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**UMTRA PROJECT**

180 HOWARD ST. SAN FRANCISCO, CA 94105

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10 September 1985

Mr. John D'Antonio  
Department of Energy  
c/o Jacobs Engineering Group, Inc.  
5301 Central Avenue N.E., Suite 1700  
Albuquerque, New Mexico 87108

Subject: UMTRA PROJECT - GEN

Review Comments on 4/4/85 - 5/7/85 Draft of "Methodology for  
Evaluating Long Term Stabilization Designs of Uranium Mill  
Tailings Impoundments", CSU Report to NRC

WM Red578 File

WM Project

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Distribution:

Ted Johnson w/enc. G. Grunwald  
LBH, JEM, H. Smith, w/enc. DGillen  
(Return to WM, 623 SS)

Sac

Dear Mr. D'Antonio:

Enclosed are my comments marked on a copy of the subject draft document, which was distributed at the 24 July 1985 meeting of Working Group 1 of the DOE, NRC, TAC and RAC in Denver, Colorado.

By copy of this letter, I am forwarding my comments to Berg Keshian of the TAC and Ted Johnson of the NRC.

Sincerely,

*G. R. Thiers*

Gerald R. Thiers  
Principal Engineer

GRT:kfb

cc w/enclosure:  
Berg Keshian  
Jim Oldham  
Ted Johnson

Enclosure: Marked copy of 4/4/85 - 5/7/85 Draft of "Methodology for  
Evaluating Long Term Stabilization Designs of Uranium Mill  
Tailings Impoundments", CSU Report to NRC.

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Methodology for Evaluating Long Term Stabilization Designs  
of Uranium Mill Tailings Impoundments

1. Introduction

Design considerations for long term stability 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 that regardless of the stability period the appropriate design flood would be the PMF. The design of various elements of protection systems for the different failure modes should therefore be based on the PMF. *where? Not in this report.*

At the end of that investigation it was noted that the successful application of the results to the evaluation of designs for the uranium mill tailings impoundment reclamation schemes would require the development of a methodology to facilitate its application. The purpose of this investigation reported herein was to develop such a methodology. The second phase of the investigation will involve the application to selected impoundments to illustrate its use.

Chapter 2 discusses the factors to be taken into consideration in determination of a PMF and discusses the different types of PMF's that can be encountered. It does not, however, develop means of predicting PMF's but rather discusses the various methods commonly used.

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Chapter 3 discusses the probabilistic risk analysis of long term stabilization and is intended to address the nature of the risk or hazard that is imposed due to failure of different elements of the impoundment. These factors that are important in evaluating risk on the basis of probability of failure are discussed in the context of consequences of failure.

Chapter 4 discusses the potential for failure to occur due to fluvial geomorphic aspects. This chapter considers the potential for failure to occur due to intrusion of a stream and changes in the nature of streams and rivers.

Chapter 5 discusses the design of impoundment surfaces to avoid gully erosion. A methodology of predicting when gully erosion can initiate is presented. Means of predicting stable slopes and threshold values at which gully erosion to begin are developed.

Chapter 6 evaluates the potential for surface sheet erosion. This chapter develops a methodology for evaluating erosion potential from a surface which is sufficiently flat that gully erosion would not exist.

Chapter 7 discusses the selection of riprap and addresses the question of evaluating appropriate durability of a riprap material.

## Design Flood Estimation

### 2.1 Introduction

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 <sup>the</sup> Probable Maximum Flood (PMF). The PMF has been defined by the U.S. Corps of Engineers (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" (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 PMF flood plain of a major stream or wash, the PMF of concern would be that <sup>caused by</sup> ~~corresponding to~~ the occurrence of the PMP <sup>in the major stream or wash.</sup> ~~over appropriate offsite areas of the main watershed causing a PMF to flow down the canyon or valley.~~ On the other hand, some sites are located on the valley or canyon walls or on ridges such that they are outside the PMF flood plain of a major stream. For these cases, the PMF of concern is that corresponding to occurrence of the PMP on only that area above the site that is tributary to the immediate impoundment area.

The main PMF flows down the main channel and may or may not impact the toe or face of the impoundment, depending on its magnitude. A smaller PMF results from the PMP occurring on the watershed just above the impoundment. This flood would have primary influence on the potential for surface ero-



sion of the cover system if flows are not diverted around the impoundment. When flows are diverted around the reclaimed impoundment, the cover design must withstand the PMP and subsequent PMF impacting directly on the site. Therefore, regional, local and on-sight PMF's must be accounted for in the comprehensive long-term stability analysis.

## 2.2 Design Storm

### 2.2.1 PMP Design Storm

The design storms that are traditionally used to estimate the PMF are an envelopment of maximized intensity-duration values formulated for orographic and non-orographic 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, topographic influences, potential for convergence, moisture potential and meteorological transportation. Commonly, the one-hour thunderstorm and six-hour 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 values have been compiled for areas east and west of the 105° meridian for general type storms and for areas west of the 105° meridian for thunderstorms as presented by the U.S. Bureau of Reclamation (1973). An example of the PMP one-hour thunderstorm values is presented in Fig. 2-1. The charts

portraying the PMP thunderstorm values and PMP general-type storm values were originally derived from Hydrometeorological Report No. 33. (1976) for the U.S. Army Corps of Engineers. Subsequent <sup>Hydrometeorological</sup> reports have been published for various regions throughout the United States, which have adjusted these precipitation estimates ~~such as Hydrometeorological Report No. 49 (1977)~~ which provides PMP estimates for the Southwestern United States. Figure 2.2 presents the regions in which updated PMP studies have been conducted and the reports in which these precipitation estimates are published. When more than one set of precipitation estimation values are presented, it is recommended the more conservative value be used to estimate the PMF.

Regions east of the 105° Meridian in the United States 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. Application of the general-type storm in areas west of the 105° 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 general-type storm will usually yield peak runoff and runoff volumes values greater than the thunderstorm in the eastern United States.

Regions west of the 105° meridian in the United States must evaluate the PMF with both the general-type storm and the thunderstorm. The thunderstorm will generally produce a PMF with peak runoff greater than the general-type storm. However, if the volume of runoff is a consideration,

it is recommended that both PMF values be estimated as the general-type storm volume of runoff exceeds the volume of runoff from the thunderstorm.

### 2.2.2 PMP Rainfall Intensity

*In order to estimate runoff*  
~~When applying the Rational Method~~ (see Section 5.8) *The Rational Method may be used* for determining the PMF for a watershed or reclaimed site with drainage area less than <sup>200</sup> ~~one~~ <sub>acres</sub> square mile, the PMP rainfall intensity must be formulated. *with extrapolation to 5-minute duration (45% of 1-hr. for 1-hr/6-hr. = 1.3)*

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-hr thunderstorm and the 6-hr general-type storm. Table 2.2 presents the rainfall duration and percent PMP values for determining appropriate rainfall depths as recommended by the NRC Staff Technical Position (1983). *Now use Hydromets [see additions to ref. list]*

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

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

where the percent PMP is obtained <sup>as described above</sup> from Table 2.2 and the PMP is obtained from the appropriate PMP design storm presented in Section 2.2.1.

Table 2.2 Why Table 2.2 before Table 2.1?

# Rainfall Depths for Variable Rainfall Durations

Rainfall Duration min.	% of 1-hr PMP	% of 6-hr PMP
2.5	25	10
5	38	14
10	45	17
15	53	20
20	67	26
30	80	30
40	91	34
50	100	38
60	-	53
120	-	80
240	-	100
360	-	-

Use  
HMR

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

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

(in/hr.)                      (in minutes)

The rainfall intensity determined in Eq. 2.2 is generally a conservative value and represents the peak rainfall intensity of the design storm. The resulting rainfall intensity is the input value to Eq. \_\_\_\_\_ in Section 5.3.

When the rainfall duration or time of concentration is less than 5 minutes, it is recommended that a rainfall intensity versus rainfall duration curve be plotted on semilogarithmic paper. The upper end of the curve can then be extrapolated to a duration of less than 5 minutes. Because of the extremely conservative rainfall intensity values obtained for short durations, it is recommended that the minimum rainfall duration be 2.5

minutes.

### 2.3 PMP Comparison Storms

A Comparison of Generalized Estimates of the Probable Maximum Precipitation with Greatest Observed Rainfall and estimates of the 100 year events for areas both east and west of the 105th meridian was prepared by the National Weather Service (1980). It was reported that in the eastern U.S., there are 6500 precipitation reporting stations while in the west there are about 2100 stations. 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 105th meridians. Only 66 separate storm events were recorded in the western U.S. where rainfalls were greater than or equal to 50 percent of the PMP. This study included storm durations of 6 to 72 hours.

The National Weather Service also reported the number of storm events which met or exceeded the 100 years rainfall values and compared them with the regional PMP values. Table 2.1 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 in the western U.S. 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 2.1 Comparison of PMP with 100-year Rainfalls

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

(From NOAA Technical Report WWS 25)

A comparison of the 6 hour PMP to the 100-yr, 10 square miles <sup>rainfalls in</sup> ~~in~~ presented for areas west of the Continental Divide in Figure 2.3. The map indicates that the mountain and other topographic masses significantly <sup>affect</sup> ~~effects~~ the regional variation in rainfall magnitudes. The PMP to 100 yr rainfall ratios range from 3 to 8. Ratios of 3-5 prevail in the uranium mining areas.

## 2.5 PMF Estimation

The Probable Maximum Flood is an estimate of the rainfall-runoff relationship for a particular drainage basin with site specific conditions. The precipitation can be estimated as presented in Section 2.2. Therefore, the determination of the magnitude and volume of the PMF resulting from an extensive assessment of the appropriate watershed parameters can be performed.

Input parameters commonly used in a PMF determination include but are not limited to the watershed area, average slope, elevation differential, 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 be used for evaluating the long term stability of the reclamation of uranium mill tailing impoundments due to the radioactive nature<sup>of the tailings.</sup>

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 as presented in Design of Small Dams (1973). The SCS procedure is readily available and is incorporated as a design option in HEC-1. Another procedure frequently used <sup>is</sup> in the U.S. Army Corps of Engineers Procedure for PMF determination. (Ref. ?)

The Rational Method can be applied to determine the PMF peak discharge for drainage basins or covers with area less than <sup>200 acres.</sup> ~~one square mile.~~ ~~How-~~  
~~For larger areas~~  
~~ever,~~ it is recommended that one of the state-of-the-art procedures be used when possible since the Rational Method does not directly account for many of the basin parameters.

#### References

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Fig. 2.1 Probable maximum thunderstorm 1-hour rainfall (point values in inches) for area west of 105° meridian. 288-D-2760, 288-D-2761.

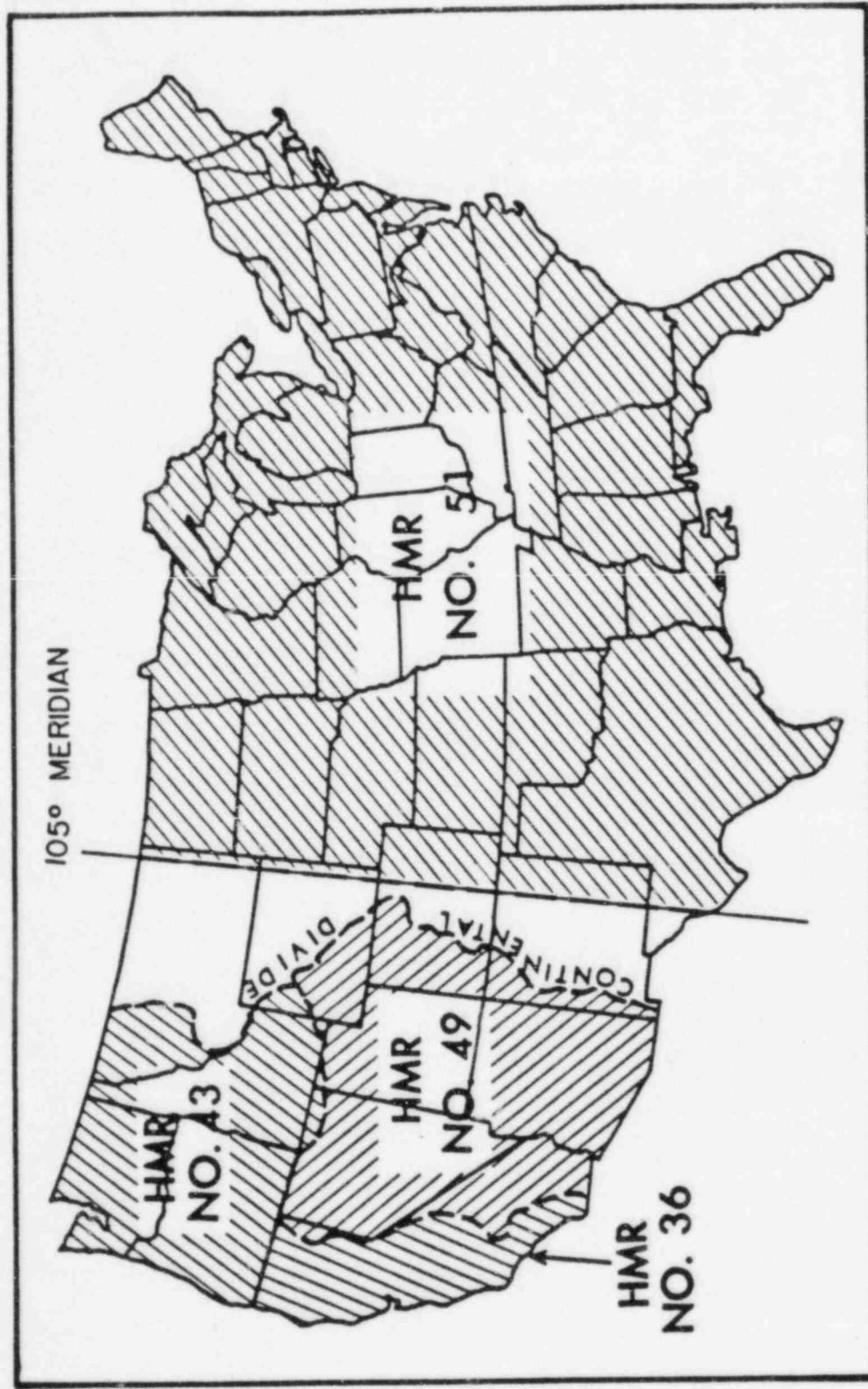


Fig. 2.2 Regions covered by generalized PMP studies used in comparisons.

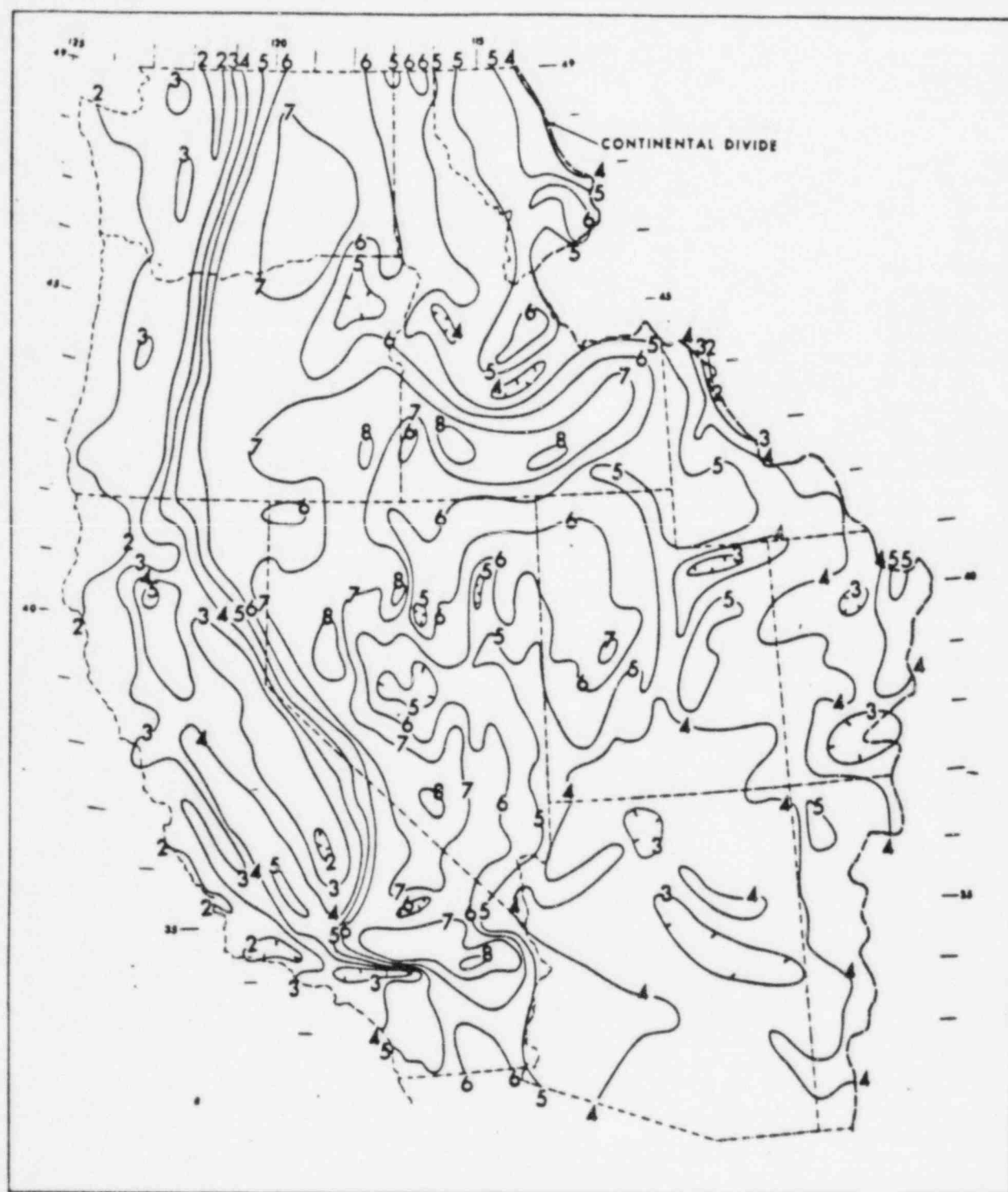


Fig. 2.3 Ratios of 10 mi<sup>2</sup> PMP (HMR Nos. 36, 43, and 49) to 100-yr rainfalls (NOAA Atlas 2) for 6 hours.



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### 3. PROBABILISTIC RISK ANALYSIS OF LONG TERM STABILIZATION

#### 3.1 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, 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 tensile strength. Geological materials and processes show much greater variability and to neglect such variability in any engineering design is unreasonable.

This mostly  
out of context  
to be included  
in the SRP

X

Variability  
is not  
neglected,  
it is to be  
accounted  
for in  
F.S.,  
sometimes  
called  
Factor  
of  
Variability

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).

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 and the mean and is

So  
include  
examples  
and present  
methodology  
in a step by  
step manner  
in the text.

definition  
are provided  
in section:

O.K.

Quote from EPA Standard, 40 CFR Part 192, page 598: "... That fact, which on the one hand warrants our making responsible societal efforts to limit risk to future generations, also warrants our refraining from actions undertaken mere in the name necessarily artificial levels of statistical certainty."

Is all this  
justification  
necessary?  
Here & in  
paragraphs  
after next.



expressed as a percentage.

The final result of a probabilistic analysis is expressed as a reliability or probability of failure. This is a much more realistic measure upon which to base decisions than a 'factor of safety' which can only be compared to some 'acceptable' value.

Probabilistic analyses are not a replacement for engineering judgement. On the contrary, considerable judgement 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 a analysis aids greatly in understanding the major sources of risk (Whitman, 1984).

Probabilistic analysis today can at best supplement & complement the deterministic analysis.

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 judgement. 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.

This observation is correct. The engineers go through data analysis and statistical manipulation for development of design parameters from test data.

It is the purpose of this section to investigate the potential application of probabilistic risk analysis in the long-term stabilization planning of uranium tailings impoundments. This section will review some of the definitions and principles of a probabilistic or

Chapter

→ Should not SRP include more than that? Won't you indicate the level of sophistication of the risk analysis or risk model which will be acceptable to NRC? Any statistical inference based on wrong models has lead many investigators astray. Because of the unconventional nature of the study and analysis, the SRP\* (See bottom of next page)

Why investigate only the failure modes identified in a single study? Has it included all the conceivable modes? I don't think so. Should consider all the possible with multidisciplinary input.

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.

### 3.2 Definitions and Concepts

or

The variability/uncertainty in the value of any parameter  $x$  is

expressed by a probability density distribution, as shown in Figure

3.1. 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 3.1.

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. For example, the shaded area will be the probability that cracking will take place when the differential settlement (or demand on the structure) is  $x = a$  or less. Therefore, if the demand is considered to be a deterministic value such as  $x = a$  in Figure 3.1 and the capacity is assumed to have some distribution, then the shaded area is the probability of failure.

any?

The demand on the structure, e.g. the predicted differential settlement, can also be a variable such as  $x$  in Figure 3.1. In this case one can consider a 'capacity-demand model' as shown in Figure 3.2. 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 central factor of safety is defined as:

$$CFS = \bar{C}/\bar{D}$$

\* Should include some examples of working models and on risk analysis which will exhibit the level of sophistication acceptable to the NRC; a complete case study with all statistical formulations and mathematics could be included in an appendix to the SRP. The SRP text could then confine discussions on input parameters, methodology and results and acceptance criteria. Should such analysis be performed for all sites? Or should it be performed based on classification of the sites into low, intermediate and high hazard categories?

Are you proposing any minimum confidence interval or Max. min. of value

is included as Attachment - A. If the list is too long it could be enclosed in the form of an appendix.

Very few definitions are presented here.

