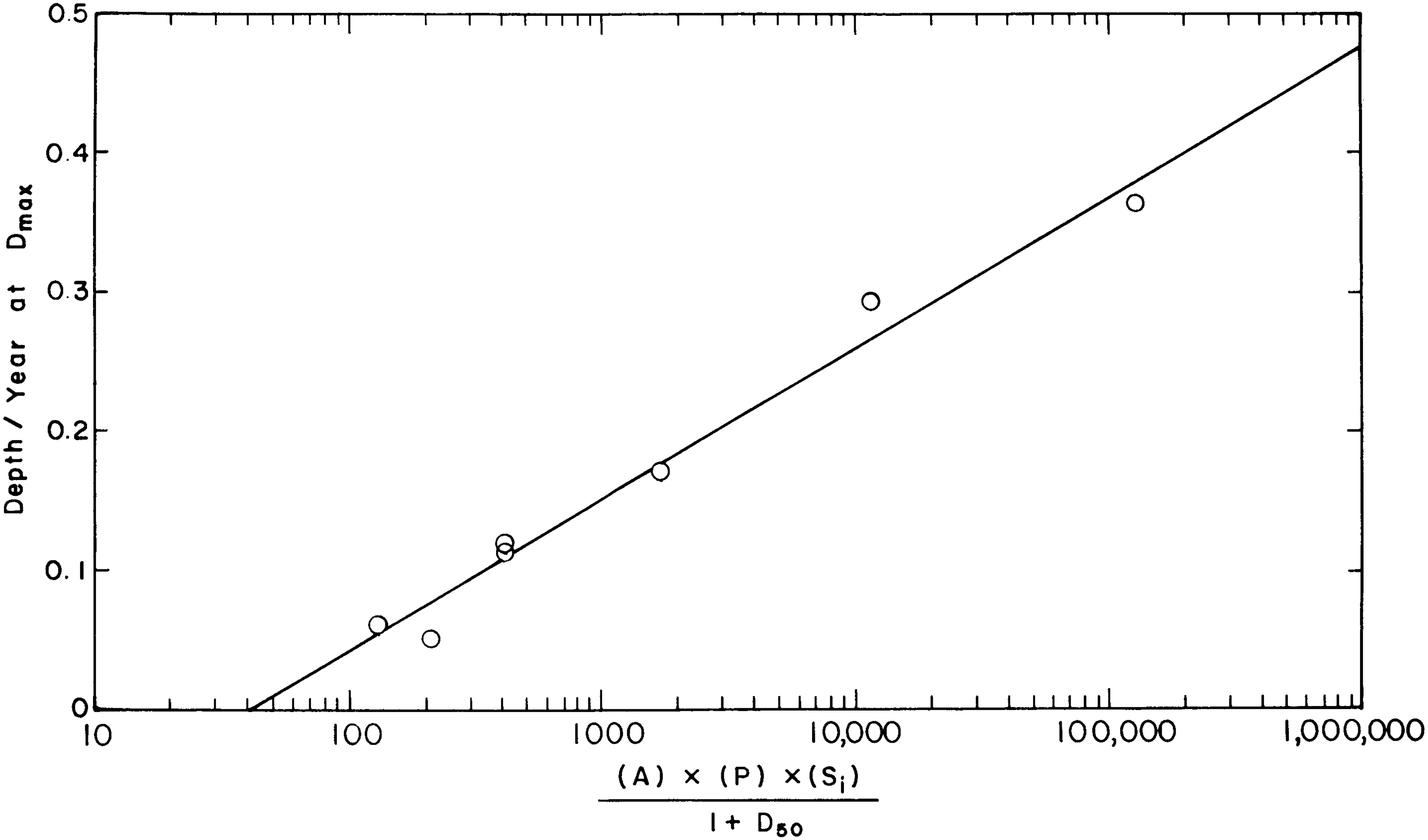


Fig. 4.2. Relation of variables to the average rate of gully incision on reclaimed slopes.

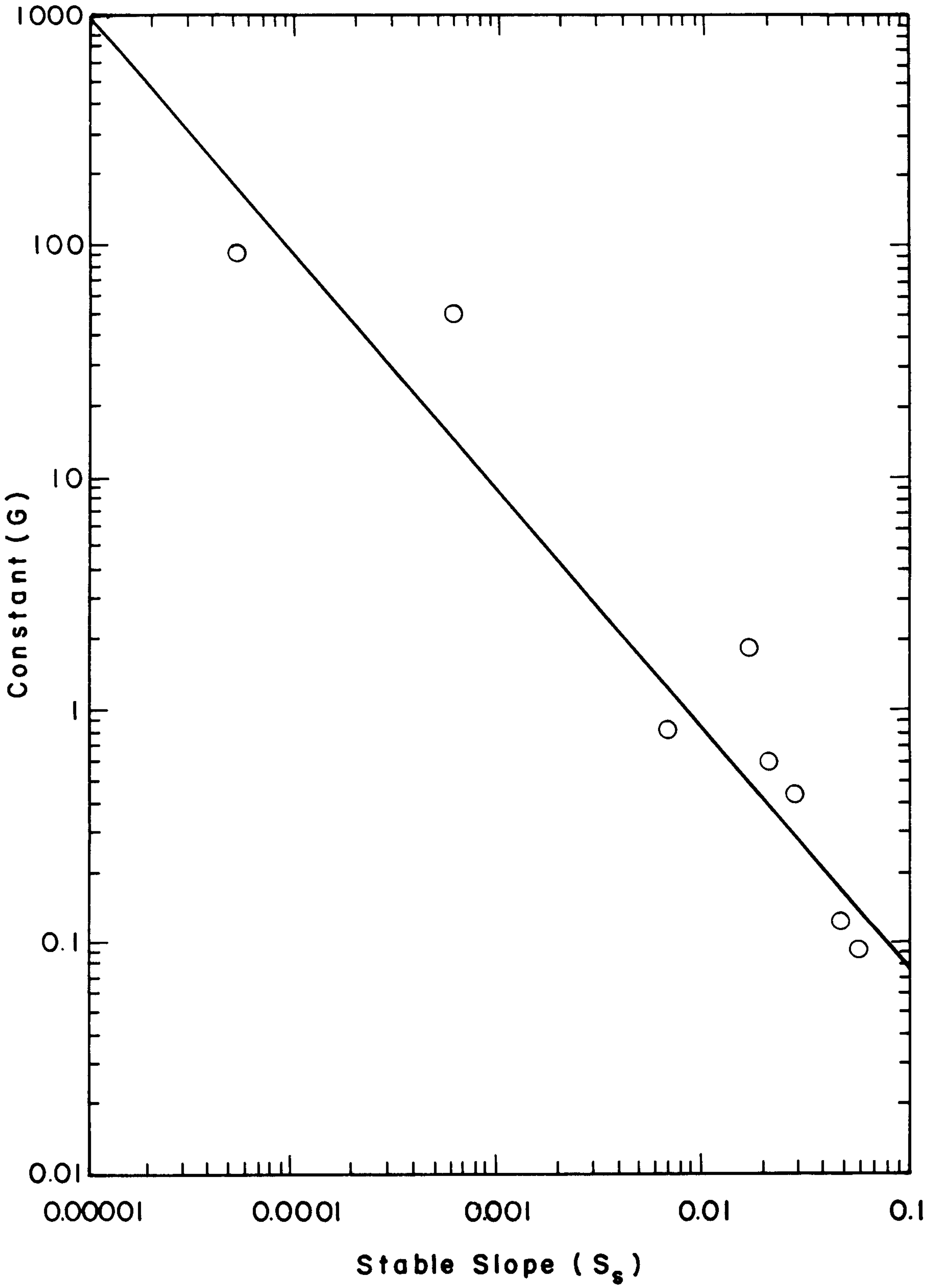


Fig. 4.3. Relation of stable slope to the calibration constant, *G*.

found to be a function of the transitional slope, S_t, and the soil uniformity coefficient, C_u, as presented in Figure 4.4.

The maximum depth of gully incision, D_{max}, is estimated as

$$D_{max} = \frac{L_D}{L} H - (X)(S_t + S_s) \qquad (4.5)$$

where D_{max} is the maximum depth of gully incision, L_D/L is the occurrence of D_{max} from the toe-of-the-slope and X is the horizontal distance from the toe-of-the-slope to the crest of the reclaimed embankment. Since the maximum gully depth and the location of occurrence can be estimated, it is possible to determine whether the gully can penetrate the cap and cover into the tailings.

Once the maximum depth of incision is computed, the gully top width can be estimated at the point where D_{max} occurs. Figure 4.5 presents the gully top width relationship to the predicted maximum depth of incision and uniformity coefficient. The approximate gully top width can be estimated as

$$W_T = 4.936 + 2.923 \text{ LOG}(D_{max}/C_u) \qquad (4.6)$$

where W_T is the top width at D_{max} and C_u is the uniformity coefficient of the cover material.

4.1.4 Limitations

The gully erosion estimation procedure presented was developed under the assumptions as stated in Section 4.1 and formulated with a limited data base from sites in the arid, western United States (Falk, et al., 1985). Care must be taken to not apply this procedure to sites where the assumptions are not applicable. The intent of this procedure is to provide a rough estimate of gully development potential applied to reclaimed tailings piles. It is recommended that the time of analysis not extend beyond 200 years.

4.1.5 Example Problem

A uranium tailings pile is to be reclaimed. It is proposed to cover the pile with a soil such that the side slopes of the pile along the main embankment will be reclaimed at a 1V:5H slope. The top of the pile will be contoured so that there will not be any area tributary to the embankment above the crest. The soil was analyzed and found to have median grain diameter (d_{50}) of 1.2 mm and a uniformity coefficient (C_u) of 10. The height of the embankment at the crest is 43.1 feet resulting in a slope length of 220 feet around the perimeter. The area receives an average of

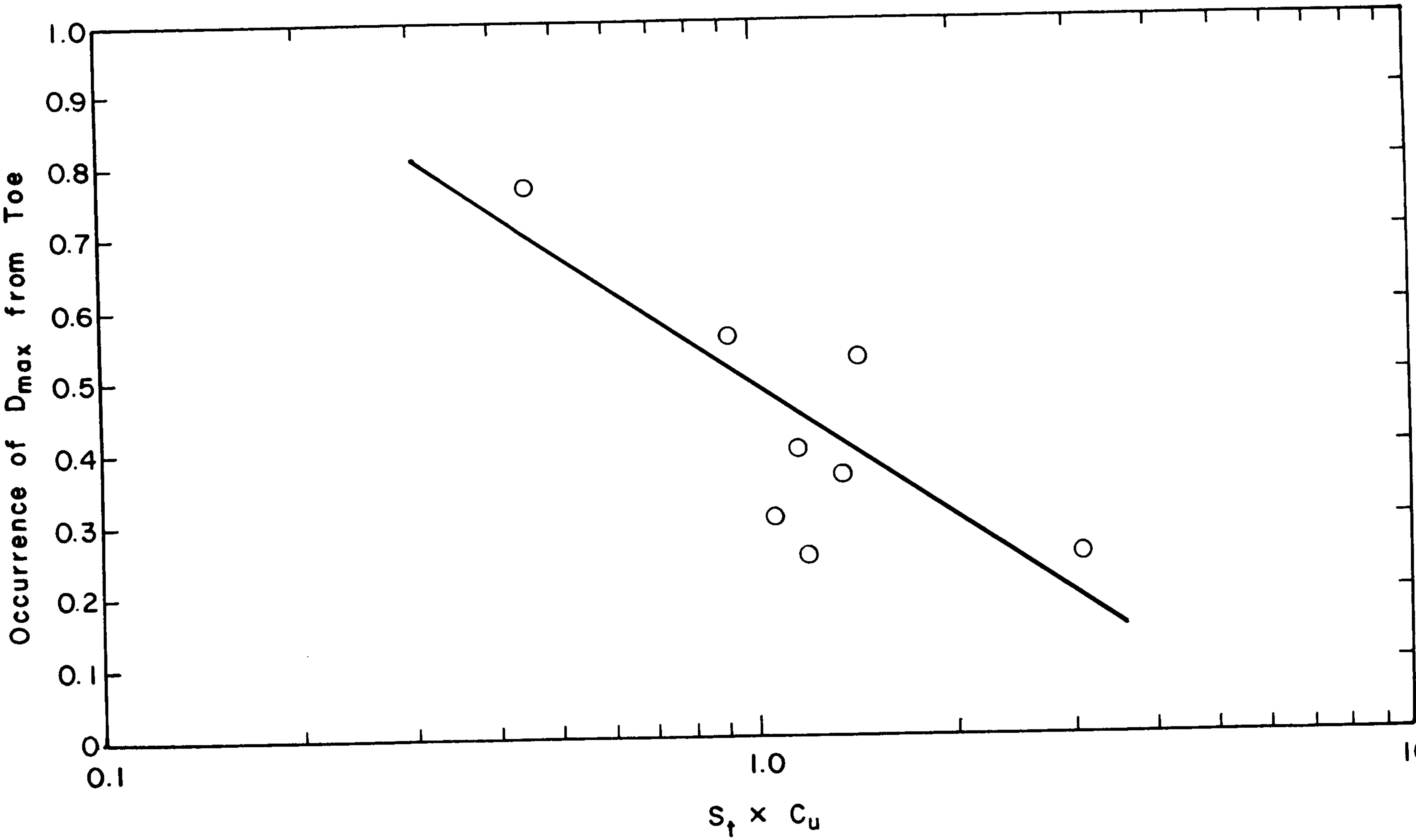


Fig. 4.4. Relation of time-dependent slope and uniformity coefficient vs occurrence of maximum gully depth from toe-of-slope.

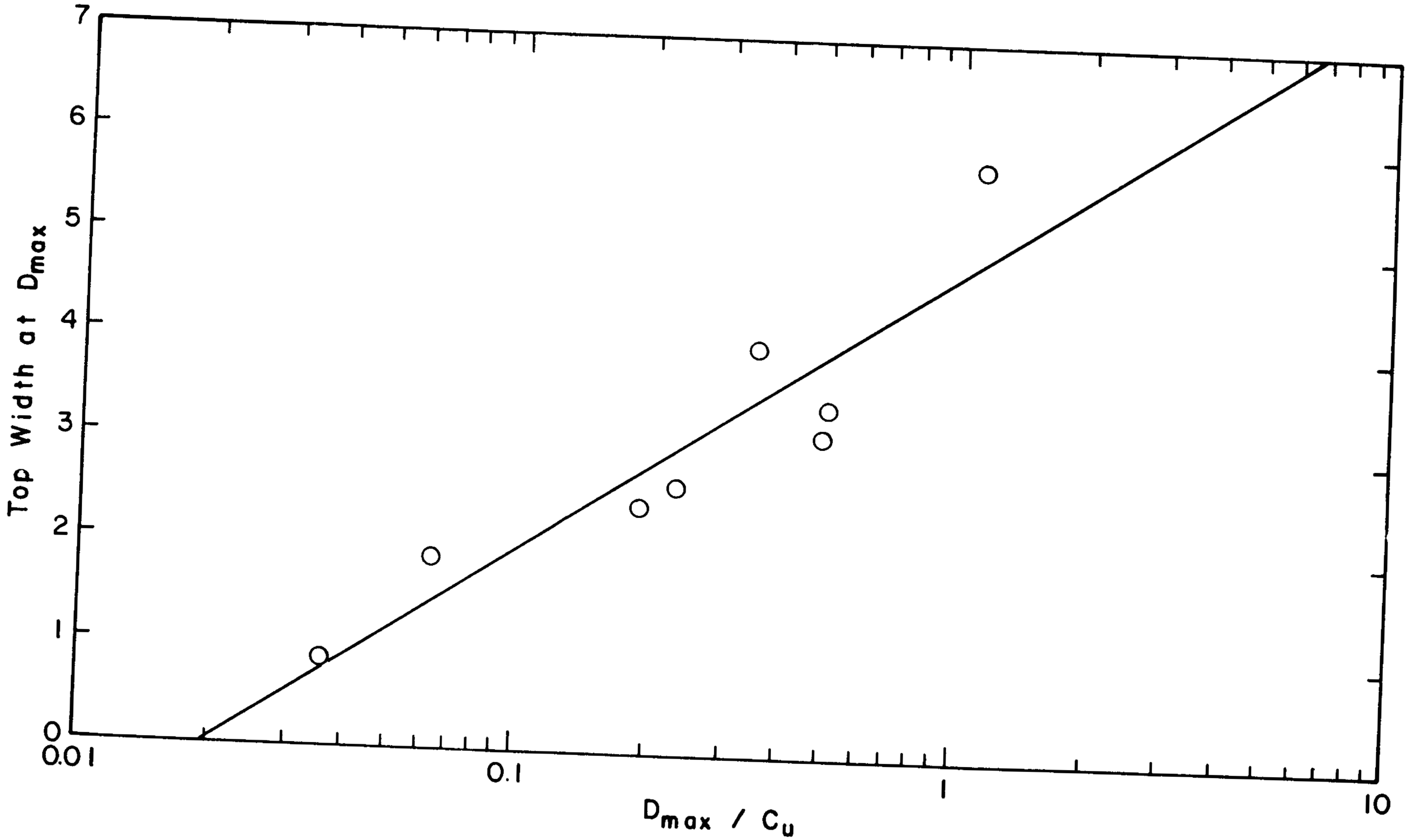


Fig. 4.5. Depth-uniformity parameter vs top width at maximum gully depth for reclaimed sites.

2.8 precipitation events annually in which 0.5 inches or greater of rainfall is expected. The site will be revegetated where 25% of the area is expected to be covered and stabilized.

Given these proposed reclamation parameters, determine the gully incision potential after 200 years.

4.1.5.1 Drainage Path

Since there is not a tributary drainage area above the slope, it is assumed that the longest drainage path will be directly down the face of the slope. Therefore, the drainage path is approximately 220 feet long.

4.1.5.2 Tributary Area

The tributary drainage area can be determined using the Mosley relation presented in Equation 4.1. The drainage density can be estimated as

$$D = 0.909 + 22.418\ (0.2) \tag{4.7}$$

therefore,

$$D = 5.39\ \text{ft}^2/\text{ft} \tag{4.8}$$

The potential drainage area is the product of the drainage density and longest drainage path and is calculated as

$$\text{Drainage Area (A)} = 5.39 \times 220 \tag{4.9}$$

$$A = 1186\ \text{ft}^2 \tag{4.10}$$

4.1.5.3 Stable Slope

The stable slope, S_s, or slope at which gully erosion is not expected to occur can be estimated using Equation 4.3 where

$$S_s = \frac{(41.2)(1 + 1.2)}{(1186)(2.8)} \tag{4.11}$$

and

$$S_s = 0.027 \qquad (4.12)$$

Therefore, the stable slope is estimated to be approximately 2.7%.

4.1.5.4 Transitional Slope

The transitional slope can be estimated after 200 years of erosion and potential gullying. However, the G constant must first be determined utilizing Figure 4.3. The G constant for a stable slope of 2.7% is approximately 0.31. Equation 4.4 can be solved to estimate the transitional slope as

$$S_t = (0.20)\ e^{-[0.31\ (0.027)\ 200]} \qquad (4.13)$$

Therefore, the transitional slope is

$$S_t = 0.038 \qquad (4.14)$$

4.1.5.5 Depth of Incision

The potential maximum depth of gully incision can be approximated using Equation 4.5. However, the location of the headcut must be estimated from Figure 4.4. Knowing the transitional slope, S_t, and the soil uniformity coefficient, C_u, the occurrence of D_{max} from the toe (L_D/L) is 0.74. Equation 4.5 can be solved to determine the maximum depth of headcutting as

$$D_{max} = [0.74]\ [43.1 - (215.7)(0.038 + 0.0270] \qquad (4.15)$$

and

$$D_{max} = 21.5 \text{ ft} \qquad (4.16)$$

which occurs at a point L_D, where

$$L_D = 0.74 \times 220 = 162.8 \text{ ft} \qquad (4.17)$$

from the toe-of-the-slope.

4.1.5.6 Gully Width

The width of the gully across the top of the gully at the point of maximum depth can be estimated from Figure 4.5. Having computed the maximum depth, D_{max}, and knowing the uniformity coefficient, C_u, the top width is estimated to be approximately 5.6 feet. However, the gully width will widen over time to where the gully side wall stands at an angle less than the angle of repose of the cover material.

4.2 EMBANKMENT AND SLOPE STABILIZATION USING RIPRAP

Rock riprap is one of the most economical materials that is commonly used to provide for cover and slope protection. Factors to consider when designing rock riprap are: (1) rock durability, density, size, shape, angularity, and angle of repose; (2) water velocity, depth, shear stress, and flow direction near the riprap; and (3) the slope of the embankment or cover to be protected. Through the proper sizing and placement of riprap on any impoundment cover, rill and gully erosion can be minimized to ensure long term stabilization.

The primary failure mechanism of concern is the removal of material from the impoundment due to shear forces developed by water flowing parallel and/or adjacent to the cover as described by Nelson et al. (1983). One purpose of the cover is to expedite the removal of precipitation and tributary waters away from the cover to minimize seepage and percolation. However, when surface waters are not properly managed, extreme erosion may result and endanger the impoundment stability. For example, slopes are often designed and constructed to develop sheet flow conditions. After many years of exposure, sheet and rill erosion, and localized settlement, the hydraulic conditions have significantly altered causing flows to merge or concentrate into drainage channels. The greater the concentration of flow into the drainage channels, the greater the erosion potential.

4.2.1 Zone Protection

The design requirements for placing riprap rock on a cover vary depending upon cover location. It is suggested that four areas exist on the cover in which different failure mechanisms can result from tributary drainage. The four areas or zones of concern are presented in Figure 4.6 and include:

1. Zone I: This zone is considered the toe-of-the-slope of the reclaimed impoundment. The riprap protecting the slope toe must be sized to stabilize the slope due to flooding in the major watersheds and dissipate energy as the flow transitions from the impoundment slope into the natural terrain. Zone I is considered a zone of frequent saturation.

2. Zone II: This is the area along the side slope which remains in the major watershed flood plain (PMF). The rock protection must resist not only the flow off the cover, but also floods. The

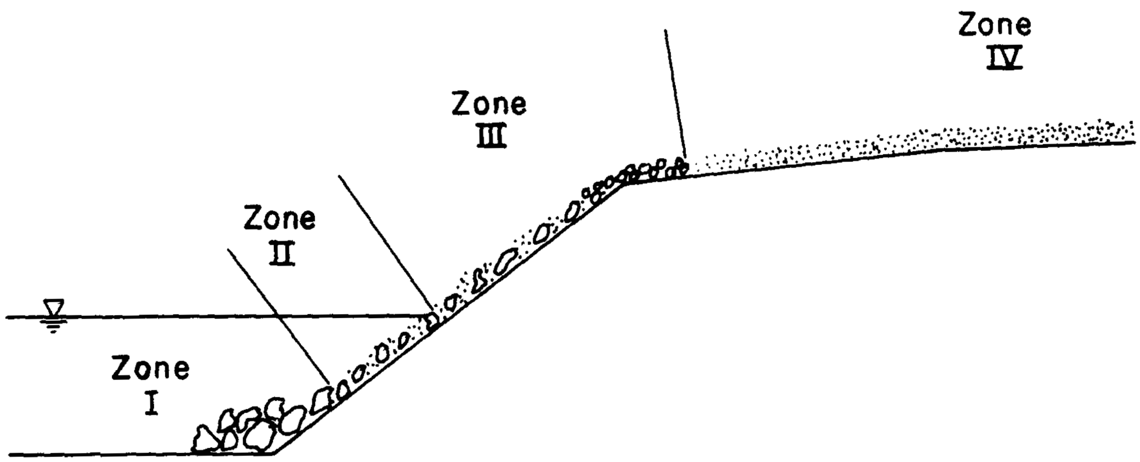


Fig. 4.6. Zones of a reclaimed impoundment requiring riprap protection.

riprap must serve as embankment protection similar to river and canal banks. Zone II is considered a zone of occasional saturation.

3. Zone III: Riprap should be designed to protect steep slopes and embankments from potential high overtopping velocities and excessive erosion. Flows in Zone III are derived from tributary drainage and direct runoff from the reclaimed site. Zone III is considered a seldom saturated zone.

4. Zone IV: Rock protection for Zone IV is generally designed for flows from mild slopes. Zone IV will usually be characterized by sheet flow with low flow velocities. Zone IV is considered a zone of seldom saturation.

Since the rock protection requirements are significantly different on various locations on the cover, it should be apparent that each riprap design procedure available was formulated to address a specific application. Since a single riprap design procedure does not necessarily meet all of the cover protection requirements, recommendations will be made indicating which zone(s) each riprap design procedure best addresses.

Because the frequency of wetting or saturation varies by zone, the durability requirements of the riprap may vary by zone. The concept of durability and oversizing will be addressed in Chapter 6 of this report.

4.2.2 Design Procedures

Presently, several methods are available to assist the designer in determining the appropriate rock size for protection of impoundment covers, embankments and unprotected slopes from the impact of drainage waters. Alternative riprap design methods summarized herein are

1. Safety Factors Method
2. The Stephenson Method
3. Corps of Engineers Method
4. The U.S. Bureau of Reclamation Method

These riprap design procedures are but examples of the many methods available.

4.2.2.1 Safety Factors Method

The Safety Factors Method (Richardson et al., 1975) for sizing rock riprap is quite versatile in that it allows the designer to evaluate rock stability from flow parallel to the cover and adjacent to the cover. The Safety Factors Method can be used by assuming a rock size and then calculating the safety factor (S.F.) or allowing the designer to determine a S.F. and then computing the corresponding rock size. If the S.F. is greater than unity, the riprap is considered safe from failure; if the S.F. is unity, the rock is at the condition of incipient motion; and if S.F. is less than unity, the riprap will fail.

The following equations are provided for rock riprap placed on a side slope or embankment where the flow has a non-horizontal (downslope) velocity vector. The safety factor, S.F., is:

$$\text{S.F.} = \frac{\cos\theta \tan\phi}{\eta' \tan\phi + \sin\theta \cos\beta} \tag{4.18}$$

where

$$\eta = \frac{\eta\left[1 + \sin(\lambda+\beta)\right]}{2} \tag{4.19}$$

$$\eta' = \frac{21\ \tau o}{(S_s-1)\gamma D} \tag{4.20}$$

$$\tau_o = \gamma ds \tag{4.21}$$

and

$$\beta = \tan^{-1}\left[\frac{\cos\lambda}{\dfrac{2\sin\theta}{\eta\tan\phi} + \sin\lambda}\right] \tag{4.22}$$

The angle, λ, is shown in Figure 4.7 and is the angle between a horizontal line and the velocity vector component measured in the plane of the side slope. The angle, θ, is the side slope angle shown in Figure 4.7 and β is the angle between the vector component of the weight, W_s, directed down the side slope and the direction of particle movement. The angle, ϕ, is the angle of repose of the rock riprap, τ_o is the bed shear stress (Simons and Senturk, 1977), D is the representative rock size, S_s is the specific weight of the rock, d is the depth of flow, γ is the specific weight of the liquid, s is the slope of the channels, and η' and η are stability numbers. In Figure 4.7, the forces F and F_d are the lift and drag forces, and the moment arms of the various forces are indicated by the value e_i as i = 1 through 4. Figure 4.8 illustrates the angle of repose for riprap material sizes.

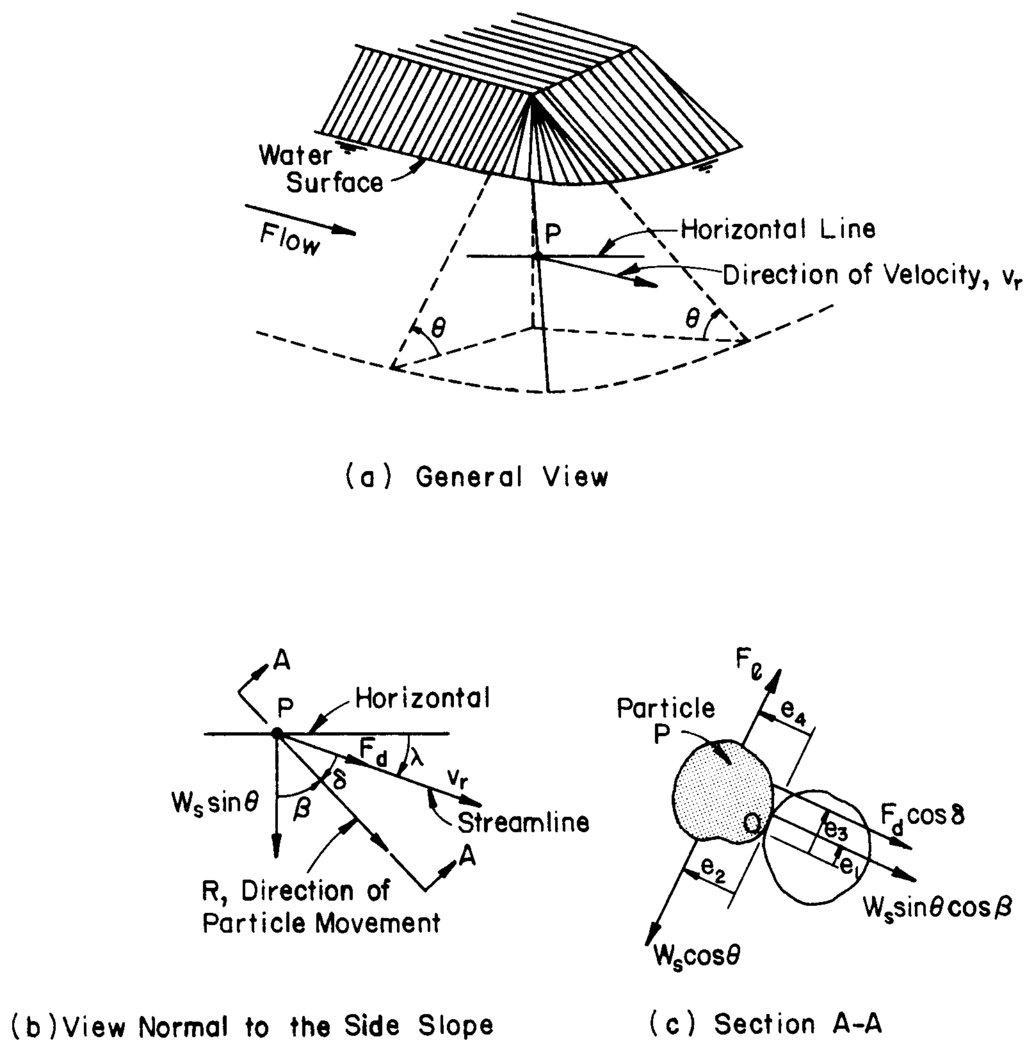


Fig. 4.7. Riprap stability conditions as described in the Safety Factors Method.

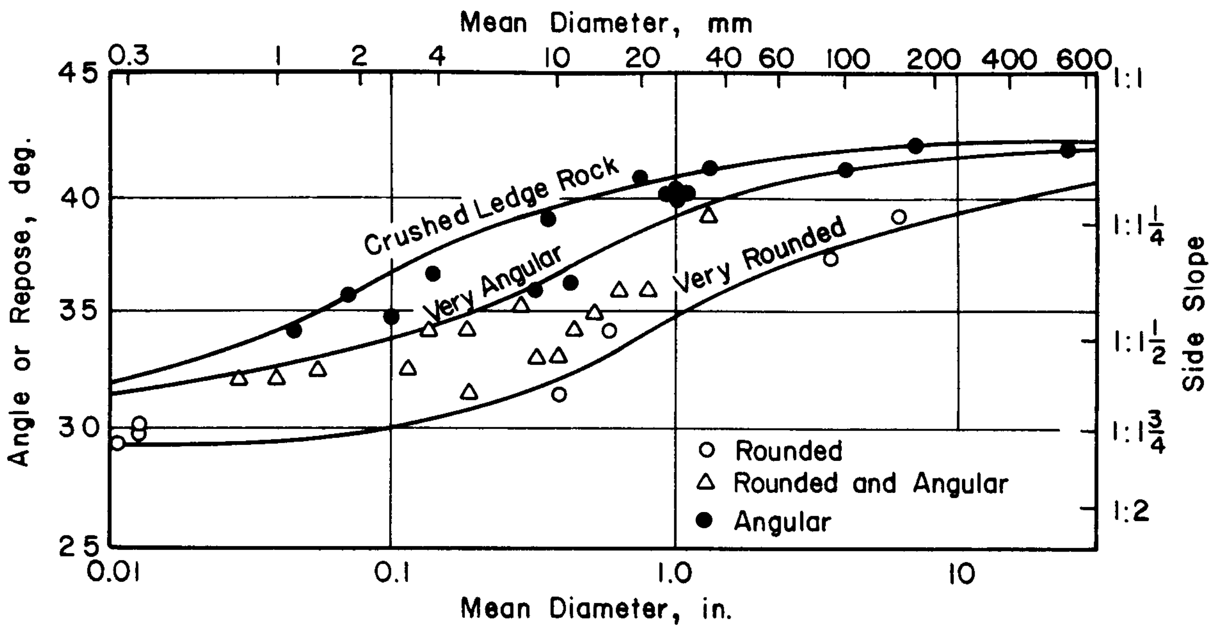


Fig. 4.8. Angle of repose as a function of mean rock diameter and shape.

Riprap is often placed along side slopes where the flow direction is close to horizontal or the angularity of the velocity component with the horizontal is small (i.e., $\gamma = 0$). For this case, the above equations reduce to:

$$\tan \beta = \frac{\eta \tan \phi}{2 \sin \theta} \tag{4.23}$$

and

$$\eta = \left[\frac{S_m^2 - (S.F.)^2}{(S.F.)\ S_m^2}\right] \cos \theta \tag{4.24}$$

where

$$S_m = \frac{\tan \phi}{\tan \theta} \tag{4.25}$$

The term S_m is the safety factor of the rock particles against rolling down the slope with no flow. The safety factor, S.F., for horizontal flow may be expressed as:

$$S.F. = \frac{S_m}{2} \left[(S_m^2 \eta^2 \sec^2 \theta + 4)^{0.5} - S_m \eta \sec \theta\right] \tag{4.26}$$

Riprap may also be placed on the cover or side slope. For a cover sloping in the downstream direction at an angle, α, with the horizontal, equations reduce to:

$$S.F. = \frac{\cos \alpha \tan \phi}{\eta \tan \phi \sin \alpha} \tag{4.27}$$

Historic use of the Safety Factors Method has indicated that a minimum S.F. of 1.5 for non-PMF applications (i.e. 100-year events) provides a side slope with reliable stability and protection (Simons and Senturk , 1977). However, a S.F. of slightly greater than 1.0 is recommended for PMF or maximum credible flood circumstances. It is recommended that the rock riprap thickness be a minimum of 1.5 times the d_{50}. Also, a bedding or filter layer should underlay the rock riprap. The filter layer should minimally range from 6 inches to 12 inches in thickness. In cases where the Safety Factors Method is used to design riprap along embankments or slopes steeper than 4H:1V, it is recommended that the toe be firmly stabilized. The Safety Factors Method is ideally suited for Zone I and Zone II riprap design.

4.2.2.2 Stephenson Method

The Stephenson Method for sizing rockfill to stabilize slopes and embankments is an empirically derived procedure developed for emerging flows (Stephenson, 1979). The procedure is applicable to a relatively even layer of rockfill acting as a resistance to through and surface flow. It is ideally suited for the design and/or evaluation of embankment gradients and rockfill protection for flows parallel to the embankments, cover or slope.

The sizing of the stable stone or rock requires the designer to determine the maximum flow rate per unit width (q), the rockfill porosity (n), the acceleration of gravity (g), the relative density of the rock (s), the angle of the slope measured from the horizontal (θ), the angle of friction (ϕ), and the empirical factor (C). The unit discharge can be estimated as indicated in Section 4.8 of this report.

The stone or rock size, d, is expressed by Stephenson as

$$d = \left[\frac{q(\tan\theta)^{7/6}\, n^{1/6}}{C\, g^{1/2}\, [(1-n)(s-1)\cos\theta\,(\tan\phi - \tan\theta)]^{5/3}} \right]^{2/3} \qquad (4.28)$$

where the factor C varies from 0.22 for gravel and pebbles to 0.27 for crushed granite. The stone size calculated in Equation 4.28 is the represention diameter, d_{50} at which rock movement is expected for unit discharge, q. The representative median rock diameter (d), is then multiplied by Oliviers' constant to insure stability. Oliviers' constants are 1.2 for gravel and 1.8 for crushed rock. The rockfill layer should be well graded and at least two times the d_{50} in thickness. A bedding layer or filter should be placed under the rockfill.

The Stephenson Method does not account for uplift of the stones due to emerging flow. This procedure was developed for flow over and through rockfill on steep slopes. Therefore, it is recommended that the Stephenson Method be applied as an embankment stabilization for overflow or sheetflow conditions. Alternative riprap rockfill design procedures should be considered for toe and stream bank stabilization. The Stephenson Method is best suited for Zone III protection and is considered an acceptable design procedure for overtopping flow conditions in this zone.

4.2.2.3 Corps of Engineers Method

The U.S. Army Corps of Engineers has developed perhaps the most comprehensive methods and procedures for sizing riprap revetment. Their criteria are based on extensive field experience and practice (COE, 1970 and 1971). The Corps of Engineers Method is primarily applicable to embankment toe and bank protection and has been developed to protect the embankment from local shear forces and localized velocities.

The toe of a slope or embankment is generally subjected to the greatest concentration of erosive forces and therefore must be protected. The effective stone size, d_{50}, can be estimated after the depth of flow, y, is determined. The local boundary shear, $\bar{\tau}_0$ can be computed as

$$\bar{\tau}_0 = \frac{\gamma \bar{v}^{\,2}}{(32.6 \log_{10} \frac{12.2y}{k})^2} \tag{4.29}$$

where γ is the unit weight of water in pounds per cubic foot, $\bar{v}$ is the average cross-sectional velocity in fps, k is the equivalent channel boundary surface roughness in feet, and y is the depth of flow in ft. By substituting d_{50} for k, the local boundary shear at any point on the wetted perimeter can be determined. The design shear stress, τ_0, should be based on critical local velocities and shall serve as the design shear for the toe and channel bottom. A graphic solution to Equation 4.29 is presented in Figure 4.9.

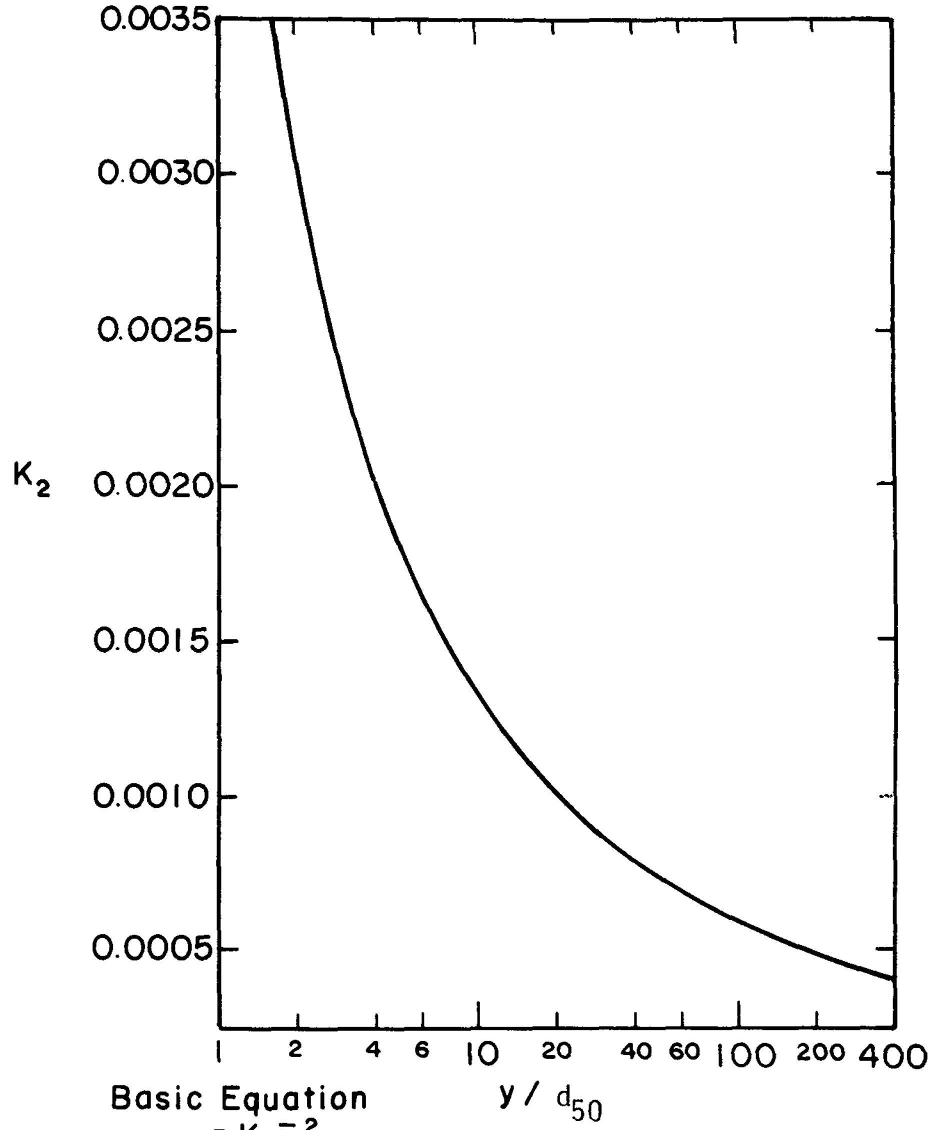
The design shear for riprap placed on the channel slope or bank can be determined as

$$\tau_0 = \tau \left(1 - \frac{\sin^2\phi}{\sin^2\theta}\right)^{0.5} \tag{4.30}$$

as

$$\tau = a\ (\gamma_s - \gamma)\ d_{50} \tag{4.31}$$

where ϕ is the angle of the side slope with the horizontal, θ is the angle of repose of the riprap (normally about 40^o), γ_s is the unit weight of surface dry, but saturated stone and the value of a is 0.04. The side slope shear, τ_0, is the design shear for sizing the riprap revetment.



Basic Equation

$$\tau_0 = K_2 \bar{v}^2$$

where

$$K_2 = \frac{\gamma}{\left[32.6 \operatorname{Log}_{10}(12.2\, y/d_{50})\right]^2}$$

γ = Specific Weight of Water

y = Flow Depth

d_{50} = Theoretical Spherical Diameter of Average Stone Size

Fig. 4.9. Graphical solution to Eqn 4.29. *Source:* COE, 1970.

The average stone size can then be determined as

$$d_{50} = \frac{\tau}{0.04\ (\gamma_s - \gamma)} \tag{4.32}$$

for the toe and channel bottom and

$$d_{50} = \tau_o / 0.04\ (\gamma_s - \gamma) \tag{4.33}$$

for the channel side slopes where γ_s and γ are the specific weights of the stone and water, respectively. The same procedure can be used for bank protection. A graphic representation of Equation 4.33 is provided in Figure 4.10.

The Corps of Engineers Method was developed for channelized flows. Therefore, this procedure should be used to evaluate and/or design rock protection for the portions of the cover or embankment that are in the floodplain. This method is ideal for stabilizing cover and embankment toes and is, therefore, best applied for Zone I and Zone II protection.

Riprap Layer Thickness

Riprap layer thickness criteria are based on ranges of acceptable rock weights and gradations. The gradation of stone weight may be expressed in terms of descriptions W_{100}, W_{50}, etc., wherein all stones from a given source weigh less than or equal to W_{100}, 50% of the stones weigh less than or equal to W_{50}, etc. The ranges of acceptable W_{100}, W_{50}, etc. weight gradations are described in terms of upper and lower limits. A graphical representation relating rock weight to rock spherical diameter was presented by Nelson et al. (1983).

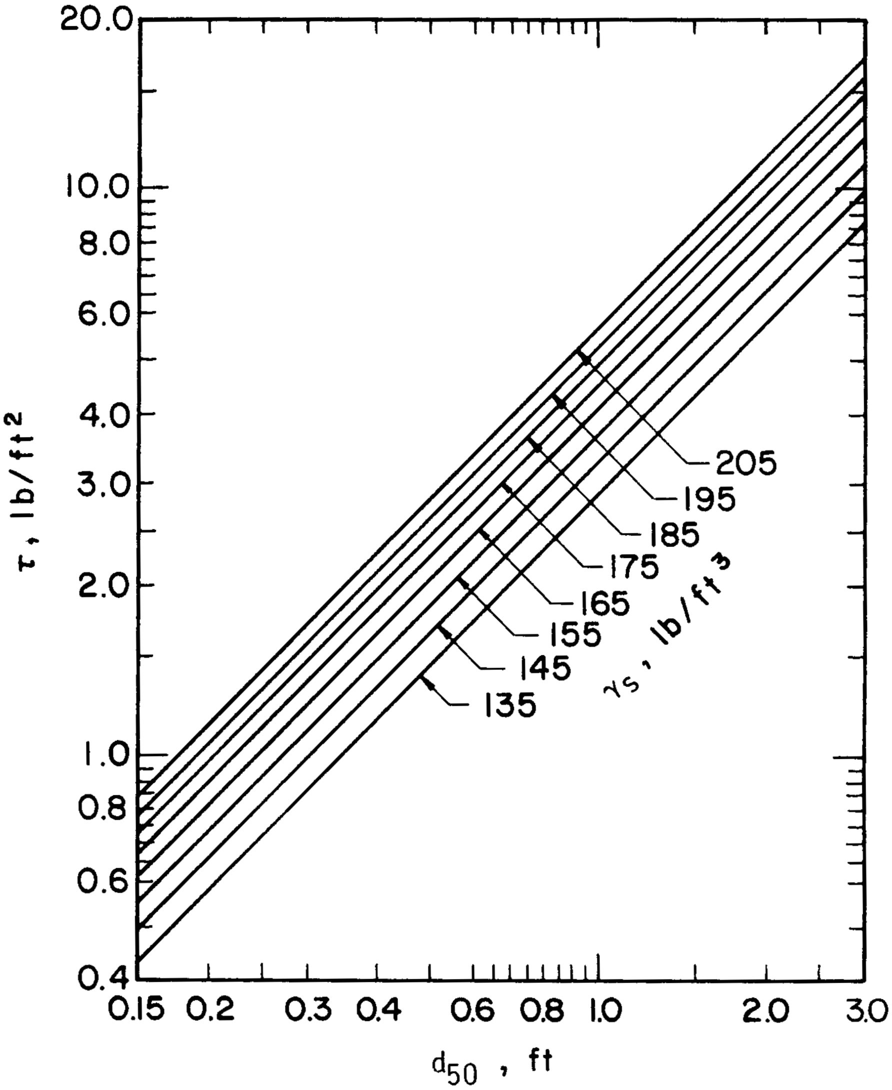
The Corps of Engineers Method (COE, 1970) presents criteria based on stone weight to determine the riprap layer thickness.

1. The thickness should not be less than the spherical diameter of the upper limit W_{100} stone or less than 1.5 times the spherical diameter of the upper limit W_{50} stone, whichever results in the greater thickness.

2. The thickness should not be less than 12 inches.

3. The thickness determined in 1 or 2 should be increased by 50% when the riprap is placed underwater.

4. The thickness should be increased by 6-12 inches and an appropriate increase in stone sizes should be provided where riprap will be subject to attack by large floating debris.



<u>Basic Equation</u>

$\tau = 0.040(\gamma_s - \gamma)\,d_{50}$

where

τ = Design Shear Stress on Bottom of Channel

γ_s = Specific Weight of Stone

γ = Specific Weight of Water (62.4 lb/ft^3)

d_{50} = Theoretical Spherical Diameter of Average Size Stone

Fig. 4.10. Sizing of riprap as a function of design shear stress. *Source:* COE, 1970.

The riprap layer thickness should be underlain with a gravel filter for channel, toe and side slope applications. The filter will serve to stabilize the rock layer, allow drainage and prevent the movement of fine embankment materials. Filter criteria are presented in Section 4.4 of this report.

Rock Gradation

The gradation of rocks in riprap revetment affects the riprap's resistance to erosion. The stone should be reasonably well graded throughout the layer thickness. The following criteria provide guidelines for establishing gradation limits.

1. The lower limit of W_{50} rock should not be less than the weight of rock required to withstand the design shear forces.

2. The upper limit of W_{50} rock should not exceed that weight which can be obtained economically from the quarry or that size which will satisfy layer thickness requirements.

3. The lower limit of W_{100} rock should not be less than two times the lower limit of W_{50} rock.

4. The upper limit of W_{100} rock should not exceed: (a) five times the lower limit of W_{50} rock, (b) that size which can be obtained economically from the quarry, or (c) that size which will satisfy layer thickness requirements.

5. The lower limit of W_{15} rock should not be less than one-sixteenth the upper limit of W_{100} rock.

6. The upper limit of W_{15} rock should be less than the upper limit of W_{50} rock as required to satisfy criteria for graded stone filters.

7. The bulk volume of rock lighter than the W_{15} rock should not exceed the volume of voids in the revetment without this lighter rock.

8. W_0 to W_{25} rock may be used instead of W_{15} rock in criteria 5, 6, and 7 if desirable to better utilize available rock sizes. Design memoranda and specifications should indicate the permissible stone gradation limits.

4.2.2.4 U.S. Bureau of Reclamation Method

The U.S. Bureau of Reclamation (USBR) Method (DOI, 1978) for riprap design was developed for the prevention of damage in and near stilling basins. The USBR procedure is empirically based upon extensive laboratory testing and field observations. Riprap failure was determined to occur because alternative design procedures underestimate the required stone size in highly turbulent zones, and there is a tendency for inplace riprap to be smaller and more stratified than specified. The USBR method is a velocity based design procedure.

Stone-Size Determination

The USBR method estimates the maximum stone size, d_{100}, as a function of the localized bottom velocity of flow, V_b, in feet per second. One means of predicting the maximum stone size is using the Mavis and Laushey (1948) procedure where

$$d_{100} = \left[\frac{V_b}{0.5\ (s - 1)^{0.5}} \right]^2 \qquad (4.34)$$

as d_{100} is the maximum stone size in mm and s is the particle specific gravity. If the bottom velocity can not be determined, local velocity may be substituted to size the rock. The local velocity can be determined using Corps of Engineers procedures (COE, 1970).

The stone size and stone weight can be determined from Figure 4.11 for a given bottom velocity, V_b. The resulting stone size is conservative. The riprap should be composed of a well-graded mixture of stone. Riprap should be placed on a filter blanket or bedding layer. The riprap layer should be 1.5 times as thick as the largest stone diameter. The filter blanket should be at least 6 inches thick.

It is recommended that the USBR method be considered only for design of rock along the toe-of-the-slope (Zone I) or where flow concentrations require substantial energy dissipation. This method would be well suited in areas where a hydraulic jump may occur. The USBR method is not necessarily recommended for bank and cover protection due to its conservatism.

4.3 SLOPE TRANSITION PROTECTION

Observation of several reclaimed tailings impoundments in which gully erosion occurred indicated that cover protection is warranted at major slope transitions (Nelson et al. 1984; Falk et al. 1985). Most of the sites were characterized by covers with flat slopes (i.e. gradients less than 0.05) transitioning to steep slopes (i.e. gradients 0.10 - 0.30) around the impoundment perimeter. In most of these cases, the gully intrusion extended 2-5 feet up-gradient from the transition. It was evident that there exists a potential for long-term gully development.

A series of tests were conducted by Powledge and Dodge (1985) evaluating the effects of overtopping on earthen dam embankments. They indicated that despite the care exercised during the construction and inspection of the structure, low areas will always occur along the crest. The low areas result from crest traffic, nonuniform settlement and a lack of maintenance.

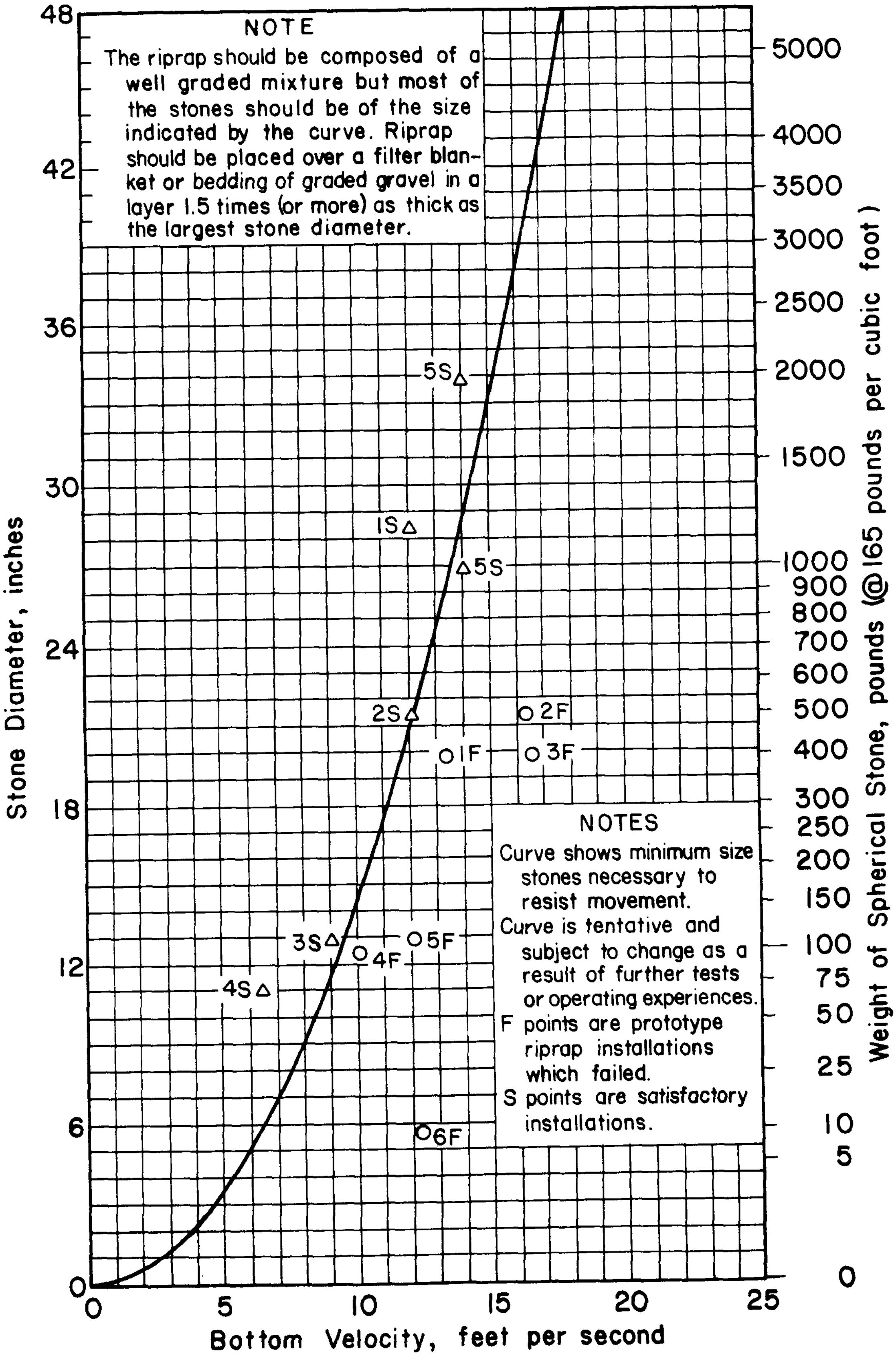


Fig. 4.11. Parametric curve used to determine maximum stone size in riprap mixture as a function of channel flow velocity.

Furthermore, Powledge and Dodge indicated that with slowly rising or low constant flow, erosion may start and remain at one low point causing gully-type erosion.

It is recommended that the slope transition areas, particularly along the embankment crest, be protected (i.e., riprap, rock mulch, etc.) at least 8-10 feet up-grade and down-grade of the slope break. The slope transitional area is vulnerable to sheet and concentrated flows. Design discharges will often transition from subcritical to critical or supercritical flows resulting in a high potential for erosion. The recommended protection will provide an armoring that will help resist degradation, particularly from unexpected, concentrated flows.

4.4 FILTER CRITERIA

It is recommended that a layer or blanket of well-graded rock material be placed over the embankment or cover slope prior to riprap placement. Sizes of gravel in the filter blanket should be from 3/16 inch to an upper limit of approximately 3 to 3 1/2 inches, depending on the gradation of the riprap. The filter thickness shall vary depending upon the riprap thickness and riprap design procedure, but should not be less than 6 to 9 inches. Filter thicknesses one-half the riprap layer thickness are recommended. Suggested specifications for gradation of the filters are as follows:

$$\frac{D_{15}\ (\text{Filter})}{D_{85}\ (\text{Base})} < 5 \tag{4.35}$$

and

$$\frac{D_{15}\ (\text{Filter})}{D_{85}\ (\text{Base})} < 10 \tag{4.36}$$

The criteria presented in Equation 4.35 will prevent migration of the bedding into the riprap. When the filter meets the criteria as specified in Equation 4.36, erosion of the radon barrier below the bedding shall be prevented (Sherard et al., 1984).

4.5 FLOW THROUGH RIPRAP ROCKFILL

When a riprap layer is used to stabilize a sloped cover, it is advantageous to determine the discharge through the rockfill. The analysis of flow through a riprap rockfill is complex and does not comply with Darcy's

Law except at extremely low gradients. The following design guideline for estimating flow through riprap rockfill closely conforms to the laws of turbulent flow.

Flow through granular material is dependent on the geometry, structure and flow properties of the porous media. Leps (1973) presented a basic equation for turbulent flow through rockfill as

$$V_v = Wm^{0.5} i^{\ 0.54} \tag{4.37}$$

where V_v is the average velocity of water (inches/sec) in the voids of the rockfill, W is an empirical constant for a specific riprap material, m is the hydraulic mean radius and i is the hydraulic gradient. Table 4.1 presents a series of empirically derived values for the hydraulic mean radius, m, and the $Wm^{0.5}$ parameter as presented by Leps.

Table 4.1. Empirically derived values for equation 4.37.

Rock size (inch)	m (inch)	$m^{0.5}$ (inch $^{1/2}$)	$Wm^{0.5}$ (inch/sec.)
3/4	0.09	0.30	10
2	0.24	0.49	16
6	0.75	0.87	28
8	0.96	0.98	32
24	3.11	1.76	58
48	6.43	2.54	84

Source: Leps, 1973.

The hydraulic gradient will range from 0 to 1.0. The dominant rock size for flow calculations was considered to be the 50% size, d_{50}. Although Equation 4.37 was derived for a uniformly graded rockfill, the procedure is considered applicable to well graded rockfill provided that the material smaller than one inch is less than 30%. Leps indicated that if more than 30% of the less than one inch material is present, the rockfill should be treated as earthfill. A series of tests were conducted at Colorado State University by Abt and Ruff (1985) and Equation 4.37 was found to be accurate within ± 15%.

The unit discharge, q, per foot of width can be estimated as

$$q = \frac{V_v}{12}\ d_r \tag{4.38}$$

where d_r is the rock thickness in feet and q is expressed in cubic feet per second (cfs).