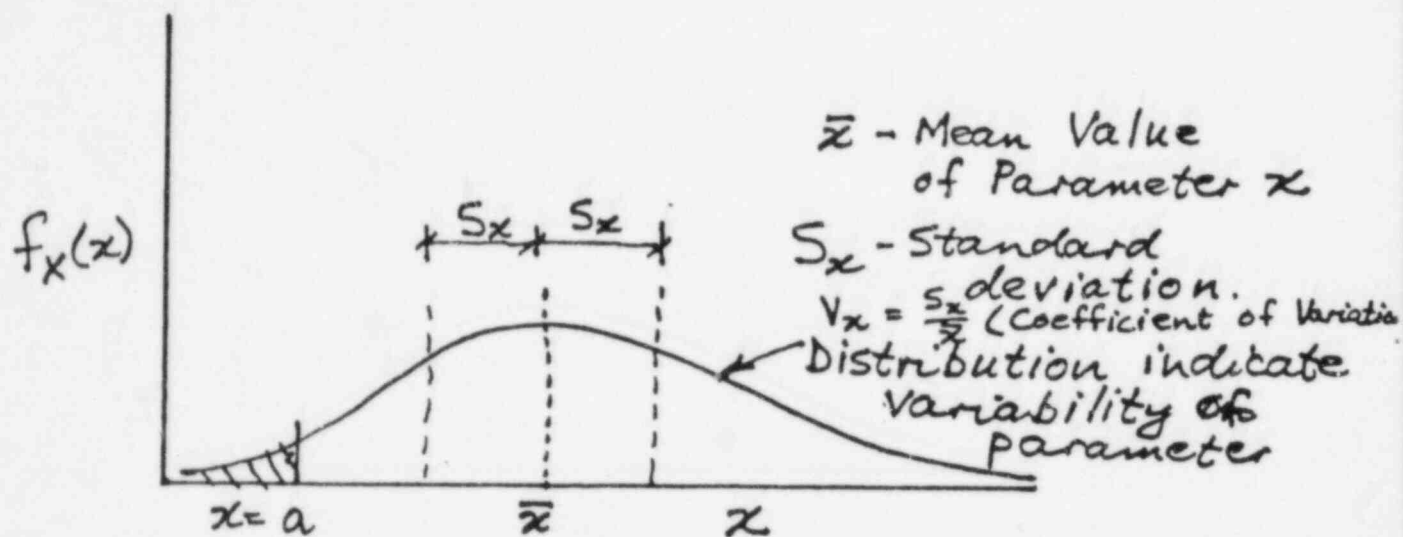


Fig. 3.1 Probability Density Distribution of Parameter  $x$ .

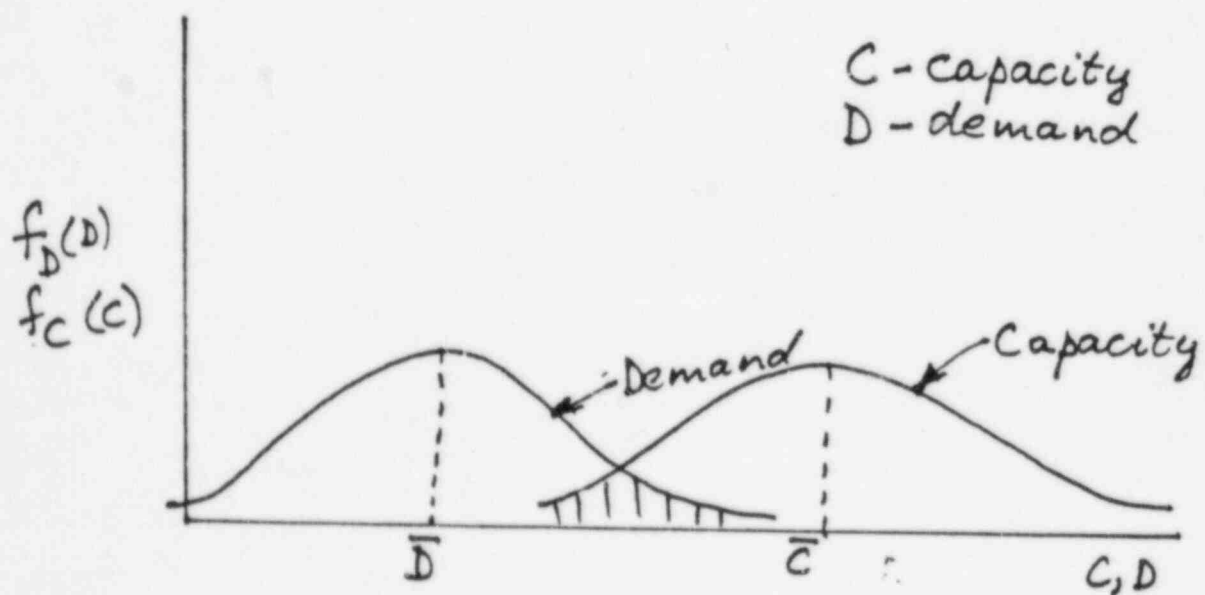


Fig. 3.2 Capacity - Demand Model.

Instead of using 'all the information' contained in the probability distributions the factor of safety approach only uses two deterministic values.

There is one more approach to calculate the probability of failure on the basis of factor of safety. Consider the distribution in Figure 3.1 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 appropriate for a particular application. Historical precedent or experience can be used to select 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 (Whitman, 1984). 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/<sup>or</sup>uncertainty of the capacity (e.g. strength) of the system and the magnitude of the mean capacity  $\bar{c}$  (to) the demand placed upon the system.

*ratio?  $\bar{c}$  ?*

For applications to uranium tailings impoundments the design demand would be the PMF and forces associated with the PMF. That has been discussed previously. 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.

### 3.3 RISK

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

$$\text{Risk} = \text{Uncertainty} \cdot \text{Damage}$$

Other definitions of risk are 'the chance of encountering harm or loss' or the 'degree of probability of such loss' (Stanford Workshop, 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

$$\text{Risk} = \text{Hazard/Safeguards}$$

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 (Stanford Workshop, 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).
- (ii) Hazard evaluation (disposal methods, prior treatment, failure modes, transport mechanism, processes acting on wastes through time).
- (iii) Risk evaluation (probability of a failure, concentration and population at risk, toxicological and epidemiological



levels of potential and actual human exposure, and information, effects and consequences of dose).

- (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 and response as outlined above involve a large number of areas where judgements are required. Some of the results are often qualitative instead of quantitative in nature.

#### <sup>4</sup> 3.4 Probabilistic Risk Assessment

A probabilistic risk assessment (PRA) is an analysis that (NUREG-1050, 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 <sup>to</sup> the precise absolute magnitude of the numbers generated. A PRA study is multidisciplinary. Depending on its scope, a PRA may require analyses of containing 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 (NUREG-1050, 1984). This is obviously a concern and further underscores the necessity for qualitative results.

Based on the schematic outline of the offsite consequences from the Reactor Safety study given in NUREG-1050, 1984), the schematic in Figure 3.3 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 a final property damage or health risk. It is therefore not proposed to evaluate these in the probabilistic risk analysis presented here. Instead it will be more realistic to expend the effort on a probabilistic analysis of tailings release mechanisms.

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 and the profession for other failure modes. This approach is schematically shown in Figure 3.4. 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 3.4, 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 informa-

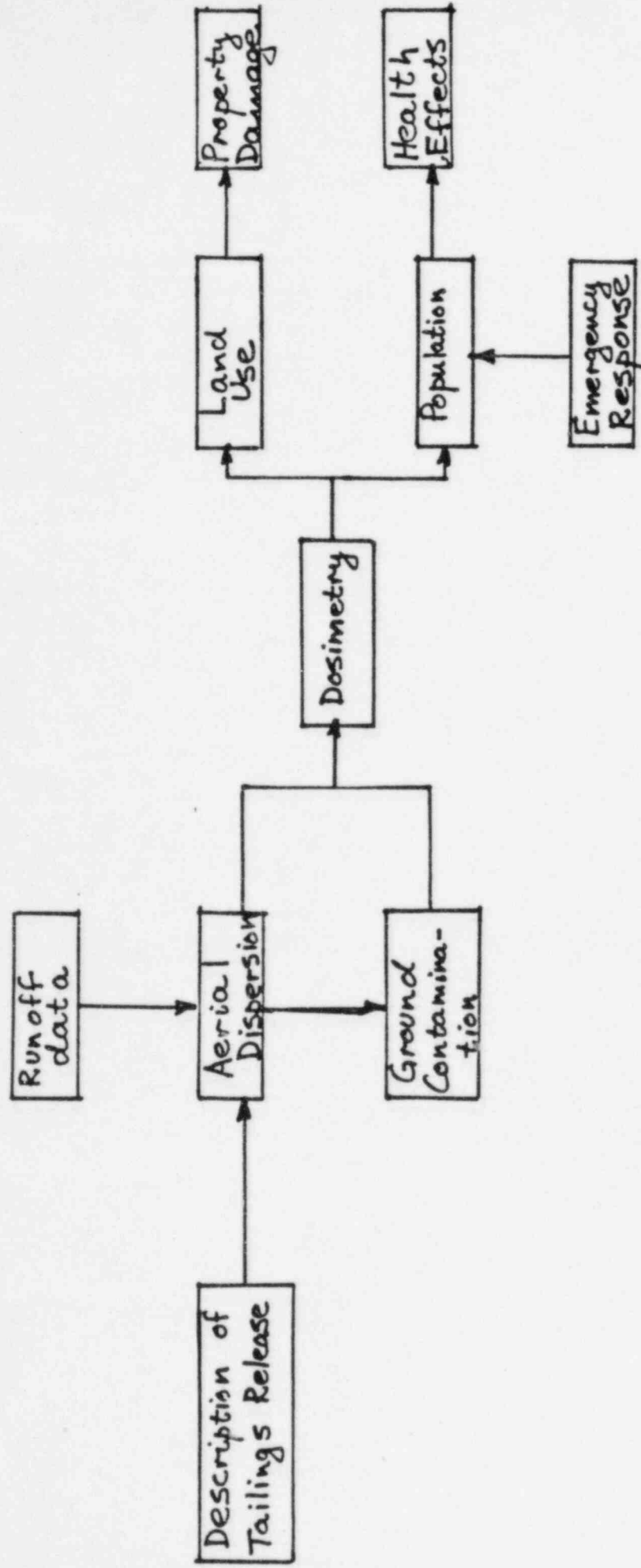


Figure 3.3 Schematic Outline of Offsite Consequences Model from Uranium Tailings Release.

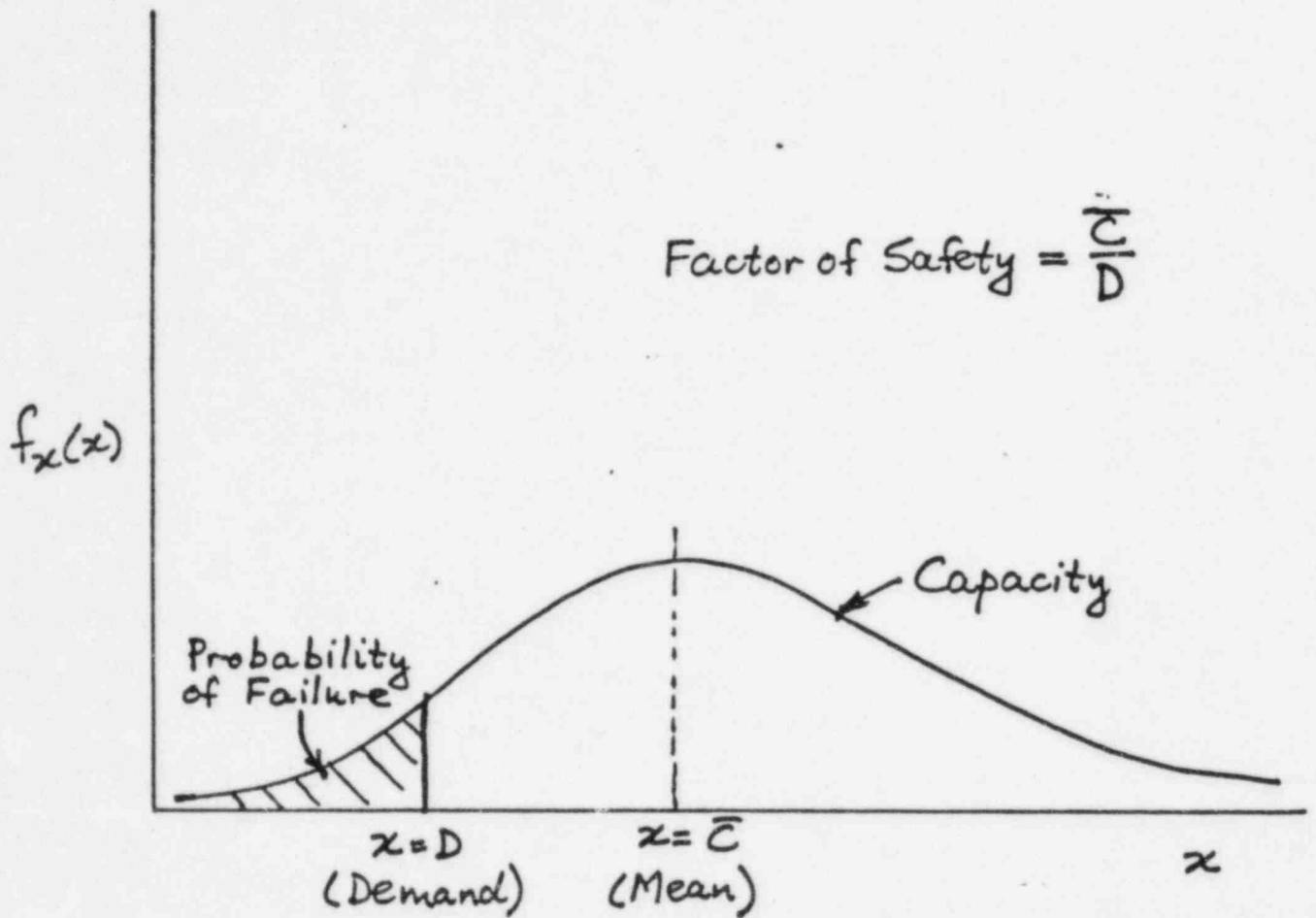


Figure 3.4 Relationship Between Capacity, Demand, Probability of Failure and Factor of Safety.

tion' as it does not include the variability of the capacity function.

However, an important point is the fact that if the safety factor is chosen as unity the probability of failure occurring is 50%. Thus, even if the design demand is the PMF, a factor of safety greater than unity must be utilized to maintain the factor of safety at an acceptably low level. The appropriate value of factor of safety will depend upon the variability or uncertainty of the system capacity and its standard deviation. Also, the acceptable/probability of failure corresponding to an acceptable level of risk must be taken into account.

Maybe  
50% is  
acceptable?

What is  
your criteria  
for acceptable  
system capacity  
considering all  
the uncertainty  
or variability  
associated  
with the  
system?  
or what is  
the % probability  
of non-exceedance  
of capacity  
acceptable

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

### 3.5 Failure Modes in Long-Term Stabilization of Impoundments

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

In ~~mostly 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.

An event tree can be used to indicate the various sequences of events which may lead to a failure by any of the failure modes. Fig-

It does  
not include  
all the modes  
of failures.  
Therefore other  
modes of failure  
should also  
be included.

ure 3.5 presents an event tree for the failure modes identified by Nelson, et al. (1984) in evaluating the long term stability of a uranium tailings impoundment. This event 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.

What about the impact of the failure modes not included in the event tree

The overall probability of failure of a structure is given by the sum of all the probabilities ( $p_{f1}$  to  $p_{f7}$ ) obtained from the event tree. Neglecting the probabilities of failure due to floods smaller than the PMF will therefore 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 event tree in Figure 3.5, 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 the PMF will occur with certainty. Thus, the probability of failure due to flood intrusion given that a PMF has occurred is equal to:

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

What is this?



Must consider all modes  
of failure adopting  
systems approach.

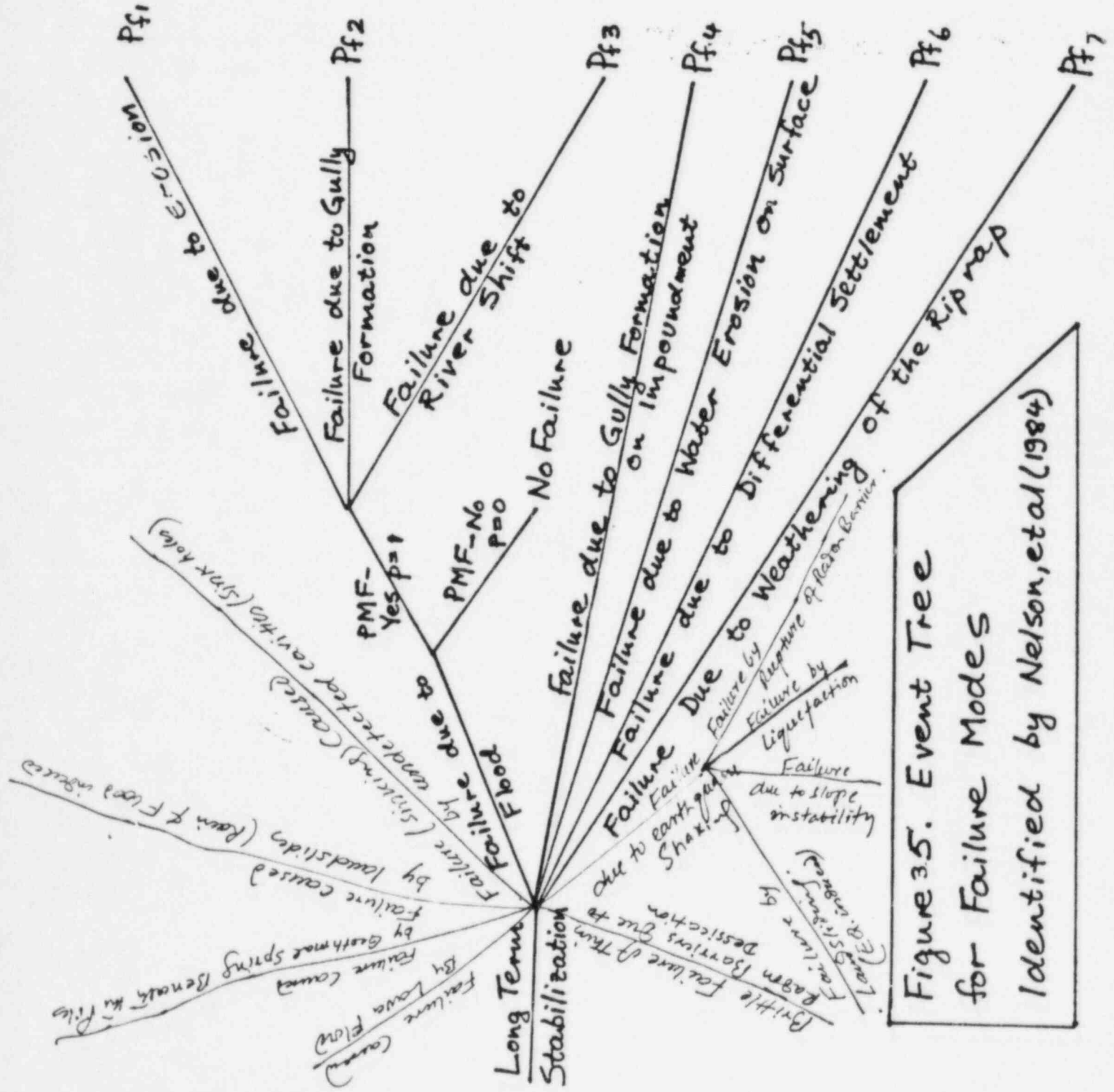


Figure 35. Event Tree for Failure Modes Identified by Nelson, et al (1984)

<sup>5</sup>  
3.4.1 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 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 flood plain. Failure of the impoundment will take place if the erosional forces of the intruded flood are of sufficient magnitude to cause damage.

The flood magnitude used is the PMF, and it is therefore, a deterministic value. One would therefore know (deterministically)

whether the flood has intruded or not. This implies that the 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 <sup>are</sup> 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. (At the time of this writing, these procedures have not been finalized and this section will therefore be completed in the near future).

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 to flow-width ratio vs. drainage basin area presented in Nelson, et al. (1983) be used as a basis for the analysis. This plot establishes a geomorphic threshold zone separating ungullied conditions from gullied conditions. This plot is repeated in Figure 3.6.

The scatter in data on Figure 3.6 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 3.6. It is assumed that this distribution is normal. A mean value and a coefficient of variation can be obtained from the original data. (Note that the plot is on a log-log scale and it may therefore be more reasonable to assume a lognormal distribution. This will be investigated). Starting with a probability of failure one can therefore obtain a allowable factor of safety for design purposes.

Flood intrusion can also take place due to river shift. 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

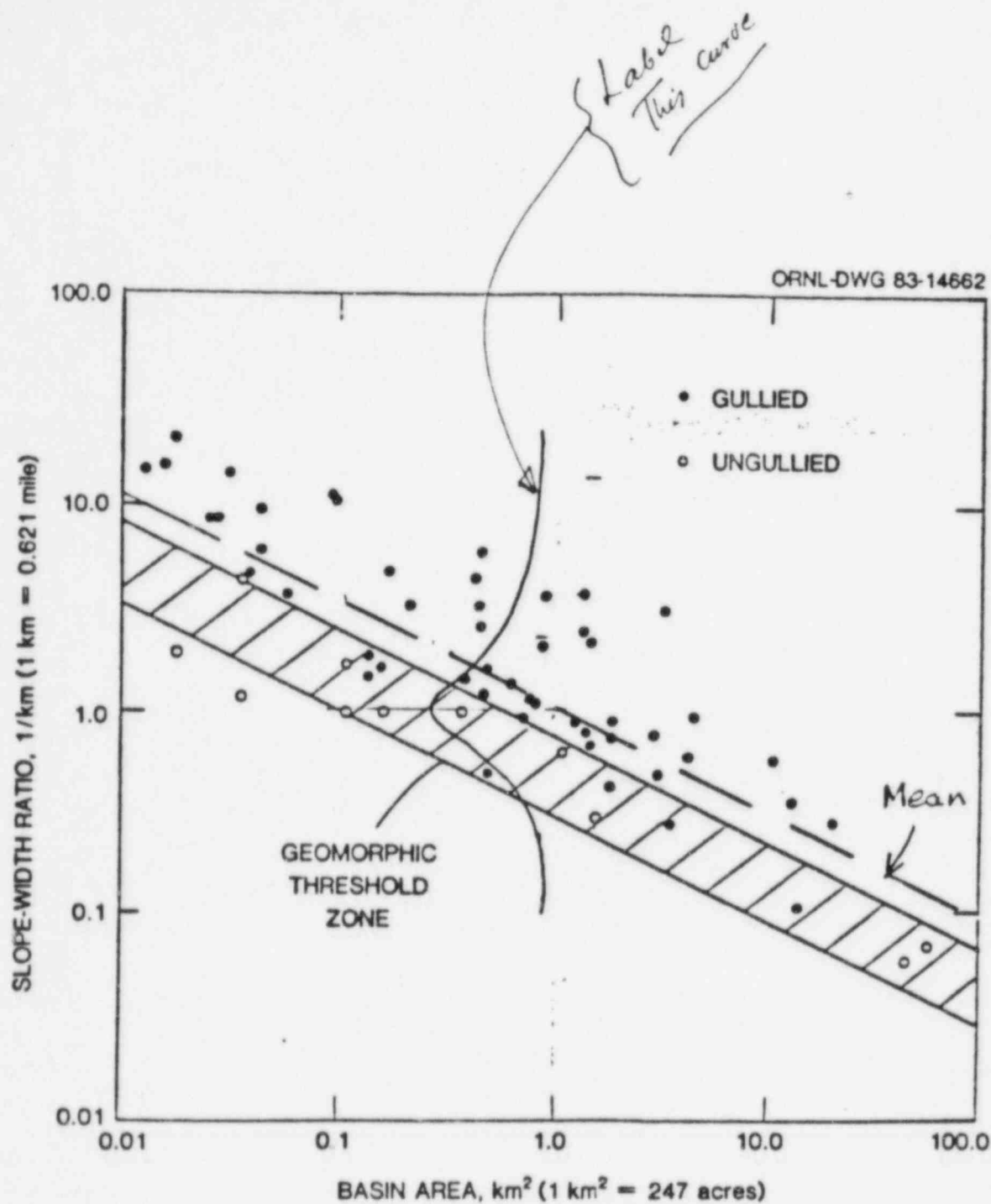


Fig. 3.6 Plot of critical slope to flow-width ratio vs drainage basin area, illustrating the geomorphic threshold zone separating ungullied area from gullied area. Source: Bradley 1980.

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 hazards encountered with each pattern are shown in Figure 3.7 (from Appendix C, Nelson, et al., 1983). Significant engineering judgement will be required to predict possible changes in channel pat-

tern over the time period, say 200 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.

### 3.4.2 Failure Mode 2. Gully Formation on Impoundment Surface

The methodology described in the next sections will be used to evaluate the most reasonable factor of safety for this failure mode. The main task will again be to develop the variability of the capacity curve.

### 3.4.3 Failure Mode 3. Water Erosion on Impoundment Surface

It was suggested that to protect a cover against surface erosion, the Unified Soil Loss Equation be used, and a factor of safety be applied to protect the surface. This approach will guard only against

*Should the risk assessment be performed for the design life of 200 years or up to 1000 years? It is desirable that this criteria be explicitly stated in the SRP.*

*Provide mathematical formulation so that the suggested method could be followed without ambiguity.*

*Provide equation or cite reference*



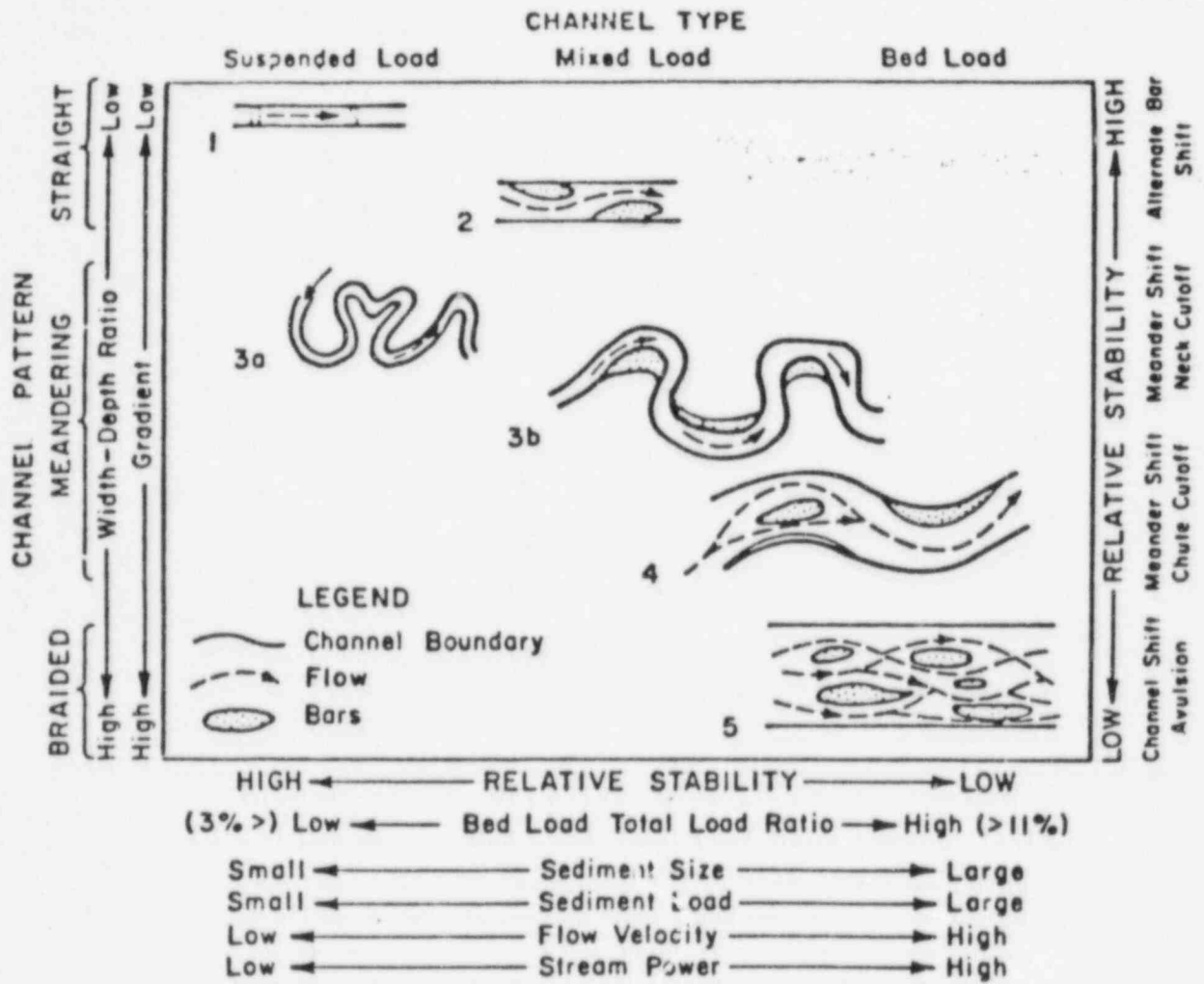


Fig. 3.7 Channel classification showing relative stability and types of hazards encountered with each pattern.

*Please Cite Reference 2*

sheet erosion. Considerations for gully formation are presently under development.

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%.

Is this term defined?

very very rare. Some damage and loss of effectiveness of radon barrier. Should check % loss of effectiveness for acceptability or otherwise.

#### 5 3.4.4 Failure Mode 4. Differential Settlement

Post assurance against this problem is adequate investigation to identify soft & compressible mat'l; appropriate deterministic analysis with site specific data & remedial action during construction, if necessary.

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 3.2 can then be used.

Present mathematical or statistical formulae

#### 5 3.4.5 Failure Mode 5. Weathering of Riprap

This failure mode is the most difficult to evaluate quantitatively. However, qualitative discussions will be presented to evaluate a factor of safety approach.

Not clear what are you suggesting

#### References

Whitman, Robert V. (1984). Evaluating Calculated Risk in Geotechnical Engineering, 17th Terzaghi Lecture, Journ. of Geot. Eng., ASCE, Vol.

- a) May perform some accelerated tests to simulate the effects of ageing and select comparatively better material available.
- b) Must accept some risks as the ageing effect is not verifiable.
- c) use best-judgement.
- d) Cost-benefit analysis may dictate the provision for long term maintenance.

110, No. 2, pp. 145-188.

Stanford Workshop on Risk-Based Approach to Dam Safety Assessment (1984) Sponsored by Federal Emergency Management Agency in cooperation with Department of Civil Engineering, University of Colorado at Denver, Jan. 9-10, Denver, Colorado.

NUREG-1050 (1984). Probabilistic Risk Assessment (PRA) Reference Document, Final Report, United States Nuclear Regulatory Commission, September.

Dreith, R.H. (1982) An Industry's Guidelines for Risk Assessment, In: Risk Assessment at Hazardous Waste Sites, F.A. Long and G.E. Schweitzer (Eds), Am. Chem. Soc. Symp. Series 204, pp. 45-53.

### 3.5 Decision Criteria

The results above have been expressed as a probability of failure. These can also be expressed in terms of an annual frequency of failure if some recurrence interval is assumed. As the demand function consists of a deterministic value, the PMF, a recurrence interval can not be obtained. It has been suggested that a recurrence interval of  $10^6$  years can be adopted for the PMF (Stanford Workshop, 1984). However, to assign a recurrence interval to the design event is not compatible with the philosophy presented regarding the adoption of the PMF as the design flood.

Other decision criteria can be established, these include (Stanford Workshop, 1984):

o Life loss risk. This criterion is very difficult to apply to the cases described above as direct fatalities are not expected. Indirect fatalities due to radon emanation, etc. may occur but it is impossible to quantify this.

o Economic risk. The major economic consequence of a failure is the cost of cleaning up the failure and restoring the cover to the original condition. This approach seems the most reasonable criterion to use. It is also a useful way of comparing various stabilization techniques. One can then use the following expressions to obtain total cost:

Total cost = original construction cost + (probability of failure) (cost of clean up).

## General Comments

This report which will form Chapter 3 or Part 3 of SRP of Nuclear Regulatory Commission (NRC) is not a reviewable quality report. Its principal weaknesses are:

- it lacks sense of direction and purpose
- too much of subjective discussion at rudimentary level
- unintentional but strong denouncement of deterministic analysis thus contradicts the guidelines in the EPA standard and the LMTRA project procedures manual; overview of deterministic analysis is included <sup>incorrectly</sup>
- does not clearly state the nature of risk analysis expected of the project, and how the problem of unpredictability of some input parameters could be resolved.
- Contains many unwanted and unnecessary comments and discussion which should have no place in a SRP, which will act like a regulatory guidelines.
- Any statistical inference based on wrong models has lead many investigators far astray; it may lead to very misleading conclusions far worse than matured qualitative judgements.

### Introduction:

On account of the unconventional nature of probabilistic risk analysis, it is alright in <sup>(the introduction to)</sup> ~~stress~~ the importance of the subject and state clearly the author's purpose in dealing with it; however, to be truly effective, it must be 'relevant, clear and concise'. In addition an introduction should include an "anticipatory Summary", primarily concerned with preparing the reader to what is to follow.

Presenting <sup>briefly</sup> both similarities and differences between the probabilistic and the deterministic analyses, if done with clear objectivity should also be welcome.

From considerations referred above, I believe the introduction needs rewriting, as it lacks sense of direction and purpose. The writer's overview and criticisms of the deterministic analysis are out of place, incorrect, and in essence contradict regulatory guidelines. A sophisticated statistical risk model, built with multidiscipline input, can at best supplement and complement the deterministic analysis and can not adopted as a substitute.

### Definitions:

Very few definitions are included under the section on definitions. The definitions of terms should be expanded (Attachment-A - partial list) and included in form of an appendix, if considered convenient. Please see also additional marginal remarks.



### Concepts & Methodology:

On account of the unconventional nature of this study, it is important to provide complete details of the risk analysis model including mathematical formulations. Possibly this could be best done by including as an appendix a case study based on an working model, acceptable to the NRC. It will be then easy to perform the analysis and present the results conforming to the SRP.

### Contents

Besides stressing the need for probabilistic risk analysis and its merits, the content of the paper provides lot of fragmentary information, which makes interesting reading but lacks consistency and focus. The report content appears more like a progress report than a SRP, which should clearly tell the reader what the NRC expects from the implementing agencies of the project and how the results should be presented for compliance with regulatory standard. None of the complex issues, such as collecting of verifiable input data for risk analysis has been discussed at length.

I believe the type of probabilistic risk analysis of long term stabilization which will enjoy reasonable confidence could only be developed by a multidisciplinary team in the University of National Laboratories and other public agencies, with input from the project engineers.

Conclusions:

(writer) It appears this chapter was concluded very abruptly. Here the writer could have used the opportunity to summarize the main points and exhort the readers to pursue some desired course of action. It should have ended with some parting question referring to various constraints which need to be resolved so that the probabilistic risk analysis procedure could give meaningful results.

# ATTACHMENT-A

## Bayesian updating

Include complex mathematical theory and formulations, which is unavoidable in a complex statistical <sup>model</sup> in the form of appendices.

Random variables,

Probability Density Function (PDF)

Cumulative Density Function (CDF)

Normal or Gaussian Distribution

Lognormal distribution

Histogram

mean, or expected value

Variance

Standard Deviation

Coefficient of Variation

moments

Covariation

Correlation coefficient.

or scale of fluctuation

Probabilities and Conditional Probabilities.

Capacity (in statistical term)  $\approx$  allowable value

Demand ( " " " )  $\approx$  Computed value

Reliability Theory

Standard Normal Distribution Function

Reliability Index,  $\beta$

Systematic errors

#### 4. FLUVIAL GEOMORPHIC INFLUENCES

Schumm (1977) considers an ideal fluvial system in three parts. These are indicated in Figure 4.1. They include three distinct zones. Zone 1 is the production zone which essentially forms the drainage basin which supplies a major part of the water for the river. Zone 2 is a transfer zone wherein the water collected in the drainage basin is being transferred to another point which is known as Zone 3 where deposition would be occurring.

With the current emphasis on minimizing the upstream drainage area above uranium mill tailings impoundments, most of the active sites exist in a Zone 1. The inactive sites may be in either Zone 1 or Zone 2. There are essentially no uranium mill tailings impoundments existing in a Zone 3.

In Zone 1 the major factor of concern is erosion and instability of the system such that localized erosion or gulying could encroach on the impoundment. On the other hand in Zone 2, a major area of concern would be flood intrusion. Zone 1 considerations are covered in Chapter 5 which discusses gully formation and geomorphic stability of the site.

If the impoundment however is located in Zone 2, the main concern revolves around the potential for the river channel to move laterally and intrude upon the impoundment. Appendix D of Nelson, et al. (1983) presents a brief discussion of methods of field investigation to estimate river stability. This Appendix was prepared by S.A. Schumm based on material from "Geomorphic Controls on the Management of Nuclear Waste" by S.A. Schumm and R.J. Chorley (1983). Three distinct phases of investigation are noted therein.

The first is reconnaissance and field inspection. It is stressed therein that both upstream and downstream reaches of the river must be examined in detail even at locations several miles away from the site under consideration. Instability of nick points or aggradation can change the slope and sediment characteristics of the river which can influence its

stability as will be discussed below.

The second phase involves site inspections in the immediate area under consideration. This would involve a study of the channel morphology, bank erosion, sediment characteristics, and vegetation type for some distance, both 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 review the past and present channel behavior. This phase includes a review of the history of nearby bridges which indicate channel width changes over the previous years by comparison of the present cross-section characteristics with those indicated to have been in existence when the old bridge was constructed. Old photographs and conversations with long time residents of the valley provide indications as to the past behavior of the river. Newspaper reports, railroad company files, gauging station records and aerial photography all provide additional other important data. Rates of channel shift can also be assessed by determining the age of vegetation and trees on the flood plain.

#### 4.1 Identification of Fluvial Instability

A quantitative method for assessing 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. These include straight, meandering, and braided channels. Factors influencing whether a channel will be of one type or another include channel slope, mean annual discharge, sediment load, and whether the channel is a bed load, mixed

?

load, or sediment load channel. Figure 4.2 shows these three types of channels.

In assessing the potential hazard from a river to a tailings impoundment, the potential for the channel to invade the impoundment is the main item of concern. Thus, the methodology must consider both horizontal and vertical stability.

#### 4.1.1. Horizontal Stability

Horizontal stability relates to the potential for a river to change from one type to another with accompanying change in location.

Figure 4.3 shows the general effect of slope on the sinuosity 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 for example, the sediment would deposit and the channel would aggrade. Consequently, the channel will remain as a straight channel in that region.

As the slope of the channel increases, the river is capable of transporting more sediment and meanders can develop increasing the sinuosity. *Greater slope generally means less meanders!*  
However, if the sinuosity increases to a point that is too great, the river may become unstable again. As the slope increases more, 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 4.4 shows the effect of slope and sediment load on channel type for a given discharge rate. These data have been combined in Figure 4.5 which shows sinuosity as a function of stream power. Stream power is the product of tractive force and velocity. It is a function of hydraulic radius, slope, and specific weight of the fluid. Thus, stream power is a parameter which takes into account the above hydrologic variables.

In Figure 4.5, zones that would delineate straight, meandering, and braided streams are evident by comparison with Figure 4.2.

Another form of the data is shown in Figure 4.6 which plots slope vs. mean annual discharge. The experimental points shown in Figure 4.6 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 4.5 and 4.6 for comparison with threshold values. This will provide an indication of the stability of the river and its potential to change.

#### 4.1.2 Vertical Stability

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

Rivers may be separated into two major groups depending upon their freedom to adjust their shape and gradient. Bedrock controlled channels are those where the slope of the river is controlled by nick points and outcrops of the bedrock. The slope of these rivers generally is fairly

stable.

The other type of channels are alluvial channels. Alluvial channels flow through a channel having a bed and banks composed of the material transported by the river under present flow conditions.

The vertical stability of a bedrock controlled channel will be dependent primarily upon the resistance of the bedrock forming the nick points. Alluvial channel stability, however depends 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 4.1 provides a classification of alluvial channels and indicates the relative stability of the different types.

#### 4.2 Impact of Flood Intrusion

The impact of flood intrusion will depend to a large extent upon the flow of the river, the velocities associated with it, and the extent to which downcutting can occur causing release of tailings or other elements of the impoundment. In general, however, it may be concluded that if the river channel can come into contact with parts of the impoundment, localized erosion will occur to the extent that release of tailings is probable. Thus, if the main channel can be shown to encroach upon an impoundment, this should be considered unacceptable. On the other hand, if flood waters having relatively low velocities encroach upon the impoundment, this type of flood intrusion may be acceptable. This situation will be discussed in Chapter 5.

#### 4.3 Mitigative Procedures

If vertical instability is a question, stable base levels can be manufactured or created artificially. These base levels must be sufficiently durable and stable so as to 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 of concern, structures may be constructed to cause the river to be diverted around the impoundment, even under PMF conditions. This is possible only under conditions where the PMF flow and velocities are of reasonably small size. For large flows, such as would occur on major rivers in the western United States, this would be virtually impossible to accomplish.

On the other hand, if the river is not large artificial controls can be instituted to reroute the river channel. In so doing however, the engineer must be cognizant of the variables presented in Section 4.1 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 channel instability to occur at a later point.

#### 4.4 Methodology

Application of the methodology would consist of initial gathering of data in accordance with Appendix D by S.A. Schumm (Nelson et al., 1983). After this has been accomplished, the horizontal stability of the site can be determined by plotting of the appropriate parameters in Figures 4.3, 4.5, or 4.6. If the nature of the river channel agrees with that shown by the regions on which it plots in these figures the river may be considered to be stable. If the data points indicate an unstable condition, the nature of the instability should be assessed and the potential for river intrusion into the pile must be determined.

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 nick points and stable base levels at locations both above and below the impoundment. The stability of these nick points and the

ability of the river to migrate laterally and bypass the nick points must be determined by a competent geologist.

In conclusion, 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.

## 5. Gully Erosion

### 5.1 Description

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 floods resulting from precipitation events occurring on the major watersheds <sup>in the vicinity of</sup> ~~associated with~~ 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 <sup>on or</sup> immediately adjacent to the impoundment area.

The development of gullies on the impoundment is associated with erosional forces on immature surfaces. Since reclaimed impoundment covers are comprised of locally derived materials which 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 <sup>?</sup> ~~more~~ vulnerable to gully intrusion than an in situ material with similar site conditions. *Doesn't compaction "mature" a soil at all?*

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 <sup>gully</sup> 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 channel surfaces~~ <sup>lineations of flow concentration</sup>, which <sup>could</sup> 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. Protective measures based on runoff from a probable maximum precipitation (PMP) event should prevent gully formation for periods of 200 years and greater 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.

#### 5.1.1. Gully Intrusion Prediction Procedure

In order to determine the gully intrusion potential of an impoundment cover, an extensive field investigation was conducted by Colorado State University (1985). A series of reclaimed tailings sites were visited in which gullies developed into and in some instances through the cover material. Data collection included cover soil samples, gully dimension, pile dimensions, precipitation records and the reclaimed site age.

Based upon an extensive analysis, <sup>? [Describe.]</sup> a gully intrusion prediction procedure was developed to estimate the maximum depth of gully intrusion, 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 the toe of the slope is relatively stable, the vegetation cover is less than 30% and the slope is marginally protected. The gully intrusion prediction procedure is as follows:

5.1.2 Site Specific Information: It is necessary to determine the site specific characteristics. These characteristics include:

- a) Cover or cap soil mean grain size,  $d_{50}$ ,  
in mm and uniformity coefficient,  $C_u$ .
- b) Pile dimensions, <sup>including</sup> of the slope length, L, slope

- height, H, slope base distance, X, and the  
gradient of the initial slope,  $S_i$  (prior to gullying)?
- c) Precipitation records and estimates?
- d) Time in years over which the potential gully  
intrusion is to be evaluated.

5.1.3 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 ultimately result. <sup>\* ← Under what conditions?</sup> Since incipient gully initiation and subsequent development are a function of the drainage area, an estimate of the area is required. Tributary area determination is as follows:

- a) Estimate the largest drainage area tributary to the outlet slope derived from the reclamation plan contour map. ?

} Need a Figure.

- 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 (1972) as

Define ?

$$D = 0.909 + 22.418 (S_i)$$

[site page]

(5.1)

where D is the density of drainage area per unit length of channel <sup>(units?)</sup> and  $S_i$  is the design or initial slope of the cover.

Eqn. 5.1 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

$$A_{\text{total}} = D_j L_j + D_{j+1} L_{j+1} + \dots \quad (5.2)$$

when  $L_j$  is the potential watercourse  
length at a slope of  $S_j$  for drainage  
density  $D_j$ .

5.1.4 Maximum Depth of Intrusion: Once the site specific characteristics and drainage area is determined, it is possible to estimate the maximum depth of gully intrusion, 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 intrusion. The slope limits are the initial slope gradient,  $S_i$ , and the stable slope gradient which can be predicted as

$$S_s = \frac{0.411 (1+d_{50})}{(PP70.5) (A)} \quad PP > 0.5 ? = ? \quad (5.3)$$

which is derived from Figure 5.1. The estimated stable slopes generally agree with the slope-drainage area relationships derived by Schumm as presented by Nelson et al. (1983). Page — ?

The slope,  $S$ , is the slope of the tangent extending from the toe-of-the-slope to the deepest point in the gully. The slope can be determined at a desired point in time that can be calculated as

$$S = S_i e^{(-k S_s t)} \quad (5.4)$$

where  $k$  is a coefficient and  $t$  is the estimated time in years. The coefficient,  $k$ , was determined to be a function of the stable slope,  $S_s$ , as presented in Figure 5.2. The exponential

is a decaying function that accounts for the decreasing rate of erosion that occurs as the gully slope decreases over time.