4.6 HYDRAULIC COMPUTATIONS

In order to appropriately analyze general flow conditions for overland or sheet flows and open channel flows, it is generally recommended that the Manning formula be used. The Manning formula was developed for steady, uniform, incompressible flow and can be applied to a variety of field situations and conditions. The Manning formula is expressed empirically as

$$V = \frac{1.486}{n} R^{2/3}\ S^{1/2} \tag{4.39}$$

where V is the average velocity at a specified cross section, R is the hydraulic radius, S is the slope of the channel bottom or loss per unit length of channel, and n is a surface roughness coefficient. Representative values of Manning coefficients are presented in Section 4.7. To determine the discharge, Q, Equation 4.39 can be modified to

$$Q = \frac{1.486}{n} AR^{2/3}\ S^{1/2} \tag{4.40}$$

where A is the cross sectional area of flow.

When the area of flow is limited to unit width, the unit discharge, q, can be determined. A unit discharge approach is often used for application to sheet or overland flows.

Although the Manning formula is a simplistic procedure for computing the normal depth of flow, it yields a good estimate of the discharge and/or velocity of flow for channels of constant slope and unvarying cross-section. However, alternative procedures such as the Chezy formula or other sophisticated numerical models may also be used.

An alternative procedure for estimating flow velocity is the HEC-2 computer program. The HEC-2 computer program was developed for calculating water surface profiles for steady, gradually varying flow in natural or man-made channels (COE, 1982). Both subcritical and supercritical flow profiles can be estimated. The computational procedure is based on the solution of the one-dimensional energy equation with energy losses due to friction. The HEC-2 procedure is similar to the Standard Step Method for computing water surface elevations. The program was developed for flood plain management, floodway encroachment evaluation and flood hazard designations.

4.7 DETERMINATION OF THE MANNING ROUGHNESS COEFFICIENT

The greatest difficulty in applying the Manning formula and other flow models such as HEC-2 is the determination of the boundary roughness coefficient, n. The n value is an estimate of flow resistance. There is not an exact procedure or method for determination of flow resistance. It is imperative to recognize that the selection of an appropriate n value requires careful judgment and reason.

The n values commonly available were formulated for flows in natural and artificial channels. Factors affecting Manning's roughness coefficient include surface roughness, vegetation, channel irregularity, channel alignment, flow depth, silting and scouring, obstructions and channel shape. Chow (1959) and Barnes (1967) present a comprehensive list of n values for open channel applications. Values of n range from 0.017 for smooth channels free from growth to 0.07 for cobble bed streams (Chow, 1959). Equations 4.39 and 4.40 are extremely sensitive to the n value. Therefore the selection of an appropriate n value may require several iterations.

The Manning formula is commonly used to estimate discharge for overland flow, particularly over large areas in which runoff channelization has not yet initiated. Overland or sheet flow is characterized by a flow depth less than 1.0 ft. and is significantly influenced by the boundary shear or resistance to flow. The n value may vary with flow depth.

Morris and Wiggert (1972) published a list of n values that have been adopted by the U.S. Bureau of Reclamation and are presented in Table 4.2. These values apply to well-seasoned, straight channels on mild slopes with flow depths less than 3.0 ft.

A series of values for the Manning Coefficient, n, were adopted by the Department of the Interior (DOI, 1975) for natural channels and streams. These values are presented in Table 4.3.

One of the most difficult Manning's roughness values to determine is for riprap. Riprap serves as an alternative surface stability technique that provides considerable resistance to flow resulting in velocity and energy dissipation. An expression for determining the value of the Manning coefficient, n, for riprap was presented by the Corps of Engineers (COE, 1970) and by Anderson et al. (1970) is:

$$n = 0.0395\ (d_{50})^{1/6} \qquad (4.41)$$

where d_{50} is the mean rock size in feet. A graphical representation for determining n is presented in Figures 4.12 and 4.13. However, these values were developed for uniform flow condition over submerged riprap. When overtopping flows on steep slopes begin to cascade, n values will increase and may range from 0.07 to 0.09 or higher. (Abt and Ruff, 1985 and COE, 1970).

Table 4.2. Manning Coefficient, n.

Channel Material	Manning Coefficient, n
Fine sand, colloidal	0.020
Sandy loam, non-colloidal	0.020
Silt loam, non-colloidal	0.020
Alluvial silts, non-colloidal	0.020
Ordinary firm loam	0.020
Volcanic ash	0.020
Stiff clay, very colloidal	0.025
Alluvial silts, colloidal	0.025
Shales and hardpans	0.025
Fine gravel	0.020
Graded loam to cobbles, non-colloidal	0.030
Graded silts to cobbles, colloidal	0.030
Coarse gravel, non-colloidal	0.025
Cobbles and shingles	0.035

Source: Morris and Wiggert, 1972.

4.8 COVER EROSION RESISTANCE EVALUATION

The cover design should be evaluated to determine if the unprotected slopes(s) can withstand overland or sheet flow with a minimum of erosion. Based upon the site-specific cover and precipitation parameters, the design sheet flow velocity should be estimated. A comparison of the design flow velocity with the cover permissible flow velocity can be performed. Furthermore, the design velocity can be used to determine the sediment discharge using the Universal Soil Loss Equation (Chapter 5) and for sizing stone protection (Section 4.2).

The design velocity will usually be determined from the peak discharge generated from the Probable Maximum Flood (PMF). The PMF can be estimated by

(a) Using computer models, i.e., HEC-1 (COE, 1974), that are widely accepted by the engineering profession.

Table 4.3. Manning Coefficient, n, for natural channels.

Natural Channel Conditions	Value of n
Smoothest natural earth channels, free from growth with straight alignment	0.017
Smooth natural earth channels, free from growth, little curvature	0.020
Average, well-constructed, moderate-sized earth channels in good condition	0.0225
Small earth channels in good condition, or large earth channels with some growth on banks or scattered cobbles in bed	0.025
Earth channels with considerable growth, natural streams with good alignment and fairly constant section, or large floodway channels well maintained	0.030
Earth channels considerably covered with small growth, or cleared but not continuously maintained floodways	0.035
Mountain streams in clean loose cobbles, rivers with variabel cross-section and some vegetation growing in banks, or earth channels with thick aquatic growths	0.050
Rivers with fairly straight alignment and cross-section, badly obstructed by small trees and underbrush or aquatic growth	0.075
Rivers with irregular alignment and cross-section, moderately obstructed by small trees and underbrush	0.100
Rivers with fairly regular alignment and cross-section, heavily obstructed by small trees and underbrush	0.100
Rivers with irregular alignment and cross-section, covered with growth of virgin timber and occasional dense patches of bushes and small trees, some logs and dead fallen trees	0.125
Rivers with very irregular alignment and cross-section, many roots, trees, large logs, and other drift on bottom, trees continually falling into channel due to bank caving	0.200

Source: DOI, 1975.

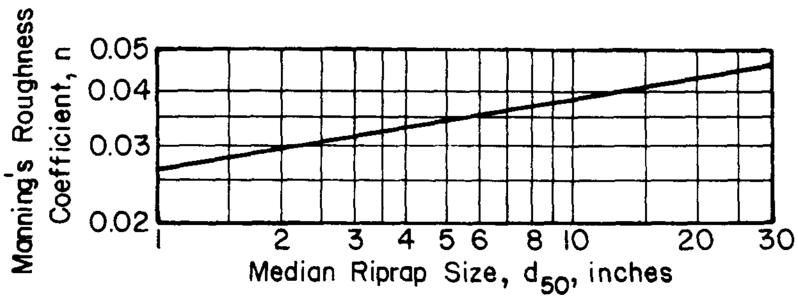


Fig. 4.12. Manning's coefficient for riprap. *Source:* SCS, 1975.

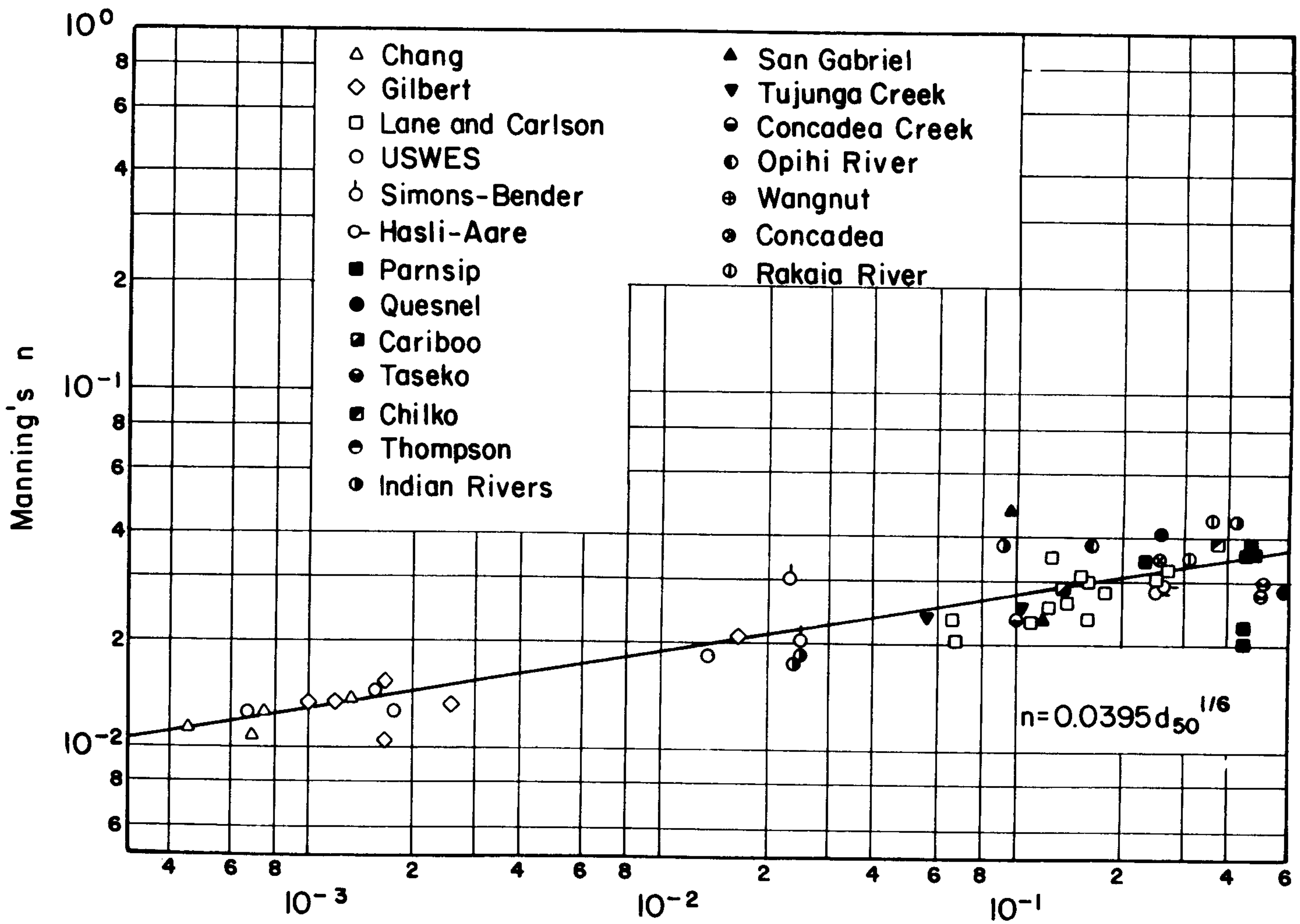


Fig. 4.13. Variation of Manning's *n* with size of stone comprising the bed.

(b) Applying the Rational Method for tributary areas that are less than approximately one square mile in area.

The Rational formula is commonly expressed as

$$Q = CiA \qquad (4.42)$$

where Q is the maximum or design discharge in cfs, C is a runoff coefficient dependent upon the characterization of the drainage basin, i is the rainfall intensity expressed in inches per hour and A is the tributary area expressed in acres. When a unit width approach is taken, the area A_W is the slope(s) length times the unit width. Therefore, Equation 4.42 would be presented as

$$q = CiA_W \qquad (4.43)$$

for a unit width analysis.

4.8.1 Runoff Coefficient

The runoff coefficient, C, is related to the climatic conditions and type of terrain characteristic of the watershed including soil materials, permeability and storage potential. Values of the coefficient C are presented in Table 4.4 (Lindsley et al., 1958), Table 4.5 (Chow, 1964), and Table 4.6 (ASCE, 1970 and Seelye, 1960).

Table 4.4. Values of Coefficient C.

Type Area	Value of C
Flat cultivated land, open sandy soil	0.20
Rolling cultivated land, clay-loam soil	0.50
Hill land, forested, clay loam soil	0.50
Steep, impervious slope	0.95

Source: Lindsley, et al, 1958.

The selection of a coefficient value requires considerable judgment as it is a tangible aspect of using the rational formula. It is recommended that a conservative value of C be applied for PMF estimation since infiltration and storage comprise a low percentage of the runoff. Furthermore, the C values presented were derived for storms of 5-100 year frequencies. Therefore, less frequent, higher intensity storms will require the use of a higher C value (Chow, 1964). It is recommended that a runoff coefficient of 1.0 be used for PMF applications in very small watersheds since the effects of localized storage and infiltration will be small.

Table 4.5. Values of C for Use in Rational Formula.

Soil Type	Watershed Cover		
	Cultivated	Pasture	Woodlands
With above-average infiltration rates; usually sandy or gravelly	0.20	0.15	0.10
With average infiltration rates; no clay pans; loams and similar soils	0.40	0.35	0.30
With below-average infiltration rates; heavy clay soils or soils with a clay pan near the surface; shallow soils above impervious rock	0.50	0.45	0.40

Source: Chow, 1964.

4.8.2 Rainfall Intensity

In order to determine the rainfall intensity, i, the time of concentration, t_c must be estimated. The time of concentration can be approximated by:

(a) Applying one of the many accepted empirical formulae such as

$$t_c = 0.00013 \frac{L^{0.77}}{S^{0.385}} \qquad (4.44)$$

where L is the length of the basin in feet measured along the watercourse from the upper end of the watercourse to the drainage basin outlet and S is the average slope of the basin. Time of concentration is expressed in hours. This procedure is not applicable to rock covered slopes. This expression was

Table 4.6. Values of runoff coefficient C.

Character of Surface	Runoff Coefficients Range	Runoff Coefficients Recommended
Pavement--asphalt or concrete	0.70-0.95	0.90
Gravel, from clean and loose to clayey and compact	0.25-0.70	0.50
Roofs	0.70-0.95	0.90
Lawns (irrigated) sandy soil		
Flat, 2 percent	0.05-0.15	0.10
Average, 2 to 7 percent	0.15-0.20	0.17
Steep, 7 percent or more	0.20-0.30	0.25
Lawns (irrigated) heavy soil		
Flat, 2 percent	0.13-0.17	0.15
Average, 2 to 7 percent	0.18-0.22	0.20
Steep, 7 percent	0.25-0.35	0.30
Pasture and non-irrigated lawns		
Sand		
Bare	0.15-0.50	0.30
Light vegetation	0.10-0.40	0.25
Loam		
Bare	0.20-0.60	0.40
Light vegetation	0.10-0.45	0.30
Clay		
Bare	0.30-0.75	0.50
Light vegetation	0.20-0.60	0.40
Composite areas		
Urban		
Single-family, 4-6 units/acre	0.25-0.50	0.40
Multi-family, >6 units/acre	0.50-0.75	0.60
Rural (mostly non-irrigated lawn area)		
<1/2 acre - 1 acre	0.20-0.50	0.35
1 acre - 3 acres	0.15-0.50	0.30
Industrial		
Light	0.50-0.80	0.65
Heavy	0.60-0.90	0.75
Business		
Downtown	0.70-0.95	0.85
Neighborhood	0.50-0.70	0.60
Parks	0.10-0.40	0.20

Source: ASCE, 1970 and Seelye, 1960.

designed for and applicable to small drainage basins (Kirpich, 1940).

(b) Using the Soil Conservation Service (SCS) Triangular Hydrograph Theory (DOI, 1977), the time of concentration is

$$t_c = \frac{11.9\ L^3}{H}\left(0.385\right) \tag{4.45}$$

where L is the length (miles) of the longest watercourse from the point of interest to the tributary divide, H is the difference in elevation (feet) between the point of interest and the tributary divide. The time of concentration will be expressed in hours. The SCS procedure is most applicable to drainage basins of at least 10 square miles.

Once the rainfall duration or time of concentration is determined, the rainfall depth can be computed based on the PMP intensity values estimated in Section 2.1.2.

4.8.3 Tributary Area

The tributary area may be expressed in a unit width format for design of rock protection on an embankment. Therefore, the area is the length of the longest expected or measured water course multiplied by the unit width. This procedure is primarily applicable to Zones I, II, and III and is not applicable for drainage ditch design. It should be noted that a unit width approach to drainage and diversion ditch design is not effective. Ditch design requires an entire basin analysis in which a composite inflow hydrograph is determined and is routed along the channel. From the inflow hydrograph, water surface profiles (i.e., HEC-2) can be estimated to determine flow depth and velocities for riprap design (COE, 1982).

4.8.4 Sheet Flow Velocity

The design velocity for sheet flow on an embankment slope can be estimated by solving the Manning formula presented in Equation 4.39. It is assumed that the hydraulic radius, R, is approximately equal to the flow depth, y, and that the design discharge is equal to that estimated by the Rational Method. Therefore, the depth of flow is

$$y = \left[\frac{Qn}{1.486\ S^{1/2}}\right]^{3/5} \tag{4.46}$$

where Q is the discharge, S is the slope, and n is the Manning coefficient.

Therefore, the design velocity can be estimated as

$$V_{Design} = Q/A \text{ (feet/sec)} \tag{4.47}$$

where A is the cross-sectional area of flow.

4.9 FLOW CONCENTRATIONS

Despite the extensive efforts of the impoundment reclamation designer, reviewer, contractor and inspector, the topographic features of the cover will alter over time without continual maintenance (Powledge and Dodge, 1985). Cover modifications will result from differential settlement, collapsing soils, marginal quality control in cover placement, erosion, major hydrologic events and monitoring disturbance. Because of these unpredictable and generally uncontrollable events, tributary drainage areas evolve that were not originally designed or constructed. The result is that the peak discharge and volume of runoff exceed design levels and increase the erosion potential.

Abt and Ruff (1985) conducted a series of flume experiments on a 1V:5H prototype embankment protected by riprap with median rock sizes of 2 inches to 6 inches in diameter. It was observd that 2-4 inch diameter riprap were highly susceptible to sheet flows converging along the face of the embankment into channels. The discharge in the channel(s) was compared to the total discharge over the embankment by

$$CF = \frac{1}{1 - (Q_c - Q)} \tag{4.48}$$

where CF is the concentration factor, Q_c is the discharge in the channel and Q is the total discharge over the embankment. The concentration factors ranged from 1.1 to 3.2 where flows were less than the failure discharge. These preliminary results indicate that riprap designed for sheet flow conditions may be subjected to flow channelizations that concentrate 3 times the discharge in a single location.

The peak discharge along a crest or at a design point is a function of the amount of precipitation, the tributary drainage area, the slope of the drainage basin, the basin contouring, the cover material and cover protection. Any modification in one or more of these parameters can impact the outlet peak discharge. The cover design must account for these potential changes in the form of a concentration or safety factor. Therefore, a flow concentration factor may be incorporated into the design process to adequately evaluate the soil resistance to erosion, to adequately select and evaluate alternative protective measures and to size riprap when warranted.

It is difficult to accurately predict the value of the flow concentration factor since limited information is currently available to substantiate design limits. However, it is reasonable to assume that values between 2 and 3 are attainable with only a slight evolutionary change in cover. Unless it can be shown that design procedures such as overbuilding can compensate for differential settlement, it is recommended that a conservative concentration factor be used until additional research can justify a more reasonable range of values.

To incorporate the flow concentration factor into the stone sizing procedure of any riprap design method, multiply the design peak discharge by the flow concentration factor. All subsequent computations, i.e., velocity and depth estimate, stone size determination, etc., will reflect the influence of the flow concentration.

4.10 PERMISSIBLE VELOCITIES

Evaluation of proposed reclamation alternatives should include an analysis of the critical erosion potential of the cover material. Erosion potential can be determined based upon the properties of the reclamation materials as well as the degree of compaction in which the material is placed. The permissible velocity approach consists of specifying a velocity criterion that will not erode the cover or channel and will prevent scour. A comparison of the actual or design flow velocities to the permissible velocities associated with overland flows, sheetflows or channel flows determines the erosion potential. When the design flow velocity meets or exceeds the permissible velocity, cover protection should be considered.

The permissible velocity values presented were developed from experiments performed primarily in canals and stream beds. Therefore, the following permissible velocities should provide a conservative estimate for evaluating the erosion resistance of the reclaimed covers over long term periods. In cases where a range of permissible velocities are presented, it is recommended that the lower velocity be used for determining erosion potential.

A series of permissible maximum canal velocities was developed by Fortier and Scobey (1926) and adapted by Lane (1955). The maximum permissible velocities presented in Table 4.7 are applicable to colloidal silts. These velocity values were developed for channels without sinuosity. Lane recommended a reduction of the velocities in Table 4.7 by 13 percent if the canal/channel is moderately sinuous. The maximum allowable velocities for sandy-based materials are given in Table 4.8. Table 4.9 provides limiting velocities for cohesive materials according to compactness for materials with less than 50 percent sand content. The Soil Conservation Service maximum permissible velocities (SCS, 1984) for well maintained grass covers are presented in Table 4.10.

It is important to recognize that limited information is available pertaining to permissible velocities on covers under sheet flow conditions.

Table 4.7. Maximum permissible velocities in erodible channels.

Channel Material	Water Transporting Colloidal Silts v (ft/sec)
Fine sand, colloidal	2.50
Sandy loam, non-colloidal	2.50
Silty loam, non-colloidal	3.00
Alluvial silts, non-colloidal	3.50
Firm loam	3.50
Volcanic ash	3.50
Stiff clay, colloidal	5.00
Alluvial silts, colloidal	5.00
Shales and hardpans	6.00
Fine gravel	5.00
Graded loam to cobbles, non-colloidal	5.00
Graded silts to cobble, colloidal	5.50
Coarse gravel, non-colloidal	6.00
Cobbles and shingles	5.50

Source: Lane 1955.

Table 4.8. Maximum allowable velocities in sand-based material.

Material	Velocity (ft/sec)
Very light sand of quicksand character	0.75 to 1.00
Very light loose sand	1.00 to 1.50
Coarse sand to light sandy soil	1.50 to 2.00
Sandy soil	2.00 to 2.50
Sandy loam	2.50 to 2.75
Average loam, alluvial soil, volcanic ash	2.75 to 3.00
Firm loam, clay loam	3.00 to 3.75
Stiff clay soil, gravel soil	4.00 to 5.00
Coarse gravel, cobbles and shingles	5.00 to 6.00
Conglomerate, cemented gravel, soft slate, tough hardpan, soft sedimentary rock	6.00 to 8.00

Source: Lane, 1955.

Therefore, the permissible velocities developed for channels is usually extended to overland flow situations. When design velocities reach or exceed those indicated in Tables 4.7 through 4.10, protection is warranted.

Table 4.9. Limiting Velocities in Cohesive Materials.

	Compactness of Bed			
	Loose	Fairly Compact	Compact	Very Compact
Principle Cohesive Material	Velocity (ft/sec)	Velocity (ft/sec)	Velocity (ft/sec)	Velocity (ft/sec)
Sandy clay	1.48	2.95	4.26	5.90
Heavy clayey soils	1.31	2.79	4.10	5.58
Clays	1.15	2.62	3.94	5.41
Lean clayey soils	1.05	2.30	3.44	4.43

Source: Lane, 1955.

The materials presented in Tables 4.7 through 4.9 can be referenced to the Unified Soil Classification System as presented by Wagner (1957). An engineering analysis of the cover material can provide an approximation of the permissible velocities that the alternative cover materials may withstand without supplemental protection.

4.11 PERMISSIBLE VELOCITY EXAMPLE

A tailings disposal site located in the northwest corner of New Mexico has prepared a reclamation plan for review. The reclamation plan indicates that a 10 foot thick cap will be placed atop the tailings at a slope of 2.4% with a compaction of 95% of optimum. The cap will be graded as shown in Figure 4.14 and shall transition into side slopes of 1V:10H. It is proposed that the cap will be composed of a sandy clay with a coarse gravel cover. Along the crest, a 12 inch thick layer of riprap will be placed for at least 8 feet upslope and downslope of the crest to stabilize the transition. The riprap will have a median stone size of 6 inches. The gravel cover will have a median rock size of 1.5 inches. The design reviewer must verify that the gravel cover will resist the potential velocities that may result on the cap.

Table 4.10. Maximum Permissible Velocities in Feet per Second (fps) for Channels Lined With Uniform Stands of Various Well-Maintained Grass Covers.

Cover	Maximum Permissible Velocities[a]		
	Slope Range %	Erosion-Resistant Soils	Easily-Eroded Soils
Bermudagrass	0-5	8	6
	5-10	7	5
	Over 10	6	4
Buffalograss	0-5	7	5
Kentucky bluegrass	5-10	6	4
Smooth brome	Over 10	5	3
Blue grama[b]	0-5	5	4
Grass mixture[b]	5-10	4	3
Lespedeza sericea			
Weeping lovegrass			
Yellow bluestem[c]	0-5	3.5	2.5
Kudzu			
Alfalfa			
Crabgrass			
Common lespedeza[c,d]	0-5	3.5	2.5
Sudangrass[d]			

[a]Use velocities over 5 fps only where good covers and proper maintenance can be obtained.

[b]Do not use on slopes steeper than 10 percent.

[c]Use on slopes steeper than 5 percent is not recommended.

[d]Annuals are used on mild slopes or as temporary protection until permanent covers are established.

Source: SCS, 1984.

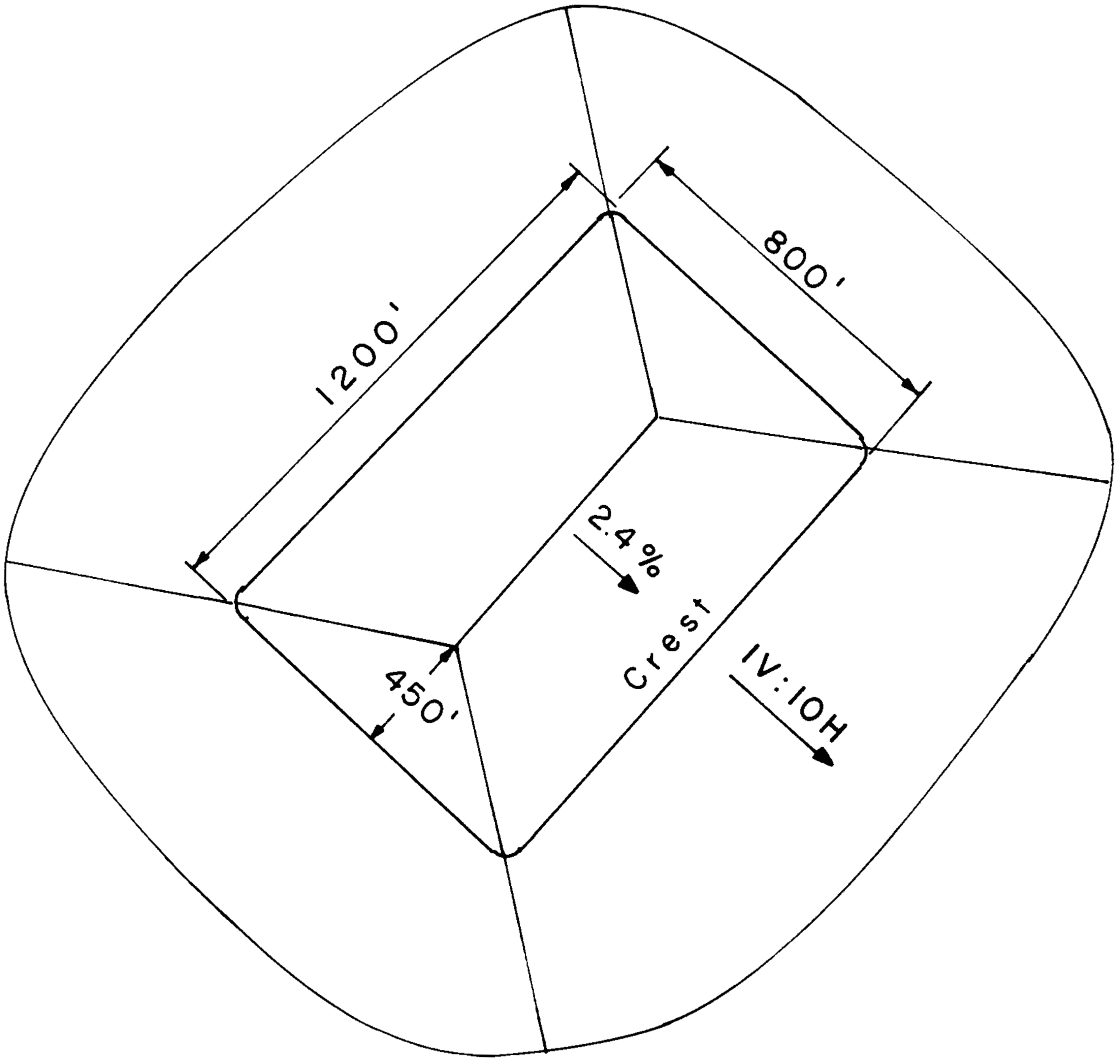


Fig. 4.14. Representative reclaimed tailing pile—example problem.

In order to assess the stabilization of the cap against erosion due to overland flow, information provided in Sections 4.6 through 4.10 of this report must be utilized. One alternative means of reviewing the design is presented in the following analysis.

4.11.1 Estimation of Peak Runoff

The peak runoff can be estimated using the Rational formula presented in Equation 4.43. The three components of the Rational formula that require consideration are: the runoff coefficient, C; the rainfall intensity, i; and the tributary area, A.

The runoff coefficient can be estimated by examining Tables 4.4 through 4.6. Since the cap will be composed of a compacted clay, the infiltration and localized storage will be low. The peak runoff is a direct function of the estimated localized PMF. Therefore, a reasonable C value is 1.0.

The rainfall intensity can be estimated by determining the 1-hr, 1-mi^2 local storm PMP value and adjusting the rainfall depth in accordance with the percentages presented in Table 2.1. For northwest New Mexico, the 1-hr, 1-mi^2 PMP is estimated to be 9.5 inches after the appropriate elevation and area adjustments are performed.

The time of concentration, t_c, should be estimated. Using Equation 4.44, the t_c can be estimated where the longest flow path is approximately 450 feet as

$$t_c = 0.00013 \frac{(450)^{0.77}}{(0.024)^{0.385}} \tag{4.49}$$

and

$$t_c = 0.06 \text{ hrs} = 3.62 \text{ minutes} \tag{4.50}$$

The rainfall depth for variable rainfall durations can be estimated using the values presented in Table 2.1 which are applicable to northwest New Mexico. Since the time of concentration is 3.6 minutes, the percent of the 1-hr PMP can be interpolated to be approximately 35 percent. The rainfall depth is computed using Equation 2.1 to be

$$\text{Rainfall depth} = (0.35) \times 9.5 \text{ inch} = 3.33 \text{ inches} \tag{4.51}$$

A conservative estimate of the rainfall intensity is determined by applying Equation 2.2.

$$i = 3.33 \text{ inches} \times \frac{60}{3.6} = 55.5 \text{ inches/hr} \qquad (4.52)$$

The tributary area, A, can be estimated using a unit width approach presented in Section 4.8. Since the longest flow path is 450 feet with a unit width of one foot, the tributary area is 450 square feet. The tributary area can be converted to acres by dividing by 43,560 square feet/acre resulting in an area of 0.0103 acres.

The peak sheet flow unit discharge at the transition can be computed by using the Rational formula presented in Equation 4.43.

$$q = (1.0)\ (55.5)\ (0.0103) = 0.57 \text{ cfs} \qquad (4.53)$$

4.11.2 Sheet Flow Velocity

The sheet flow design velocity can be estimated by first determining the depth of flow. The depth of flow, y, can be calculated using Equation 4.46. However, the Manning surface roughness coefficient, n, must be determined. From Equation 4.41, the Manning n value can be calculated as

$$n = 0.0395\ (0.125)^{1/6} = 0.028 \qquad (4.54)$$

The depth of flow is then computed to be

$$y = \left[\frac{(0.57)\ 0.028}{1.486\ (0.024)^{1/2}}\right]^{3/5} = 0.202 \text{ feet} \qquad (4.55)$$

or

$$y = (0.202 \text{ ft})\ (12 \text{ in/ft}) = 2.42 \text{ inches} \qquad (4.56)$$

The design sheet flow velocity is calculated using Equation 4.47.

$$V = \frac{0.57}{(1.0)(0.20)} = 2.82 \text{ feet/sec} \qquad (4.57)$$

where 0.57 is the unit discharge, 1.0 is the width of flow in feet and 0.20 is the depth of flow in feet. It should be noted that the flow concentration factor was not incorporated into this computation.

4.11.3 Cover Permissible Velocity

The permissible velocity for the clay cap covered with gravel has been determined to be 5.0-6.0 feet/sec as presented in Table 4.8. Since the design sheet flow velocity was calculated to be 2.9 feet/sec, the cover should be able to withstand the design flow.

5. EVALUATING THE POTENTIAL FOR SURFACE SHEET EROSION

Due to their fine-grained noncohesive nature, uranium mill tailings have a high potential for sheet erosion when subjected to the forces of wind and water. Sheet erosion is defined and limited to that erosion which occurs as a result of: (1) the impact of raindrops striking the ground surface, (2) the lifting and transporting of material due to wind forces, or (3) the transporting of material due to water flowing in small ephemeral rills. Any of these forces acting individually, or in combination with each other, can detach and transport significant quantities of material from an area. The potential for sheet erosion and subsequent transportation of the eroded tailings away from the impoundment area are the principal concerns dictating the need for sound engineering design and proper construction of a stable cover material over abandoned tailings. This chapter presents a discussion of the analytical techniques that will be used in a followup study to evaluate sheet erosion and a recommended approach to estimate the life of different protective covers.

One of the existing methods used to estimate soil loss due to sheet erosion is based on experimental observation of soil erosion occurring primarily on agricultural lands. This observational approach lacks the mathematically rigorous concept of, for example, methods involving a tractive shear force. Nevertheless, it is believed that the extended period of time over which the observational approach of sheet erosion has been developed and used lends credibility to its consideration in evaluating protective covers for uranium tailings.

Historically, sheet erosion and gully erosion have not been viewed or treated separately. Indeed, the natural forces associated with gully and sheet erosion are similar; however, it is now recognized that the damage associated with gully erosion is potentially much greater than that of sheet erosion. Given the same potentially erosive material, both erosion phenomena are possible but usually only one will dominate the erosion process primarily as a function of ground slope and the frequency and intensity of rainfall. It is important, therefore, to put in proper perspective these different types of erosion phenomena and to treat them separately in the stabilization design process.

5.1 METHODOLOGIES

Soil particles can become detached when the impact of rainfall and/or the forces caused by wind or flowing water exceed the combination of factors that contribute to soil cohesion or stability. Factors which tend to stabilize the soil or resist such erosive forces include natural vegetation (ground cover) and protective rock covers. The design of any protective soil cover over uranium tailings must consider the soil particle detachment process and the erosion potential over the entire reclamation period.

Two basic approaches exist for the design of suitable erosion-resistant covers for a tailings impoundment surface as originally described by Nelson et al. (1983). The first approach consists of providing a cover material that will resist material transport by flowing water using the concept of critical shear stress. The second approach is based on the Universal Soil Loss Equation, an empirical method originally developed during the 1930's. The methodologies involved with both of these methods are discussed below.

5.1.1 Critical Shear Stress Approach

The critical shear stress approach consists of providing a cover material with a d_{30} grain size (i.e., 70% of the material by weight is coarser than the d_{30}) that will resist movement when subjected to the sheet flow maximum permissible velocity resulting from the application of the PMP over the entire impoundment surface. Minimum d_{30} grain sizes should be determined using the critical shear stress approach similar to the procedures discussed in Simons and Senturk (1977) applicable to over-land flow. A numerical solution for selecting an appropriate d_{30} to provide armoring has been developed by Shen and Lu (1983).

The design approach described above, in which the critical grain size is selected to resist the onset of sheet erosion, should evaluate the run-off from PMP storms of different durations, such as 0.5, 1, 2, 4, and 6 hours to select the maximum d_{30} required. Rainfall depths will usually be based on 2.5 to 15 minute durations for small drainage basins as presented in Section 2.1.2. Typically, the minimum construction layer thickness is specified to be at least two times the maximum particle size. If the above approach results in a cover thickness less than about 6 inches, then other considerations - such as nonuniform placement of cover and particle breakdown due to handling, placement and weathering - would suggest that a minimum cover thickness of 10 inches should be considered. If a self-armoring cover can be provided, and there is no major concern for weathering of the cover material, the design is independent of time and the cover should remain intact indefinitely.

5.1.2 Soil Loss Equation Approach

The concept of sheet erosion was recognized by early researchers and the Universal Soil Loss Equation (USLE) was developed in the late 1930's by the Agricultural Research Service to evaluate soil conservation practices for cropland throughout the United States. After its inception, the soil loss procedure was used and modified as field experience and data were obtained incorporating the basic parameters of field slope and length, precipitation, and crop management to estimate soil losses on an annual basis. Application of the USLE to non-cropland areas and specifically for construction sites became feasible when Wischmeier et al. (1971), using basic soil loss characteristics, developed and implemented a soil erodibility factor (K) in the soil loss computation. Subsequent efforts refined the parameters used in the USLE for mining and construction activities in the interior western United States.

The Modified Universal Soil Loss Equation (MUSLE) was developed by the Utah Water Research Laboratory in 1978 for the principal objective of estimating soil losses due to highway construction activities. Alterations were made to the USLE to accomodate unique or special conditions encountered in highway construction, including steep and deep cuts and fill slopes that could cause erosion affecting adjacent or nearby roadways, streams, lakes, or inhabited areas. It is apparent that the modifications made to the USLE extend to many construction and mining sites beyond the scope of highway construction.

The Modified Universal Soil Loss Equation (MUSLE) is a mathematical model based on field determined coefficients and provides the most rational approach to evaluate the long-term erosion potential from an upland area similar to that of the area covering a reclaimed tailings pond. Recent investigations into appropriate methods of modeling major types of sheet erosion (Abt and Ruff, 1978; Nelson et al. 1983; Nyhan and Lane, 1983; and NRC, 1983), indicate that although more rigorous mathematical models are available to simulate erosion as a function of time, the use of the USLE has a strong precedent because it has a 40-year history of runoff and soil loss data.

The MUSLE is used to evaluate average soil losses for certain types of slopes as a function of time. The MUSLE does not consider the potential for gully development or intrusion as discussed in Chapter 4 because the topographic features of the tailings area are assumed to remain constant with time. Also, the MUSLE does not incorporate the concept of the PMP but rather a rainfall factor based on historical rainfall values. The MUSLE is defined by Clyde et al. (1978) as follows:

$$A = R\ K\ (LS)\ (VM) \tag{5.1}$$

where,

A = the computed loss per unit area in tons per acre per year with the units selected for K and R properly selected;

R = the rainfall factor which is the number for rainfall erosion index units plus a factor for snowmelt, if applicable;

K = the soil erodibility factor, which is the soil loss rate per erosion index unit for a specified soil as measured on a unit plot that is defined as a 72.6-ft length of uniform 9% slope continuously maintained as clean tilled fallow;

LS = the topographic factor, which is the ratio of soil loss from the field slope length to that from a 72.6-ft length under otherwise identical conditions;

VM = the dimensionless erosion control factor relating to vegetative and mechanical factors. This factor replaces the cover management factor (C) and the support factor (P) of the original USLE.

5.1.2.1 The Rainfall and Runoff Factor (R)

As noted by previous research at Los Alamos National Laboratory (Nyhan and Lane, 1983), the R factor as used in the MUSLE is often misinterpreted only as a rainfall factor. In reality, it must quantify both the raindrop impact and provide information on the amount and rate of runoff likely to be associated with the rain. More specifically, the R factor is described in terms of a rainfall storm energy (E) and the maximum 30-minute rainfall intensity (I_{30}). Generalized R factors applicable to the interior western United States are given in Table 5.1. For R factors in specific areas of the United States, it is recommended that erosion index distribution curves be obtained from local SCS offices.

Table 5.1. Generalized Rainfall and Runoff (R) Values.

State	Eastern Third	Central Third	Western Third
N. Dakota	50 - 75	40 - 50	40
S. Dakota	75 - 100	50	40
Montana	30 - 40	20	20 - 50
Wyoming	30 - 50	15 - 30	15 - 25
Colorado	75 - 100	40 - 50	20 - 40
Utah	20 - 30	20 - 50	15 - 40
New Mexico	75 - 100	40 - 50	20 - 40
Arizona	20 - 50	20 - 50	25 - 40

5.1.2.2 The Soil Erodibility Factor (K)

The soil erodibility factor (K) recognized the fact that the erodibility potential of a given soil is dependent on its compositional makeup, which in turn reflects the grain size distribution of the soil. To predict soil erodibility, five soil characteristics that include the percent silt and fine sand, percent sand greater than 0.1 mm, percent organic material, general soil structure and general permeability are determined. The K factor is then found by using the Wischmeier nomograph presented in Figure 5.1.

The makeup of the various soil fractions presented in Figure 5.1 is based on separating sand and silt at the 0.1 mm size. This differs from the Unified Soil Classification System which uses the No. 200 sieve size (0.075 mm) for the separation between sand and silt. The value to enter Figure 5.1 with should be the percentage of material finer than 0.1 mm in size, not the percentage passing the No. 200 sieve. Also, the determination of the Soil Erodibility Factor (K) as shown on Figure 5.1 does not specifically reference the percentage of clay (finer than 0.002 mm) contained in the material. The percentage of silt plus very fine sand to be used for Figure 5.1, therefore, is the percentage of material contained between 0.002 mm and 0.1 mm.

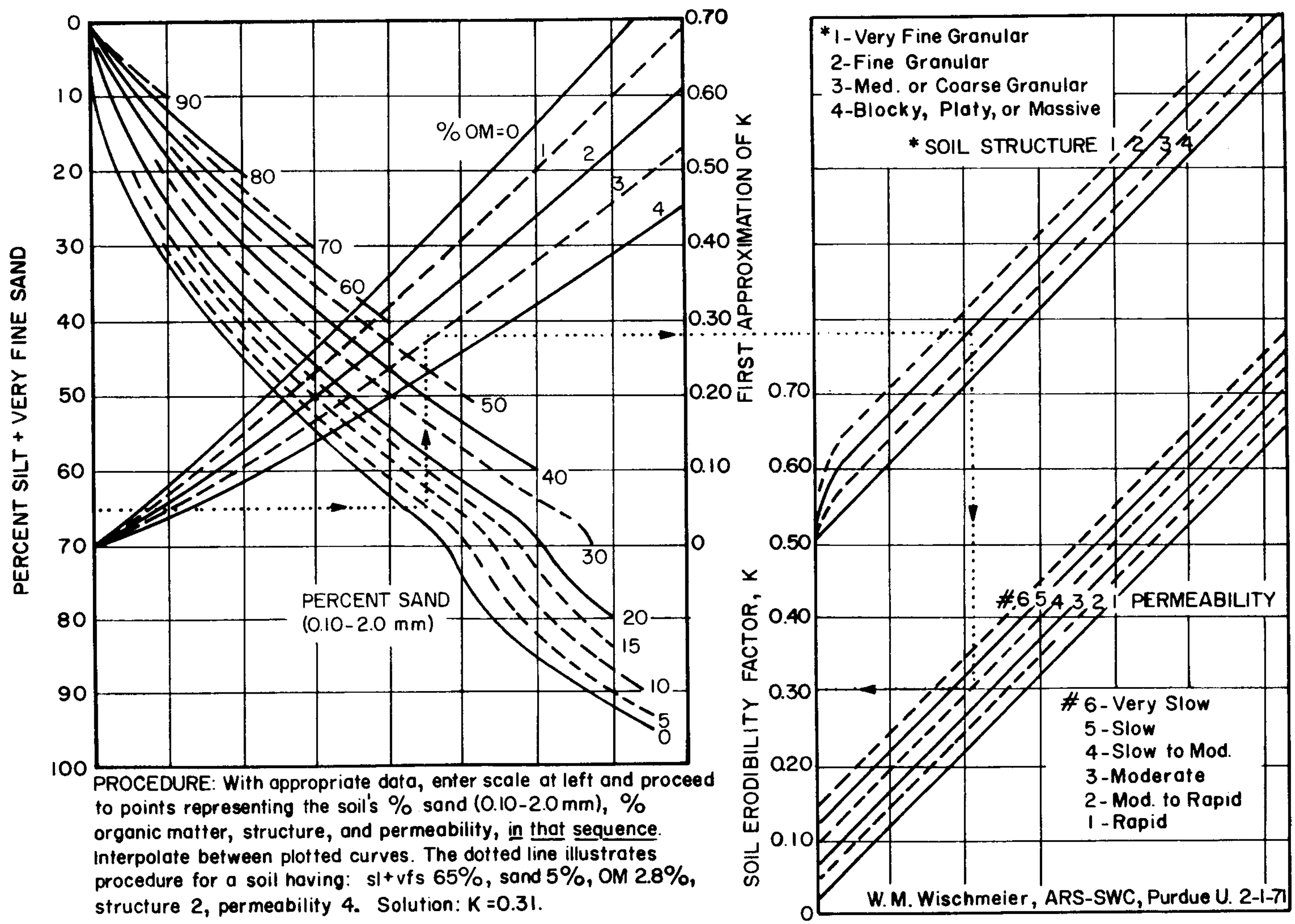


Fig. 5.1. Nomograph for determining soil erodibility factor *K*. *Source:* after Wischmeier et al., 1971.

5.1.2.3 The Topographic Factor (LS)

Although the effects of both length and steepness of slope have been investigated separately in different research efforts, it is more convenient for analytical purposes to combine the two into one topographic factor, LS. Wischmeier and Smith (1978) developed plots correlating the topographic factor for slopes up to 500 meters in length at slope inclinations from 0.5% up to 50%. Note that flat, short slopes will have less erosion than long, steep slopes and it is to the benefit of the design engineer to optimize slope length and gradients to fit the topography.

The equation to determine the LS factor is as follows:

$$LS = \frac{650 + 450s + 65s^2}{10{,}000 + s^2} \quad \frac{L}{72.6}^{\;m} \tag{5.2}$$

where LS = topographic factor
L = slope length in feet
s = slope steepness in percent
m = exponent dependent upon slope steepness

The slope dependent exponent m is presented in Table 5.2.

Table 5.2 Slope Dependent Exponent

Slope (percent)	m
$s \leq 1.0$	0.2
$1.0 < s \leq 3.0$	0.3
$3.0 < s \leq 5.0$	0.4
$5.0 < s \leq 10.0$	0.5
$s > 10.0$	0.6

5.1.2.4 The VM Factor

The VM factor is the erosion control factor applied in place of the cover and erosion control factors found in the USLE. The erosion control factor accounts for measures implemented at the construction site to include vegetation, mulching, chemical treatments and sprayed emulsions to impede or reduce erosion due to the overland flow of water. Values of the VM factor relative to site-specific conditions are presented in Table 5.3.

The VM factor is perhaps the most sensitive factor to effect the computed erosion loss for a given site. As shown by the values presented on Table 5.3, the development of a permanent vegetative cover can have a significant impact in reducing the computed erosion loss. However, the effectiveness of a vegetative cover over long-term periods should be questioned unless other protective schemes, such as armoring of the cover with the proper size material, are also included in the design.

5.1.2.5 Example Problem

An example problem in how to use the MUSLE is provided below.

Assumptions:

Site location:	Western Colorado
Site description:	Uncovered tailings pond
Pond size:	160 acres
Slope:	3%
Length:	2500 ft
Material:	42% sand greater than 0.10 mm; 58% fine sand and silt less than 0.10 mm; 5% clay less than 0.002 mm; 0% organics; (53% silt plus fine sand less than 0.1 mm); Consistency - fine granular; Permeability - slow to moderate.

The following factors have been determined for use in Equation 5.1.

R = 20 from Table 5.1

K = 0.50 from Figure 5.1

LS = 0.747 from Equation 5.2 and Table 5.2

VM = 1.0 (average from Table 5.3 based on an undisturbed surface)

Using Equation 5.1, the annual soil loss (A) from the tailings pond due to sheet erosion caused by flowing water is computed to be 7.47 tons/acre/year, or 1195 tons/year from the facility. Therefore, the cover is estimated to erode at a rate of 0.003 ft per year, or 0.3 ft/century.

5.2 SUMMARY AND FUTURE STUDIES

The main application of the soil loss equation approach in the evaluation of cover integrity is to determine whether it is possible for sheet erosion to penetrate the tailings cover, thereby exposing bare tailings and constituting a failure of the cover. The followup study will concentrate

Table 5.3. Typical VM Factor Values Reported in the Literature.[a]

Condition	VM Factor
1. Bare soil conditions	
freshly disked to 6-8 inches	1.00
after one rain	0.89
loose to 12 inches smooth	0.90
loose to 12 inches rough	0.80
compacted bulldozer scraped up and down	1.30
same except root raked	1.20
compacted bulldozer scraped across slope	1.20
same except root raked across	0.90
rough irregular tracked all directions	0.90
seed and fertilizer, fresh	0.64
same after six months	0.54
seed, fertilizer, and 12 months chemical	0.38
not tilled algae crusted	0.01
tilled algae crusted	0.02
compacted fill	1.24 - 1.71
undisturbed except scraped	0.66 - 1.30
scarified only	0.76 - 1.31
sawdust 2 inches deep, disked in	0.61
2. Asphalt emulsion on bare soil	
1250 gallons/acre	0.02
1210 gallons/acre	0.01 - 0.019
605 gallons/acre	0.14 - 0.57
302 gallons/acre	0.28 - 0.60
151 gallons/acre	0.65 - 0.70
3. Dust binder	
605 gallons/acre	1.05
1210 gallons/acre	0.29 - 0.78
4. Other chemicals	
1000 lb. fiber Glass Roving with 60-150 gallons asphalt emulsion/acre	0.01 - 0.05
Aquatain	0.68
Aerospray 70, 10 percent cover	0.94
Curasol AE	0.30 - 0.48
Petroset SB	0.40 - 0.66
PVA	0.71 - 0.90
Terra-Tack	0.66
Wood fiber slurry, 1000 lb/acre fresh[b]	0.05
Wood fiber slurry, 1400 lb/acre fresh[b]	0.01 - 0.02
Wood fiber slurry, 3500 lb/acre fresh[b]	0.10
5. Seedings	
temporary, 0 to 60 days	0.40
temporary, after 60 days	0.05
permanent, 0 to 60 days	0.40
permanent, 2 to 12 months	0.05
permanent, after 12 months	0.01
6. Brush	
7. Excelsior blanket with plastic net	0.04 - 0.10

[a]Note the variation in values of VM factors reported by different researchers for the same measures. References containing details of research which produced these VM values are included in NCHRP Project 16-3 report, "Erosion Control During Highway Construction. Vol. III. Bibliography of Water and Wind Erosion Control References," Transportation Research Board, 2101 Constitution Avenue, Washington, DC 20418.

[b]This material is commonly referred to as hydromulch.

on using the MUSLE for several alternate cover designs in order to evaluate whether the proposed analytical approach can be successfully used to measure the long-term integrity of protective soil covers for uranium tailings reclamation. Alternative designs will be compared, both from a standpoint of overall integrity and construction difficulty. The covers will also be evaluated using the critical shear stress approach to determine, based on a given PMP, the minimum particle size necessary to protect the cover against long-term degradation.

6. SELECTION OF RIPRAP

This chapter provides a methodology for selecting and oversizing riprap. Long-term performance objectives may be achieved with either high quality, weather resistant rock or suitably oversized marginal quality rock that is less resistant to weathering processes. Oversizing may be accomplished in either of two ways: 1) by increasing the design size of individual stones or 2) by increasing the thickness of the riprap blanket. However, marginal quality rock should be excluded from use in certain critical areas (Section 6.2).

There are limitations to oversizing individual stones of marginal quality rock. Rocks larger than 1 m^3 generally cannot be quarried, transported, and emplaced without considerable waste. Size limitations are imposed by the spacing of joints and bedding planes in quarried rock and by the maximum size of cobbles or boulders in channel or talus deposits.

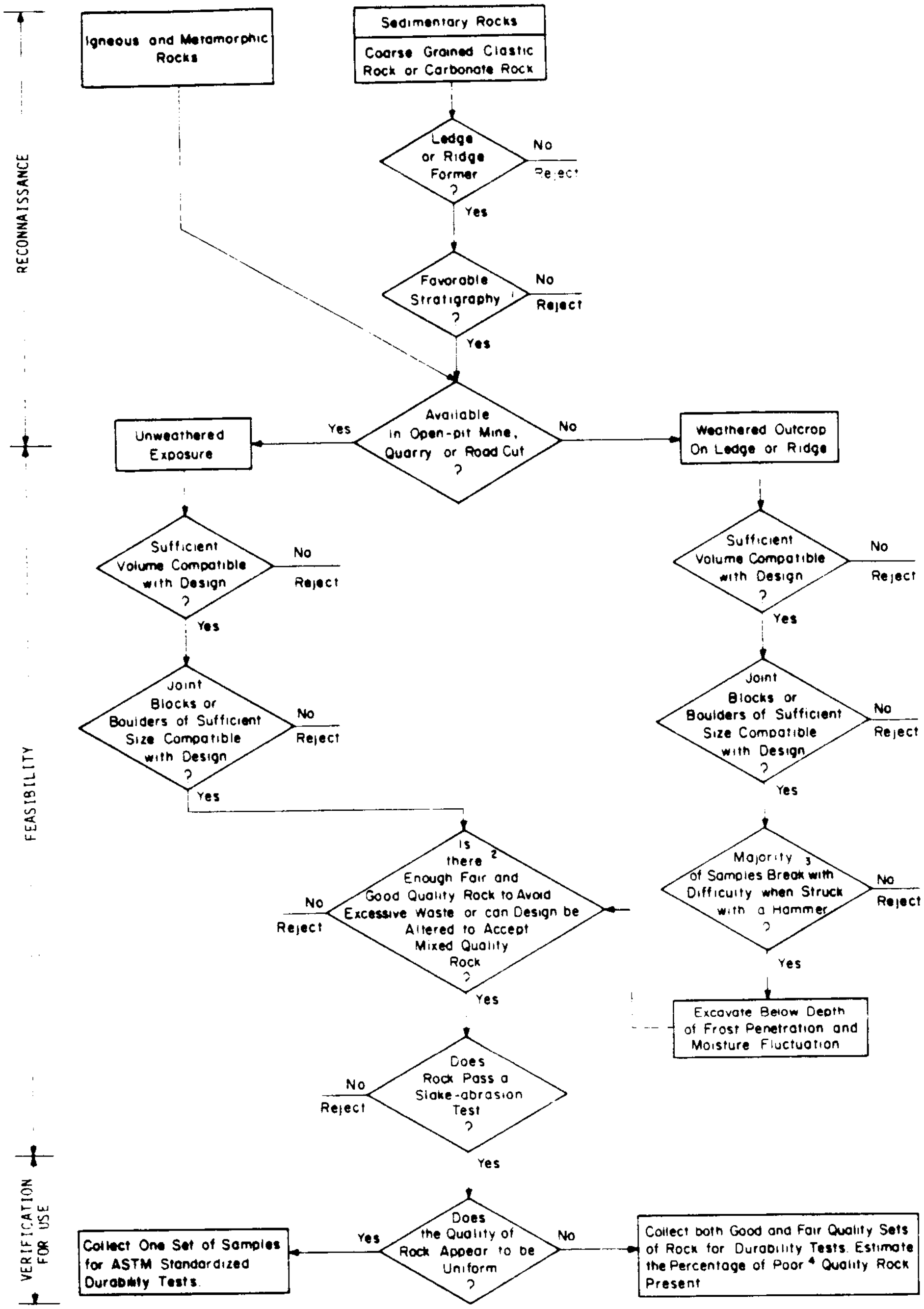
There is evidence to suggest that increasing the thickness of a riprap blanket is an effective alternative to oversizing the stones. Hanegan (1984) discussed the use of marginal quality riprap on the outer shell of a dam. The outer meter (3 feet) of the shell had deteriorated badly within 7 years after placement. The next meter (3 feet) showed only hairline cracking and below that little or no deterioration was observed. Hanegan believes that the buried rock experienced little alteration because temperature and moisture content fluctuations were minimal. However, the long-term fate of the deteriorated outer shell is uncertain. Erosion may eventually expose protected riprap to the same weathering processes that damaged the outer shell.

The riprap selection methodology developed in this chapter assumes reasonable care in quarrying, transportation, and placement of rock. Performance of riprap is as much related to handling practices as it is to selection of raw materials. It is the responsibility of the licensee to exercise reasonable care in the handling of riprap. Without proper handling even the most carefully selected rock may fail to perform well.

6.1 GENERALIZED INVESTIGATIONS FOR RIPRAP SOURCES

This section describes procedures followed by the U.S. Bureau of Reclamation (USBR) when investigating potential sources of riprap, and includes guidelines to be followed in sampling and testing of rock materials (DOI, 1974).

The complexity and extent of investigations conducted to determine suitable sources of riprap material will be governed by the size and design requirements of the project features and the quantity and quality of material required. These investigations occur in three stages: (a) reconnaissance, (b) feasibility, and (c) verification for use. Additional investigations are sometimes required immediately before or during construction. Figure 6.1 is a rock durability flow chart for riprap selection, encompassing reconnaissance and feasibility, and initiating the verification for use stage.



[1] Cobble or Boulder Conglomerate, Concretionary Sandstone, or other Favorable Characteristic.

[2] Generally Below Depth of Frost Penetration and Moisture Fluctuation.

[3] Approximate Sample Quality of Joint Blocks or Boulders:

Good - Breaks with Difficulty and Rings when Struck with a Hammer

Fair - Breaks with Difficulty

Poor - Breaks Easily

[4] It is not Necessary to Test Poor Quality Rock because it will Inevitably Fail Durability Tests; however, an Estimate of its Percent by Volume may be Needed to Assess Oversizing the Thickness of a Riprap Blanket.

Fig. 6.1. Rock durability flow chart for feasibility study. ***Source:*** **DOI, 1974.**

Rock sources must satisfy two main requirements: (a) the rock fragments must be produced in suitable sizes for the required usage and the (b) rock fragments should be hard, dense, and durable enough to withstand procurement and placement, and the processes involved in weathering. If material of required quality is available in sufficient quantity in the immediate vicinity of the project, it will be unnecessary to investigate more distant sources. If, however, there is a deficiency of suitable rock in the immediate area, it will be necessary to explore further. In this case, prospecting for rock should extend radially outward from the site until a deposit of rock is located which is suitable in quality and sufficient in quantity to fulfill the anticipated requirements.

6.1.1 Reconnaissance

This initial or preliminary exploration involves field surface reconnaissance using topographic, geologic, and agricultural soil maps and aerial photographs with supplemental information provided by records of known developed sources of material. A study of maps and aerial photographs may reveal possible sources of material. Contours are often an indication of the type of material: sharp breaks ususally indicate hard rock and slopes below cliffs often have talus deposits. During field reconnaisance, the countryside should be examined for exposed rock outcrops or cliffs. Road cuts, ditches, and open-pit mines may also reveal useful deposits. Data obtained should define the major advantages or disadvantages of potential materials sources within reasonable (economic) haul distance to the project site.