Knowing the stable slope, it is possible to estimate the location of the maximum depth of intrusion measured from the toe-of-the-slope. The maximum depth of gullying was found to be a function of the slope,  $S$ , and the soil uniformity coefficient,  $C_u$ , as presented in Figure 5.3.

The maximum depth of gully intrusion,  $D_m$ , at  $L_m D/L$  is estimated as

$$D_m = \left( \frac{L_m}{L} \right) \times (S_i - S) \quad (5.5)$$

Since the maximum gully depth and the location of occurrence is known, it is possible to determine whether the gully has potentially penetrated the cap and cover into the tailings.

Once  $D_m$  is computed, the gully top width can be estimated at the point where  $D_m$  occurs. Figure 5.4 presents the gully top width relationship to the predicted maximum depth of intrusion and uniformity coefficient.

Falk, J., Abt, S.L., and Nelson, J.D., "Gully Intrusion Prediction", working paper, Colorado State University, 1985.

(303) 482-0583 Nisley & Schumm Ref.

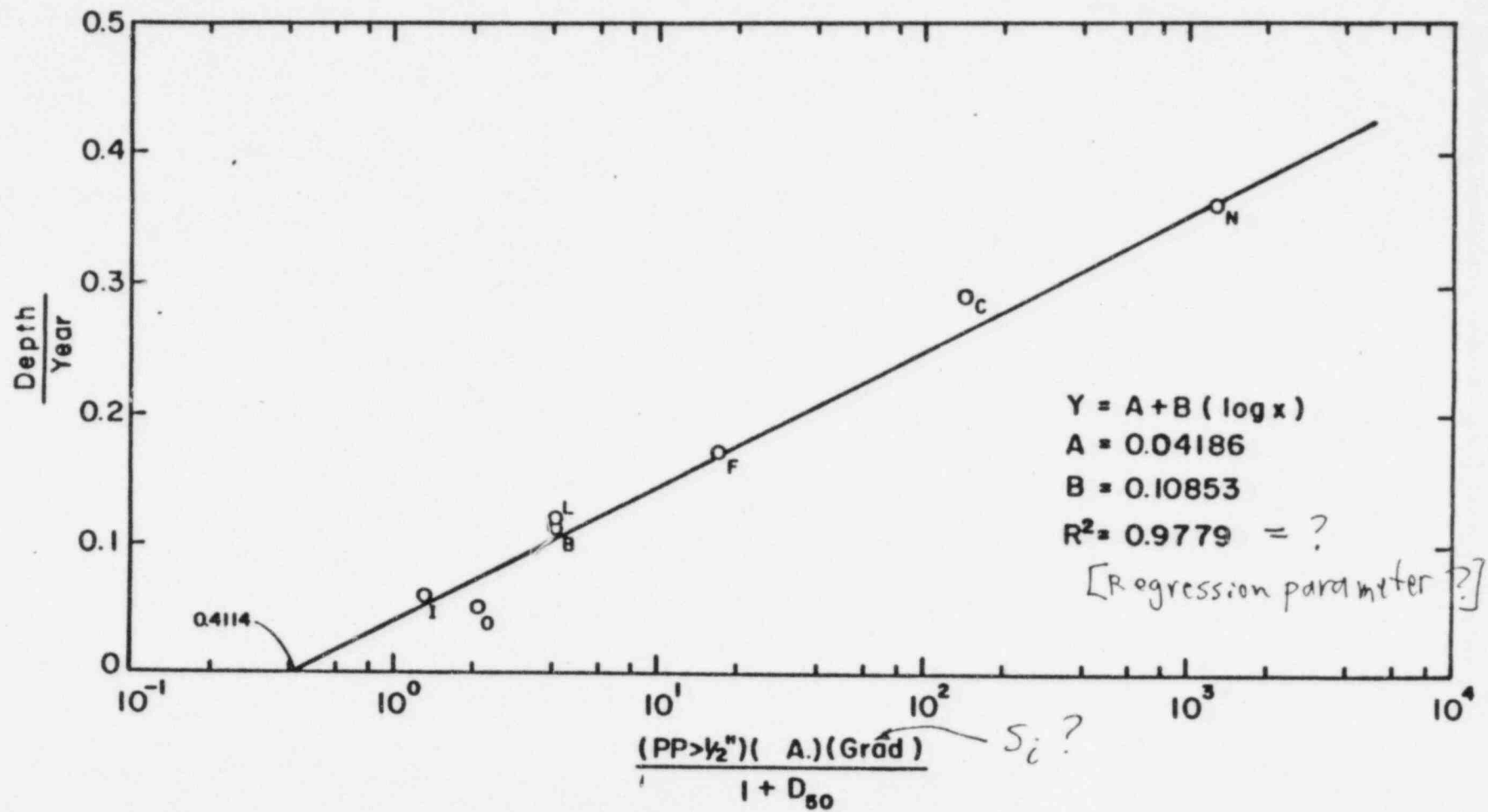


Fig. 5.1 Title? Ref.?

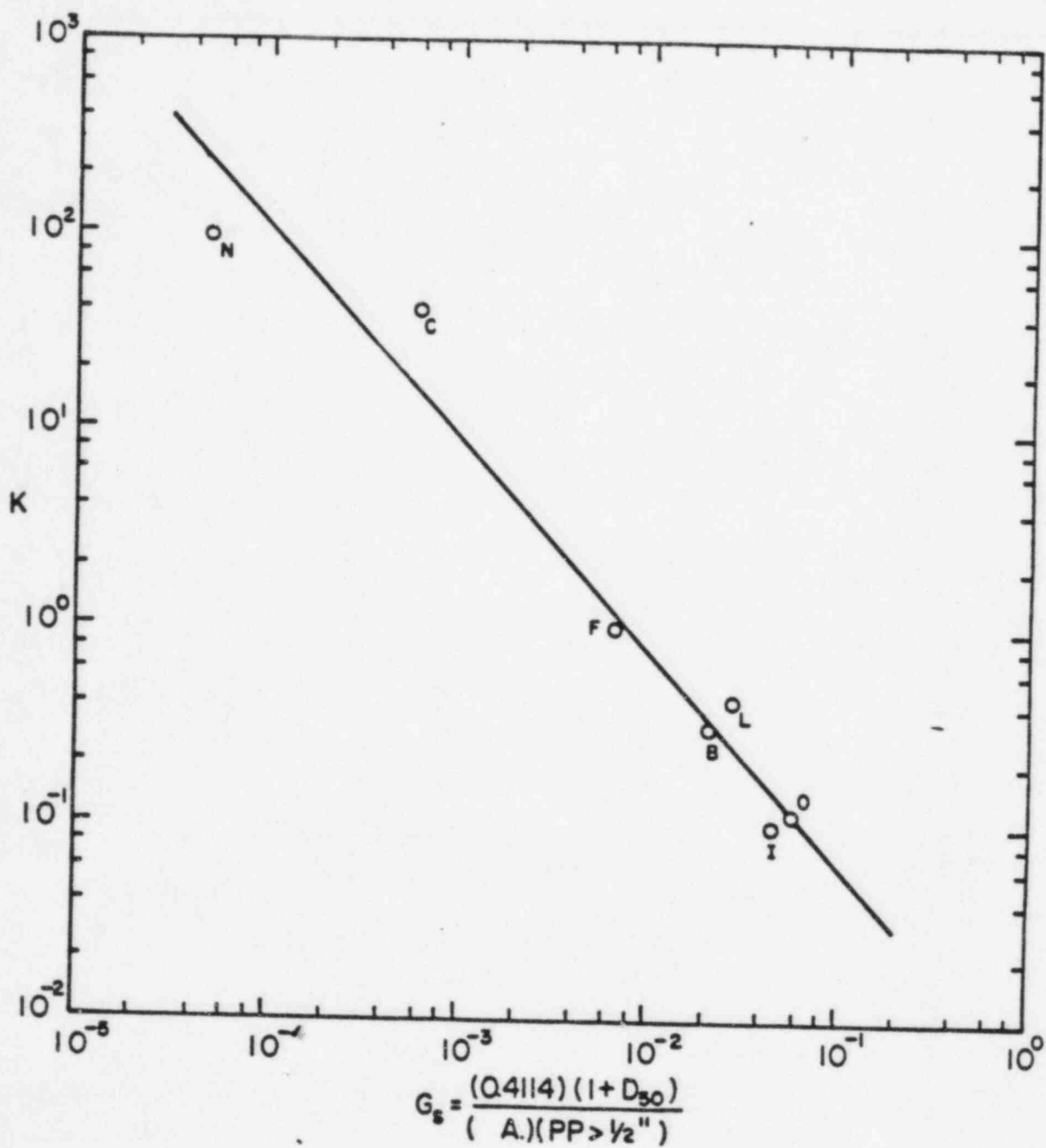


Fig. 5.2

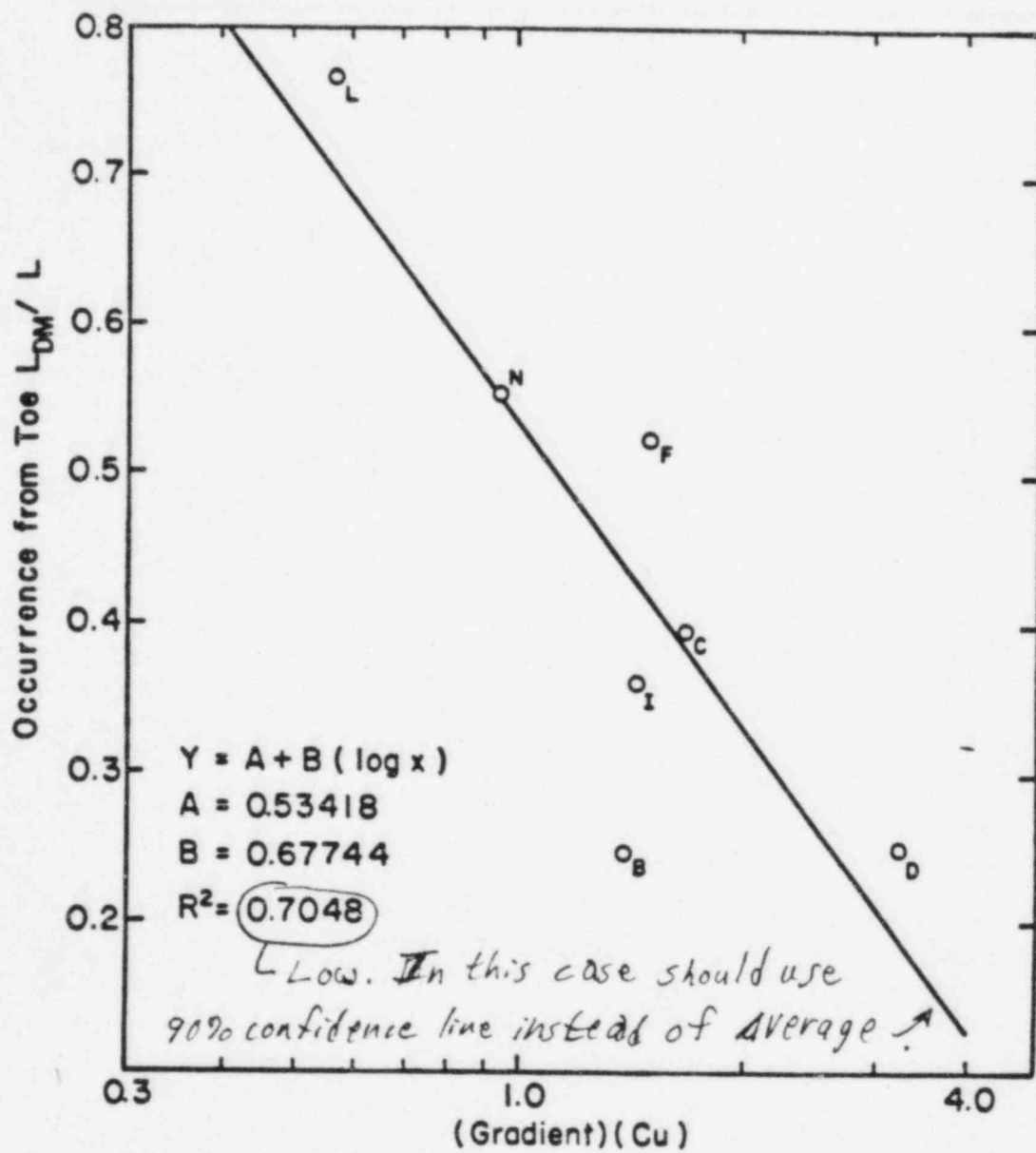
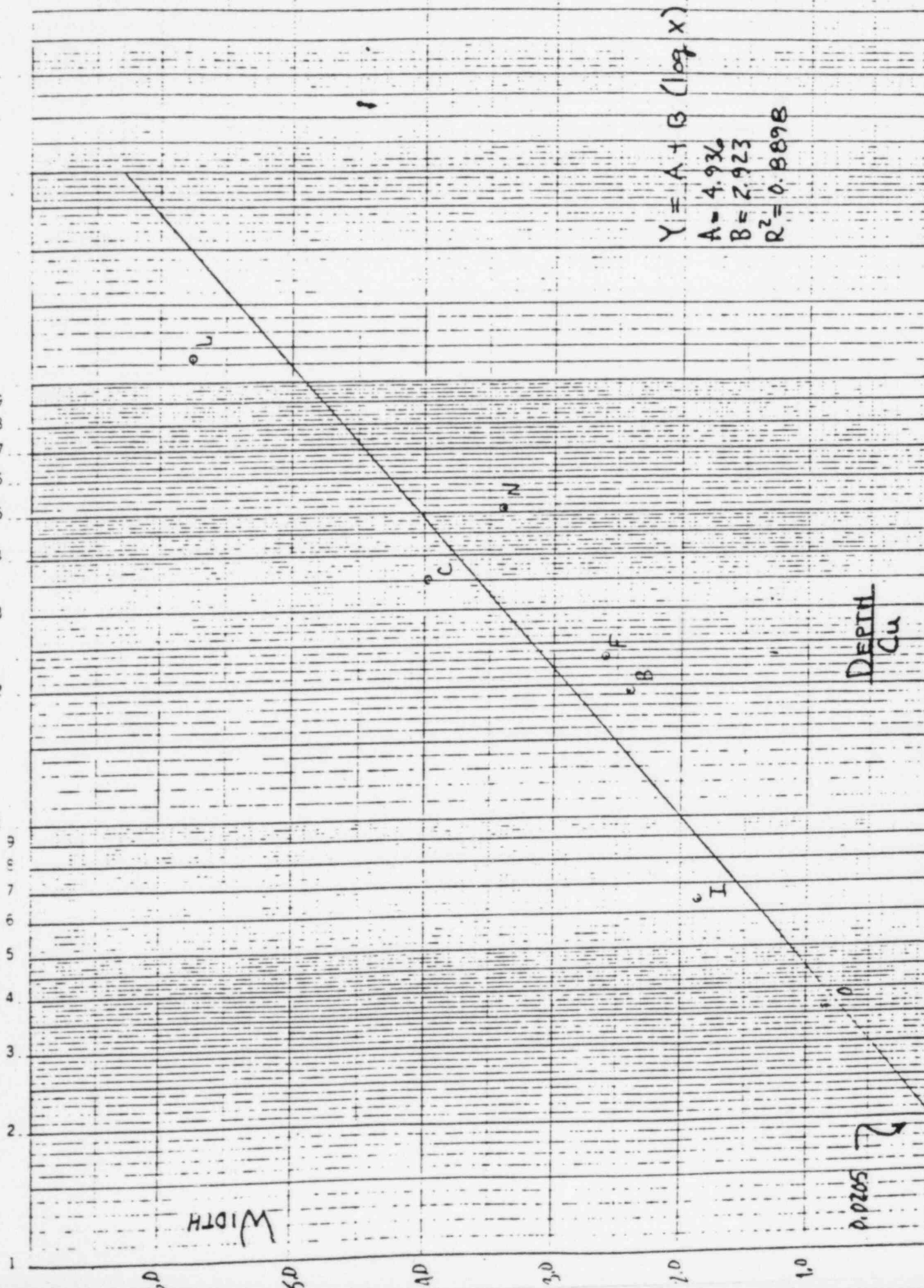


Fig. 5.3





## 5.2 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 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 describe 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 results which may 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.

### 5.2.1 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 5.5 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 and dissipate energy as the flow transitions from the impoundment slope into the natural terrain.
2. Zone II: Zone II is the area along the slope which remains in the major watershed flood plain. The rock protection must resist not only the flow off the cover, but also floods. The riprap must serve as embankment protection similar to river and canal banks.
3. Zone III: Riprap should be designed to protect steep slopes and embankments from potential high velocities and excessive erosion. Flows in Zone III are derived from tributary drainage and direct runoff from the site.
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.

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.

#### 5.2.2 Design Procedures

Presently, several design 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 (1975)
2. The Stephenson Method (1979)
3. The Proposed Nuclear Regulatory Commission Method (1984)
4. U.S. Army Corps of Engineers Method (1970, 1971)
5. The U.S. Bureau of Reclamation Method (1970)

These riprap design procedures are but examples of the many methods available. The impoundment cover designer may utilize any available method for determining the appropriate rock size to ensure cover stabilization.

#### 5.2.2.1 Safety Factors Method

Ref. ?

The Safety Factors Method 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 calculate the Safety Factor (S.F.) or allowing the designer to determine a S.F. and then compute 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; if S.F. is less than unity, the riprap will fail.

The following equations are given 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:

$$S.F. = \frac{\cos \theta \tan \phi}{\eta' \tan \phi + \sin \theta \cos \beta} \quad (5.6)$$

where

$$\eta' = \eta \left[ \frac{1 + \sin(\lambda + \beta)}{2} \right] \quad (5.7)$$

$$\eta = \frac{21 \tau_0}{(S_s - 1) \gamma D} \quad (5.8)$$

and

$$\beta = \tan^{-1} \left[ \frac{\cos \lambda}{\frac{2 \sin \theta}{n \tan \phi} + \sin \lambda} \right] \quad (5.9)$$

The angle,  $\lambda$ , is shown in Figure 5.6 and is the angle between a horizontal line and the velocity vector component measured in the plane of the side slope. The angle,  $\theta$ , is the side slope angle shown in Figure 5.6 and  $\beta$  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,  $\phi$  is the angle of repose of the rock riprap,  $\tau_0$  is the bed shear stress,  $D$  is the representative rock size,  $S_s$  is the specific weight of the rock, and  $\eta'$  and  $\eta$  are stability numbers. The forces  $F$  and  $F_d$  are the lift and drag forces, and the moment arms of the various forces are indicated by value  $e_i$ . Figure 5.7 gives the angle of repose for riprap material sizes.

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.  $\lambda \approx 0$ ). For this case, the above equations reduce to:

$$\tan \beta = \frac{n \tan \phi}{2 \sin \theta} \quad (5.10)$$

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



(5.11)

where

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

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

$$S.F. = \frac{S_m}{2} \left[ (S_m^2 \eta^2 \sec^2 \theta + 4)^{0.5} - S_m \eta \sec \theta \right] \quad (5.12)$$

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

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

*Cite reference(s).*

Historic use of the Safety Factors Method has indicated that a minimum S.F. of 1.5 provides a side slope reliable stability and protection. It is recommended that the rock riprap thickness be a minimum of twice the  $d_{50}$ . *C. of E. recommends  $1.5 d_{50, \max} \approx 2 d_{50, \min}$ .* 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 4 H:IV, it is recommended that the toe be firmly stabilized. The S.F. Method is ideally suited for Zone I and Zone II riprap design.

#### 5.2.2.2 Stephenson Method

The Stephenson Method (1979) for sizing rockfill to stabilize slopes and embankments is a empirically derived procedure developed for emerging flows. The theory is applicable to a relatively even layer of rockfill acting as a resistant to through and surface flow. It is ideally suited

*\*Stephenson ( ) says,*

for the design and/or evaluation of embankment gradients and rockfill protection for flows <sup>down</sup> 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,  $\theta$ , the angle of friction,  $\phi$ , and the empirical factor,  $c$ . The unit discharge can be estimated as indicated in Section \_\_\_ of this report. ?

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

$$d = \frac{q(\tan\theta)^{7/6} n^{1/6}}{C_3^{1/2} [(1-n)(S-1) \cos \theta (\tan \phi - \tan \theta)]^{5/3}} \quad (5.13)$$

where the factor  $C$  varies from 0.22 for gravel and pebbles to 0.27 for crushed granite. The stone size calculated in Eqn. 5.13 is the representative diameter,  $d_{50}$ . The rockfill <sup>o.k.</sup> ~~layer~~ should be well graded, ~~the layer thickness~~ and at least two times the  $d_{50}$  in thickness. A bedding layer or filter should be placed under the rockfill. ?  
 $C_u > 2$  ?

The Stephenson Method does not account for uplift of the stones due to <sup>?</sup> ~~emerging~~ flow. This procedure was developed for flow over and through rockfill. Therefore, it is recommended that the Stephenson Method be applied as a cover and embankment stabilization for overflow or sheetflow conditions. Alternative riprap rockfill design procedures should be considered for toe and bank stabilization. The Stephenson Method is best suited for Zone III protection. ?

#### 3.2.2.3 Proposed NRC Riprap Method

The proposed NRC riprap method has been developed to size rock or





Project \_\_\_\_\_

UMTRA - Gen Criteria.

Contract No. \_\_\_\_\_

File No. \_\_\_\_\_

Feature \_\_\_\_\_

Designed APHDate 8/28/85

Item \_\_\_\_\_

Checked \_\_\_\_\_

Date \_\_\_\_\_

SHP? Type A / Select Rock (Embankment Top)

$$D_{10}(\text{lower lim}) \approx 18 \text{ mm}$$

$$D_{60}(\text{low lim}) \approx 40 \text{ mm}$$

$$D_{60}(\text{up lim}) \approx 65 \text{ mm}$$

$$C_u = \frac{D_{60}}{D_{10}}$$

$$= \frac{40}{18} \text{ to } \frac{65}{18}$$

$$= 2.2 \text{ to } 3.6 < 4.0$$

Hence poorly graded  
(GP)

$$D_{30}(\text{low lim}) \approx 24 \text{ mm}$$

$$D_{30}(\text{up lim}) \approx 39 \text{ mm}$$

by Unified Soil Class

$$C_c = \frac{(D_{30})^2}{(D_{10})(D_{60})}$$

$$= \frac{(24)^2}{(18)(65)} \text{ to } \frac{(39)^2}{(18)(65)}$$

$$= 0.49 \text{ to } 1.30$$

Gradation Parameters for Rock Riprap (Ship rock)Note

The rock riprap gradation used is that based on  
Cops of Engg's criteria.



Type B / Select Rock (Embankment Sides & Ditches)

$$D_{10} (\text{low lim}) \approx 85 \text{ mm.}$$

$$D_{10} (\text{up lim}) \approx 175 \text{ mm.}$$

$$D_{60} (\text{low lim}) \approx 140 \text{ mm.}$$

$$D_{60} (\text{up lim}) \approx 210 \text{ mm.}$$

$$C_u = \frac{D_{60}}{D_{10}}$$

$$= \frac{210}{175} \text{ to } \frac{210}{85}$$

$$= 1.2 \text{ to } 2.5 < 4.0 \text{ Hence poorly graded (GP)}$$

$$D_{30} (\text{low lim}) = 105 \text{ mm.}$$

$$D_{30} (\text{up lim}) = 190 \text{ mm.}$$

by Unified Soil Classif

$$C_c = \frac{(D_{30})^2}{(D_{10}) \cdot (D_{60})}$$

$$= \frac{(105)^2}{(85) \cdot (210)} \text{ to } \frac{(190)^2}{(85) \cdot (210)}$$

$$= 0.62 \text{ to } 2.0$$

Table 3.5 Unified Soil Classification

Field Identification Procedures (Excluding particles larger than 3 in. and basing fractions on estimated weights)				Group Symbols	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria	
Coarse-grained soils More than half of material is larger than No. 200 sieve size (The No. 200 sieve size is about the smallest particle visible to naked eye)	Gravels More than half of coarse fraction is larger than No. 7 sieve size (For visual classification, the 1/2 in. size may be used as equivalent to the No. 7 sieve size)	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes	GW	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbols in parentheses  For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics  Example: <i>Silty sand, gravelly</i> ; about 20% hard, angular gravel particles 1/4-in. maximum size; rounded and subangular sand grains coarse to fine, about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (SM)	Determine percentages of gravel and sand from grain size curve Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse soils are classified as follows: Low than 5% GW, GP, SW, SP More than 5% GM, GC, SM, SC Borderline cases requiring use of dual symbols	
			Predominantly one size or a range of sizes with some intermediate sizes missing	GP	Poorly graded gravels, gravel-sand mixtures, little or no fines			
			Nonplastic fines (for identification procedures see ML below)	GM	Silty gravels, poorly graded gravel-sand-silt mixtures			
			Plastic fines (for identification procedures, see CL below)	GC	Clayey gravels, poorly graded gravel-sand-clay mixtures			
	Sands More than half of coarse fraction is smaller than No. 7 sieve size (For visual classification, the 1/2 in. size may be used as equivalent to the No. 7 sieve size)	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	SW	Well graded sands, gravelly sands, little or no fines			
			Predominantly one size or a range of sizes with some intermediate sizes missing	SP	Poorly graded sands, gravelly sands, little or no fines			
			Nonplastic fines (for identification procedures, see ML below)	SM	Silty sands, poorly graded sand-silt mixtures			
			Plastic fines (for identification procedures, see CL below)	SC	Clayey sands, poorly graded sand-clay mixtures			
		Gravels with fines (appreciable amount of fines)						
Fine-grained soils More than half of material is smaller than No. 200 sieve size (The No. 200 sieve size is about the smallest particle visible to naked eye)	Identification Procedures on Fraction Smaller than No. 40 Sieve Size							
	Silty sands and clays Liquid limit less than 50	Dry Strength (crushing characteristics)	None to slight	Quick to slow	None	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses  For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions  Example: <i>Clayey silt, brown</i> ; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (ML)	Use grain size curve in identifying the fractions as given under field identification	
			Medium to high	None to very slow	Medium			
			Slight to medium	Slow	Slight			
		Dilatancy (reaction to shaking)	None to slight	Quick to slow	None			
			Medium to high	None to very slow	Medium			
			Slight to medium	Slow	Slight			
	Silty sands and clays Liquid limit greater than 50	Toughness (consistency near plastic limit)	Slight to medium	Slow to none	Slight to medium			
			High to very high	None	High			
			Medium to high	None to very slow	Slight to medium			
Readily identified by colour, odour, spongy feel and frequently by fibrous texture								
Highly Organic Soils				Pt	Peat and other highly organic soils			

$C_u = \frac{D_{60}}{D_{10}}$ Greater than 4	
$C_u = \frac{(D_{60})^3}{D_{10} \times D_{30}}$ Between 1 and 3	
Not meeting all gradation requirements for GW	
Atterberg limits below "A" line, or $P_L$ less than 4	Above "A" line with $P_L$ between 4 and 7 are borderline cases requiring use of dual symbols
Atterberg limits above "A" line, with $P_L$ greater than 7	
$C_u = \frac{D_{60}}{D_{10}}$ Greater than 6	
$C_u = \frac{(D_{60})^3}{D_{10} \times D_{30}}$ Between 1 and 3	
Not meeting all gradation requirements for SW	
Atterberg limits below "A" line or $P_L$ less than 5	Above "A" line with $P_L$ between 4 and 7 are borderline cases requiring use of dual symbols
Atterberg limits below "A" line with $P_L$ greater than 7	

Plasticity index

60

50

40

30

20

10

0

0 10 20 30 40 50 60 70 80 90 100

Liquid limit

Plasticity chart

for laboratory classification of fine grained soils

Comparing soils at equal liquid limit

Toughness and dry strength increase with increasing plasticity index

A line

CH

CL

OL

CL-MI

ML

OH or MH

From Wagner, 1957.

<sup>a</sup> Boundary classifications. Soils possessing characteristics of two groups are designated by combinations of group symbols. For example GW-GC, well graded gravel-sand mixture with clay binder.<sup>b</sup> All sieve sizes on this chart are U.S. standard.

## Field Identification Procedure for Fine Grained Soils or Fractions

These procedures are to be performed on the minus No. 40 sieve size particles, approximately 1/4 in. For field classification purposes, screening is not intended, simply remove by hand the coarse particles that interfere with the tests.

## Dilatancy (Reaction to shaking):

After removing particles larger than No. 40 sieve size, prepare a pat of moist soil with a volume of about one-half cubic inch. Add enough water if necessary to make the soil soft but not sticky.

Place the pat in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pat which changes to a livery consistency and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the pat stiffens and finally it cracks or crumbles. The rapidity of appearance of water during shaking and of its disappearance during squeezing assist in identifying the character of the fines in a soil.

Very fine clean sands give the quickest and most distinct reaction whereas a plastic clay has no reaction. Inorganic silts, such as a typical rock flour, show a moderately quick reaction.

## Dry Strength (Crushing characteristics):

After removing particles larger than No. 40 sieve size, mould a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun or air drying, and then test its strength by breaking and crumbling between the fingers. This strength is a measure of the character and quantity of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity.

High dry strength is characteristic for clays of the CH group. A typical inorganic silt possesses only very slight dry strength. Silty fine sands and silts have about the same slight dry strength, but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty whereas a typical silt has the smooth feel of flour.

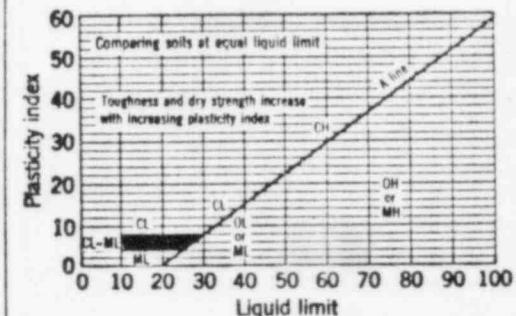
## Toughness (Consistency near plastic limit):

After removing particles larger than the No. 40 sieve size, a specimen of soil about one-half inch cube in size, is moulded to the consistency of putty. If too dry, water must be added and if sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation. Then the specimen is rolled out by hand on a smooth surface or between the palms into a thread about one-eighth inch in diameter. The thread is then folded and re-rolled repeatedly. During this manipulation the moisture content is gradually reduced and the specimen stiffens, finally loses its plasticity, and crumbles when the plastic limit is reached.

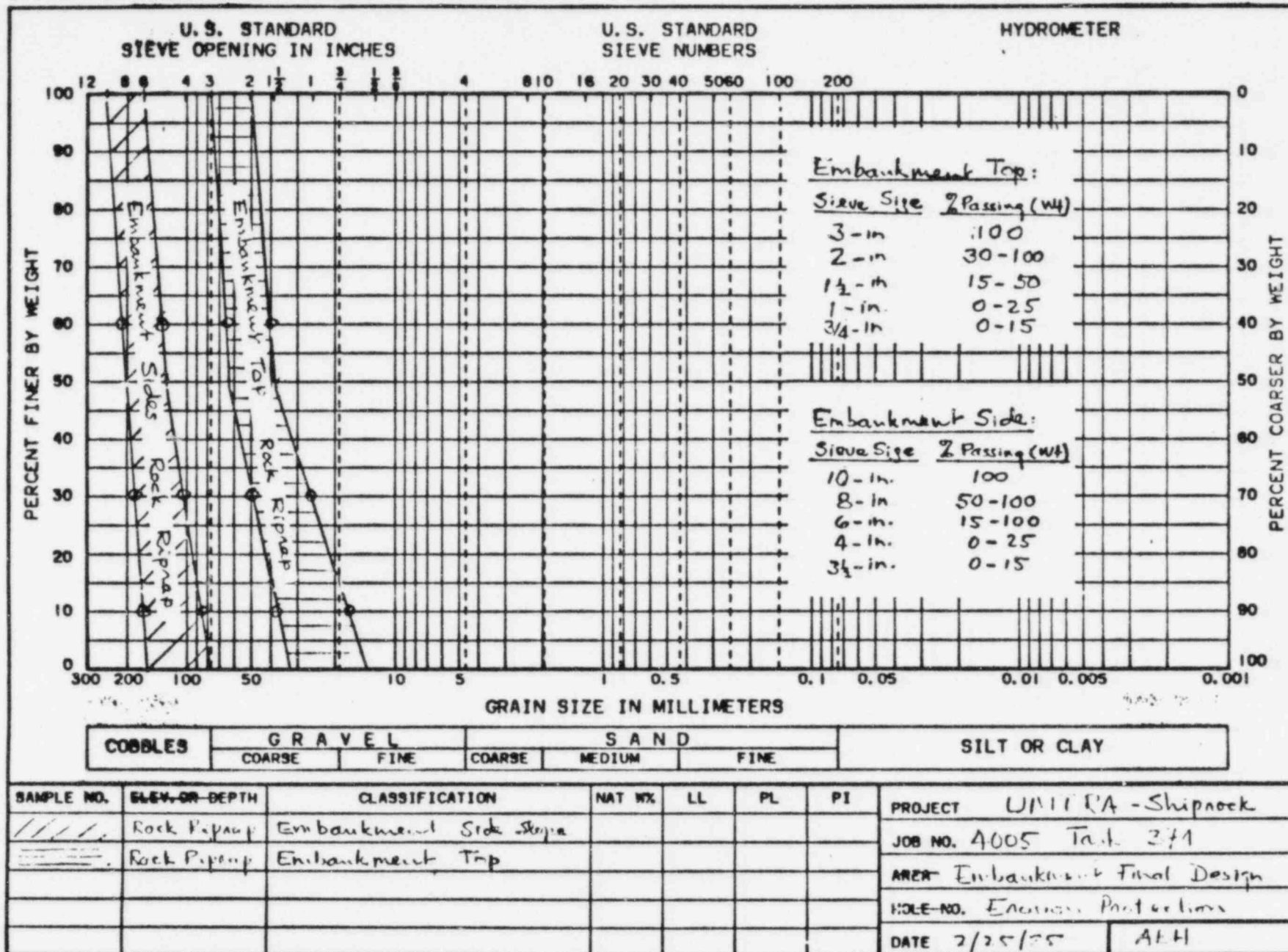
After the thread crumbles, the pieces should be lumped together and a slight kneading action continued until the lump crumbles.

The tougher the thread near the plastic limit and the stiffer the lump when it finally crumbles, the more potent is the colloidal clay fraction in the soil. Weakness of the thread at the plastic limit and quick loss of coherence of the lump below the plastic limit indicate either inorganic clay of low plasticity, or materials such as kaolin-type clays and organic clays which occur below the A-line.

Highly organic clays have a very weak and spongy feel at the plastic limit.

Plasticity chart  
for laboratory classification of fine grained soils

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riprap to protect earthen embankments and covers. It is well suited for the protection of rock covers from sheet and overland flow thereby minimizing general erosion and gully initiation.

It is assumed that the design flood depth and peak discharge can be estimated. Section 5.8 of this report presents an a method for determining the discharge and flow depth. Once the design flow depth is quantified, the average boundary shear that the in-place riprap must resist can be computed. The shear stress,  $\tau$ , may be expressed as

$$\tau = \gamma R S \quad (5.14)$$

Since  $R = y$  for overland and sheetflow, Eqn. (5.14) can be amended to

$$\tau = \gamma y S \quad (5.15)$$

where  $\gamma$  is the unit weight of water,  $y$  is the depth of flow and  $S$  is slope expressed in decimal form. A safety factor,  $SF$ , should be introduced to adjust the design shear stress. The safety factor will range from 1.0 to 3.0 compensating for potential flow concentration, potential uplift of the stones and rock layer stratification. Recommended safety factor values are 1.5 to 2.0. Therefore, the design shear stress,  $\tau_o$ , can be expressed as

$$\tau_o = SF \times \tau \quad (5.16)$$

The representative riprap size,  $d_{50}$ , required to resist the design shear stress may be determined with Lane and Carlson's relation (1953) by

$$d_{50} = \frac{\tau_o}{0.04 (\gamma_s - \gamma)}$$

*{ How does this method differ from C. of E. method? (see pgs. 9 & 10.) }*  
(5.17)

*Eqns. 5.18 & 5.20.*

where  $\gamma_s$  is the rock specific weight.



The proposed NRC Method is applicable to a uniformly deep, well graded rock layer. Furthermore, it is recommended that a filter layer or gravel bed underlie the riprap layer, particularly in areas where critical or super critical flows are expected. It is recommended that the layer thickness range from 1.5 times the  $d_{50}$  to 2.0 times the  $d_{50}$ . Consideration must be given to the size of the available rock source. Generally, the proposed NRC method will provide a more conservative effective stone size than the Stephenson Method. *What does Stevens say about this approach?*

*How does it compare to safety Factors Method?*  
The proposed NRC method was developed to protect earth cover and embankments for overland and sheet flow. It is recommended that this procedure not be used for toe and bank protection applications. Furthermore, this procedure should be extensively tested to refine the safety factor.

The other method is designed for Zone III applications. *1?!*

*[c. of E. & Bur. of Rec.]*

#### References

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- Stephenson, D., Rockfill in Hydraulic Engineering, Developments in Geotechnical Engineering, 27, Elsevier Scientific Publishing Company, pp. 50-60, 1979.

#### 5.2.2.4 U.S. Army Corps of Engineers Method

The U.S. Army Corps of Engineers, (USACE) has developed perhaps the most comprehensive methods and procedures for sizing riprap revetment. Their criteria are based on extensive field experience and practice (1970). The USACE method is primarily applicable to embankment toe and bank protection.

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,  $y$ , and the slope of the energy gradient,  $S$ , is determined. The average boundary shear,  $\tau$ , can be computed as

$$\tau = \gamma R S \quad (5.18)$$

where  $\gamma$  is the unit weight of water in pcf and  $R$  is the hydraulic radius in ft. The design shear stress,  $\tau$ , shall serve as the design shear for the toe and channel bottom.

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

$$\tau_o = \tau \left( 1 - \frac{\sin^2 \phi}{\sin^2 \theta} \right)^{0.5} \quad (5.19)$$

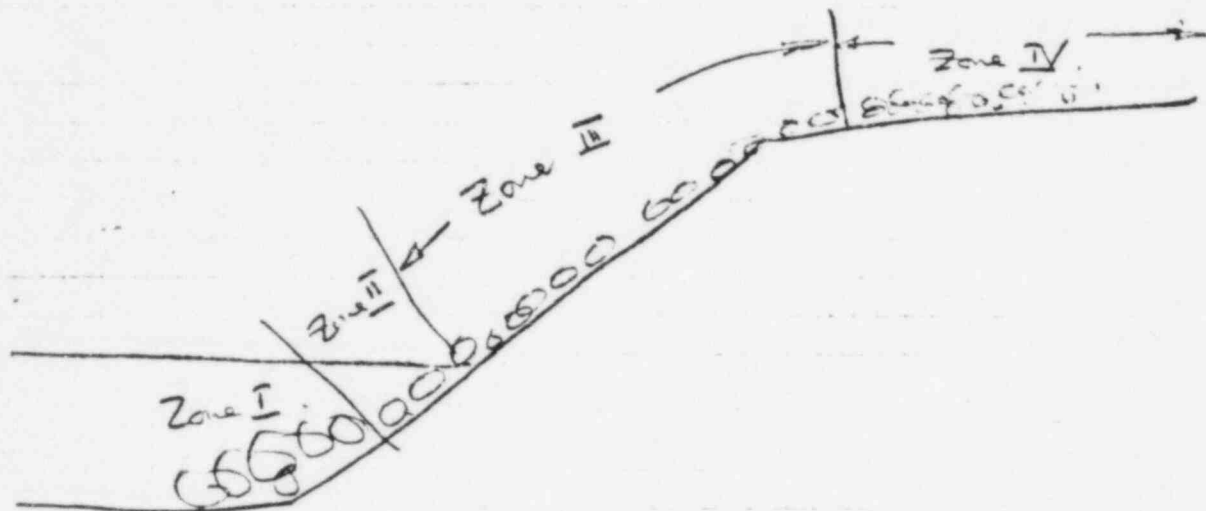
where

$\phi$  = the angle of the side slope with the horizontal

$\theta$  = the angle of repose of the riprap (normally about  $40^\circ$ )

The side slope shear,  $\tau_o$ , is the design shear for sizing the riprap revetment.





Zone III Stephenson's method.

Zone I } Corp of Eyr's method - for channel bank  
 Zone II } also Safety Factor method

Zone IV ?

Reacts & Junctions, USBR method  
 [hydraulic jump  
 & high turbulence area]

NRC method is same as Corp of Eyr's method, but  
 with safety factor applied to shear stress.

The average stone size can then be determined as

$$(5.20) \quad d_{50} = \frac{\tau}{0.04 (\gamma_s - \gamma)}$$

for the toe and channel bottom and

$$(5.21) \quad d_{50} = \tau_o / 0.04 (\gamma_s - \gamma)$$

for the channel side slopes

where  $\gamma_s$  is the specific weight of the stone. The same procedure can be used for bank protection. A graphic representation of Eqn. 5.21 is provided in Fig. 5.8.

The USACE 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 is in the flood plain. The USACE Method is ideal for stabilizing cover and embankment toes. However, the USACE Method is not necessarily recommended for overland and sheetflows due to its conservatism and cost. Therefore, the USACE Method is best applied for Zone I and Zone II protection.

Riprap Layer Thickness:

*\* But as the "NRC method it is! (see p. 8)*

The USACE Method presents the following criteria 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 in.
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 in<sup>ch</sup>, <sup>and an</sup> accompanied by the appropriate increase in stone sizes, should be provided where riprap will be subject to attack by large floating debris.

The riprap layer thickness should be <sup>underlain</sup> ~~unlain~~ with a gravel filter for channel, toe and side slope applications. Filter criteria <sup>are</sup> ~~is~~ presented in Section 5.4 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: five times the lower limit of  $W_{50}$  rock, that size which can be obtained economically from the quarry, or 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 <sup>the</sup> ~~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 memorandum and specifications should indicate the permissible stone gradation limits.

A graphical representation relating rock weight to rock spherical diameter was presented by Nelson et al. (1983).

#### References

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

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

Richardson, E.V., et al., 1975, Highways in the River Environment-Hydraulics and Environmental Design Considerations, U.S. Department of Transportation. Available from Superintendent of Documents, Washington, D.C.

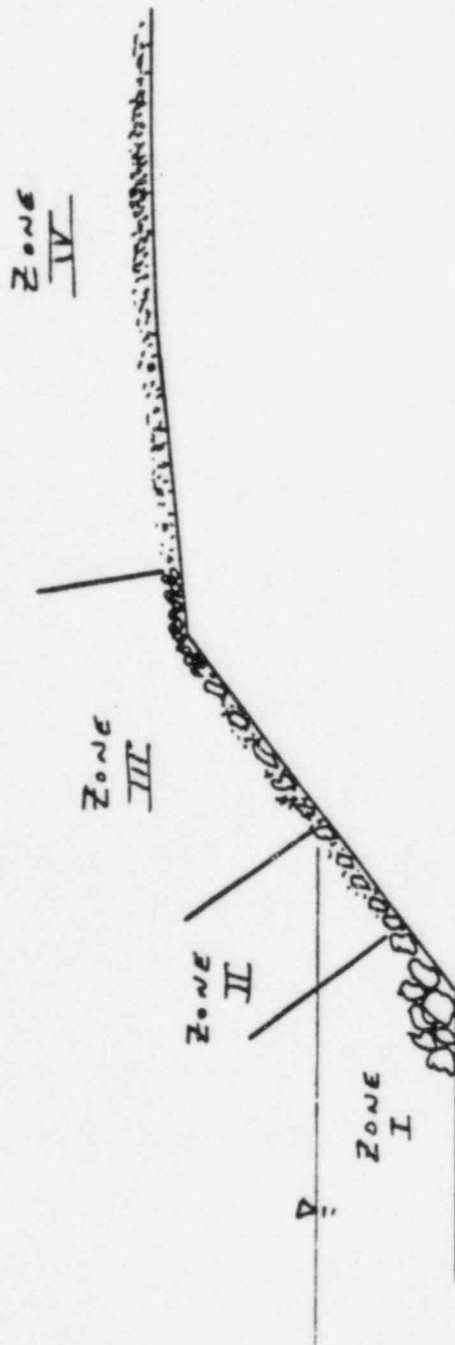
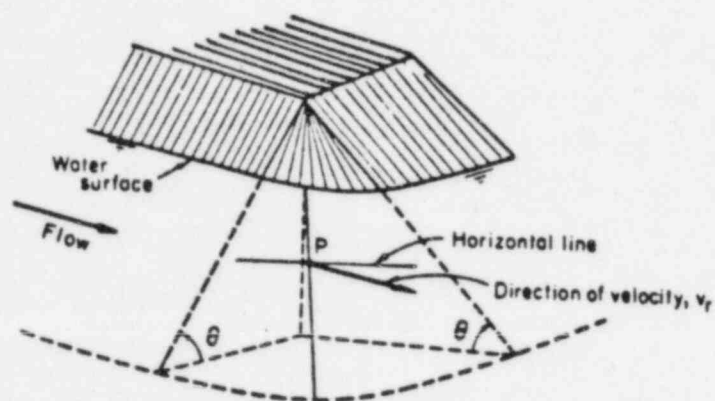
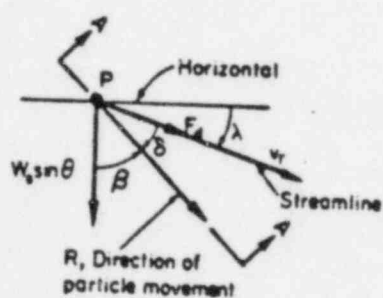


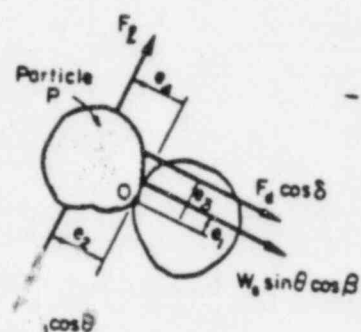
Fig. 5.5



(a) General view



(b) View normal to the side slope



Section A-A

Fig. 5.6 Diagram describing riprap stability conditions.