6.1.2 Feasibility

Information accumulated during this stage is needed to prepare preliminary designs and cost estimates. A complete survey of possible material sources located within economical haul range of the project site is made at this time. Field work should be done jointly by a geologist and a materials engineer. The potential material sources are examined to determine size and character, and particularly to observe joint and fracture spacing, resistance to weathering, and variability of the rock. The spacing of joints, fractures, and bedding planes will control the size of rock fragments obtainable from the deposit. Observation of weathering resistance of rock in situ along with resistance to fracturing by hammer blows will provide good indications of its durability. Rock that produces a ringing sound when struck with a hammer is generally durable. Particular attention should be given to location and distribution of weak seams or strata which must be avoided or wasted during quarrying operations. A general location map and a report describing the potential sources are prepared. The report should include estimates of quantity and uniformity of resource, amount and type of overburden, and accessibility by haul roads.

Representative samples of riprap material from the most promising potential sources are required for quality evaluation tests. The extent and detail of information necessary at this stage is described in Appendix B.

6.1.3 Verification for Use

The purpose of investigations at this stage is to verify a given rock source's durability and its suitability for use as riprap on tailings embankments and covers or on outfall areas of diversion channels as described in subsequent sections. Durability test data are also used to determine whether individual stones should be oversized or the riprap blanket should be overthickened.

Core drilling may be required, if dictated by geologic conditions, to verify the volume and uniformity of source material available. Such core drilling should be done on a grid system, if appropriate, and should include both vertical and angled holes as directed by the geologist or materials engineer. Blast testing, if appropriate, should also be done at this time.

6.1.4 Construction

This investigation stage is sometimes required to provide field and design personnel with additional detailed information for proper source development. This information should be obtained as project construction proceeds to provide advice to the quarry operators and to provide for proper processing and placing of quarried material. If unforeseen changes occur in quality of material being removed from the source, sampling and testing of the rock may be required to confirm material suitability or to delineate unsuitable rock areas.

Details of sampling, testing, and reporting are presented in Appendix B.

6.2 MICRO-ENVIRONMENTAL CONSIDERATIONS

Foley et al. (1985) describes a slake-abrasion test that is appropriate for tentatively determining the suitability of rock for use in protecting diversion channels and embankments from erosion during flood impingement or overland flow during intense storms. The combined effects of slaking and abrasion during a flood event can destroy marginal quality riprap, quickly exposing channel floors, walls, and embankments to catastrophic erosion. Foley et al. (1985) provides test data showing that durabilty is highly dependent on flow velocity. However, neither acceptance criteria, nor a standardized testing procedure based on predicted flow velocities have been established. The slake-abrasion tests should be considered.

Slow disintegration and decay may be more important in the long-term than catastrophic failure by slake and abrasion. The more common slow acting failure mechanisms in a semi-arid environment are (a) cyclic wetting and drying for some types of rock and (b) cyclic freezing and thawing of most rock types when more than 91% saturated.

The quality of riprap required for long-term stability depends on the frequency of saturation. Frequently saturated areas will experience cyclic deterioration by freezing and thawing far more frequently than seldom saturated areas. Hence, frequently saturated areas will require higher quality riprap than seldom saturated areas.

The validity of the assumption that areas above the 100-year floodplain are seldom saturated depends on good drainage. Good drainage can be provided by a filter blanket of gravel or sand underlying the riprap. The blanket would provide free drainage for overland flow from rain or snow melt. Thus, riprap located above flood waters would remain saturated for only a brief period of time. With proper drainage, riprap placed above the 100-year floodplain may occasionally be stressed by freezing and thawing but it would not be seriously stressed for more than one or two 24-hour cycles, once or twice per year. On the other hand, rock placed on a floodplain may be stressed by 24-hour cycles of freezing and thawing ranging from weeks to months during floods and accumulation of water from melting snow farther up the slope.

Rocks that are susceptible to disintegration by cyclic wetting and drying are considered to be nondurable even in seldom saturated areas. These rocks can be physically stressed with moisture contents throughout the partially saturated range. Expanding lattice clay minerals (smectites) swell and shrink as the moisture content rises and falls. Thus, rocks containing smectites are physically stressed by cycles of wetting and drying throughout the year and they are only marginally more durable in seldom saturated areas than they are in frequently saturated areas. If used, they should be restricted to areas above the PMF because of their tendency to disintegrate through the combined effects of slaking and abrasion during flood events.

Basically there are three distinctly different environments affecting the long-term durability of riprap in the uranium mill tailings management area. They are: 1) the relatively small but frequently (seasonal to 5-year intervals) saturated areas at and near the shoreline of a river and in the floor of a diversion ditch, 2) the somewhat larger areas that are occasionally saturated during 5 to 100-year flood events, and 3) the much larger areas (a) farther up an embankment face that may be saturated during rare but extreme flood events such as a 500-year flood or PMF, (b) on the tailings cap, or (c) in upstream areas of diversion ditches used infrequently to divert run-off water away from the tailings.

Diversion channels that drain small area watersheds deserve special consideration. Such channels are usually designed so that flood waters remain in them for only brief intervals. In this case, the channel is not necessarily considered as frequently saturated even though it lies within the 5-year floodplain. Thus, most of a properly designed diversion channel would not experience repeated freeze/thaw cycles despite periodic flooding. On the other hand, it may be more difficult to prevent repeated freeze/thaw cycles in the stilling basin area at the lower end of a division channel.

Diversion channels that drain large area watersheds would be subject to the same stringent requirements as natural rivers. In this case, it is unlikely that the diversion channel would dry out sufficiently after flood flow to prevent freeze/thaw cycling.

Figures 6.2 through 6.4 illustrate how frequently, occasionally, and seldom saturated areas may be identified. In Figure 6.2, a diversion channel completely contains the PMF so that tailings do not require channel erosion protection. Good quality riprap may be required for lining the channel floor as presented in Figure 6.3 whereas intermediate quality riprap would be suitable for the lower slope up to the height of the 100-year flood. Relaxation of these stringent requirements may be justified for a small watershed. The diversion channel should be designed to permit rapid drainage of peak flood flow. Slake-abrasion resistant but otherwise marginal quality riprap would be suitable on the upper slopes to the height of the PMF. Because of their gentle design slopes the tailings embankment and cap (10H:1V and 100H:1V, respectively) could be protected from gully erosion by overland flow with either vegetation or slake-abrasion resistant but otherwise marginal quality rock ranging in size from cobblestones to small boulders.

Figure 6.4 illustrates a diversion channel which is not designed to contain a PMF. In this case the lower slope of the tailings embankment must be riprapped with large stones to protect it from channel erosion. The riprap must be slake-abrasion resistant but may otherwise be of marginal quality because it lies above the 100-year floodplain. Slake-abrasion resistant but otherwise marginal quality rock ranging in size from cobblestone to small boulders would be required from the height of the PMF to the embankment crest because of its steepness (10H:3V). The tailings cap could be vegetated or riprapped with marginal quality small boulders and cobblestones.

Sections 6.3 through 6.5 describe variations in selection methodology in terms of the micro-environment. These environments differ substantially from the environments existing around flood control dams (spillway stilling basins and upstream embankment slopes being protected from wave action). However, the required duration of protection is an order of magnitude longer for the mill tailings environment (1000 years rather than 100 years) and it must endure without the benefit of periodic maintenance. Hence the riprap selection methodology for mill tailings stabilization must be more innovative and flexible than that of a typical reservoir embankment.

6.3 RIPRAP FOR FREQUENTLY SATURATED AREAS

Generalized methodologies for selecting riprap were addressed in Sections 6.1 and 6.2. Riprap durability scoring systems, oversizing and overthickening methodologies, and special considerations are discussed in terms of the micro-environment in this and subsequent sections.

6.3.1 Recommended Rock Types

Only highly durable rock should be considered for use in frequently saturated areas. Jahns (1982) suggests that rocks meeting the specifications of superior building stone for exterior use should be

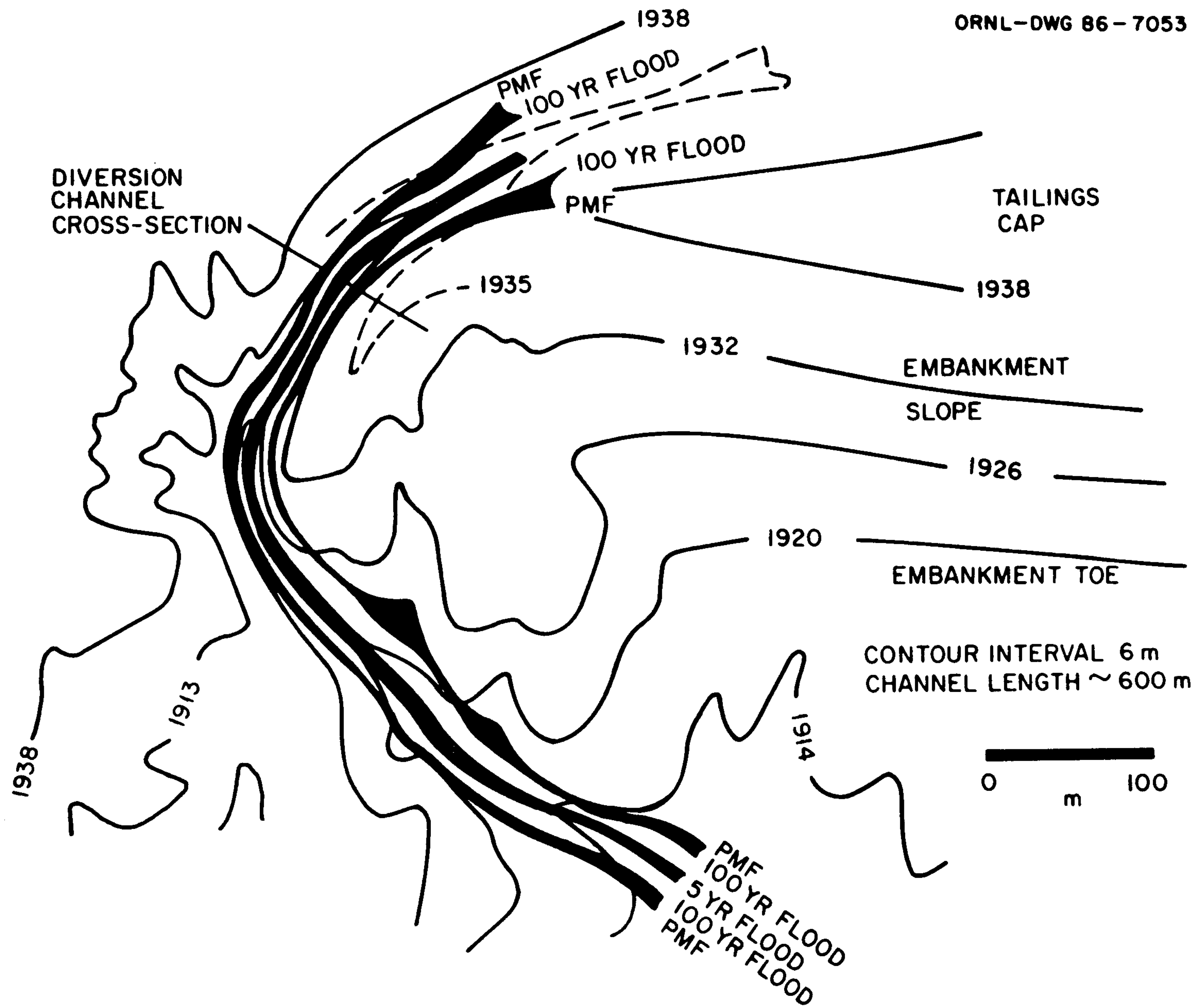


Fig. 6.2. Diversion channel showing frequently (5-year flood), occasionally (100-year flood), and seldom saturated areas; PMF totally contained.

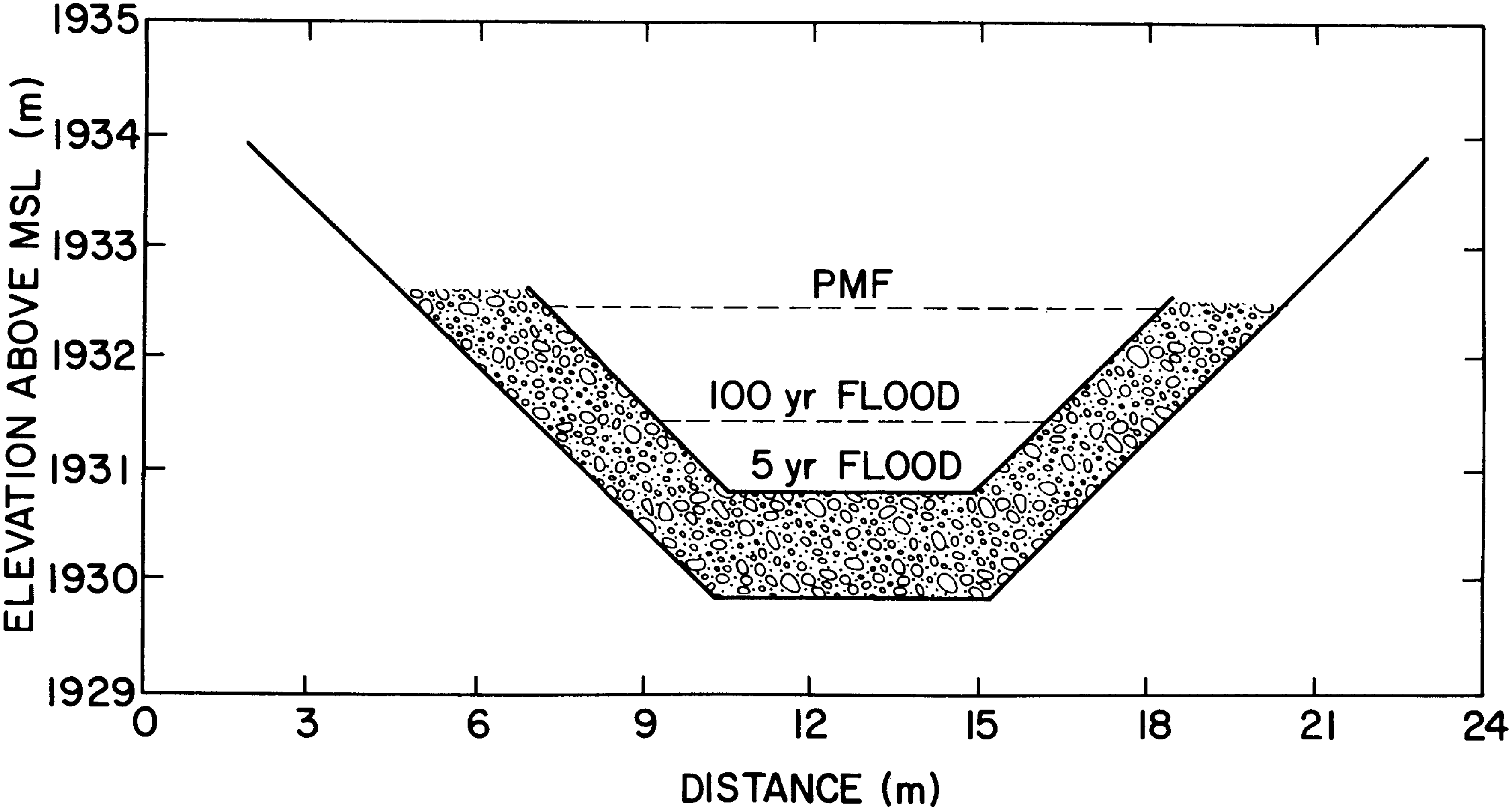


Fig. 6.3. Cross section of diversion channel showing frequently (5-year flood), occasionally (100-year flood), and seldom saturated areas; PMF totally contained.

ORNL-DWG 86-7051

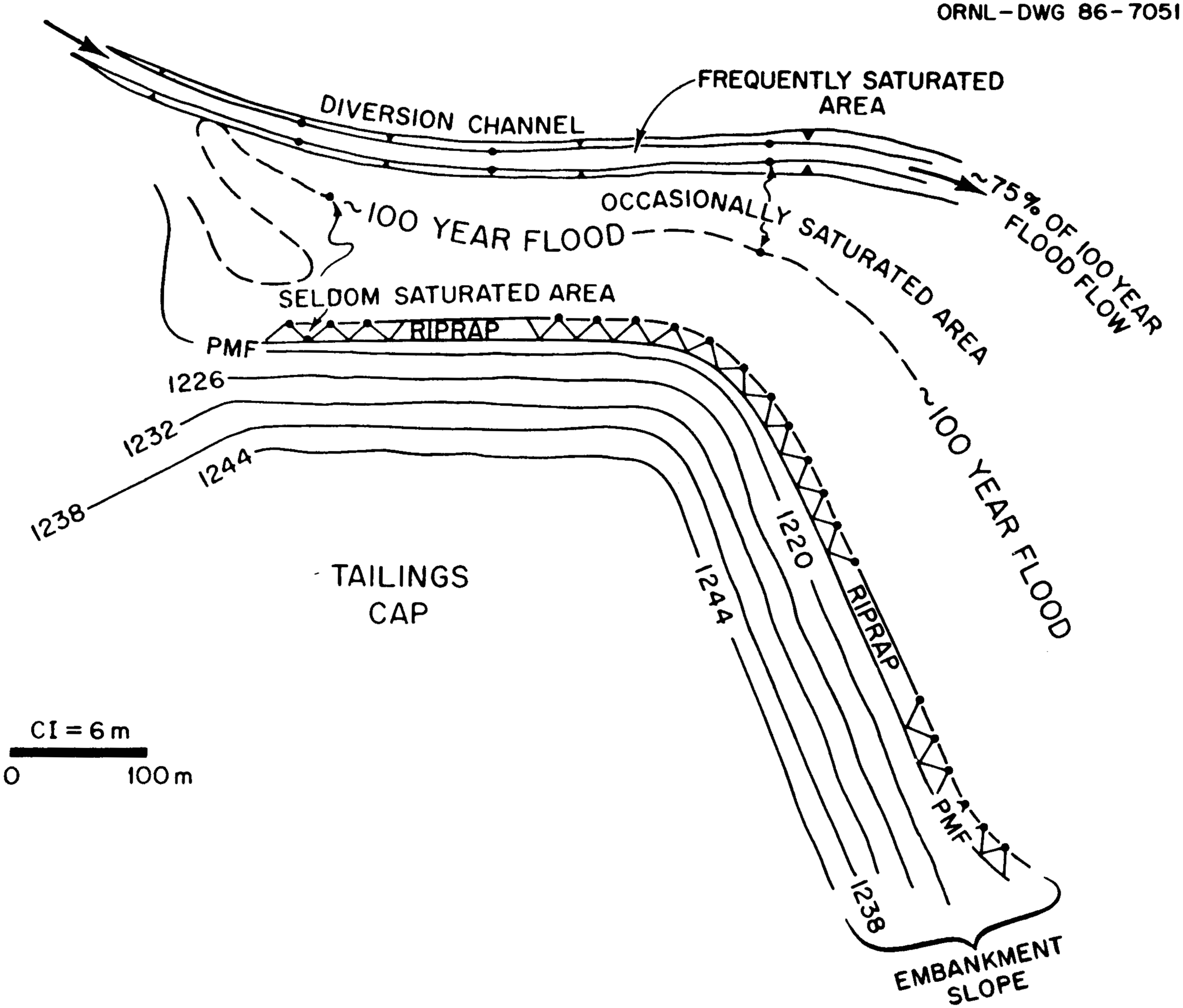


Fig. 6.4. Diversion channel showing frequently (5-year flood), occasionally (100-year flood), and seldom saturated areas; 100 -year flood uncontained.

relatively resistant to weathering. Table 6.1 lists these rocks in three priority groupings. Groups 1 and 2 are igneous and metamorphic rocks of preferred and acceptable rank, respectively. Group 3 rocks are carbonates which are vulnerable to decomposition in an acidic environment and are not generally recommended for frequently saturated areas.

Table 6.1 Rock Priority Groupings for External Use as Building Stone

Group	Type
1	Quartzites, noncalcareous slates, fine- to medium-grained felsic granites or granitic gneisses
2	Coarser grained granites or gneisses, dense basalts/or diabases
3	Marbles, limestones, dolomites

Source: Jahns, 1982

6.3.1.1 Prospecting

Extensive data files are available for locating suitable and accessible igneous and metamorphic rock quarries in the western United States. Among them are the open-file data of the U.S. Army Corps of Engineers and the U.S. Bureau of Reclamation (USBR). A limited amount of data may also be available from various state highway departments. These data provide quarry location, petrographic analyses, results of various durability tests, and intended uses for the rock. Also, Esmiol (1968) provides an analysis of performance of riprap at 149 USBR dams. It should be possible to identify several candidate sources of durable riprap within 100 km of a mill tailings site.

It may not be practical to open a new quarry closer than an existing quarry in cases where relatively small quantities of riprap are required. Exploration and development costs would likely exceed the savings in transportation costs that might be achieved from hauling a relatively small volume of rock.

6.3.1.2 Selection

Foley's slake-abrasion test should be used to qualify rock for more extensive testing for long-term durability. Candidate sources of riprap can then be compared with one another by examining the results of standard durability tests. At the present time the USBR routinely performs petrographic analysis, specific gravity, absorption, the sulfate soundness, freeze-thaw, and Los Angeles abrasion tests (see Appendix B for details). Table 6.2 is a list of acceptance criteria for USBR routine tests (DePuy and Ensign, 1965). The Corps of Engineers also performs the above tests but generally places less reliance on the sodium sulfate soundness tests. Also, some variation in testing is allowed between Corps districts. For example, in the southeastern states (including Texas) a wetting and drying test is substituted for the freeze-thaw test. Furthermore, the Corps often performs ethylene glycol tests on samples suspected of containing smectites (clay minerals that shrink and swell during drying and wetting cycles). Unfortunately specialized tests used by the Corps have never been standardized.

Table 6.2 U.S. Bureau of Reclamation Standards for Judging Riprap Durability

Test	Quality		
	Poor (N=1)[a]	Intermediate (N-2)	Good (N=3)
Bulk specific gravity	<2.5	2.5 to 2.65	>2.65
Absorption (% weight gain)	>1.0	0.5 to 1.0	<0.5
Freeze-thaw weight loss, %[b]	>5	0.5 to 5	0 to 0.5
Na_2SO_4 weight loss, %[c]	>10	5 to 10	<5
Los Angeles abrasion weight loss, %[d]	>10	5 to 10	<5

(a) Quality scores
(b) 250 cycles
(c) 5 cycles
(d) 100 revolutions

Source: Modified after DePuy and Ensign, 1965

If a licensee chooses to develop a new source of riprap, routine petrographic analysis and the durability tests (using ASTM, USBR or equivalent standard procedures) in Table 6.2 should be used to evaluate its suitability. If a sample meets acceptance criteria in a sodium sulfate soundness test, there should be no need to perform a freeze-thaw test. However, since the sodium sulfate soundness test is typically performed on crushed samples, failure sometimes results from stress fractures induced by crushing. If a sample fails this test, a freeze-thaw test should be performed on 7.3 cm cubes prepared with a rock saw.

Results of these tests can be directly compared with existing data from region-wide but less convenient sources of riprap. Estimated costs for work performed by the USBR are $2000 (1985 dollars) for durability tests and $1500 for petrographic analysis per sample set. The cost of freeze-thaw testing is included in petrographic analysis. Standard durability tests as described in USBR publications (DOI, 1974 and 1977) are presented in Appendix B. The USBR's standardized tests do not differ significantly from ASTM procedures.

The acceptance criteria of Table 6.2 require modification in response to special environmental conditions along a diversion channel or an embankment toe. It is expected that such areas will be chronically subject to greater salt crystallization, tensile stresses from frost wedging, absorption, and greater chemical weathering relative to stilling basins or reservoir embankments being protected from waves. On the other hand, impact, abrasion, and compressive stresses will be less important. Table 6.3 contains a suggested weighting system. Table 6.4 provides acceptance criteria for petrographic analysis in addition to other criteria listed by DePuy and Ensign (1965). Although petrographic analysis is a more appropriate indicator of long-term resistance to chemical weathering than are physical durability tests, the latter can be used to infer how much a rock has been physically weakened by prior weathering.

Overall quality test scores (Q) for candidate sources of riprap can be determined from Tables 6.2, 6.3, and 6.4. Quality scores (N_i, N=1, 2, and 3 for poor, fair, and good, respectively) from Tables 6.2 and 6.4 are multiplied by their weighting factors (W_i) for a given test (Table 6.3) and summed to obtain their overall quality test scores:

$$Q = \sum_{i=1}^{n} N_i \cdot W_i \tag{6.1}$$

where n is the number of petrographic and durability tests performed.

Table 6.3 Comparative Ratings and Weighting Factors of Selected Riprap Durability Tests

Category	Test Method	Weighting Factor
General weathering potential	Bulk composition	1.00
	Secondary mineralization and weathering	1.00
	Specific gravity	1.00
Tensile strength	Sodium sulfate soundness	0.75
	Freeeze-thaw	0.75
	Absorption	0.75
Compressive strength, impact, and abrasion	Los Angeles abrasion	0.50

Source: Modified after DePuy, 1965

Table 6.4 Additional Petrographic Analysis Acceptance Criteria

Criteria	Quality Poor (N=1)[a]	Fair (N=2)	Good (N=3)
Bulk composition [b]	Group 3, other	Group 2	Group 1
Secondary minerals and weathering	Smectites and thick weathering rinds[c]	Other clays and thin weathering rinds	No clays no weathering rinds

[a]Quality scores
[b]Groups 1, 2, and 3 rocks, see Table 6.1
[c]Greater than 1 cm thick

Acceptance criteria are tentative at this time. The maximum test score for the complete set of seven tests in Tables 6.2 to 6.4 is 17.25. It is suggested that if a riprap source has a test score exceeding 80% of the maximum possible score, it would be considered conditionally acceptable for use on frequently saturated areas. To be accepted, a sample would be required to score higher than 16.2 for the complete set of tests in Tables 6.2 to 6.4. A sample calculation is presented in Appendix C.

X-ray diffraction analysis should be performed on all candidate sources of riprap being seriously considered for use in frequently saturated environments. If smectite clay minerals or carbonate minerals are identified by X-ray diffraction analysis, further chemical tests may be necessary. The ethylene glycol test is used in many Corps of Engineer districts when the presence of smectites is suspected (Lutton et al, 1981). Joints in rocks are often sealed by secondary mineralization. Carbonate mineralization is the second most common form of secondary mineralization (quartz veins being most common). Their presence could be ascertained by placing fairly large rock specimens in a strongly acidic solution. Reaction to either ethylene glycol or acid and marginally acceptable performance in physical durability tests should result in exclusion from frequently saturated areas.

6.3.1.3 Design Modifications

For frequently saturated areas, project design modifications are sometimes possible to make use of rock containing carbonates or rock that is marginally acceptable as indicated by physical durability tests. Table 6.5 lists design modifications for various test results.

Table 6.5 Design Modifications Based on Results of X-ray Diffraction Analysis, Chemical Tests, and Physical Durability Tests

Test Outcome	Design Modifications
Marginally acceptable physical durability	Oversize stones
Well defined X-ray diffraction peaks for expanding lattice clay minerals (smectites) or reaction to ethylene glycol	None available
Reaction to acid or identification of carbonates	Emplace crushed limestone between blocks of riprap
No reaction to acid, few smeck- tites or carbonates present, superior physical durability	None required

An oversizing methodology is based on the assumption that the controlling failure mechanism is cyclic freezing and thawing. This is a reasonably good assumption provided the rock contains an insignificant amount of smectite clay minerals. Insignificant amounts of these clays are evidenced by the absence of well defined X-ray diffraction peaks used in their identification or failure of the rock to react to ethylene glycol. Presence of smectite minerals suggests that the rock is already in an advanced stage of chemical weathering and that further mechanical weathering is controlled by cyclic wetting and drying, or by slaking and abrasion, rather than by cyclic freezing and thawing. Hence, the following oversizing methodology does not apply to smectite-rich rocks.

Oversizing factors can be determined by the use of either the freeze-thaw test or the sodium sulfate soundness test. The latter is recognized by the USBR as a substitute for the freeze-thaw test. However, the use of the sodium sulfate soundness test may lead to more conservative oversizing than that of the freeze-thaw test.

Fresh samples of Groups 1 and 2 rocks (Table 6.1) generally perform well in durability tests and have good performance records when used as riprap. There is no reason to believe that these rocks would undergo substantial weathering during a 1000 year performance period. For example, weathering rinds on Eocene (40 million years old) granite boulders in the Wind River Formation of Wyoming often consume less than 50% of the boulders' total diameter and many Eocene boulders display no weathering rinds at all. Futhermore, the most accelerated period of weathering is believed to have taken place during Eocene time when the climate was sub-tropical and humid (Harshman, 1972). Oligocene and younger boulder conglomerates of the Shirley Basin are much less weathered, not only because of their relative youth but also because of the more arid climate since the beginning of Oligocene time. Therefore a 10% loss in size over a 1000-year performance period may be considered as a highly conservative estimate of weathering rate in semi-arid regions where most of the United States uranium mills are located. This suggested weathering rate would be less conservative for mills located in south Texas or Virginia. Oversizing fresh granite by 10% would generally provide a substantial factor of safety against weathering. Rocks that do not perform as well as USBR's good quality rock in durability tests should be oversized in proportion to their weight loss during sodium sulfate soundness or freeze-thaw tests. Groups 1 and 2 fresh rocks in Wyoming lose an average of about 1.3 percent weight during sodium sulfate soundness tests (USBR open-file data). Thus a rock that loses twice as much weight as the above tested rocks could be safely oversized by about 20%.