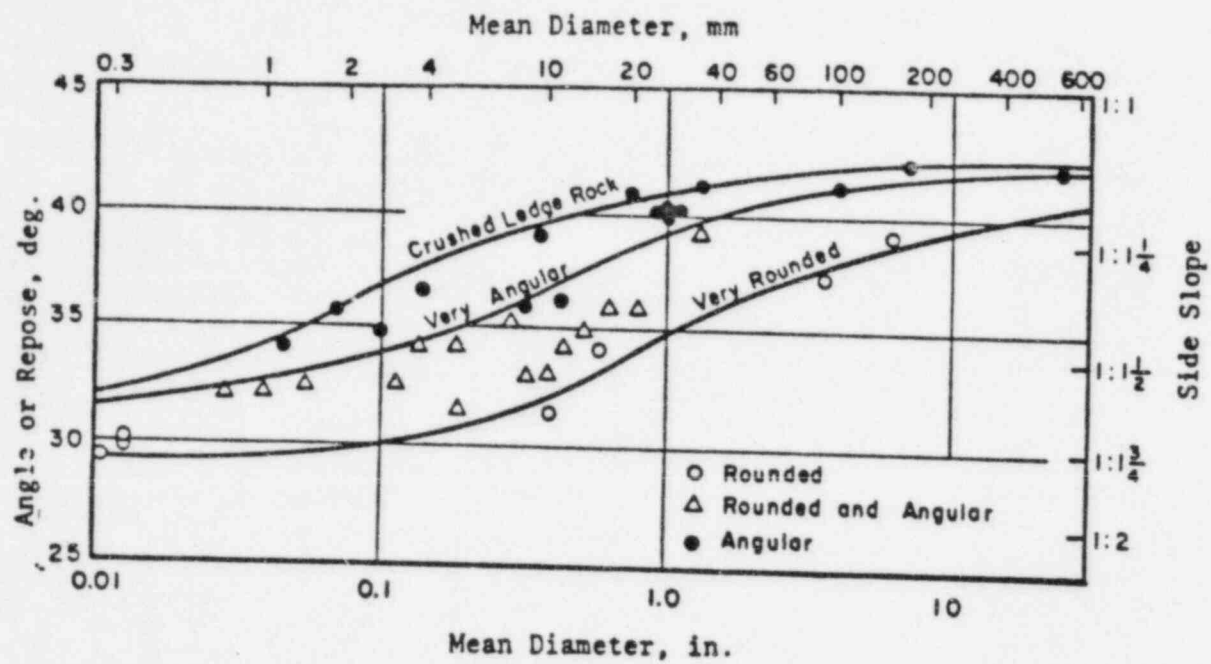


Fig. 5.7 Angle of repose.

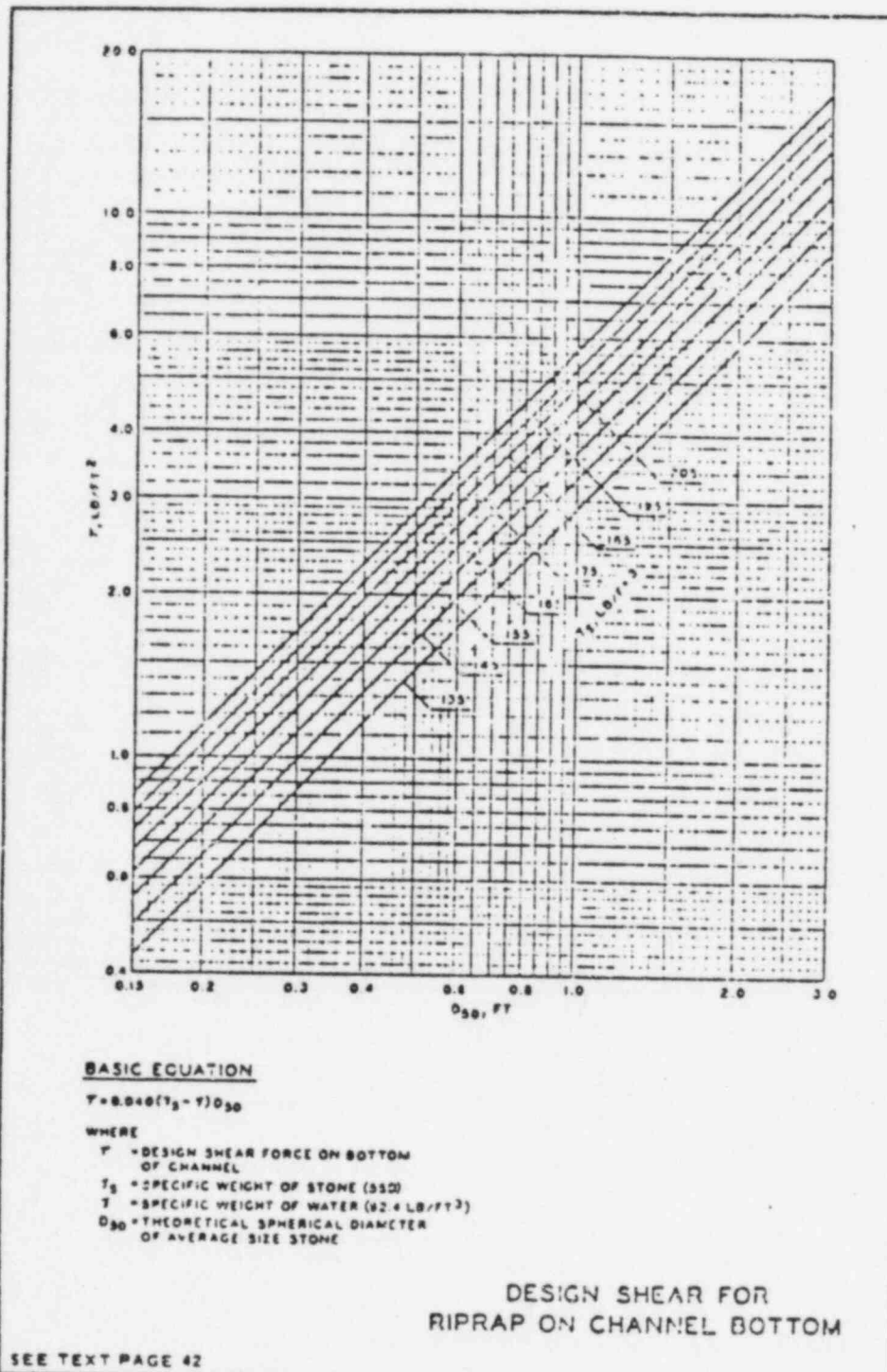


Plate 35

III-37

Fig. 5.8

5.2.2.5 U.S. Bureau of Reclamation Method

The U.S. Bureau of Reclamation (USBR) Method (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 underestimates the required stone size in highly turbulent zones and that there is a tendency for in-place 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 bottom velocity of flow,  $V$  in feet per second. One means of predicting the velocity impacting the stones is using the Mavis and Laushey (1948) procedure where

$$V_b = 0.5 (d_1)^{0.5} (s-1)^{0.5}$$

Why should the velocity be a function of  $G_s$ ?  
& not of slope &  $\phi$ ? (5.2.2)

as  $d_1$  is the particle diameter in mm and  $s$  is the particle specific gravity.

The stone size and stone weight can be determined by entering Fig. 5.9 with the 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 inch thick.

of rock  
It is recommended that the USBR method be considered only for <sup>design</sup> use along the toe-of-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 conservation.

References

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

Mavis, F.T., and Laushey, L.M., 'A Reappraisal of the Beginnings of Bed Movement-Competent Velocity', Proceedings of the International Association for Hydraulic Structures Research, Stockholm, Sweden, 1948.

# SIZE OF RIPRAP TO BE USED DOWNSTREAM FROM STILLING BASINS

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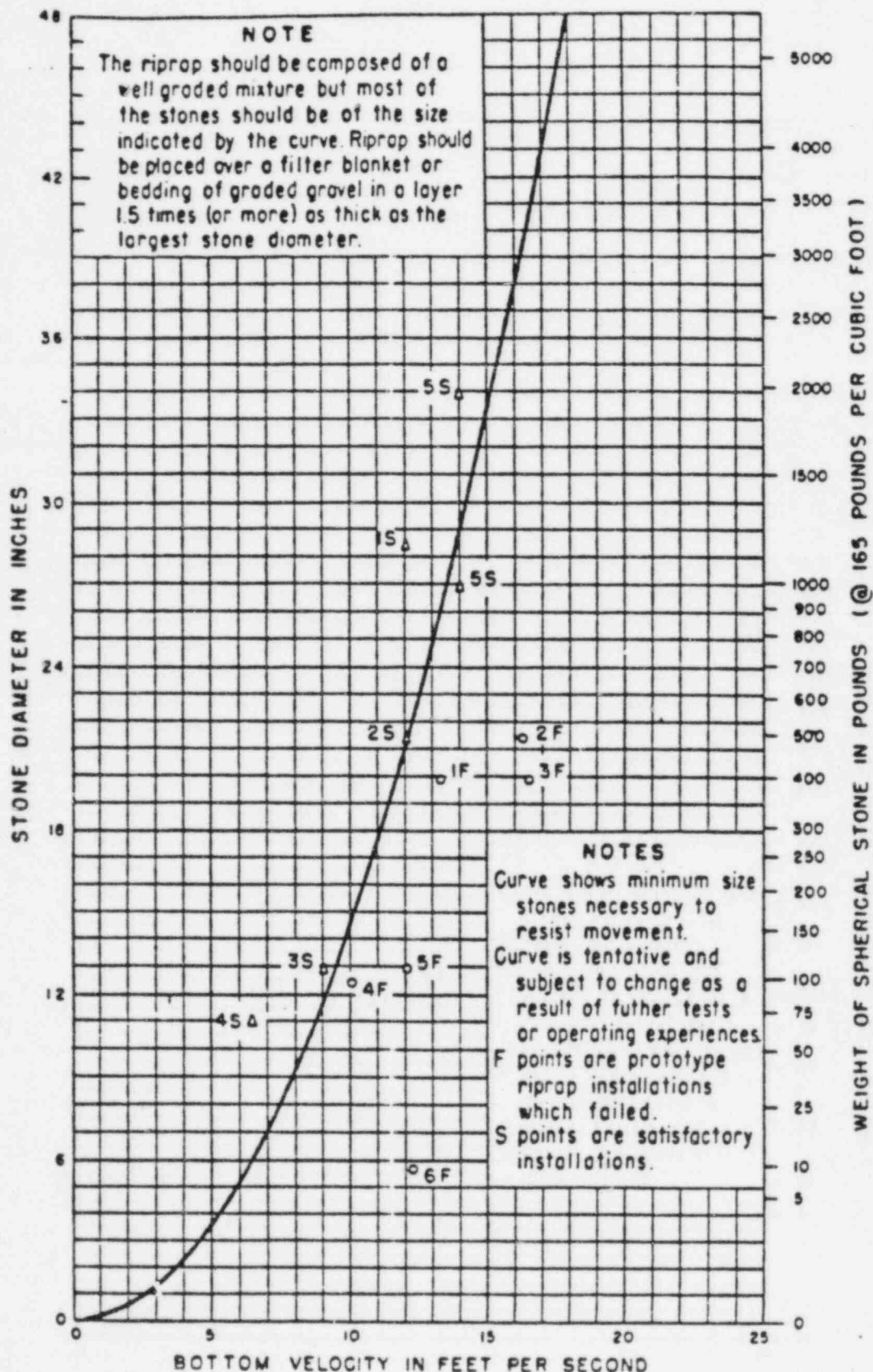


Fig. 5.9 Curve to Determine Maximum Stone Size in Riprap Mixture

(Ref. ?)

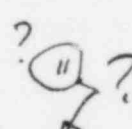
#### 5.2.2.6 Alternative Design Methods


The riprap or stone design methods presented are but a few of the procedures available to the cover designer. Alternative methods include:

1. Bureau of Public Roads Method
2. California Division of Highways Method (1970)
3. Lane Method (1953)
4. Shen and Lu Method (1983)


It is recommended that each method be evaluated as to its applicability to an appropriate segment of the cover design. However, cost effectiveness of cover stabilization must be taken into account in the design process.

#### References

Shen, H.W. and Lu, J.Y., Development and Prediction of Bed Armoring, 

ASCE J. of the Hyd. Division, Vol. 109, No. 4, pp 611-629, April 1983. 

California Division of Highways, Bank and Shore Protection in California

Highway Practices, Business and Transportation Agency, Department of Public Works, Sacramento, Cal., 1970. 



### 5.3 Slope Transition Protection

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

It is recommended that the slope transition areas be protected (i.e. riprap, rock mulch, etc.) at least ten (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 armouring that will resist degradation, particularly from unexpected, concentrated flows.

### 5.5 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. (Hirschfeld et al. 1973) presented a basic equation for turbulent flow through rockfill as

$$V_v = W_m^{0.5} i^{0.54} \quad (5.24)$$

where  $V_v$  is the average velocity of water in the voids of the rockfill,  $W_m$  is an empirical constant for a specific riprap material,  $m$  is the hydraulic mean radius and  $i$  is the hydraulic gradient. The void velocity,  $V_v$ , determined in Eqn. 5.24 is presented in in./sec. Table 5.1 presents a series of empirically derived values for the hydraulic mean radius,  $m$ , and the  $W_m^{0.5}$  parameter as presented by Hirschfeld et al. (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 Eqn. 5.24 was derived and applicable to a uniformly graded rockfill, the procedure is considered applicable to well graded rockfill provided that the minus 1-in. material is less than 30%. Hirschfeld indicated that if more than 30% of the minus 1-in. material is present, the rockfill should be treated as earthfill.

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

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D - NRC f - rpt.abt.a

$$q = \frac{V_y}{12} d_r$$

(5.25)

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

(Editors)

Hirschfeld, R.C. and Poulos, S.J., Embankment-Dam Engineering, John Wiley  
Sons, pp. 87-107, 1973. X

Table 5.1

Empirical Derived Values for Eqn. 5.24

Rock size (in.)	m (in.)	$m^{0.5}$ (in. <sup>1/2</sup> )	$W_m^{0.5}$ (in./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

#### 5.4 Filter Criteria

It is recommended that a layer or blanket of well-graded gravel should be placed over the embankment or cover slope prior to riprap placement. Sizes of gravel in the filter blanket should <sup>have</sup> ~~be from 3/16 in. to~~ an upper limit depending on the gradation of the riprap with maximum sizes of approximately 3 to 3 1/2 inches. The filter thickness shall vary depending upon the riprap thickness, but should not be less than 6 to 9 inches. Filters <sup>thickness equal to</sup> that are <sup>one-half</sup> the riprap layer thickness is recommended. Suggested specifications for gradation of the filters are as follows:

$$\frac{D_{15} \text{ (Filter)}}{D_{85} \text{ (Case)}} \leq 5 \quad \text{to prevent migration of bedding into riprap.} \quad (5.23)$$

$\leq 10$  to prevent erosion of radon barrier below bedding.  
(See Sherard, 1984.)

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Sherard, J.L., Dunnigan, L.P., and Talbot, J.R., "Basic Properties of Sand and Gravel Filters," ASCE, Journal of the Geotechnical Engineering Division, Vol. 110, No. 6, June, 1984.

### 5.10 Permissible Velocities

Evaluation of proposed reclamation alternatives should include an analysis of the cover material critical erosion potential. 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 criteria 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 determine 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 the long term. 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 Scooby (1926), <sup>and</sup> ~~which were~~ adapted by Lane (1955). The Lane maximum permissible velocities ~~are presented~~ in Table 5.6, ~~and~~ are applicable to colloidal silts. These velocity values were developed for channels without sinuosity. Lane recommended a reduction of the velocities in Table 5.6 by 13 percent if the canal/channel is moderately sinuous.

Lane also presented a series of maximum allowable velocities for sandy based materials. Table 5.7 presents the allowable velocity ranges for these materials.

Lane adapted limiting velocities for cohesive materials according to compactness for materials with less than 50 percent sand content. Table 5.8 presents these limiting, or upper bound velocities.

The U.S. Department of Agriculture (1984) ~~presented a series of~~ maximum permissible velocities for well maintained grass covers <sup>are</sup> as presented in Table 5.9. It should be noted that these velocity limits pertain to slopes ranging from 0 to 10 percent.

Fortier, S., and Scobey, F.C., 'Permissible Canal Velocities', Transactions, ASCE, Vol. 99, paper no. 1533, 1926.

Lane, E.W., 'Design of Stable Channels', Transactions, ASCE, Vol. 120, paper no. 2276, pp. 1234-1279, 1955.

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



Table 5.6

Maximum Permissible Velocities in Erodible Channels

<u>Channel Material</u>	<u>Water Transporting Colloidal Silts</u>  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

\* Above material descriptions do not correspond to the Unified Classification Syst., eg. colloidal & non-colloidal are not of standard usage. Also terms like loam and shingles should not be used - equivalent Unified terms should be substituted.

Table 5.7

Maximum Allowable Velocities	
Material	Velocity (ft/sec)
<u>very light sand of quicksand character</u> *	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 shingles	5.00 to 6.00
conglomerate, cemented gravel, soft slate, tough hardpan, soft sedimentary rock	6.00 to 8.00

\* this is not a material type, rather a condition.  
see also note on ~~page~~ bottom of preceding page.

Table 5.8

Limiting Velocities in Cohesive Materials

Principle cohesive material	<u>Compactness of Bed</u>			
	Loose	Fairly Compact	Compact	Very Compact
	Velocity (ft/sec)	Velocity (ft/sec)	Velocity (ft/sec)	Velocity (ft/sec)
sandy clay	1.48	2.95	4.26	5.90
high plasticity heavy clayey soils	1.31	2.79	4.10	5.58
clays	1.15	2.62	3.94	5.41
low plasticity lean clayey soils	1.05	2.30	3.44	4.43

Table 5.9

Maximum Permissible Velocities in Channels Lined with Uniform Stands  
of Various Well-Maintained Grass Covers (5)

COVER	MAXIMUM PERMISSIBLE VELOCITIES IN FT./SEC. FOR:		
	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 . . . . .	0-5 (2)	5	4
Grass mixture . . . . .	5-10 (2)	4	3
Lespedeza sericea . . . . .			
Weeping lovegrass . . . . .			
Yellow bluestem . . . . .	0-5 (3)	3.5	2.5
Kudzu . . . . .			
Alfalfa . . . . .			
Crabgrass . . . . .			
Common lespedeza (4) . . . . .	0-5 (3)	3.5	2.5
Sudangrass (4) . . . . .			

- (1) Use velocities over 5 f.p.s. only where good covers and proper maintenance can be obtained.
- (2) Do not use on slopes steeper than 10 percent.
- (3) Use on slopes steeper than 5 percent is not recommended.
- (4) Annuals, used on mild slopes or as temporary protection until permanent covers are established.
- (5) U.S. Department of Agriculture, Soil Conservation Service Engineering Field Manual, 1984.

### 5.6 Hydraulic Computations

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

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \quad (5.26)$$

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 present in Section       . To determine the discharge, Q, Eqn.        can be modified to

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2} \quad (5.27)$$

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.

It is evident that although the Manning formula is simplistic, it yields a good estimate of the discharge and/or velocity of flow. However, alternative procedures such as the Chezy formula or many other sophisticated numerical models may also be used.

### 5.7 Determination of the Manning Roughness Coefficient

The greatest difficulty in applying the Manning formula is the determination of the boundary roughness coefficient,  $n$ . The  $n$  value is an estimate of the flow resistance, to which there is not an exact procedure or method for determination ~~x~~ of this resistance.

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, silting and scouring, obstructions and channel shape. Chow (1959) and the U.S. Geologic Survey (1967) present a comprehensive list of  $n$  values for open channel applications. Manning's  $n$  values range from 0.017 for smooth channels free from growth to 0.07 for cobble bed streams.

The Manning Equation 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 flow depth less than 1.0 ft. and is significantly influenced by the boundary shear or resistance to flow. Coefficients of roughness for overland flow are presented in Table 5.2 .

Morris and Wiggert (1972) published a list of  $n$  values that have been adopted by the U.S. Bureau of Reclamation, and <sup>are</sup> presented in Table 5.3. These values apply to well-seasoned, straight channels on mild slopes, with flow depths less than 3.0 ft.

A series of Manning Coefficient,  $n$ , values were adopted by the U.S. Department of the Interior (1975) for natural channels and streams. These values are presented in Table 5.4.

One of the most difficult Manning's roughness values to determine is for riprap. Riprap serves as an alternative surface stability technique



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that provides considerable resistance to flow resulting in velocity and energy dissipation. An expression for determining the Manning coefficient,

n, value for riprap (1975) is <sup>Author</sup>

$$n = 0.0395 (d_{50})^{\frac{1}{4}} \quad \text{Independent of flow depth?} \quad (5.28)$$

where  $d_{50}$  is the mean rock size <sup>in feet</sup>. A graphical representation for determining 'n' is presented in Fig. 5.10 .

What about C. of E. formula?



Project

UMTRA - Criteria

Feature

Item

Contract No.

Designed ABH

Checked

Sheet 1/2

File No.

Date 8/28/85

Date

Manning's Roughness Coefficient  $n$ calculated  
by various  
methods.  
for comparison.

$D_{50} = \begin{cases} \text{inches} \\ \text{feet} \end{cases}$		1	3	6	12	18	30
		0.0833	0.25	0.5	1.0	1.5	2.5
Manning's $n$	by Strickler's formula	0.023	0.027	0.031	0.0342	0.037	0.040
	by CSU/NRC formula	0.026	0.031	0.035	0.0395	0.042	0.046
	by Corps of Eng'rs formula						
	for $R/k = 0.2$	0.059	0.071	0.080	0.090	0.096	0.105
	0.5	0.034	0.041	0.046	0.052	0.055	0.060
	1.0	0.028	0.033	0.037	0.042	0.045	0.049
	5.0	0.022	0.027	0.030	0.033	0.036	0.039
	10.0	0.021	0.025	0.029	0.032	0.034	0.037

Strickler's formula:

$$n = 0.0342 k^{1/6}, \text{ where } k \text{ is in feet.}$$

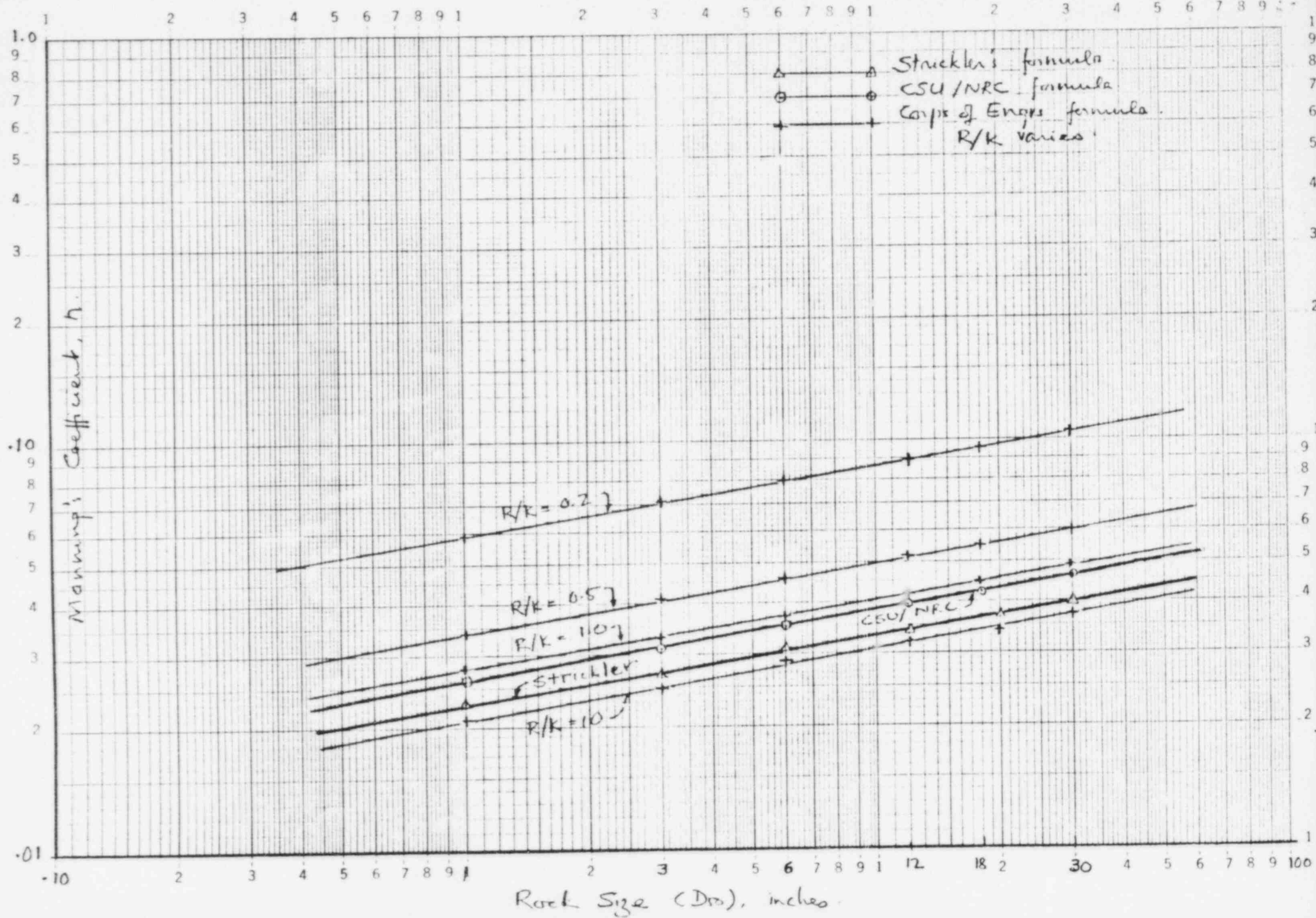
CSU/NRC formula:

$$n = 0.0395 k^{1/6}, \text{ } k \text{ is in feet.}$$

Corps of Engineer's formula:

$$n = \frac{R^{1/6}}{23.85 + 21.95 \log(R/k)}, \text{ } R \text{ \& } k \text{ in feet.}$$



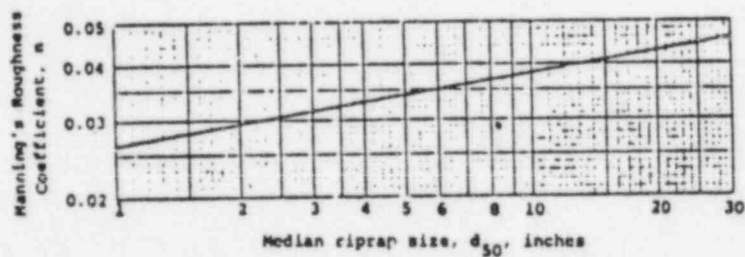


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Chow, V.T., 'Open-Channel Hydraulics', McGraw-Hill Book Company, 1959.

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

Fig. 5.10 - Manning's Coefficient for Riprap



USDA, SCS "Standards And Specifications for Soil Erosion  
and Sediment Control in Developing Areas" 1975 APPENDIX A

Table 5.2

Coefficients of Roughness for Overland Flow

Surface	Value of n
Pavements and paved shoulders	0.01
Bare packed soil free of stone	0.10
Sparse grass cover, or moderately rough bare surface	0.20
Average grass cover	0.40
Dense grass cover	0.80

(~) U.S. Army and Air Force 'Surface Drainage Facilities for  
Airfields and Heliports' TM 5-820-1 or AFM 88-5, Chapter 1, 19

?

Table 5.3

Manning Coefficient, n for rough calcs. ?

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

Morris, H.M. and James M. Wiggert 'Applied Hydraulics in Engineering'  
2nd edition 1972 publ. Wiley Sons.



Table 5.4 (See table 5.3)

Manning Coefficient, n, for Natural Channels

NATURAL CHANNEL CONDITION	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 variable 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

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

### 5.8 Cover Erosion Resistance Evaluation

The cover design should be evaluated to determine if the unprotected slope(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 can 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 (see Sec. 6) and for sizing stone protection (see Sec. 5.2).

#### Determination of the Design Velocity:

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

- a) Using models (HEC-1, USACE, etc.), that are widely accepted by the engineering profession
- b) Applying the Rational Method for tributary areas on the cover that are less than or equal to one square mile

The rational formula is commonly expressed as

$$Q = CiA$$

*What about  
Santa-Barbara  
Method?  
(5.29)*

where Q is the maximum or design discharge in cfs, C is a runoff coefficient, i is the rainfall intensity expressed in inches per hour and A is the tributary area expressed in square miles. When a unit width approach is taken, the area ~~shall be~~<sup>is</sup> expressed as the slope(s) length times the unit width. Therefore, Eqn.        would be presented as

$$q = CiA$$

(5.30)

for a unit width analysis.

### 5.8.1 'C' Coefficient

The runoff coefficient, C, is related to the type of terrain to include the material, cover permeability and storage potential. Example values of the coefficient 'C' values are presented in Table 5.5 (1958).

Table 5.5 Values of Coefficient C

<u>Type Area</u>	<u>Value of C</u>
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

It is recommended that a conservative estimate of, C, for PMF computations is 1.0. ~~Also recommended~~ <sup>Also consider</sup> values given by Office of Surface Mining (attached).

### 5.8.2 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:

- Applying one of the accepted mathematical models
- Assuming that the average sheet or overland flow velocity down the entire length of slope is approximately equal to 1/2 of the design velocity. The time of concentration is

$$t_c = \frac{\text{slope length}}{\text{ave. velocity}} \quad (\text{seconds}) \quad (5.31)$$

where the time of concentration,  $t_c$ , should be converted to minutes.

- Using the Soil Conservation Service (SCS) Triangular Hydrograph Theory ( ), the

Table 3.3.\* Rational Runoff Coefficients  
(after Schwab et al., 1971).

Topography and Vegetation	Values of C in $Q = CiA$		
	Soil Texture		
	Open Sandy Loam	Clay and Silt Loam	Tight Clay
<u>Woodland</u>			
Flat 0-5% slope	0.10	0.30	0.40
Rolling 5-10% slope	0.25	0.35	0.50
Hilly 10-30% slope*	0.30	0.50	0.60
<u>Pasture</u>			
Flat	0.10	0.30	0.40
Rolling	0.16	0.36	0.55
Hilly	0.22	0.42	0.60
<u>Cultivated</u>			
Flat 0-5% } slope	0.30	0.50	0.60
Rolling 5-10% }	0.40	0.60	0.70
Hilly 10-30% }	0.52	0.72	0.82

\*Values are not available for steeper slope conditions, so when applying the Rational Formula to steeper slopes this limitation must be realized.

\* Ref: Surface Water Diversion Manual  
September 1982

US Dept. of Interior  
Office of Surface Mining

time of concentration is

$$t_c = \left[ \frac{11.9 L^3}{H} \right]^{0.385} \quad (\text{Ref. ?}) \quad (5.32)$$

$$= 0.0078 L^{0.77} S^{-0.385} \quad (\text{Gray, Handbook on the Principles of Hydrology, 1970}) \quad (\text{D.H. Gray})$$

where L is the length of the longest water course from the point of interest to the tributary divide, in miles, H is the difference in elevation in feet between the point of interest and the tributary divide, in feet.

The time of concentration will be expressed in hours and should be converted to minutes.

Once the rainfall duration or time of concentration is determined, the rainfall depth can be computed based on the PMP intensity values estimated in Sec. 2.2.2.

#### 5.8.3 Tributary Area

The tributary area shall be expressed in a unit width format. Therefore, the area is the length of the longest expected or measured water course multiplied by the unit width converted to square miles.

#### 5.8.4 Sheet Flow Velocity

The design velocity can be estimated by solving the Manning formula presented in Eqn. \_\_\_\_\_. 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

$$y = \left[ \frac{Q_n}{1.486 S^{1/2}} \right]^{3/5} \quad (5.33)$$

Therefore, the design velocity is estimated as

rough draft NRC report - 4/4/85  
D - NRC f - rpt.abt.c

$$V_{\text{Design}} = Q/A \text{ (ft/sec)}$$

(5.34)

#### References

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

U.S. Bureau of Reclamation, Design of Small Dams, U.S. Government Printing  
Office, 1977.



### 5.9 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. Cover modifications will result from differential settlements, 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 <sup>may be</sup> ~~result is~~ that the peak discharge and volume of runoff exceed design levels and increase the erosion potential. The ratio of the actual peak discharge at a design point on the cover to the design peak discharge is considered the flow concentration and can be expressed as

$$F.C. = \frac{Q_{actual}}{Q_{design}}$$

(5.35)

The peak discharge 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 should 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 little information is currently available to substantiate

design upper limits. However, it is reasonable to assume that values between 2 and 3 are attainable with only a slight evolutionary change in cover. It is recommended that a conservation concentration factor be used until additional research can justify a reasonable range of values, or the

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 factor.

embankment is overbuilt to compensate for settlement.

## 6.0 EVALUATING THE POTENTIAL FOR SURFACE SHEET EROSION

### 6.1 Introduction

Due to the fine-grained noncohesive nature of uranium tailings, these materials are highly erodible when subjected to the erosive forces of wind and water. The high potential for sheet erosion and the potential for transportation of 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 the tailings material. This chapter presents a discussion of the engineering analysis techniques that will be used in the Phase II study effort to evaluate the surface erosion potential and predictive capabilities for estimating the life of protective covers. X

### 6.2 Background

Soil particles can become detached when the impact of rainfall, or the stresses caused by wind or water, are in excess of the ability of the soil to remain stable. Factors which tend to stabilize the soil and retard 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 this detachment process and properly consider this erosion potential over the entire period covered by the reclamation.

The application of the Universal Soil Loss Equation (USLE) is considered to offer 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 (Nelson, et al., 1983; Nyhan and Lane, 1983; and Walters, 1983), indicate that whereas other, more

mathematically rigorous, models do exist to simulate erosion as a function of time, the use of the USLE has a strong precedent since it has a 30 year history of basic runoff and soil loss data.

### 6.3 The Universal Soil Loss Equation

#### 6.3.1 Description of the Equation

The USLE is a mathematical model based on field determined coefficients that is used to evaluate average soil losses for certain types of slopes as a function of time. The basic development of the USLE does not consider the potential for gully development or intrusion as discussed in Chapter 5 since the topographic features of the relationship are assumed to remain constant with time. The USLE is defined as follows:

$$A = RKLSCP$$

where,

A = the computed loss per unit area, expressed in the units selected for K and for the period selected for R, usually selected so that they compute A in units of tons per acre per year;

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, which is defined as a 72.6-ft length of uniform 9% slope continuously in clear/tilled fallow;

L = the slope-length 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 *to the slope under consideration. ?*

S = the slope-steepness factor, which is the ratio of soil loss from the field slope gradient to that from a 9% slope under otherwise identical conditions *to those of the slope under consideration. ?*

C = the cover management factor, which is the ratio of soil loss

from an area with specified cover and management to that from an identical area in tilled continuous fallow;

P = the support factor, which is the ratio of soil loss with a support practice like contouring, stripcropping, or terracing to that with straight-row farming up and down slope.

Since its development in 1954, the USLE has been the focus of on-going continuous research and verification, which is the main reason for its consideration in evaluating the design of tailings covers. Since 1965, the USLE has been expanded for use in different climatic conditions, and additional land uses and management practices. The bulk of the on-going research indicates that although the variables affecting the input parameters to the USLE, in particular the erosion factors, vary considerably about their means, the effects of random fluctuations tend to average out over extended periods. These results suggest that the application of the USLE is less accurate in estimating losses for a specific storm than for the prediction over longer periods.

#### 6.3.2 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 USLE 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 as a rainfall and runoff factor and is computed as the product of rainfall storm energy (E) and the maximum 30-minute rainfall intensity ( $I_{30}$ ). However, for the Phase 2 studies, it will be assumed that rainfall intensity records are not available for the study site. Data for the R value will be chosen from an appropriate value shown on Figure 6.1, which presents contours of average annual values of the rainfall erosion index for the

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"Average" R  
may not be  
good enough

e.g. see  
Isaacson, et al.

We can get site-specific " $I_{30}$ ". Can we also calculate site-specific "E" instead of using average-<sup>3</sup>-R-values, or modify average R-values based on  $I_{30}$  (ie, what  $I_{30}$  values are used for average annual R-values)?

Not given!



continental United States.

*R should 1) be site-specific and 2) apply to the design period (i.e., be adjusted for storm recurrence interval) (SEE "EROSION CONTROL DURING HIGHWAY CONSTRUCTION", PP 5-6)*

### 6.3.3 The Soil Erodibility Factor (K)

The soil erodibility factor (K) recognizes 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. The factor also considers the percentage of organic matter present, the type of soil structure, and a class of permeability. A nomograph for use in determining the K factor is presented in Figure 6.2. ?

### 6.3.4 The Combined 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 (1973) developed plots correlating the topographic factor for slopes up to 500 meters in length at slope inclinations from 0.5% up to 50%. The slope effect chart is presented in Figure

6.3. ?

### 6.3.5 The Cover Management Factor (C)

The C factor is the most difficult factor to estimate in the USLE since it involves so many interrelated factors. In its original development, a value of C equal to unity is assumed for an area that is tilled and continuously fallow. Based on field experience and limited research data, Meyer and Ports (1976) published C factors that have direct applicability for protective cover options which are reproduced in Table 6.1. C factors for other typical types of vegetative canopy supported on the protective



continental United States.

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cover are presented in Table 6.2. ?

#### 6.3.6 The Support Practice Factor (P)

The support practice factor is the ratio of soil loss with a specific support practice to the corresponding loss of upslope and downslope straight-row farming. Since the application of the USLE for the present study does not assume that any practice will occur once a site has been reclaimed, the support factor to be used should follow that recommended by Wischmeier and Smith (1978), which is reproduced as Table 6.3. These data indicate that the P value is simply a function of the ultimate slope of the reclaimed surface.

#### 6.4 Proposed Methodology

The main application of the USLE to the evaluation of cover integrity is to evaluate whether it is possible for sheet erosion to penetrate the tailings cover, thereby exposing bare tailings and constituting a failure of the cover. The Phase 2 study effort will concentrate on using the USLE 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. ?

As mentioned previously, the application of the USLE does not enable the evaluation of average soil loss in the event of gully intrusion into the cover area. To the degree practical, an assessment will be attempted to correlate the results from the gully prediction capability (See Chapter 5), with the results from the USLE studies.

## References

- J.W. Nyhan and L.J. Lane, 1983, 'Erosion Control Technology: A User's Guide to the Use of the Universal Soil Loss Equation at Waste Burial Sites', Los Alamos National Laboratories, Los Alamos, NM
- J.D. Nelson, R.L. Volpe, R.E. Wardwell, S.A. Schumm, W.P. Staub, October 1983, 'Design Considerations for Long-Term Stabilization of Uranium Mill Tailings Impoundments', NUREG/CR-3397
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rough draft - Staub - NRC report - 3/7/85

## 7. Selection of Riprap

### 7.1 Introduction

*Also See attached typed  
comments submitted  
on 8-26-85.*

This section 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 which is less resistant to weathering processes. However, marginal quality rock should be excluded from use in certain critical areas (see Section 5.2). 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.

Oversizing individual stones of marginal quality rock will not completely remove uncertainty in meeting performance objectives. Weathering rates of marginal quality rock (especially when placed in positions of exposure) cannot be determined with any degree of reliability. Thus, oversizing adds only an unknown incremental degree of long-term erosion protection.

There is evidence to suggest that increasing the thickness of a riprap blanket is an effective alternative to oversizing the stones. Hanegan (1984) discusses the use of marginal quality riprap on the outer shell of a dam <sup>located</sup> ~~in~~ <sup>in</sup> ~~the~~ <sup>in</sup> ~~outer~~ <sup>in</sup> ~~3 feet~~ <sup>in</sup> ~~of the shell~~ <sup>in</sup> ~~had deteriorated badly within seven years~~ <sup>in</sup> ~~after placement.~~ <sup>in</sup> ~~The next 3 feet showed only hairline cracking and below~~ <sup>in</sup> ~~that little or no deterioration was observed.~~ <sup>in</sup> Hanegan believes that the buried rock experienced little alteration because temperature and moisture content fluctuations were minimal.

The riprap selection methodology developed in this section 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 up to expectation.

Is there  
really that  
much  
variation  
in  
handling?

## 7.2 Micro-Environmental ~~Considerations~~

Basically there are two distinctly different environments affecting the durability of riprap in the uranium mill tailings management area. They are: 1) the relatively small but frequently saturated areas at and near the downstream toe of an embankment dam, 2) the somewhat larger areas that are occasionally saturated during flood events, and 3) the much larger areas farther up the embankment face that are rarely saturated during extreme events on the tailings cover and in peripheral drains which are used infrequently to divert run-off water away from the tailings. The upper elevations of frequently and occasionally saturated zones may be defined as the annual and 20 year flood events, respectively.

Zoned construction of riprap blankets will be a practical necessity at most uranium mill tailings management areas in the United States. Frequently saturated areas will require erosion protection by highly durable riprap which is generally not locally available (Nelson and others, 1983). Small quantities of good riprap can be transported significant distances without placing an undue burden on the licensee. However, to hold <sup>initial</sup> costs within reason local sources of marginal quality riprap should be considered for protecting the much larger areas that are infrequently or rarely

?

saturated.

### 7.3 Riprap for Frequently Saturated Areas

#### 7.3.1 Recommended Rock Types

Only highly durable rock should be considered for use in frequently saturated areas. Jahns (1982) <sup>Not on ref. list.</sup> suggests that rocks meeting the specifications of superior building stone for exterior use should be relatively resistant to weathering. Table 7.1 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. Thus, Group 3 rocks should not be considered for use in stabilizing potential seepage zones of acidic tailings impoundments (Nelson and other, 1983).

x  
Preferred  
for  
Building  
Stone,  
but not  
necessari-  
ly as rip-  
rap.

(Not expected  
@ LMTRA Sites)

##### 7.3.1.1 Prospecting

Extensive data files are available for locating suitable and assess-  
able igneous and metamorphic rock quarries in western United States. Among  
them are the open-file of the U.S. Army Corps of Engineers (COE), the U.S.  
Bureau of Reclamation (USBR), and 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) pro-  
vides an analysis of performance of riprap at 149 USBR dams. It should be  
possible to identify several candidate sources of durable riprap within <sup>180</sup>300  
miles  
km of a mill tailings site.

?  
x  
?



Table 7.1 Rock priority groupings for external use as building stone. (Source: Jahns, 1982)

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

In general it will 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.

Candidate sources of riprap should be studied in place for comparison of lithologic and structural features, outcrop characteristics, and relative degrees and styles of weathering. These rocks should also be studied in quarry spoil piles, nearby talus accumulations, at dams, other structures and monuments where they were used.

#### 7.3.1.2 Selection

The quality of candidate sources of riprap can be compared with one another by examining the results of standard durability tests. DePuy and Ensign (1965) proposed a list of acceptance criteria (Table 7.2) for various durability tests. Results of most of the tests in Table 5.2 as well as petrographic analysis are available from COE, USBR, or state highway



department open-file data. Thus, preliminary comparisons and acceptability among candidate sources of riprap can be made without performing additional tests. DePuy (1965) also rated the ability of durability tests to accurately predict rock quality (Table 7.3). At present, the USBR places even greater importance on petrographic analysis than that indicated in Table 7.3. X

Two facts must be kept in mind regarding standard durability tests and open-file data. First, most tests are performed on small specimens intended for use as concrete aggregate. Second, DePuy and Ensign's acceptance criteria were developed for riprap to be used as upstream slope protection against erosion by waves.

*7.4 Preliminary acceptance of a candidate riprap should be followed by specific durability tests. The degree of chemical and mechanical degradation the candidate riprap will undergo should be investigated by tests that represent site conditions.*

Table 7.2 U.S. Bureau of Reclamation standards for judging  
riprap durability (DePuy and Ensign 1965).