Equation 6.2 is an equation for oversizing riprap for use in frequently saturated areas:

$$S = 10 \cdot \frac{T}{D}$$

where

S = percent increase in design diameter
T = percent weight loss for a given physical durability test
D = average percent weight loss for the same tests performed on USBR's most durable rock

This oversizing methodology assumes that smectite minerals are not present in significant quantities. A sample calculation is presented in Appendix C.

6.3.2 Unconsolidated Cobbles and Coarser Grained Pleistocene Deposits

Pleistocene age cobblestones and boulders excavated from nearby abandoned or existing stream channels are the most widely considered alternatives to quarried rock. Other Pleistocene deposits (desert armor, talus, and glacial outwash) are less common alternatives. Coarse alluvium has been used at a number of Uranium Mill Tailings Remedial Action Program (UMTRAP) sites. Examples are the Gunnison and Grand Junction tailings piles in Colorado and at the Riverton site in Wyoming.

Channel and outwash deposits and desert armor are inferior to quarried igneous and metamorphic rocks becaue of their heterogeneity and size limitations. Nevertheless, cobblestones and boulders are commonly Group 1 and Group 2 rocks of Table 6.1. For example, Wind River gravels are mainly igneous and metamorphic rocks washed downstream from distant sources high in the Wind River Mountain Range. Unfortunately, some rocks that are very susceptible to weathering and/or wear (nondurable) are almost always present (USBR open-file data) in alluvial deposits.

Some channel deposits contain substantial amounts of nondurable rock. Cobbles of nondurable rock are likely to be found in streams flowing through narrow canyons which cut through stratified rock or through conglomerate beds that are older than Pleistocene. It may be necessary to restrict the use of such rock to occasionally or seldom saturated areas unless good quality rock can be easily separated from less desirable rock.

Boulder conglomerates of the Wind River Formation provide insight into the durability of Pleistocene channel deposits consisting of igneous and metamorphic rocks. Despite their age in millions of years, many of these Eocene boulders have only thin to moderate size weathering rinds. Although they break more readily under the impact of a hammer than do their Pleistocene counterparts, they still have considerably greater strength than sandstone and siltstone facies of the same formation. This implies that Pleistocene boulders of Group 1 and 2 rocks would be durable enough to survive a 1000-year performance period under the most stringent of environmental conditions.

6.3.2.1 Prospecting

Generally, suitable alluvial deposits are found only on terraces, flood plains, and channels of major streams whose headwaters originate high in nearby mountain ranges. The three UMTRAP sites previously cited are adjacent to the Gunnison, Colorado and Wind Rivers. Many abandoned (UMTRAP) and older operating mills are located adjacent to streams. Fewer than half of these streams contain adequate riprap resources to use in protecting the impoundments. None of the newer mills is located near a major stream.

Glacial outwash, desert armor, or colluvium from pediments may be sources of riprap at a few mill sites. Glacial outwash is found in Washington and pediments are found in the desert southwest. Talus deposits are widely distributed wherever there is sharp topographic relief.

Data resources for the location of gravel pits and durability tests for coarse aggregate are the same as those listed in Section 6.3.1. The USBR has substantially more test data on file for gravel pits than for any other potential sources of riprap.

It may be worthwhile to develop local sources of alluvium or desert armor. The fluvial geomorphology of a region should be studied in an attempt to find new sources of channel deposits. Topographic maps and aerial photographs are the best sources of information. Desert armor and talus deposits are difficult to identify from maps and aerial photographs and more extensive ground reconnaissance will be required to locate them.

6.3.2.2 Selection

Several sources of coarse aggregate should be evaluated for selection as riprap. Characteristics of deposits vary from one stream to another in terms of grain size distributon and lithology. After design size criteria have been met, the lithology should be examined in more detail.

Unlike rock quarried in place, alluvial deposits are lithologically heterogeneous so that representative sampling will be difficult to achieve. In addition to collecting samples of apparently good and intermediate quality materials for durability testing as described in the previous section, it will be necessary to estimate the percentage of nondurable rock present.

An estimate of the percentage of nondurable rock can be used to determine the cost of its removal. Samples should be drawn from each potential source population and examined for the presence of Group 1 and 2 rocks (Table 6.1). Rock samples may be identified by breaking them open and observing the fresh surfaces. The percentage of durable rocks (D_p) is the sum of the number of Group 1 (R_1) and Group 2 (R_2) rocks which break with difficulty divided by the total number of rocks sampled (T_r):

$$D_p = \frac{[R_1 + R_2]}{T_r} \times 100 \qquad (6.3)$$

Pleistocene boulders, however, are often so durable that they cannot be broken with a hammer even after repeated blows. In this case, the percentage of highly durable rocks could be obtained by determining the percentage of rocks which ring when struck with a hammer. Large numbers of samples reduce the likelihood of sampling error and lowering the uncertainty also reduces the potential for overdesigning a riprap blanket.

If may be feasible to remove nondurable rocks and fine grained material before alluvium is used as riprap. Many nondurable rocks (for example, sandstone) will not survive a trip through a grizzley while others (for example, weathered granite) may be difficult to separate from durable rock. Rock fragments and fine grained material can be washed through a screen and larger fragments of organic debris can be removed by hand. Channel deposits from existing streams will require a minimum of washing.

If several sources have durable rock, selection should be based on land acquisition, excavation, and transportation economics as well as on the cost of removal of nondurable rock or accommodating for the presence of nondurable rock by overthickening the riprap blanket.

6.3.2.3 Design Modifications

Design modifications in consideration of the characteristics of available alluvial sources of riprap differ from those discussed in Section 6.3.1.3 because oversizing individual stones from an alluvial source is not generally a viable option. Different degrees of oversizing would usually be required in response to a wide range of durabilities in nonuniform deposits.

Generally, overthickening the riprap blanket is the only viable option when using nonuniform material. If the best quality alluvial material is resistant to sodium sulfate soundness or freeze-thaw tests, the amount of overthickening will depend upon the percentage of high quality material present. Thus it may be advantageous to process alluvial material through mild crushing, screening, and washing before applying it as riprap. The amount of nondurable rock remaining in the riprap will depend on the intensity of processing. The riprap blanket thickness should be increased in proportion to the amount of nondurable rock that remains in the riprap after processing. A sample calculation is presented in Appendix C.

6.4 RIPRAP FOR OCCASIONALLY SATURATED AREAS

Any rock that is acceptable for use in frequently saturated areas is acceptable for use in occasionally saturated areas. However, highly durable rock such as that described in Section 6.3 may not be locally available in sufficient quantity to protect the larger but only occasionally saturated areas.

6.4.1 Alternative Rock Types

Other common local sources of riprap are clastic sedimentary rocks such as sandstone and siltstone. Occasionally, carbonate rock may also be locally available. These rocks are generally poorer in quality in relation to those previously discussed.

6.4.1.1 Prospecting

Foley et al (1985) developed a methodology for selecting riprap where only sedimentary rock sources are available. This methodology is greatly simplified with respect to that developed by the USBR (DOI, 1974) as discussed in Section 6.2. Foley emphasizes a reconnaissance search for geomorphic features (ridges, cuestas, and ledges) which demonstrate a natural resistance to erosion. Then Foley's feasibility study includes examination of erosion resistant rocks at weathered outcrops. Foley recommends several requirements to be met before a given source can be considered further. These requirements are: 1) the rock must break with difficulty under the impact of a hammer and is not simply case-hardened, 2) bedding planes and joints are spaced far enough apart to accommodate design size requirements, and 3) little organic material is present. The above requirements are pre-conditions for selecting samples for durability tests. Finally, samples would be subjected to a slake-abrasion test (a new test for which ASTM standard procedures have yet to be developed).

Attention to stratigraphic detail is essential in selecting sedimentary rock as a source of riprap. Despite their tendency to form erosion resistant ridges, hogbacks, and cuestas, Wyoming sandstones generally perform poorly in USBR standardized riprap durability tests (USBR, open-file data). Erosion resistance is often related to high infiltration rate (Sharp and Gibbons, 1964) which prevents concentrated overland flow required for gully development. Poorly cemented and poorly graded sandstones are generally nondurable, yet they may be relatively resistant to erosion because of the high infiltration rate. Sandstones are promising sources of riprap only where they contain lenses of cobble or boulder conglomerate or they are unusually well cemented. For example, in

Wyoming, the Wind River and Shirley Basins contain large quantities of cobble and boulder conglomerate and the Powder River Basin contains localize lenticular beds of concretionary sandstone (Soister, 1968; Harshman, 1972; and Sharp and Gibbons, 1964). Concretions are usually calcite cemented but they occasionally contain ferrugenous or siliceous cements as well. Fortunately, most sandstone concretions in the Powder River Basin contain few clay minerals. Boulders and concretions may range up to a meter or more in size.

In summary, one should not examine topographic features alone during reconnaissance for sources of riprap. Not all erosion resistant rock is durable. Outcropping rock is often nondurable because of its advanced state of weathering, and durable rock is not always exposed at the surface at convenient locations. Although ridge forming strata are potentially durable, open-pit mines and road cuts provide the most convenient source areas for collecting durability test samples. When taking samples from outcrops, excavation (by blasting, if necessary) to fresh rock is recommended.

If the duration of exposure to weathering at an outcrop were known, the degree of weathering would be an excellent indicator of the long-term durability of rock. Rarely, however, will the duration of exposure be known with sufficient precision. The presence of lichens and desert polish suggest only that exposure to weathering was "not recent". Radiocarbon dating of dead tree roots in rock fractures or tree ring analyses are the only available means of determining the absolute age of exposure to weathering and even then these methods only provide a minimum age. Unfortunately, organic debris is rarely preserved in an oxidizing environment.

It is known, however, that some boulder conglomerates have been exposed to weathering at least intermittently for very long periods of time. For example, in the Wind River and Shirley Basins of Wyoming, Eocene-age boulders of the Wind River Formation were first exposed some 40 million years ago, weathered, dislodged from their areas of outcrop, rounded in transport for a distance of up to several 10's of km and finally deposited and left undisturbed until Holocene time. This process obviously required considerable time, perhaps millions of years.

Durability of cobbles and boulders is a function of both age and lithology. Although basalt and diabase cobbles of Eocene age in the Shirley Basin are badly weathered, granite and granodiorite cobbles are only moderately weathered (Harshman, 1972). Oligocene and Miocene cobbles in the Shirley Basin are much fresher even though they are also millions of years old. According to Harshman, weathering rates were greater during Eocene time in Wyoming because of the humid-subtropical climate that prevailed at that time. Since Oligocene time, the climate was more like that of today. Hence, present weathering rates are much slower than that of Eocene time.

6.4.1.2 Selection

Acceptability criteria can be relaxed for the use of marginal quality rock in occasionally saturated areas. Such areas will experience slower rates of chemical weathering and reduced deterioration from cyclic freeze-thaw. Furthermore, impact and abrasion from flood events will occur less often. To be accepted for use in occasionally saturated areas, it is tentatively suggested that a sample would be required to score higher than 65% of the maximum possible score (11.2 for the complete set of tests of Tables 6.2 to 6.4).

6.4.1.3 Design Modifications

Oversizing methodology is similar to that described in Section 6.3.1.3. Areas subject to flooding only once every 5 to 100 years are likely to be saturated one fifth to one twentieth as often as in saturated areas. Thus samples that lose twice as much weight in standard durability tests as fresh granite would require only a maximum of 4%, rather than 20%, oversizing that is required for frequently saturated areas. Samples that lose ten times as much weight would require 20% oversizing as opposed to 100% oversizing in frequently saturated areas. Greater weight losses than 25 times that of fresh granite would likely result in a sample's rejection for use as riprap. Equation 6.4 is a formula for oversizing riprap in occasionally saturated areas:

$$S = 2 \cdot \frac{T}{D} \tag{6.4}$$

where S, T, and D are defined in Section 6.3.1.3. Again, this oversizing methodology assumes that few smectites are present. A sample calculation is presented in Appendix C.

Overthickening the riprap blanket would be required for the use of cobble or boulder conglomerate because of its generally nonuniform quality. Overthickening methodology is the same as that outlined for Pleistocene deposits in Section 6.3.2.

6.5 RIPRAP FOR SELDOM SATURATED AREAS

The methodology is essentially the same as that developed in Section 6.4. The principal difference is the further relaxation of acceptance criteria in response to diminishing frequency of freeze-thaw, impact, and abrasion. Tentatively, it is suggested that durability test scores exceeding 50% of the maximum possible score (8.6 for the complete set of tests) would be acceptable for use in seldom saturated areas.

Oversizing could also be further relaxed because areas above the 100-year floodplain are saturated less than one twentieth as often as areas below the 5 year floodplain. Thus, 20 times as much weight loss during a standard durability test would require only 10% oversizing. Hence, if the rock passes acceptance criteria only nominal oversizing should be required above the 100-year floodplain.

7. CONCLUSION

This report has presented as quantitatively as possible methodologies to evaluate physical factors (precipitation, fluvial systems, erosion, and riprap sizing and selection) that can influence the long term stability of uranium mill tailings impoundments. Where technology did not exist, the authors attempted to develop procedures to the extent necessary to meet the requirement of the project.

As determined in previous studies, the design flood for the evaluation of the stability of tailings impoundments should be the PMF. This report presents methods available for determination of the PMF. These methods, which have been developed over a relatively long time, represent the current state of the art and are sufficient for the purposes of evaluation of long-term mill tailings stabilization plans.

Similarly, the state of the art for predicting fluvial geomorphic stability has been developed over a long time, and is considered adequate if used within the framework presented in Chapter 3. However, it may be difficult to obtain all of the data needed at a particular site to perform a quantitative evaluation. In that case, disciplined, professional judgment must be used to make a qualitative assessment.

The sizing of rock covers as part of the surface stabilization to protect against sheet erosion is also discussed based on using the existing state of the art. The many observations and experience gained over a long time period lends confidence to the use of the methodology presented.

The initiation of gully formation on a surface, the development of gullies, and protection against gully intrusion is an area in which the existing state of the art is inadequate for purposes of this report. Therefore, a methodology was developed based on observations of existing tailings impoundments. Although the data base is limited, the results to date have fostered a good level of confidence. Because gully erosion is a failure mechanism that can have severe consequences with regard to long term stability, it is recommended that additional efforts continue to develop and further refine the methodology.

The durability of the cover materials used at the mill tailings impoundments is of paramount importance. Selection of riprap is another parameter for which the current state of the art is lacking. This report has discussed various factors influencing weathering and durability of riprap, and has set forth procedures that can be used to evaluate durability. Nevertheless, this is another subject to which future investigative efforts should be devoted.

Throughout the development of the methodologies presented herein, consideration has been given to uncertainties in various parameters, and to the assessment of the consequences of failure. Appendix A discusses several aspects of probabilistic risk analysis. However, with regard to the design of uranium mill tailings impoundments, risk analysis is in its infancy.

8. APPENDICES

APPENDIX A. PROBABILISTIC RISK ANALYSIS OF LONG TERM STABILIZATION

A.1 INTRODUCTION

Most engineering designs are presently based on deterministic analyses. That is, single values are selected for material parameters and compared against the loads applied. In other words, the capacity of the system is compared to the demand placed on it, and this comparison is frequently expressed as a factor of safety. Such an approach neglects the importance of the variability which is inherent in all parameter values. For example, even manmade materials such as steel, which is subjected to stringent quality control, show variability in yield and tensile strength. Geological materials and processes show much greater variability and to neglect such variability in any engineering design is unreasonable.

Recent years have seen rapidly growing research into applied probability and increased interest in applications to geotechnical engineering practice. Unfortunately, probability still remains a mystery to many engineers, partly because of a language barrier and partly from lack of examples showing how the methodology can be used in the decision making process (Whitman, 1984). Probabilistic concepts are routinely used for decision making in water resources.

A probabilistic analysis considers the variability or uncertainty of the parameters. The central tendency of a parameter is usually expressed by the arithmetic mean while the coefficient of variation is a useful measure of variability or dispersion. The coefficient of variation is the ratio of the standard deviation to the mean and is expressed as a percentage.

The final result of a probabilistic analysis is expressed as a probability of failure. This linearly scaled parameter is a much more realistic measure upon which to base decisions than a "factor of safety". The latter is not linearly scaled and its 'accepted' value is based on experience. Probabilistic analyses are therefore particularly useful in the design of systems for which an accepted "factor of safety" has not yet been defined.

Probabilistic analyses are not a replacement for engineering judgment. On the contrary, considerable judgment and knowledge of a process or failure mode is required to perform a realistic probabilistic analysis. The biggest advantage of a probabilistic analysis is that it forces the engineer to investigate the variability and uncertainty of all the contributing forces or parameters to a specific failure mode. Even when a precise quantification of probability of failure is not possible, systematic formulation of the analysis aids greatly in understanding the major sources of risk (Whitman, 1984).

In a deterministic analysis, only one value is selected for a parameter. This value can be the mean or a "best estimate", very often a conservative value based on judgment. Although a large volume of data might have been gathered the variability is neglected in deterministic analyses and only one value is used. This is tantamount to "not using all the information" that was gathered during an investigation.

It is the purpose of this appendix to investigate the potential application of probabilistic risk analysis in the long-term stabilization planning of uranium tailings impoundments. This appendix will review some of the definitions and principles of a probabilistic or risk based analysis. Each of the failure modes identified by Nelson, et al. (1983) will be investigated and it will be shown in principle how these can be cast in a probabilistic framework.

A.2 DEFINITIONS AND CONCEPTS

The variability/uncertainty in the value of any parameter is expressed by a probability density distribution as shown in Figure A.1. The probability that the parameter x will have values less than x = a is given by the area under the curve shown shaded in Figure A.1. The parameter x can represent the capacity of the structure to withstand load, e.g., the differential settlement which a cover material can withstand before severe cracking will occur. In this example, the shaded area will be the probability that cracking will take place when the differential settlement (or demand on the structure) is a maximum of x = a. Therefore, if the demand is considered to be a deterministic value such as x = a in Figure A.1 and the capacity is assumed to have some distribution, then the shaded area is the probability of failure.

The demand on the structure, e.g. the predicted differential settlement, can also be a variable such as x in Figure A.1. In this case one can consider a "capacity-demand model" as shown in Figure A.2. The probability of failure is a function of the area of overlap (note the probability of failure is not equal to the area of overlap).

As a comparison, note that the factor of safety is defined as:

$$F.S. = \overline{C}/\overline{D}$$

Instead of using "all the information" contained in the probability distributions the factor of safety approach only uses two deterministic values, the statistically mean values of capacity and demand.

There is one more approach to calculate the probability of failure on the basis of factor of safety. Consider the distribution in Figure A.1 to be the distribution of factor of safety. The probability of failure will then be given by the shaded area if a = 1, i.e. the probability of failure is the probability that the factor of safety is less than unity.

Reliability theory provides a rational framework for accounting for the uncertainties in both capacity and demand. Reliability theory also offers the prospect of a systematic method for selecting the safety factor

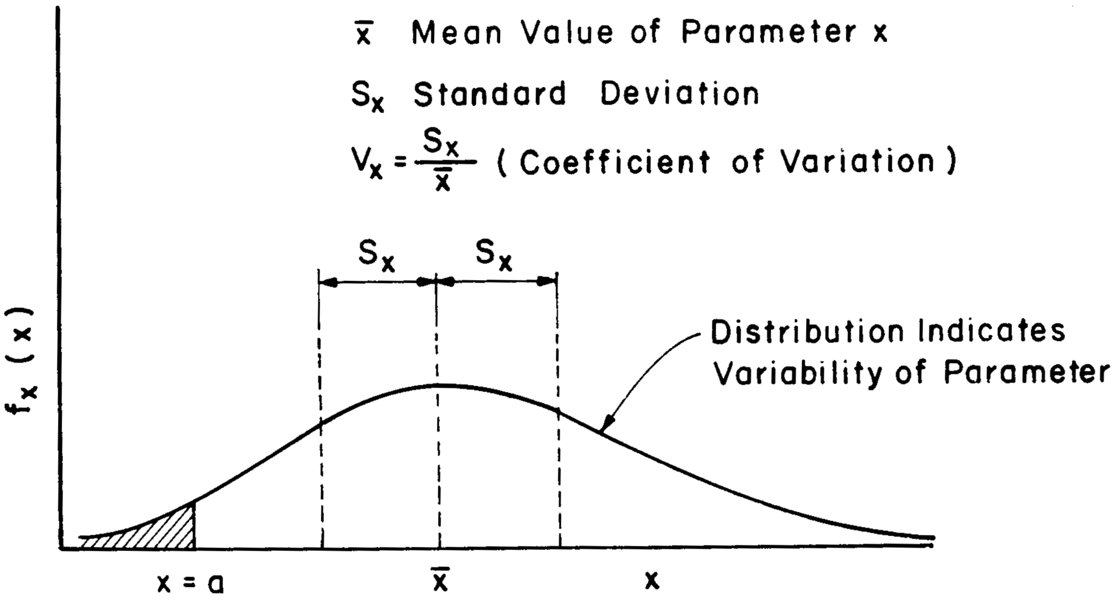


Fig. A.1. Probability density distribution of parameter x**.**

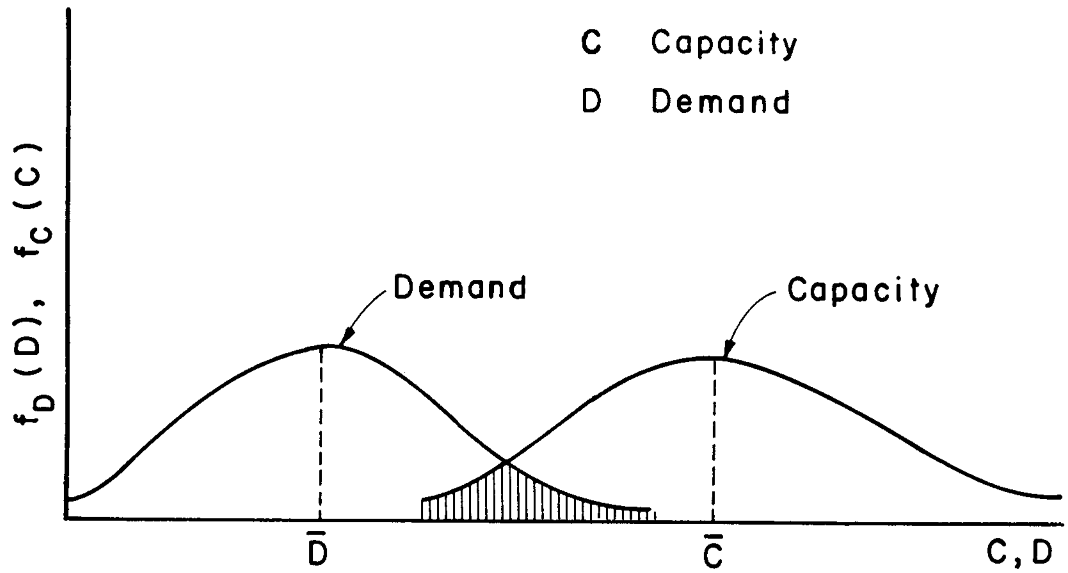


Fig. A.2. Capacity-demand model.

appropriate for a particular application. Historical precedent or experience can be used to guide the designer in selecting a suitable reliability and subsequently a safety factor. It can therefore be concluded that when there is no standard for a safety factor, but the problem is well understood and there is an adequate data base, reliability theory may be used to guide selection of a safety factor consistent with the degree of safety in other problems. However, in cases where the problem is not well understood and data sets are small, the probabilistic approach can still be applied through the use of subjective probabilities.

There are several requirements for the formal treatment of reliability (Whitman, 1984):

(i) Clear delineation of the criteria for success or failure.
(ii) Selection of a deterministic model relating the basic variables to the criteria for success or failure.
(iii) Identification of the uncertainties concerning the basic variables.
(iv) Evaluation of the distribution functions or moments of the basic variables.

Thus, there exists a probability of failure for any system that is designed. This probability of failure is a function of the variability/uncertainty of the capacity (e.g. strength) of the system and the magnitude of the demand placed upon the system.

For applications to uranium tailings impoundments, the design demand could be taken as the PMF and forces associated with the PMF. In this case the demand is a deterministic value.

Failure of an impoundment, however, can take different forms. For example, some erosion could occur and remove a small amount of the toe of an embankment with no release of tailings. On the other hand, a massive loss of a large part of the impoundment could occur releasing large volumes of tailings over a large area. Obviously these two failures would have greatly different consequences. It is necessary, therefore, to consider not only probability of failure but the consequences of this failure as well. In this regard the concept of "risk" and "hazard" should be introduced.

A.3 RISK AND HAZARD ASSESSMENT

Risk may be defined as a compound measure of the probability and magnitude of adverse effects, or

Risk = (Uncertainty) (Damage)

Other definitions of risk are "the chance of encountering harm or loss" or the "degree of probability of such loss" (FEMA 1984).

The dictionary defines hazard as "a source of danger". Hazard, therefore, simply exists as a source. Risk includes the likelihood of conversion of that source into actual delivery of loss, injury or some form of danger, or

Risk = Hazard/Safeguards

This implies that risk may be kept as small as desired by increasing the safeguards. As a matter of practical reality, however, risk can never be brought to zero (FEMA 1984).

Hazard is the possibility that some adverse effect might happen upon exposure. Risk is the probability that hazard will happen.

Dreith (1982) lists the following four steps to evaluate risks and define appropriate responses with respect to hazardous waste sites:

(i) Hazard identification (inventory composition; physical and chemical properties; biological properties; toxicity, carcinogeneity; interaction of wastes). In the case of uranium mill tailings hazard identification consists of characterization of the waste with respect to radiological parameters (e.g. potential radon emission), heavy metals, salts and other constituents which may put some population at risk.

(ii) Hazard evaluation (disposal methods; prior treatment; failure modes; transport mechanism; processes acting on wastes through time). The hazards which a uranium tailings impoundment pose would be evaluated in this step. For example, cap failure caused by high flows and the subsequent transport of the waste to a population at risk. All failure modes must be identified in this step.

(iii) Risk evaluation (probability of a failure; concentration and population at risk; toxicological and epidemiological levels of potential and actual human exposure, and information on effects and consequences of dose). Following the identification of failure modes above, the probability of such a failure must be estimated. The effects of such a failure on the population at risk must also be evaluated. Input requirements in this step are dilution of identified hazardous material during transport, doses of these materials which can lead to negative effects and their consequences. The basic question is, given a probability of release what is the probability of negative conseqences?

(iv) Risk reduction/response (determine risk situation by making comparisons with other examples of risks that society is willing to take; determine need for actions; justify benefits vs. failures; use of critical resources - costs/time). A risk assessment in the design phase can be used to decide whether the imposed risks are too high requiring a change in the design. A relationship between cost of reclamation and risk can therefore be established and used for decision making.

A risk assessment and response as outlined above involve a large number of areas where judgments are required. Subjective probabilities therefore play an important role. Some of the results may often be qualitative instead of quantitative in nature.

A.4 PROBABILISTIC RISK ASSESSMENT

A probabilistic risk assessment (PRA) is an analysis that (NRC, 1984):

(i) identifies and delineates the combinations of events that, if they occur, will lead to an undesired event;
(ii) estimates the frequency of occurrence for each combination; and,
(iii) estimates the consequences.

PRA results are useful, provided that more weight is given to the qualitative and relative insights regarding design and operations, rather than the precise absolute magnitude of the numbers generated. A PRA study is multidisciplinary, and depending on its scope, may require analyses of containment systems, human behavior, the progression of failure modes, radionuclide behavior, and health effects. However, not all the areas of analysis involved have reached the same level of development (NRC, 1984), which further underscores the necessity for qualitative results.

Based on a schematic outline of the offsite consequences of nuclear accidents from a probabilistic risk assessment of reactor safety (NRC, 1984), the schematic outline in Figure A.3 was compiled for evaluating the offsite consequences from a uranium tailings release. A review of this schematic clearly indicates the large number of unknowns associated with the determination of final property damage or health risk. The approach presented in this appendix only addresses the probabilities of failure and not the final property damage or health risk. This is done to demonstrate the direct design applications of risk assessment.

The application of a probabilistic risk analysis based on the various failure modes, as described below, will be used as a guide for selecting a safety factor consistent with the degree of safety acceptable to society. This approach is schematically shown in Figure A.4. By using an acceptable probability of failure, $x = D$ can be determined based on the information about the mean and variability of capacity. Once D is fixed, the factor of safety, as defined on Figure A.4, can be calculated. It is very important to recognize that two structures having the same factor of safety can have different probabilities of failure due to different variabilities in the capacity function. It is therefore possible to have a structure with a factor of safety = 1.3 having a lower probability of failure than another with a factor of safety = 1.5. Or, stated differently, factor of safety does not "use all the information" as it does not include the variability of the capacity function.

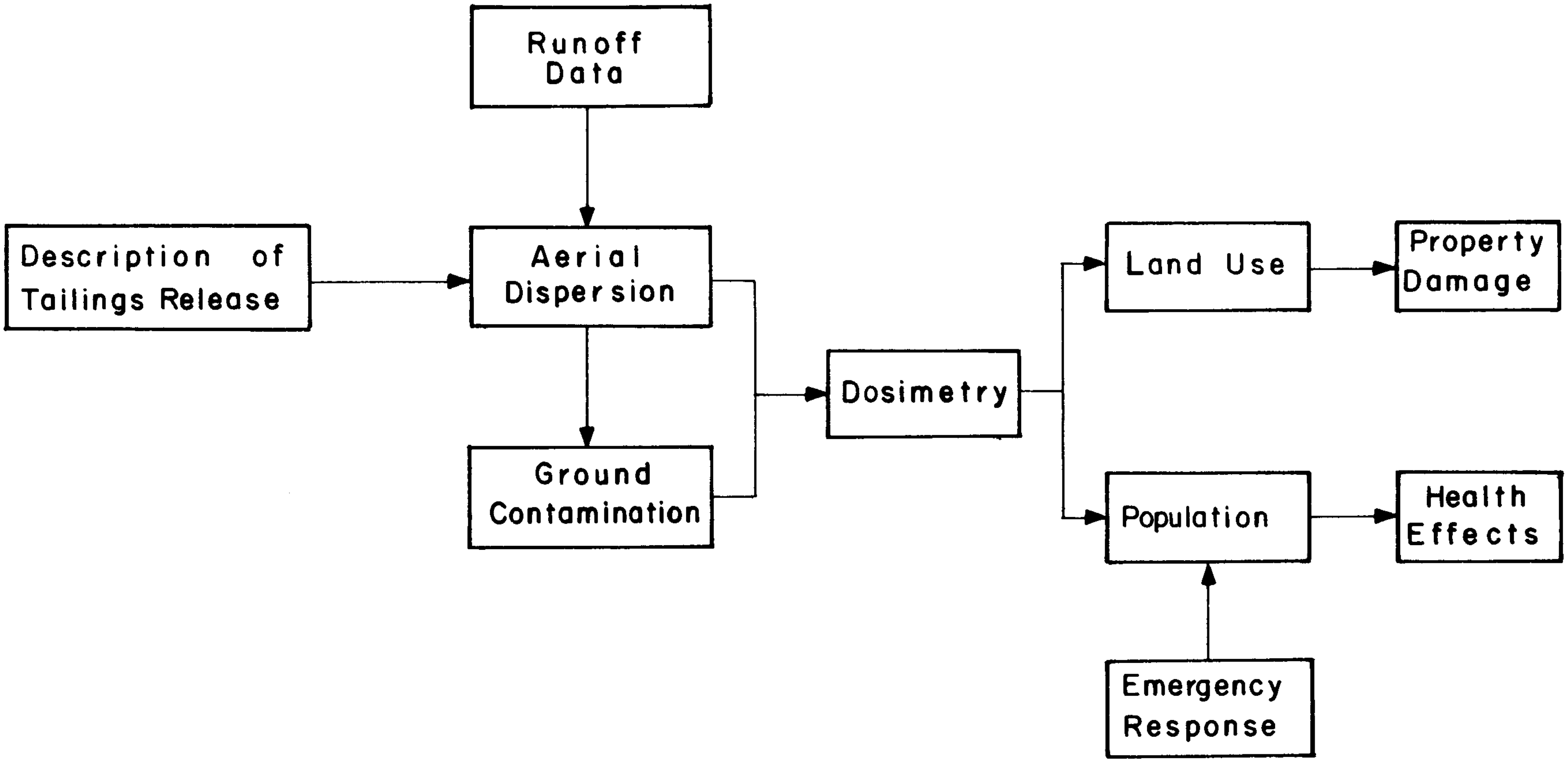


Fig. A.3. Schematic outline of offsite consequences model from uranium tailings release.

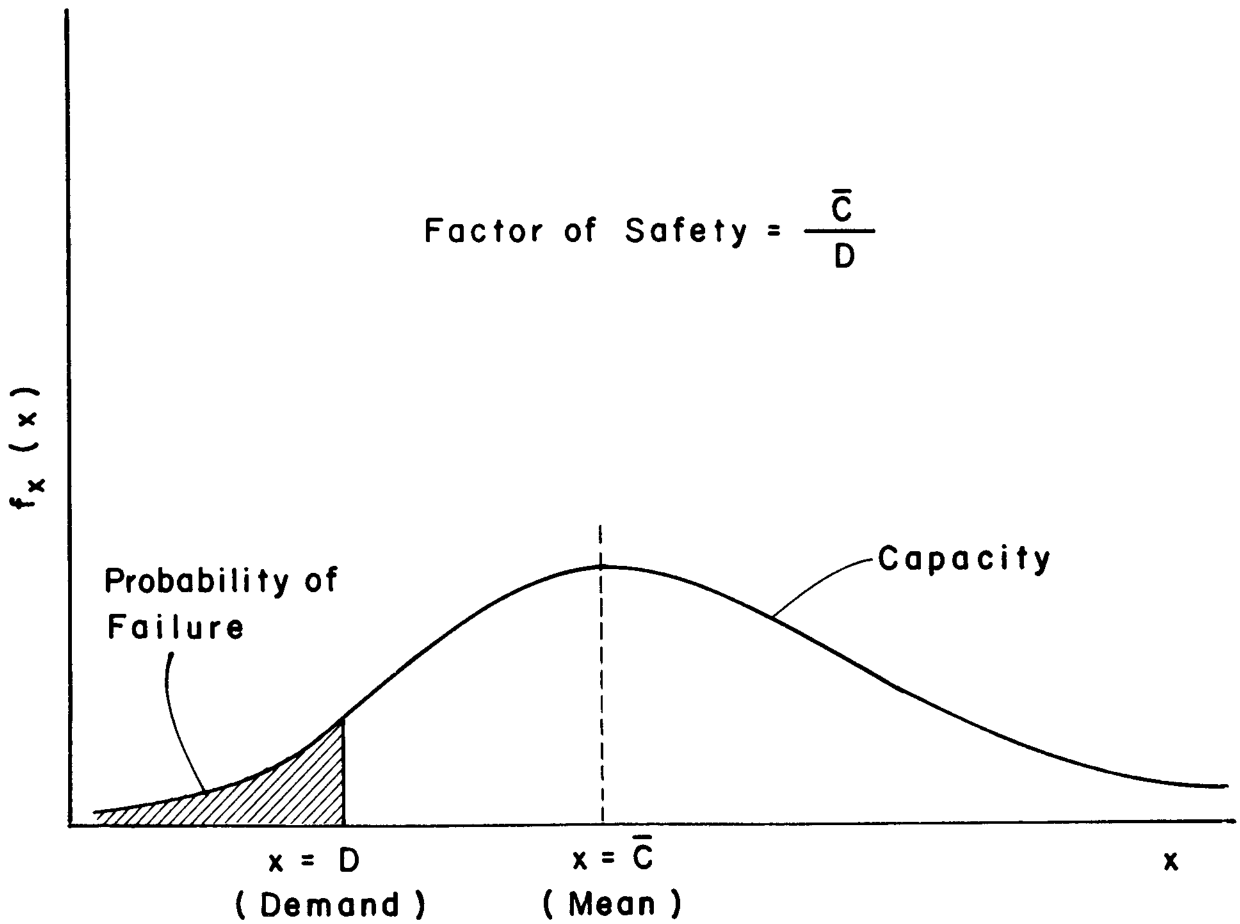


Fig. A.4. Relationship between capacity, demand, probability of failure, and factor of safety.

The value of the probabilistic risk analysis presented in this appendix, therefore, lies mainly in serving as a decision making tool to decide upon acceptable minimum factors of safety.

A.5 FAILURE MODES IN LONG-TERM STABILIZATION OF IMPOUNDMENTS

This section investigates the failure modes defined by Nelson, et al. (1983) and casts these in a probabilistic framework. The final result of this section is the ability (in principle) to calculate the probability of failure of the individual failure modes.

In almost all of the failure modes described in this section the demand function is the runoff from floods. It was concluded by Nelson, et al. (1983) that the PMF should be used as the design flood for all long-term stability evaluations.

A fault tree can be used to indicate the various sequences of events which may lead to a failure by any of the failure modes. Figure A.5 presents a fault tree for the failure modes identified by Nelson, et al. (1984) in evaluating the long term stability of a uranium tailings impoundment. This fault tree was compiled assuming that all the components meant to resist flooding were designed for the PMF and that failure will not occur if a flood smaller than the PMF occurs. This assumption is obviously not strictly correct because floods smaller than the PMF may result in a smaller probability of failure.

The overall probability of failure of a structure is given by combining all the probabilities (P_{f1}, to P_{f7}) obtained from the fault tree. Simple techniques are available to do this (Ang and Tang, 1984). Neglecting the contribution of failure probabilities due to floods smaller than the PMF will result in a lower-bound overall probability of failure.

The main purpose of the analysis here is to use probability of failure of the separate failure modes to select the most appropriate factor of safety. The overall probability of failure is not used in this approach and no information is therefore lost by making the assumption above.

Using the fault tree in Figure A.5, the separate probabilities of failure (P_{f1} to P_{f7}) can be calculated. In the concept of using the PMF as the design flood it is implicit that within the stability period used for design (200, 500 or 1000 years) either the PMF or a flood having a magnitude sufficiently close to the PMF will occur. Thus, the probability of failure due to flood intrusion given that a PMF has occurred is equal to:

$$P_{f1} = P[\text{Flood intrusion}]$$

A.5.1 Failure Mode 1. Failure Due to Flooding

The first concept is the possibility of flood intrusion if the design flood were to be exceeded and the river or stream course in question

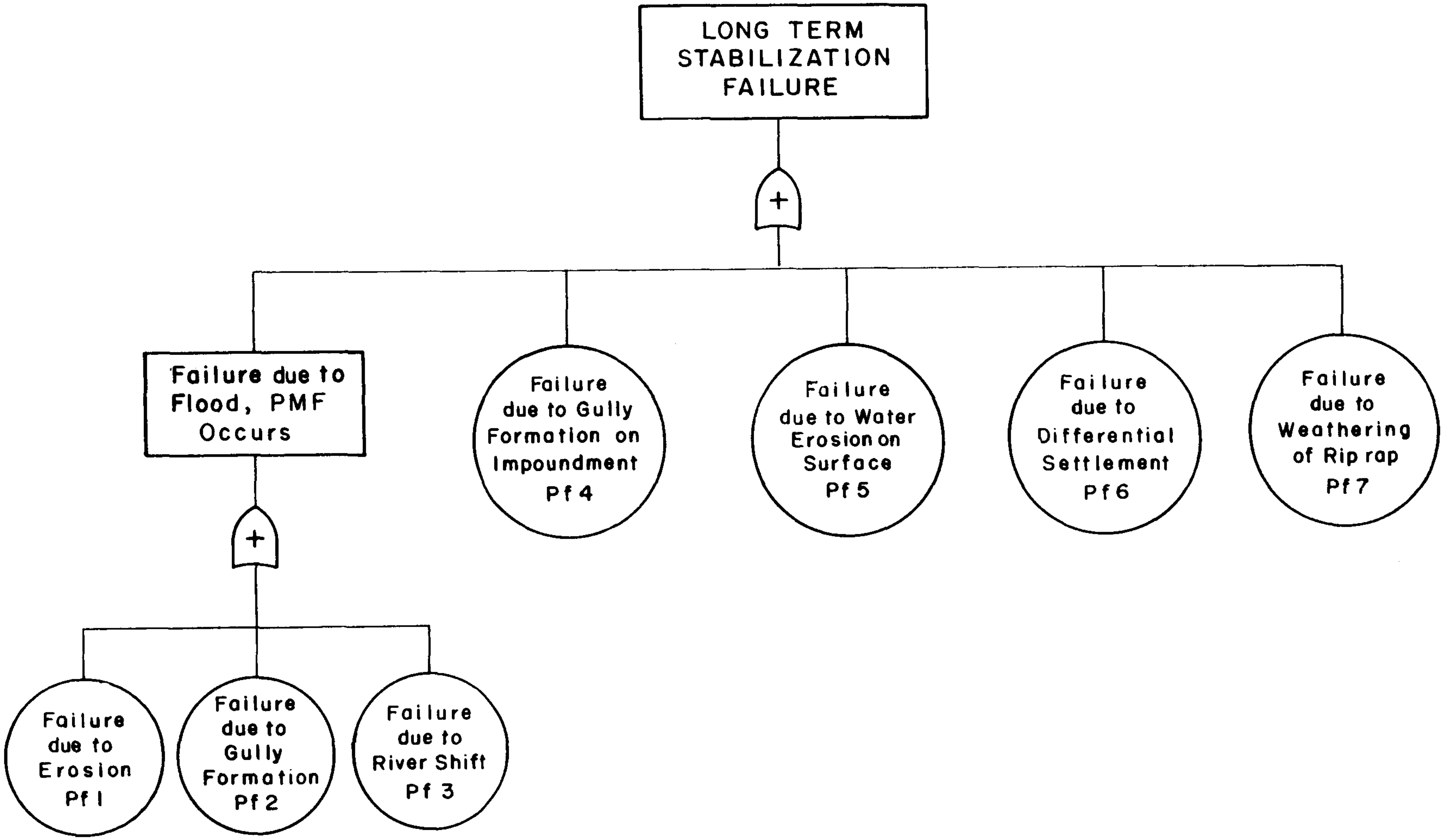


Fig. A.5. Fault tree for long-term stabilization failure based on failure modes. *Source:* Nelson et al., 1983.

remains in its present location. The second concept that must be considered is the geomorphic stability of the existing river course and the possibility that, over long-term design periods, the site may or may not remain geomorphologically stable (Nelson, et al. 1983).

Failure due to flooding can occur when:

(i) The PMF causes high water levels in streams in the vicinity of the tailings impoundment so that these overflow and erode the impoundment.
(ii) Gullies form in the landscape adjacent to the impoundment.
(iii) River shift occurs in the vicinity of the impoundment which can impact upon the impoundment.

The first failure mode should be reformulated for the case when the design is done on the basis of the PMF. In probabilistic terms it should be taken as the probability that the PMF will intrude upon the impoundment, i.e. the probability that the flood waters will leave the banks of a river and inundate portions of the impoundment located on the floodplain. Failure of the impoundment will take place if the erosional forces of the intruding flood are of sufficient magnitude to cause damage.

The flood magnitude used is the PMF, and is therefore, a deterministic value. One would therefore know (deterministically) whether the flood has intruded or not. This implies that the conditional probability of failure due to flood intrusion can be taken as the probability that erosional failure will take place.

Protection against erosional failure is designed so that the estimated flow velocity will not cause scour. Riprap design procedures are used. Riprap design methodologies were mostly developed on the basis of empirical observations and are therefore well suited for deterministic design where a "number" is required. It is clear that there must be considerable variation in the capacity function for riprap and uncertainty is therefore built into the design, although the magnitude is never stated. These are unknowns and must be investigated further to obtain a reasonable estimate of probability of failure. Only when an "acceptable" probability of failure is used can the factor of safety be selected for the design.

The main task is to develop the capacity function for each of the riprap design procedures discussed in the main body of this report.

The second potential failure mechanism due to flood intrusion is gully formation. Gully erosion may lead to tailings impoundment failure in two possible ways. First, gullies could form at a considerable distance downstream from a tailings impoundment and eventually migrate upstream until they intrude upon the impoundment area. Second, gullies could form within the vicinity of the impoundment itself and result in a similar failure mode. Because gully erosion is usually rapid and progressive, it is essential to prevent gully initiation to assure long-term stability of an area.

The probability of failure due to gully formation can therefore be taken as the probability that a gully will form. It is proposed that the plot of critical slope of flow to width (slope-width) ratio vs. drainage basin area presented in Nelson, et al. (1983) be used as a basis for the analysis. This plot (Fig. A.6) establishes a geomorphic threshold zone separating ungullied conditions from gullied conditions.

The scatter in data on Figure A.6 clearly illustrates the existence of variability. Consider now the dashed line as an "average" line, i.e. a distribution about this line will show that 50% of the time gullying will take place and 50% of the time it will not. For any given basin area then there will be a distribution of the slope-width ratio about this mean value, as is shown for 1 sq km in Figure A.6. Assume for now that this distribution is normal. A mean value and a coefficient of variation for this distribution can be obtained from the original data. The probability of failure is then the probability that the slope-width ratio exceeds a certain value. Starting with a probability of failure, one can therefore obtain an allowable factor of safety for design purposes.

Flood intrusion can also take place due to river shift, the third potential failure mechanism. Although a mill tailings site may be located some distance from a river, if the site is on a flood plain or on a low terrace, potential river shift could lead to direct river attack on the site and to increased flood damage. The primary concern with regard to the possibility of river intrusion would be lateral movement of the stream channel causing undermining or erosion of the tailings impoundment. Thus, if there is evidence of historical river shift at the site or at locations upstream or downstream from the site, potential for channel shift must be carefully evaluated on the basis of the available geomorphic evidence (Nelson, et al. 1983).

River channel classifications considering the relative stability and types of changes encountered with each channel pattern are shown in Figure A.7 (Nelson, et al., 1983). Significant engineering judgment will be required to predict possible changes in channel pattern over the time period, say 1000 years, for which the design is made. However, if the channel width is taken as a variable with estimated values of mean and coefficient of variation, probabilities can be obtained for the overall river width due to shift exceeding some value. This would be the probability of failure if the erosional forces of the river flow are sufficiently high to cause failure.

A.5.2 Failure Mode 2. Gully Formation on Impoundment Surface

The methodology described in the main body of this report can be used in a method similar to that described above to evaluate the most reasonable factor of safety for this failure mode.

A.5.3 Failure Mode 3. Water Erosion on Impoundment Cover

It was suggested that to protect a cover against surface erosion, the Unified Soil Loss Equation (USLE) be used, and a factor of safety be

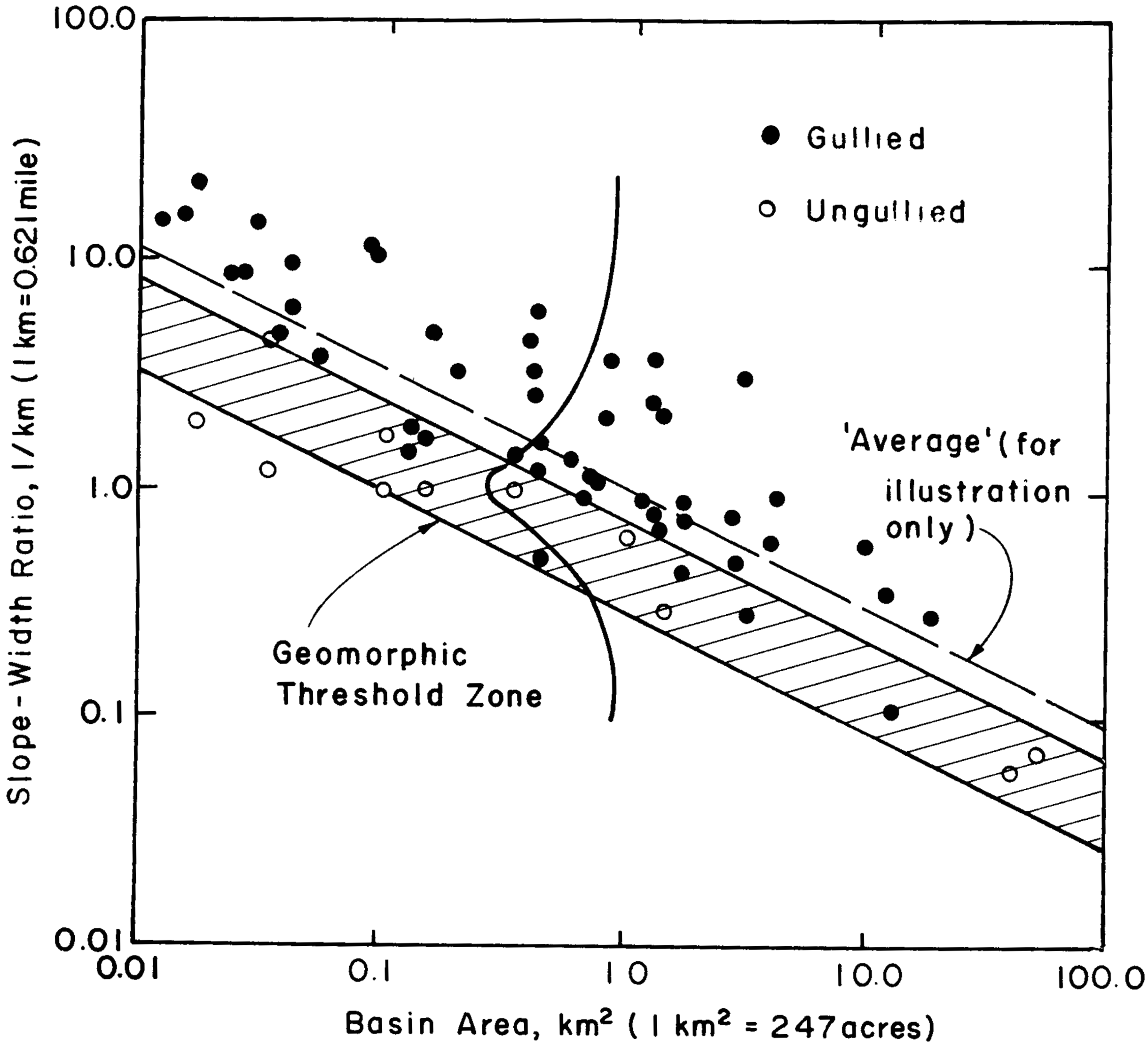


Fig. A.6. Plot of critical slope-width ratio vs drainage basin area, illustrating the geomorphic threshold zone separating ungullied area from gullied area. *Source:* after Nelson et al., 1983.

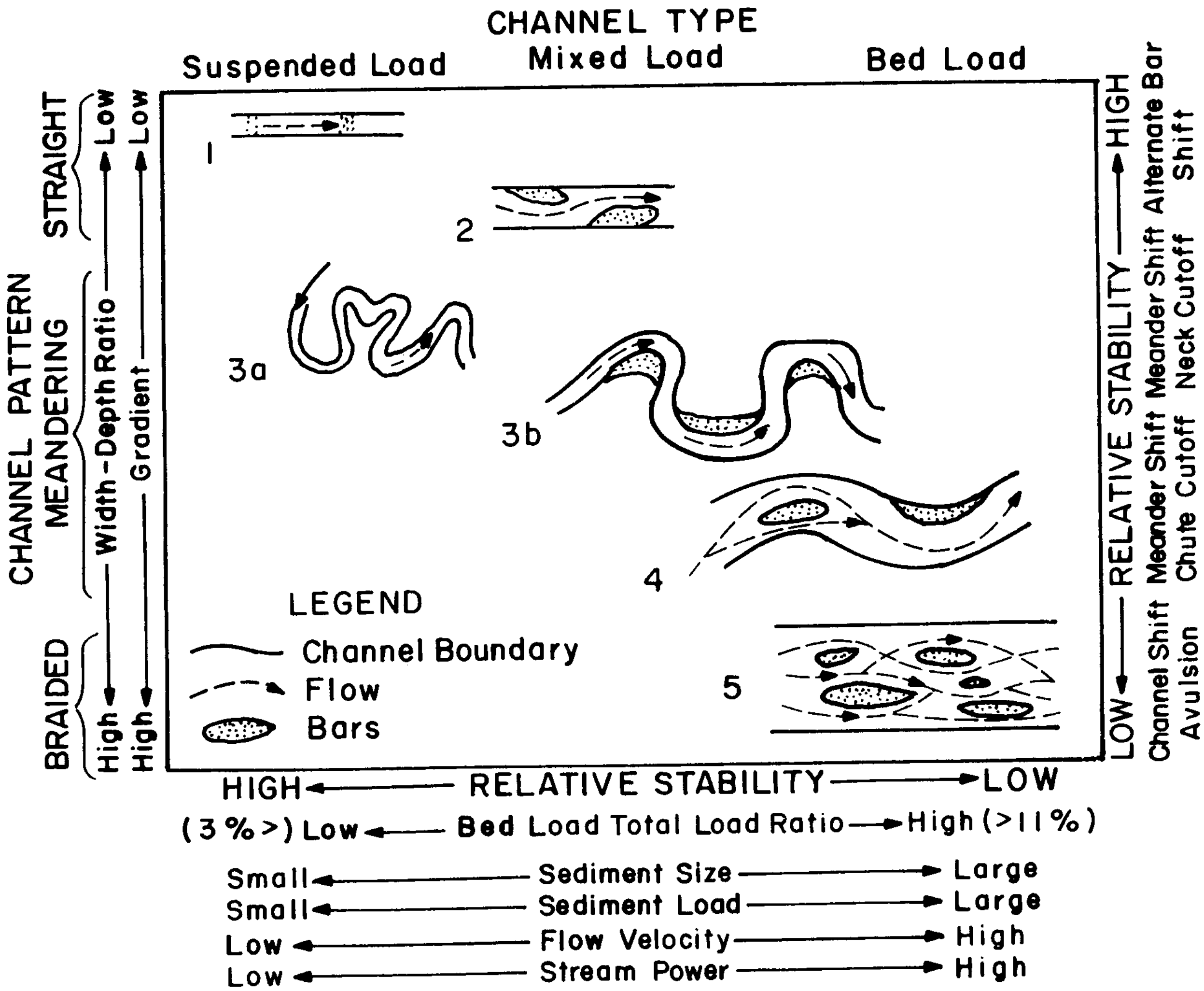


Fig. A.7. Channel classification showing relative stability and types of changes encountered with each channel pattern.

applied to protect this surface. This approach will guard only against sheet erosion.

A similar approach as that used above for the evaluation of riprap is proposed. The variability of the capacity function must be developed to select an allowable factor of safety. The factor of safety is then multiplied by the cover design thickness obtained from the USLE. Because of uncertainties in the application of the USLE a relatively large coefficient of variation is expected, e.g. equal to or larger than 30%.

A.5.4 Failure Mode 4. Differential Settlement

Differential settlement of the cover can lead to failure. The main task will again be to develop the capacity function, i.e. the capacity of the soil to resist cracking. For this failure mode it is also possible to develop the variability of the demand function, i.e. the variability in expected tailings settlement. The capacity-demand model demonstrated in Figure A.2 can then be used.

A.5.5 Failure Mode 5. Weathering of Riprap

This failure mode is the most difficult to evaluate quantitatively. However, it is suggested that qualitative approaches be used to evaluate the relative risk of riprap weathering.

A.6 SUMMARY AND RECOMMENDATIONS

This appendix presented some principles for the probabilistic risk assessment of long term stabilization of uranium mill tailings impoundments. A complete risk assessment must address the following: hazard identification, hazard evaluation, risk evaluation and risk reduction/response. There are still considerable difficulties evaluating all these aspects quantitatively and the use of subjective probabilities is often required. The major thrust of this appendix was therefore to demonstrate in principle the methodologies which can be used in a probabilistic risk assessment to help select allowable factors of safety for the various failure modes. The quantitative approaches to most all of the failure modes are based on empirical data showing considerable scatter. "Safe" and "unsafe" regions are often indicated by a best estimate line based on the empirical data. The probabilistic approach outlined here can assist the designer to select an allowable factor of safety instead of having only a division between safe and unsafe conditions.

The approach demonstrated is valid for general design and may assist the designer in some decisions. However, the large variability of natural processes and imperfect knowledge about these make it very difficult to define the failure modes, waste transport modes and impacts on populations at risk quantitatively in a formal risk assessment. It is therefore not considered feasible to perform a probabilistic risk assessment for long term stabilization of uranium mill tailings with the same level of certainty as has been done for nuclear reactors (NRC, 1984). The approach is very useful for evaluating and comparing reclamation schemes. Although the general engineering communities should be encouraged to adopt a reliability based approach, it is recognized that some stumbling blocks may remain which precludes its general application at this time. It must be emphasized that the general principles could be used in design decisions without applying a complete probabilistic risk assessment.

APPENDIX B. RIPRAP SAMPLING, TESTING, AND REPORTING

B.1 SAMPLING

Sampling can often be a weak link in the chain of investigative procedures. Thus, it must be carefully performed by qualified, experienced personnel.

The sample size should be at least 275 Kg (600 pounds). If the material quality is quite variable, three samples which represent respectively the poorest, medium, and best quality material available should be obtained. The minimum size of individual fragments selected should be at least the d_{50} design size or 20 cm (8 inches) in diameter, whichever is less. An estimate of the relative percentages of each material quality should be made and included as information relating to the source. Samples from undeveloped sources must be very carefully chosen so that the material selected will, as far as possible, be typical of the deposit and include any significant rock-type variations.

Representative samples may be difficult to obtain. Overburden may limit the area from which material can be taken and obscure the true character of a large part of the deposit. Surface outcrops wll often be more weathered than the interior of the deposit. Samples obtained from loose rock fragments on the ground or collected from weathered outer surfaces of rock outcrops are seldom representative. Fresh material may be obtained by breaking away the outer surfaces, or by trenching, blasting, or core drilling. In stratified deposits such as limestones or sandstones, vertical and horizontal uniformity must be evaluated as strata often differ in character and quality.

The dip of stratified formations must also be considered. Strata inclination with respect to surface slope will expose different strata at the surface in different parts of the area. Attention should be directed to the possibility of zones or layers of undesirable material. Clay or shale seams may be so large or prevalent as to require selective quarrying or excessive wasting of undesirable material.

B.2 TESTING

Quality evaluation investigations on representative samples submitted from the field include detailed petrographic examination and physical properties tests.

B.2.1 Petrographic Examination

The pieces of rock comprising the sample are examined individually and different rock facies and rock types, if present, are segregated. Size range is described and characteristic fragment shape studied, particularly to determine if the fragment shape is determined by joints, fractures, or shear. Surface weathering and secondary deposits of alkali salts or clay, are noted. Fracture or vein systems are described as well as the ease with which fractures or veins can be opened. Hardness, toughness, or brittleness, and visible void or pore characteristics, with their variations, are noted. Rock pieces representative of the various facies and rock types may be selected for detailed petrographic examination. The texture, internal structure, and mineralogy of the various rock facies and rock types are determined. Special attention is given to internal voids and fractures, and to the nature of cementing material in sedimentary rocks. Thin-section studies are made as required. The petrographic data are included in their entirety in the final materials report.