Test	Quality		
	Poor (1)	Fair (2)	Good (3)
Ultrasonic cavitation rating	0 to 5	5 to 7	7 to 10
Bulk specific gravity	2.5	2.5 to 2.65	2.65
Adsorption %	1.0	0.5 to 1.0	0.5
Freeze-thaw weight loss, %(a)	5	0.5	0 to 0.5
Na <sub>2</sub> SO <sub>4</sub> weight loss, %	10	5 to 10	5
Los Angeles abrasion loss, %(b)	10	5 to 10	5
Schmidt impact hammer	40	40 to 60	60
Scleroscope	30	30 to 50	50
Coefficient of restitution(c)	0.5	0.5 to 0.7	0.7
Tensile strength, psi	500	500 to 1,000	1,000
Compressive strength, psi	15,000	15,000 to 20,000	20,000
Sonic velocity, ft/sec	15,000	15,000 to 17,000	17,000

(a) 250 cycles  
(b) 100 revolutions  
(c) rebound hardness

- It seems that some of the problems could arise from the use of these standards:*
- for example, although unconfined compressive strength increases with specific gravity, a high specific gravity does not guarantee the high durability of a candidate riprap — a dolomite riprap, composed of the mineral dolomite, with a bulk specific gravity of around 2.67 would certainly be less durable than a pure quartz sandstone, with a bulk specific gravity of around 2.35, in an acidic tailings environment*
  - for example, compressive strength standard can be too restrictive — granite, which is considered to be a high performance riprap material, varies in compressive strength from about 700–1750 kg/cm<sup>2</sup> (10,000–25,000 psi) — by using the above std, a high performing riprap material could possibly be rejected.*

Table 7.2 U.S. Bureau of Reclamation standards for judging riprap durability (DePuy and Ensign 1965).

Test	Quality		
	Poor (1)	Fair (2)	Good (3)
Ultrasonic cavitation rating	0 to 5	5 to 7	7 to 10
Bulk specific gravity	2.5	2.5 to 2.65	2.65
Adsorption %	1.0	0.5 to 1.0	0.5
Freeze-thaw weight loss, %(a)	5	0.5	0 to 0.5
Na <sub>2</sub> SO <sub>4</sub> weight loss, %	10	5 to 10	5
Los Angeles abrasion loss, %(b)	10	5 to 10	5
Schmidt impact hammer	40	40 to 60	60
Scleroscope	30	30 to 50	50
Coefficient of restitution(c)	0.5	0.5 to 0.7	0.7
Tensile strength, psi	500	500 to 1,000	1,000
Compressive strength, psi	15,000	15,000 to 20,000	20,000
Sonic velocity, ft/sec	15,000	15,000 to 17,000	17,000

- (a) 250 cycles  
(b) 100 revolutions  
(c) rebound hardness

Table 7.3 Comparative rating of selected riprap durability tests for determining rock quality. The ability of a test to accurately predict rock quality is rated against a determination of rock quality using petrographic examination (DePuy 1965).

Rating	Test Method	Percent Agreement With Rock Quality
1	Petrographic examination	85
2	Tensile strength	84
3	Specific gravity	80
4	Schmidt hammer	79
5	Absorption	75
6	Freeze-thaw	70
6	Compressive strength	70
6	Scleroscope	70
6	Sulfate soundness	70
10	Coef. restitution	65
11	Pulse velocity	60
11	Cavitation	60
13	L.A. abrasion	50

The acceptance criteria of Table 7.2 and the rating system of Table 7.3 require modification in response to special environmental conditions along an embankment toe. It is expected that such areas will be chronically subject to greater tensile stresses from frost wedging, salt crystallization, absorption-desorption, and greater chemical weathering relative to reservoir embankments being protected from waves. On the otherhand, impact, abrasion, and compressive stresses will be less important. Impact and abrasive stresses will occur infrequently in response to occasional flood events. Table 7.4 is a suggested new rating system. Table 7.5 provides acceptance criteria for petrographic analysis in addition to other criteria listed by DePuy and Ensign.

Overall quality test scores ( $Q_p$ ) for candidate sources of riprap can be determined from Tables 7.2, 7.4, and 7.5. The same number (2 to 4, depending on available information) of tests should be selected from each of the three categories in Table 7.4 for comparative analysis. Then their quality scores ( $N_i$ ,  $N=1, 2$ , and  $3$  for poor, fair, and good) are multiplied by their weighting factors ( $W_i$ ) and summed to obtain their overall test scores ( $Q_p = \sum_{i=1}^n N_i W_i$ ).

If a riprap source has a test score exceeding 80% of the maximum possible score it would be considered conditionally acceptable for use at the toe of an embankment. Thus, an acceptable test score may range from 10.8 to 21.7 depending on the number of durability tests available in the scoring process.

To assure that a correct choice has been made several additional durability tests are recommended. Two of these tests are more detailed petrographic analyses and a third is a freeze-thaw test on block samples. Also, any available test results not used to compile test scores should be taken into consideration. *How?*

Table 7.4 Comparative ratings and weighting factors of selected riprap durability tests (Modified after DePuy 1965).

Category	Test Method	Weighting Factor
General weathering potential <sup>1</sup>	Bulk composition	1.00
	Secondary mineralization and weathering	1.00
	Fracture density	1.00
	Specific gravity	1.00
Tensile strength	Tensile strength	0.75
	Sulfate soundness	0.75
	Freeze-thaw	0.75
	Absorption	0.75
Compressive strength, impact, and abrasion <sup>2</sup>	Schmidt hammer	0.50
	Compressive strength	0.50
	Scleroscope	0.50
	Coef. of restitution	0.50

<sup>1</sup> Based on petrographic analysis.

<sup>2</sup> Other tests in the third category are pulse velocity, cavitation, and Los Angeles abrasion tests. These are less reliable tests according to DePuy.

Table 7.5 ~~Additional~~ petrographic analysis acceptance criteria.

	Poor (1)	Fair (2)	Good (3)
Bulk composition <sup>1</sup>	Group 3, other	Group 2	Group 1
Secondary mineralization and weathering	Carbonates and/or smectites	Other clays, if non-swelling	No clays or carbonates
Fracture density	>1/30 cm	1/30 cm > F <sub>d</sub> > 1/m	<1/m

<sup>1</sup> Groups 1, 2, and 3 rocks, see Table 7.1.

Lutton and others (1981) states that freeze-thaw tests on 5 cm (2 in) slabs often fail to predict the performance of riprap in place. Standardized freeze-thaw tests were designed to analyze the quality of rock for use as concrete aggregate. Lutton suggests that freeze-thaw tests on full sized blocks weighing up to a metric ton are more accurate indicators of

riprap performance than are similar tests on 5 cm slabs.

X-ray diffraction analysis should be performed on all candidate sources of riprap being seriously considered for use. If smectite clay minerals or carbonate minerals are identified by X-ray analysis, further chemical tests will be necessary. The ethylene glycol test is standard in many COE districts when the presence of smectites is suspected (Lutton and others, 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. Acceptance criteria for these tests are provided in Table 7.6.

*Strong?* Reaction to either ethylene glycol or acid should result in rejection.

Two alternatives to rejection are possible in cases where smectites or carbonates are identified by X-ray diffraction analysis but no chemical reactions occur. Stones could be oversized or, where carbonates are present, crushed limestone could be placed between blocks of riprap to buffer any acid solutions coming in contact with them.

Table 7.6 Acceptance criteria for X-ray diffraction analysis and chemical tests.

Reject	Conditional	Accept
<i>Strong?</i> Reaction to ethylene glycol or acid	Either smectites or carbonates are present	Neither smectites nor carbonates are present

Test scores cannot be compared on an industry wide basis. The types of durability tests run on quarry stone tend to be relatively uniform



within given USBR and COE districts or state highway departments but they may vary between districts and states. For example, freeze-thaw tests are rarely performed in southern Texas and wetting and drying tests are routinely performed only where freeze-thaw tests are not performed, generally in the southeastern states and Texas (Lutton and others, 1981).

#### 7.3.1.3 Oversizing

Riprap containing smectites may require oversizing. Groups 1 and 2 rocks containing these clay minerals may not perform well when tested by absorption, freeze-thaw or wetting and drying. Solving the problem by simply oversizing individual stones has two serious limitations. According to the USBR the maximum practical size is about one cubic meter. Furthermore, the weathering rate is unknown. Thus, increasing the stone size has no rationale on which to base a decision.

More positive protection~~s~~ can be provided by thickening the riprap blanket as well. Hageman (1984) suggests that the rate of deterioration of marginal quality riprap is a function of the depth of burial. Marginal quality rock exposed at the surface deteriorates rapidly. The same rock placed at depths where temperature and moisture fluctuations are minimal experiences little or no deterioration.

A riprap blanket utilizing marginal quality stone can be designed based on the ALARA principle. First, a design thickness base course is emplaced. Then, the base course is protected by atop course of oversized stones emplaced to a thickness sufficient to protect the base course from frost penetration. Maximum practical stone sizes should be used for this purpose. Smaller sized stones should be added to the top course to provide

additional insulation as well as to control moisture content in the base course.

### 7.3.2 Alternative Rock Types

Coarse cobblestones and small boulders excavated from nearby abandoned or existing stream channels are the most widely considered alternatives to quarried rock. Desert armor and glacial outwash deposits are less common alternatives. Coarse alluvium has been used at a number of UMITRAP sites. Examples are the Gunnison and Grand Junction tailings piles in Colorado and at Riverton, Wyoming.

Channel and outwash deposits and desert armor are inferior to quarried igneous and metamorphic rocks because of their heterogeneity and size limitations. Cobblestones are most likely to be Group 1 or Group 2 rocks of Table 7.1. For example, Wind River gravels are mainly igneous and metamorphic rocks washed downstream from distant sources high in the Wind River Mountain Range. Much coarse alluvium is reworked heterogeneous glacial outwash material that was transported downstream during more pluvial periods of the late Pleistocene. Much desert armor is a lag deposit of glacial till or alluvium wherein finer grained materials have been removed by wind erosion. Boulder sizes may range up to about 30 cm.

Although channel deposits are heterogeneous they seldom contain substantial amounts of nondurable rock. Except for a few kilometers downstream from its area outcrop nondurable rock ~~such as sandstone cobbles~~ is conspicuously absent from alluvial and outwash deposits because it rapidly disintegrates by shaking, abrasion, impact and freeze-thaw. X

#### 7.3.2.1 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 UMRAP mills previously cited are adjacent to the Gunnison, Colorado and Wind rivers. Many abandoned (UMRAP) and older operating mills are located adjacent to streams. Fewer than half of these streams contain adequate riprap resources. None of the newer mills is located near a major stream.

A few mills may be able to utilize glacial outwash or desert armor as riprap. Glacial outwash and desert armor are found in Washington and the desert southwest, respectively.

Data sources for the location of gravel pits and durability tests for coarse aggregate are the same as those listed in Section 7.3.2.

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 is difficult to identify from maps and photographs and more extensive ground reconnaissance will be required to locate it.

#### 7.3.2.2 Selection

A rigorous selection process like that described in Section 7.3.2 is unwarranted for evaluating glacial outwash deposits, alluvium, or desert armor. Unlike rock quarried in place, the above deposits are lithologically heterogeneous so that representative sampling will be difficult to achieve.

In any event these rocks survived sporadic transport over many thousands of years from outcropping source rocks located up to several hundred kilometers away. These rocks have been subjected to natural long-term durability tests in a harsh environment that could never be duplicated in the laboratory. Their lithology and degree of rounding suggests that their source beds were located at distances suggesting that considerable time was required to transport them to their present location.

Several alluvial 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 distribution and lithology. After design size criteria have been met the lithology should be examined in more detail.

The purpose of a lithologic study is to determine the percentage of nondurable rock so that the cost of its removal can be estimated. Samples should be drawn from each potential source population and examined for the presence of Group 1 and 2 rocks (Table 7.1). Rock samples can be identified by breaking 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 ( $T_K$ ):  $D_p = \frac{[R_1 + R_2]}{T_r} \times 100$ . The standard deviation should be reported to determine whether differences in composition between sources are real or are the results of sampling error. Large numbers of samples reduce the likelihood of sampling error and may be required to determine whether differences in composition are real. If there are no real differences in composition, selection should be based on land acquisition, excavation, and transportation economics.

*How is it to be removed on a productive basis? (See p. 15.)*

Nondurable rocks, organic debris, and fine grained material must be removed before alluvium can be used as riprap. Nondurable rocks (for example, sandstone) will not survive a trip through a grizzly. Rock fragments and fine grained material can be washed through a screen and larger fragments of organic debris can be removed by hand. *On a production basis?* Channel deposits from present streams will require a minimum of washing but they may have more nondurable rocks in them depending on the downstream distance from sandstone outcrops.

#### 7.3.2.3 Oversizing

If all the nondurable rock is removed oversizing should not be necessary. If some nondurable rock remains the riprap blanket's thickness could be increased in proportion to the percent of nondurable rock present.

### 7.4 Riprap for Occasionally Saturated Areas

#### 7.4.1 Recommended Rock Types

Any rock that is acceptable<sup>a</sup> for use in occasionally saturated areas. However, highly durable rock such as that described in Section 5.3 may not be locally available in sufficient quantity to protect the larger but occasionally saturated areas.

#### 7.4.2 Alternative Rock Types

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

#### 7.4.3 Prospecting

Prospecting for suitable sandstone, siltstone or carbonate rock is outlined by Foley and others (1985). Figure 5.1<sup>?</sup> is a flowchart showing steps leading up to laboratory testing. The first step is to find ledge forming strata with talus slopes consisting of fresh rock. Geologic maps and aerial photographs are suitable for locating resistant strata.

The age of talus deposits may be difficult to determine, however, there are several methodologies. Absolute age can be determined by the use of radiocarbon dating and dendrochronology. Other features that could be used as evidence of substantial age include the presence of desert varnish or surfaces dulled by oxidation, hydrolysis, and growth of lichens.

Apparently resistant ledges of rock should be examined in detail for a number of other features. Joint spacing should be noted and the rock should be broken with a hammer. Easily broken rock and rock with closely spaced joints should be rejected. Rock containing abundant organics, clays or carbonate minerals should be considered undesirable but possibly acceptable in the event a clean indurated sandstone cannot be found.

#### 7.4.4 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 and wetting and drying. Furthermore, impact and abrasion from flood events will occur less often.

Relaxation of standards is accomplished with two modifications to the methodology developed in Section 7.3.1.2. First, the weighting factor in the tensile strength category is reduced to 0.50. Second, a test score



exceeding 60% of the maximum possible score would be considered conditionally acceptable for use in occasionally saturated areas. Thus, an acceptable test score may range from 7.2 to 14.4 depending on the number of durability tests available in the scoring process.

One difficulty arises in attempting to apply this methodology to local sources of sandstone, siltstone or carbonate rock. Established quarries will not likely be nearby. Therefore, it may be necessary to evaluate stratigraphic analogues from more distant sources.

Confirmation of acceptability of a nearby source will still be required. This can be accomplished by: 1) appraising its general weathering potential (Table 7.4), 2) either performing tensile strength tests (Table 7.4) or slake-abrasion tests as described by Foley and others (1985), and 3) performing a series of mineralogical-chemical tests.

Mineralogical-chemical tests include X-ray diffraction analysis, ethylene glycol tests, and response to immersion in acid. The same acceptance-rejection criteria would be required as described in Section 7.3.1.2. If carbonate minerals are present but the rock does not react vigorously with acid, then carbonate aggregate could be placed in interstices between blocks of riprap to buffer any acidic solutions that might be present. This would not be required for carbonate riprap which would be self-buffering. If smectite clay minerals are present but there is little reaction to ethylene glycol oversizing would be an acceptable alternative to rejection.

#### 7.4.5 Oversizing

Oversizing methodology is the same as that described<sup>b</sup> in Section



## 7.5 Riprap for Generally Unsaturated Areas

The methodology is essentially the same as that developed for Section 7.4. The only difference is the further <sup>realization</sup> relaxation of acceptance criteria in response to diminishing frequency of freeze-thaw, wetting and drying, impact and abrasion. It is suggested that durability test scores exceeding 50% of the maximum possible score would be conditionally acceptable for use in seldom saturated areas.

## 7.6 References

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Comments Regarding Chapter 7 - Selection of Riprap\*, from  
"Methodology for Evaluating Long Term  
Stabilization Designs of Uranium Mill Tailings Impoundments"

Comment No. 1 - Increasing Thickness of Riprap Blanket to Compensate for Marginal Quality of Rock (Page 1, Section 7.1): The fact that the rock below a depth of six feet showed little or no deterioration within seven years after placement does not indicate the thickness of extra rock that should be placed for a 200-year design. The deteriorated rock could be removed by storms early in the design period. Unless it can be shown that this approach will provide 200-year protection it will probably be more cost effective to use the calculated blanket thickness, based on maximum particle size, and face the problem of adding additional rock if it later becomes necessary. To place an arbitrary extra thickness of rock during remedial action may be a waste of funds and resources.

A more rational approach involving increasing the size of the rock to be provided is recommended by PNL (Ref. 1, page 5). The weight loss expected for the design period is determined from slake-abrasion tests on rocks which have been exposed to various periods of the local climate (to account for chemical effects). This approach could be augmented, as follows: For locations subjected to freeze-thaw cycles, include weight loss due to freeze-thaw cycles on full size block samples; for locations subjected to sulfate in the local environment include weight loss determined from sulfate soundness tests; and for cases involving wetting and drying without abrasion include weight loss determined by wetting-drying tests.

Comment No. 2 - DePuy and Ensign's Acceptance Criteria for Riprap (Page 5, Section 7.3.1.2): The statement is made that DePuy and Ensign's acceptance criteria were developed for riprap to be used as upstream slope protection against erosion by waves. How should the criteria be modified to apply to erosion protection for UMTRA sites?

Comment No. 3 - U.S. Bureau of Reclamation Standards for Judging Riprap Durability (Page 6, Table 7.2): These standards do not account for variations in rock type. The importance of rock type is shown in Ref. 2 (Table 6.7, Pages 53 and 54), where, for example, sandstone with a specific gravity of 2.50 has a quality rating of "good", even though this value is below the lower limit for "good" rock given in Table 7.2. If possible, different standards should be given for different rock types, possibly grouped as shown in Table 7.1.

Comment No. 4 - U.S. Bureau of Reclamation Standards for Freeze-Thaw Weight Loss Results (Page 6, Table 7.2, Item 4): The quality rating to be assigned for weight loss values between 0.5 and 5.0 is not given.

\*Rough Draft NRC Report, dated 3/7/85.

Comment No. 5 - Comparative Rating of Riprap Durability Tests (Page 7, Table 7-3): It should be made clear that the tests are rated against a rock quality which is based on petrographic evaluation, "modified as appropriate by the results of the other tests" (Ref. 3, Page 35). [This is why the petrographic examination alone can have a % agreement of 85% rather than 100%.]

Comment No. 6 - Overall Quality Test Score (Page 8, top paragraph): Why require that the same number of tests be selected from each category? This could mean omitting an important result. As long as the acceptance criterion is based on 80% of the maximum possible score this restriction is not needed.

Comment No. 7 - Additional Durability Tests (Page 8, Section 7.3.1.2, last paragraph): It would clarify the report to state in this paragraph that the additional petrographic procedures recommended are x-ray diffraction analysis and the ethylene glycol test, and that these procedures would only be required in borderline cases.

Comment No. 8 - Modified Comparative Ratings (Page 9, Table 7.4): Footnote 1 does not apply to all of the tests linked to "General Weathering Potential", only to bulk composition, secondary mineralization and weathering, and fracture density; i.e., not to specific gravity.

Comment No. 9 - Freeze-Thaw Tests on Full Size Blocks (Page 9, last paragraph): It should be stated that the block size used for testing should be the largest size riprap proposed for a site, not simply "full size blocks weighing up to a metric ton." Should the results of these tests be substituted for the results of the freeze-thaw tests on smaller samples in computing the overall quality rating, or used in some other fashion? Also, can tests on large size samples be run during construction (is it practical), or just during design?

Comment No. 10 - X-Ray Diffraction Analysis (Page 10, first full paragraph): X-Ray diffraction analysis should not be required for rocks for which the results of petrographic analysis are definitive; i.e., indicate the definite presence or absence of carbonates or smectites. X-ray analysis should only be required for suspected (borderline) cases.

Comment No. 11 - Acceptance Criteria for X-Ray Diffraction Analysis and Chemical Tests (Page 10, Table 7.6): Reaction to ethylene glycol or acid should be labeled "(chemical test)"; the other two entries should be labeled "(x-ray diffraction)".

Comment No. 12 - Weathering Rate (Page 11, Section 7.3.1.3, first paragraph): The statements, "The weathering rate is unknown," and, "Thus increasing the stone size has no rationale on which to base a decision", conflict with statements in Ref. 1 (Page 5, last paragraph) concluding that it is possible to "estimate the weight loss", and "oversize less-durable rock so that it meets riprap design requirements at the end of the containment period." (Also see Comment No. 1, second paragraph.)

Comment No. 13 - Thickening of Riprap Blanket (Page 11, Section 7.3.1.3, second and third paragraphs): See Comment No. 1.

Comment No. 14 - Channel Deposits (Page 12, Section 7.3.2, second paragraph): The statement, "Channel and outwash deposits and desert armor are inferior to quarried igneous and metamorphic rocks because of their heterogeneity and size limitations", is too strong. For example, channel deposits of metamorphic rock may be more durable than certain types of freshly quarried igneous rock.

Comment No. 15 - Sandstone as a Nondurable Rock (Page 15, Top Paragraph): Not all sandstone is "nondurable", as implied.

Comment No. 16 - Riprap for Occasionally Saturated Areas (Page 15, Section 7.4.1): First sentence is incomplete.

Comment No. 17 - Steps Leading to Laboratory Testing (Page 16, Top Paragraph, Section 7.4.3): Figure 5.1 not provided. (Same as Figure 1 on page 8 in Ref 1?)

Comment No. 18 - Addition of Carbonate Aggregate to Buffer Acidic Solutions (Page 17, Section 7.4.4, last paragraph): To implement the suggestion that carbonate aggregate could be placed between blocks of riprap to buffer any acidic solutions present would require research beyond the scope of design for the UMTRA Project.

#### References

1. PNL (Pacific Northwest Laboratory), (1985), "The Selection and Testing of Rock for Armoring Uranium Tailings Impoundments", NUREG/CR-3747, May.
2. PNL (Pacific Northwest Laboratory), (1982), "Long-Term Survivability of Riprap for Armoring Uranium Mill Tailings and Covers: A Literature Review", NUREG/CR-2642, June.
3. DePuy, G. W., (1965), "Petrographic Investigations of Rock Durability and Comparisons of Various Test Procedures", Engineering Geology, Vol. 2, No. 2, Pages 31-46.

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