For freeze-thaw durability testing, 7.3 cm (2.875 inches) rock cubes are sawed from rock fragments selected to represent the poorest, medium, and best quality rock (for each facies or rock type) on the basis of visual inspection. The actual number of rock cubes tested may vary from sample to sample. After the rock cubes have been obtained, they are weighed in ovendry condition and photographed. The cubes are immersed in water for 24 hours, saturated surface dry weights and weights in water obtained, and wet bulk, dry bulk, and apparent specific gravities determined. The cubes are then inserted in 7.6 cm (3 inches) square rubber sheaths, sufficient water is added to cover the specimens, and the rubber sheaths containing the specimens are placed in automatically controlled freezing and thawing cabinets where the cubes are alternately frozen and thawed at the rate of 50 cycles per week by circulating calcium chloride brine around the sheaths. Each cycle consists of 1 1/2 hours freezing at -12^{o} C (10^{o} F) and 1 1/2 hours of thawing at 23^{o} C (70^{o} F). Throughout the tests, the appearance and manner of deterioration of the cubes are noted. Termination of the test is 250 cycles or when the rock fails (failure criterion is 25 percent weight loss), whichever is sooner. Type of failure - splitting or crumbling - is noted, photographs taken, and weight loss determined. Weight loss (in percent) is computed as difference in oven-dry weight between the largest piece of the cube remaining after testing and original oven-dry weight of the cube. The weight of material lost by splitting of the rock cube along fractures, seams, and bedding planes is considered weight loss and appropriate notation is made to aid in obtaining minimum-size riprap required by specifications.

Material remaining after petrographic examination of rock samples (excluding any pieces selected for more detailed petrographic analysis and freeze-thaw durability tests) is crushed, separated into 1 1/2- to 3-inch, 3/4- to 1 1/2-inch, 3/8- to 3/4-inch, and No. 4 to 3/8-inch-size fractions, and representative samples obtained for further physical properties tests.

B.2.2 Physical Properties

Samples consisting of different rock types or radical facies changes should be tested and examined separately. Physical properties tests performed are: (1) specific gravity, (2) absorption, (3) sodium sulfate soundness, and (4) Los Angeles abrasion.

B.2.2.1 Specific Gravity and Absorption

The specific gravity of riprap (crushed to 1 1/2-inch maximum size) is determined by washing the sample to remove dust and other coatings from the surface of the particles, drying to a constant weight and immersing in water at room temperature for 24 hours, blotting with a towel, and weighing. After weighing, the material is placed in a wire basket and is weighed again in water having a temperature of 23° C (73.4° F). The sample is then dried to a constant weight in an oven, cooled to room temperature, and weighed again. If A is the weight of the oven-dried sample in air, B the weight of the saturated, surface-dried sample, and C the weight of the sample in water, then:

the specific gravity on a dry basis = $\frac{A}{B - C}$

the specific gravity on a saturated, surface-dry basis = $\frac{B}{B - C}$;

the absorption on a dry basis = $\frac{B - A}{A}$;

and the absorption on a saturated, surface-dry basis = $\frac{B - A}{B}$.

Absorption is usually expressed as a percentage. ASTM Designation C 127-68 describes the detailed procedures for these tests.

B.2.2.2 Abrasion

This test determines the abrasion resistance of crushed rock and natural and crushed gravel. The Los Angeles abrasion machine, which consists of a hollow steel cylinder closed at both ends is used. The cylinder has a diameter of 71 cm (28 inches) and a length of 51 cm (20 inches). The abrasive charge consists of cast-iron or steel spheres approximately 4.7 cm (1.9 inches) in diameter.

The test sample of 5,000 grams and the proper abrasive charge are placed in the Los Angeles abrasion testing machine, and the machine is rotated for 100 revolutions at about 30 rpm. The material is then removed from the machine and, screened on a No. 12 sieve. The material retained on the screen is weighed. The entire sample including the dust of abrasion is returned to the testing machine and rotated an additional 400 revolutions. The screening and weighing are repeated. The differences between the original weight of the test sample and the weight of the material retained on the screen at 100 revolutions and at 500 revolutions are expressed as percentages of the original weight of the test sample. These values are reported as percentages of wear. ASTM Designation C 131-69 describes detailed procedures for this test.

B.2.2.3 Sodium Sulfate Soundness

The most commonly used soundness test is the sodium sulfate test. The results of this test are used as an indication of the ability of riprap to resist weathering. A carefully prepared saturated solution of sodium sulfate is kept at a temperature of 21° C (70° F). After washing and drying in an oven, the material to be tested is screened to provide a specified gradation, usually in the range from 3.8 cm (1.5 inches) to the No. 50 sieve size. Specified weights of the various grades of the material are placed in separate containers resistant to the action of the solution, and sufficient sodium sulfate is poured into the containers to cover the samples. The material is permitted to soak for not less than 16 hours or more than 18 hours, during which the temperature is maintained at 21° C (70° F).

After the immersion period, the samples are removed from the solution and dried to constant weight (about 4 hours) at a temperature of 105° to 110° C (221° to 230° F). After drying, the sample fractions are cooled to room temperature and the process is repeated. At the end of five cycles, the test sample is inspected and observations are recorded. Each fraction is then washed thoroughly to remove the sodium sulfate from the material, and is dried and cooled. Each fraction is screened and the quantities of material retained are weighed. The weighted average loss for each fraction is computed and reported. ASTM Designation C 88-69 describes the detailed procedure for this test.

B.3 REPORTING

Reporting of information and data accumulated during any investigation stage is most important. Although detailed information requirements increase with each successive stage, adequate information must be available by the feasibility stage to develop realistic cost estimates and properly select sources for possible use. For feasibility studies, the designers should have sufficient information to supplement laboratory test data to determine whether other types of embankment protection should be considered. A suggested outline for riprap reports for rock obtained from a quarry is as follows:

a. Ownership
b. Location, indicated by map, with reference to section, township, and range
c. General description
d. Geologic type and classification
e. Joint spacing and fracture systems
f. Bedding and planes of stratification
g. Manner and sizes in which rock may break on blasting as affected by jointing, bedding, or internal stresses
h. Shape and angularity of rock fragments
i. Hardness and density of rock
j. Degree of weathering
k. Any abnormal properties or conditions not covered above
l. Thickness, extent, estimated volume, and average depth of deposit
m. Type, extent, and thickness of overburden
n. Accessibility (roads, giving distance, load limitations, required maintenance, whether privately owned, and other pertinent information)

o. Photographs and any other information which may be useful or necessary.

If commercial quarry deposits are considered, the following information should be obtained and included in the report:

a. Name and address of plant operator - if quarry is not in operation, a statement relative to ownership or control
b. Location of plant and quarry
c. Age of plant (if inactive, approximate date when operations ceased)
d. Transportation facilities and difficulties
e. Extent of deposit, plant capacty, and stockpile size
f. Plant description (type and condition of equipment for excavating, transporting, crushing, classifying, loading) and operating restrictions, if any
g. Approximate percentages of various sizes of material produced by the plant
h. Location of scales for weighing shipments
i. Approximate prices of materials at the plant
j. Principal users of plant output
k. Service history of material produced
l. Any other pertinent information.

APPENDIX C. SAMPLE CALCULATIONS FOR CHAPTER 6, SELECTION OF RIPRAP

C.1 SUITABILITY TESTS, OVERSIZING METHODOLOGY, AND OPTIONS

C.1.1 Petrographic Analysis, Durability Test Results, and Original Design d_{50}

a. Analysis and Tests

1. Rock Type: Fossiliferous fine-grained limestone
2. Weathering and Fractures: Bedding and joints range from tightly closed to open. Rocky size fragments controlled by joints often less than 3 inches apart, trace amounts of smecktite clay minerals are present. Available rock size d_{50} = 5.5 inches
3. Specific Gravity: 2.71
4. Absorption: 0.2%
5. Sodium Sulfate Soundness: 9.7% (5 cycles)
6. Freeze-thaw: Not tested
7. Los Angeles Abrasion: 9.8% (100 revolutions)

b. Original Design d_{50} = 5.0 inches

C.1.2 Sample Calculations

	a. Maximum Possible Score	b. Actual Score
	$Qm = \sum_{i=1}^{7} N_i W_i$	$Q = \sum_{i=1}^{7} N_i W_i$
$N_1 \cdot W_1$ =	3 • 1.00	1 • 1.00
$N_2 \cdot W_2$ =	3 • 1.00	2 • 1.00
$N_3 \cdot W_3$ =	3 • 1.00	3 • 1.00
$N_4 \cdot W_4$ =	3 • 0.75	3 • 0.75
$N_5 \cdot W_5$ =	3 • 0.75	3 • 0.75
$N_6 \cdot W_6$ =	0	0
$N_7 \cdot W_7$ =	3 • 0.50	2 • 0.50
	Qm = 15.00	Q = 10.75

Note: Refer to Table 6.4 for values of N_1 and N_2; Table 6.2 for values of N_3 through N_7; and Table 6.3 for values of W_1 through W_7.

C.1.3 Percent of Maximum Score

$$\frac{Q}{Qm} \times 100 = \frac{10.75}{15.00} \times 100 = 72\%$$

C.1.4 Tentative Suitability

1. Frequently saturated areas

$$\frac{Q}{Qm} \geq 80\%$$

2. Occasionally saturated areas

$$\frac{Q}{Qm} \geq 65\%$$

3. Seldom Saturated Areas

$$\frac{Q}{Qm} > 50\%$$

4. Result of Sample Calculations

$$\frac{Q}{Qm} = 72\%$$

Implies that the sample rock is not suitable for use in frequently saturated areas but may be used in occasionally or seldom saturated areas.

C.1.5 Limitations

1. Crushed limestone would have been required between riprap blocks had this material been judged as suitable in frequently saturated areas.

2. Size Limitation: Rock d_{50} > original design d_{50} (5.5 inches > 5.0 inches) but oversizing may be required in occasionally saturated areas.

C.1.6 Oversizing Calculations

(A viable alternative because of the absence of smecktite clay minerals)

1. Frequently saturated areas
(Already disqualified; see C.1.4, above. Oversizing calculation carried out only for illustrative purposes)

$$S = (10)\ \frac{T}{D} = (10)\frac{9.7}{1.3} = 75\%$$

Modified design d_{50} = 5 x 1.75 = 8.75 inches

Implies Available rock d_{50} << modified design d_{50}
(5.5 inches << 8.75 inches)

Implies Considerable screening would be required to bring rock size up to design specifications. Modified design d_{50} of 8.75 inches may not be achievable.

2. Occasionally saturated areas

$$S = (2)\ \frac{T}{D} = (2)\frac{9.7}{1.3} = 15\%$$

Modified design d_{50} = 5 inches x 1.15 = 5.75 inches

Implies Available rock d_{50} < modified design d_{50} but only by a narrow margin (5.5 inches < 5.75 inches).

C.1.7 Options

The licensee may be able to attain modified design specifications by one of three alternatives.

Alternative 1: Light to moderate screening could bring D_{50} up to modified design specifications.

Alternative 2: Discard the results of the sodium sulfate soundness test and perform a freeze-thaw test to determine the need and magnitude of oversizing.

Alternative 3: Abandon the attempt to use this riprap source on occasionally saturated slopes. Use on seldom saturated slopes where no oversizing is necessary. Seek another riprap source for use on occasionally saturated slopes.

C.2 OVERTHICKENING A RIPRAP BLANKET CONSISTING OF HETEROGENEOUS BOULDER CONGLOMERATE

C.2.1 Assumptions

1. Material is suitable for use in seldom saturated areas.
2. Design thickness (T_D) of riprap blanket = 6 inches.

C.2.2 Durability Test Results

Rock Size (inch)	Sample Grade by Weight %	Resistance to Hammer Blows	Na_2SO_4 Soundness Test Weight Loss %
1.00 to 1.75	5	Poor	42
1.75 to 2.25	14	Poor	33
2.25 to 3.25	41	Fair	15
3.25 to 4.25	40	Fair	10

C.2.3 Calculations

To find: 1) Percentage of acceptable material (D_p)
2) Modified Design Thickness (T_{MD})

$D_p = 81\%$

$$\frac{T_D}{T_{MD}} = 0.81$$

$$T_{MD} = \frac{T_D}{0.81} = \frac{6}{0.81} = 7.5 \text{ inches}$$

C.2.4 Options

1. Construct riprap blanket to a thickness of 7.5 inches.

2. Separate poorer quality, fine-grained material by screening. Construct riprap blanket to a thickness of 6.0 inches.

9. REFERENCES

Abt, S. R. and Ruff, J. F., 1978. Three Procedures for Estimating Erosion from Construction Areas. Uranium Mill Tailings Management, Vol. II, Civil Engineering Dept., Colorado State University, Fort Collins, Colorado, November 1978.

Abt, S. R. and Ruff, J. F., 1985. Preliminary Results of Interstitial Flow and Overtopping Flow Tests on Riprap. Presented to the Nuclear Regulatory Commission, Silver Spring, Maryland, November 21, 1985.

Ackers, P. and Charlton, F. G., 1971. The Shape and Resistance of Small Meandering Channels. Inst. Civil Engin. Proc. Suppl., Paper 73625.

AMS, 1959. Glossary of Meteorology, American Meteorological Society, Boston, Massachusetts.

Anderson, A. G., Paintal, A. S. and Davenport, J. T., 1970. Tentative Design Procedure for Riprap Lined Channels. NCHRP Report No. 10.

Ang, A. H-S. and Tang, W. H., 1984. Probability Concepts in Engineering Planning and Design, Decision, Risk and Reliability, John Wiley and Sons, New York.

ANSI, 1985. Catalog of American National Standards, 1985. Standard ANSI/ANS 2.8-1981, Flooding at Power Reactor Sites, Determining Design Basis. American National Standards Institute, Inc., New York.

ASCE, 1970. Design and Construction of Sanitary and Storm Sewers. American Society of Civil Engineers and the Water Pollution Control Federation, 345 E. 47th St., New York, New York.

Barnes, H. H., 1967. Roughness Characteristics of Natural Channels. Geological Survey Water-Supply Paper 1849.

CDPH, 1970. Bank and Shore Protection in California Highway Practices. California Division of Highways, Department of Public Works, Sacramento, California.

Chow, V. T., 1959. Open-Channel Hydraulics, McGraw-Hill Book Company.

Chow, V. T., 1964. Handbook of Applied Hydrology, McGraw-Hill.

Clyde, G. G., Israelson, C. E., Packer, P. E., Farmer, E. E., Fletcher, J. E., Israelson, E. K., Haws, F. W., Rao, N. F. and Hansen, J., 1978. Manual on Erosion Control Principles and Practices, Hydraulics and Hydrology Series Report H-78-002, Utah Water Research Laboratory, June 1978.

COE, 1970. Hydraulic Design of Flood Control Channels, EM 1110-2-1601, U.S. Army Corps of Engineers, July 1970.

COE, 1971. Engineering and Design, Additional Guidance for Riprap Channel Protection, ETL-1110-2-120, Office of the Chief of Engineers, U.S. Army Corps of Engineers, May 1971.

COE, 1974. Flood Hydrograph Package, HEC-1, No. 72-010, U.S. Army Corps of Engineers.

COE, 1975. National Program of Inspection of Dams, (see Appendix D), U.S. Army Corps of Engineers, Washington, D.C., 1975.

COE, 1982. Water Surface Profiles, HEC-2, No. 723-02A, U.S. Army Corps of Engineers.

DePuy, G. W., 1965. Petrographic Investigation of Rock Durability and Comparisons of Various Test Procedures, Association of Engineering Geologists, Vol. 2(2).

DePuy, G. W. And T. F. Ensign, 1965. Progress Report - Riprap Testing Research Comparison of an Ultrasonic Cavitation Test with Petrographic and Physical Tests to Evaluate Rock Durability, Bureau of Reclamation Report No. ChE-23, U.S. Dept. of Interior, Denver, Colorado.

DOI, 1974. Earth Manual, Bureau of Reclamation, U.S. Department of Interior.

DOI, 1975. Engineering and Design Manual for Coal Refuse Disposal Facilities, U.S. Department of Interior.

DOI, 1977. Design of Small Dams, Bureau of Reclamation, U.S. Department of Interior.

DOI, 1978. Hydraulic Design of Stilling Basins and Energy Dissipators, Engineering Monograph No. 25. Bureau of Reclamation, U.S. Department of Interior.

DOI, 1985. Open-File Data at the Bureau of Reclamation, U.S. Department of Interior, Federal Center, Denver, Colorado.

Dreith, R. H., 1982. An Industry's Guidelines for Risk Assessment. Published in Risk Assessment at Hazardous Waste Sites, Am. Chem. Soc. Symp. Series 204, F. A. Long and G. E. Schweitzer (eds).

EPA, 1983. Standards for Remedial Actions at Inactive Uranium Sites, Environmental Protection Agency, Federal Register, Vol. 48, No. 3, January 1983.

Esmiol, E. E., 1968. Rock as Upstream Slope Protection for Earth Dams - 149 Case Histories, Bureau of Reclamation, U.S. Dept of Interior, Denver, Colorado.

Falk, J. A., Abt, S. R. and Nelson, J. D., 1985. Prediction of Gully Incision on Reclaimed Tailings Slopes, Report No.: CER85-86JAF-SRA-JDN6, Submitted to the U.S. Nuclear Regulatory Commission, September 1985.

FEMA, 1984. Standard Workshop on Risk-Based Approach to Dam Safety Assessment, sponsored by Federal Emergency Management Agency in cooperation with Department of Civil Engineering, University of Colorado at Denver, January 9-10, 1984, Denver, Colorado.

Foley, M. G., Kimball, C. S., Myers, D. A., and Doesburg, J., 1985. The Selection and Testing of Rock Riprap for Uranium Tailings Impoundments (Draft), NUREG/CR-3747, U.S. Nuclear Regulatory Commission, Washington, D.C.

Fortier, S., and Scobey, F. C., 1926. Permissible Canal Velocities, Transactions, ASCE, Vol. 99.

Hanegan, 1984. Is That Rock Source as Durable as it Looks? Association of Engineering Geologists Newsletter, Vol. 27, No. 3.

Harshman, E. N., 1972. Geology and Uranium Deposits, Shirley Basin Area, Wyoming, U.S. Geological Survey Professional Paper 745, Denver, Colorado.

Hirshfeld, R. C. and Poulos, S. J., 1973. Embankment-Dam Engineering, John Wiley & Sons.

Jahns, R. H., 1982. Weathering and Long-Term Durability of Rock Masses, Appendix to Long-Term Survivability of Riprap for Armoring Uranium Mill Tailings and Covers: A Literature Review, NUREG/CR-2642 (PNL-4225), U.S. Nuclear Regulatory Commission, Washington, D.C..

Khan, H. R., 1971. Laboratory Study of River Morphology, Ph.D. Dissertation, Colorado State University.

Kirpich, Z. P., 1940. Time of Concentration of Small Agricultural Watersheds, Civil Engineering, Vol. 10, No. 6, June 1940.

Knapp, M. R., 1985. PMP Rainfall Duration Intensities for Rock Armor Design, Memorandum position document, U.S. Nuclear Regulatory Commission, Silver Spring, Maryland, September 30, 1985.

Lane, E. W., 1955. "Design of Stable Channels, Transactions, ASCE, Vol. 120.

Lane, E. W., 1957. A Study of the Shape of Channels formed by Natural Streams Flowing in Erodible Material, M.R.D. Sediment Series No. 9, U.S. Army Corps of Engineers, Omaha, Nebraska.

Lane, E. W., and Carlson, E. J., 1953. Some Factors Affecting the Stability of Canals Constructed in Coarse Granular Material. Proceedings of the Minnesota International Hydraulic Convention, Minneapolis, Minnesota, September 1, 1953.

Leopold, L. B. and Wolman, M. G., 1957. River Channel Patterns: Braided, Meandering, and Straight, Professional Paper 282-B, U.S. Geological Survey.

Leps, T. M., 1973. Flow Through Rockfill, in Embankment Dam Engineering, John Wiley and Sons.

Lindsey, C. G., Long, L. W., and Begei, C. W., 1982. Long-Term Survivability of Riprap for Armoring Uranium Mill Tailings and Covers: A Literature Review, NUREG/CR-2642, U.S. Nuclear Regulatory Commission.

Lindsley, R. K., Kohler, M. A., and Paulhus, J. L., 1958. Hydrology for Engineers, McGraw-Hill Book Company.

Lutton, R. J., Houston, B. J., and Warriner, J. B., 1981. Evaluation of Quality and Performance of Stone as Riprap or Armor, U.S. Army Engineer Waterways Experiment Station Technical Report GL-81-8, Vickburg, Mississippi.

Mavis, F. T., and Laushey, L. M., 1948. A Reappraisal of the Beginnings of Bed Movement-Competent Velocity, Proceedings of the International Association for Hydraulic Structures Research, Stockholm, Sweden.

Morris, H. M. and Wiggert, J. M., 1972. Applied Hydraulics in Engineering, 2nd Edition, Wiley and Sons, Inc.

Mosley, M. P., 1971. An Experimental Study of Rill Erosion, M.S. Thesis, Colorado State University, Fort Collins, Colorado.

Nelson, J. D., Volpe, R. C., Wardwell, R. E., Schumm, S. A., and Staub, W. P., 1983. Design Considerations for Long-Term Stabilization of Uranium Mill Tailings Impoundments, NUREG/CR 3397, U.S. Nuclear Regulatory Commission, October 1983.

Nelson, J. D., Abt, S. R., Volpe, R. L. and Staub, W. P., 1984. Uranium Mill Tailings Site Visits, October 1984.

NRC, 1983. Hydrologic Design Criteria for Tailings Retention Systems, Staff Technical Position W-8201, Uranium Licensing Branch, U.S. Nuclear Regulatory Commission, Washington, D.C., January 1983.

NRC, 1983. Overland Erosion of Uranium Mill Tailings Impoundments: Physical Processes and Computational Methods, NUREG/CR-3027, U.S. Nuclear Regulatory Commission.

NRC, 1984. Effects of Rock Riprap Design Parameters on Flood Protection Costs for Uranium Tailings Impoundments, NUREG/CR-3751, U.S. Nuclear Regulatory Commission, Washington, D.C.

NRC, 1984. Probabilistic Risk Assessment (PRA) Reference Document, NUREG-1050, U.S. Nuclear Regulatory Commission, September, 1984.

NRC, 1984. Design of Long-Term Erosion Protection Covers for Reclamation of Uranium Mill Sites, Draft Report, U.S. Nuclear Regulatory Commission.

NRC, 1985. Meeting with National Weather Service to Discuss PMP, Memorandum source document, Codell, R. and Johnson, T. C., U.S. Nuclear Regulatory Commission, Silver Spring, Maryland, August 1985.

NWS, 1980. Comparison of Generalized Estimates of Probable Maximum Precipitation with Greatest Observed Rainfalls, National Weather Service Technical Report NWS 25, NOAA, U.S. Dept. of Commerce, March 1980.

NWS, 1956, 1966, 1977, 1979, and 1984. Hydrometeorological Report Nos. 33 (1956), 43(1966), 49(1977), 51(1979), and 55(1984), National Weather Service, NOAA, Dept. of Commerce.

Nyhan, J. W. and Lane, L. J., 1983. Erosion Control Technology: A User's Guide to the Use of the Universal Soil Loss Equation at Waste Burial Sites, Los Alamos National Laboratory, Los Alamos, New Mexico.

Powledge, G. R. and Dodge, R. A., 1985. Overtopping of Small Dams - An Alternative for Dam Safety, presented at the ASCE Seminar on Roller Compacted Concrete conducted in Boise, Idaho, April 11, 1985.

Richardson, E. V., Simons, D. B., Karak, S., Mahmood, K. and Stevens, M. A., 1975. Highways in the River Environment-Hydraulics and Environmental Design Considerations, U.S. Department of Transportation, Washington, D.C.

Saltzman, U., 1975. Rock Quality Determination for Large-Size Stone Used in Protective Blankets, Ph.D. Dissertation, Purdue University, Lafayette, Indiana.

Schumm, S. A., and Khan, H. R., 1972. Experimental Study of Channel Patterns, Geol. Soc. Am. Bull., Vol. 83.

Schumm, S. A., and Chorley, R. J., 1983. Geomorphic Control on the Management of Nuclear Waste, NRC Monograph, U.S. Nuclear Regulatory Commission, Washington, D.C.

Schumm, S. A., 1977.. The Fluvial System, Wiley, New York.

SCS, 1975. Standards and Specifications for Soil Erosion and Sediment Control in Developing Areas, Soil Conservation Service, U.S. Department of Agriculture.

SCS, 1977. Preliminary Guidance for Estimating Erosion on Area Disturbed by Surface Mining Activities in the Interior United States, Soil Conservation Service, U.S. Department of Agriculture, July 1977.

SCS, 1984. Engineering Field Manual, U.S. Department of Agriculture, Soil Conservation Service.

Seelye, E. E., 1960. Design, 3rd Ed., Vol. 1, John Wiley and Sons, Inc., New York, New York.

Sharp, W. N. and Gibbons, 1964. Geology and Uranium of the Southern Part of the Powder River Basin, Wyoming, U.S. Geological Survey Bulletin 1147-D, Denver, Colorado.

Shen, H. W. and Schumm, S. A., 1981. Methods for Assessment of Stream-Related Hazards to Highways and Bridges, Report FHWA-RD-80-160, Federal Highway Administration.

Shen, H. W. and Lu, J. Y., 1983. Development and Prediction of Bed Armoring, ASCE, J. of the Hyd. Division, Vol. 109, No. 4., April 1983.

Sherard, J. L., Dunnigan, L. P., and Talbot, J. R., 1984. Basic Properties of Sand and Gravel Filters, J. of Geotech, Eng. Div., ASCE, Vol. 110, No. 6, June 1984.

Shipe, A. P. and Riedel, J. T., 1976. Greatest Known Area 1 Storm Rainfall Depths for the Continguous United States, Technical Memorandum NWS Hydro-33, National Weather Service, NOAA, U.S. Department of Commerce.

Simons, D. B. and Senturk, F., 1977. Sediment Transport Technology, Water Resources Publications, Fort Collins, Colorado.

Soister, P. E., 1968. Stratigraphy of the Wind River Formation in South-Central Wind River Basin, Wyoming, U.S. Geological Survey Professional Paper 594-A, Denver, Colorado.

Stephenson, D., 1979. Rockfill in Hydraulic Engineering, Developments in Geotechnical Engineering, 27, Elsevier Scientific Publishing Company.

USAF, 1976. Surface Drainage Facilitation for Airfields and Heliports, Chapters 1 and 19, U.S. Air Force.

Vanoni, V. A., 1977. Manuals and Reports on Engineering Practice, No. 54, American Society of Civil Engineers.

Wagner, A. A., 1957. The Use of the Unified Soil Classification System by the Bureau of Reclamation, Proc. 4th Int. Conf. Soil Mech. Found. Eng., London, Vol. I.

Whitman, R. V., 1984. Evaluating Calculated Risk in Geotechnical Engineering, 17th Terzaghi Lecture, Journ. of Geotech. Eng., ASCE, Vol. 110, No. 2.

Wischmeier, W. H., Johnson, C. B., and Cross, B. V., 1971. A Soil Erodibility Nomograph for Farmland and Construction Sites, Journal of Soil and Water Conservation, Vol. 26, No. 5, September-October 1971.

Wischmeier, W. H. and Smith, D. D., 1978. Predicting Rainfall Erosion Losses - a Guide to Conservation Planning, U.S. Department of Agriculture.

NRC FORM 335
(2-84)
NRCM 1102,
3201, 3202

U.S. NUCLEAR REGULATORY COMMISSION

BIBLIOGRAPHIC DATA SHEET

SEE INSTRUCTIONS ON THE REVERSE.

1. REPORT NUMBER *(Assigned by TIDC, add Vol. No., if any)*
NUREG/CR-4620
ORNL/TM-10067

3. LEAVE BLANK

4. DATE REPORT COMPLETED
MONTH: May
YEAR: 1986

6. DATE REPORT ISSUED
MONTH: June
YEAR: 1986

8. PROJECT/TASK/WORK UNIT NUMBER

9. FIN OR GRANT NUMBER
B0279

11a. TYPE OF REPORT
Technical

b. PERIOD COVERED *(Inclusive dates)*

2. TITLE AND SUBTITLE

Methodologies for Evaluating Long-Term Stabilization Designs of Uranium Mill Tailings Impoundments

5. AUTHOR(S)

J.D. Nelson, S.R. Abt, R.L. Volpe, D. van Zyl, CSU
N.E. Hinkle, W.P. Staub, ORNL

7. PERFORMING ORGANIZATION NAME AND MAILING ADDRESS *(Include Zip Code)*
Colorado State University
Fort Collins, CO 80523
Under Contract to:
Oak Ridge National Laboratory
Oak Ridge, TN 37831

10. SPONSORING ORGANIZATION NAME AND MAILING ADDRESS *(Include Zip Code)*
Division of Waste Management
Office of Nuclear Material Safety and Safeguards
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

12. SUPPLEMENTARY NOTES

13. ABSTRACT *(200 words or less)*

Uranium mill tailings impoundments require long-term (200-1000 years) stabilization. This report reviews currently available methodologies for evaluating factors that can have a significant influence on tailings stabilization and develops methodologies in technical areas where none presently exist. Mill operators can use these methodologies to assist with (1) the selection of sites for mill tailings impoundments, (2) the design of stable impoundments, and (3) the development of reclamation plans for existing impoundments. These methodologies would also be useful for regulatory agency evaluations of proposals in permit or license applications.

Methodologies were reviewed or developed in the following technical areas: (1) prediction of the Probable Maximum Precipitation (PMP) and an accompanying Probable Maximum Flood (PMF); (2) prediction of the stability of local and regional fluvial systems; (3) design of impoundment surfaces resistant to gully erosion; (4) evaluation of the potential for surface sheet erosion; (5) design of riprap for protecting embankments from channel flood flow and overland flow; (6) selection of riprap with appropriate durability for its intended use; and (7) evaluation of oversizing required for marginal quality riprap.

14. DOCUMENT ANALYSIS – a. KEYWORDS/DESCRIPTORS

uranium mill
tailings impoundments

b. IDENTIFIERS/OPEN-ENDED TERMS

15. AVAILABILITY STATEMENT
Unlimited

16. SECURITY CLASSIFICATION
(This page) Unclassified
(This report) Unclassified

17. NUMBER OF PAGES

18. PRICE