PROCEDURES FOR THE DESIGN OF A PROTECTIVE COVER SYSTEM OVER RADON BARRIERS FOR URANIUM MILL TAILINGS PILES

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1.0 INTRODUCTION

Design objectives for the UMTRA Project first became an issue in 1978 under Public Law 95-604 when responsibility for the remedial action of the inactive uranium mill tailings sites was placed on the Department of Energy (DOE). After an intensive research effort by the DOE, NRC and EPA to develop standards, the EPA identified the major environmental and health problems associated with inactive uranium mill tailings sites and promulgated standards to be met by proposed remedial actions, which became effective March 7, 1983 (Table 1.1). The standards establish requirements for radiation protection, for protection of water quality and for ensuring long-term containment and stability.

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The release of radon gases from uranium mill tailings piles is a major concern in the reclamation of existing tailings piles. In order to reduce the escape of the gases to acceptable levels into the atmosphere, as determined by EPA regulations, the tailings will be covered with 3 to 10 feet of clays, silts, sands or varying mixtures of the three. The type and thickness of material used will depend on the material available in close proximity to the site and the physical properties of the borrow material.

In the design of these radon barriers, it is extremely important that the integrity of the barrier be maintained for the design life of the facility (200 to 1000 years). Processes which threaten the integrity of the barrier are wind and water erosion, cracking of the soil due to differential settlement or desiccation,* roots from plants or trees, and animals burrowing into the soil. In order to protect against these potentially destructive forces, a rock cover has been incorporated into the design. This rock has previously been defined by the median particle size (D_{50}). In order to better define the size of rock required, the maximum and minimum rock size and the grading of the material have been incorporated into the proposed procedures.

Depending on the erosive forces for a particular site, a single rock layer may be the proper size and gradation to meet the criteria for erosion protection and also serve as a filter for the radon barrier. However, at other sites it may be necessary to have one or more intermediate sand or gravel layers to protect against erosion of the radon barrier due to transport of soil particles through the rock cover during a storm event.

The design of a cover system that will be effective for a specific site requires the consideration of each of the following factors:

- Grain size of radon barrier.
- Grain size of available rock.
- Velocities of runoff and floods that come in contact with cover.
- Type of vegetative cover that will establish or will be established.
- Wind and water erosion factors.
- Infiltration requirements.
- Construction requirements.

The purpose of this paper is to describe the design procedures that will be used for the remedial action at the UMTRA project sites, in order to have consistent designs from one site to another. These procedures have been adopted after a careful review of existing literature and design procedures and incorporate State-of-the-Art methodology.

* How does rock cover protect against differential settlement?

PART 192 - HEALTH AND ENVIRONMENTAL PROTECTION STANDARDS FOR URANIUM MILL TAILINGS

SUBPART A - Standards for the Control of Residual Radioactive Materials from Inactive Processing Sites

192.02 Standards

Control shall be designed to:

(a) Be effective for up to one thousand years, to the extent reasonably achievable, and, in any case, for at least 200 years, and,

(b) Provide reasonable assurance that releases of radon-222 from residual radioactive material to the atmosphere will not:

(1) Exceed an average release rate of 20 picocuries per square meter per second, or
(2) Increase the annual average concentration of radon-222 in air at or above any location outside the disposal site by more than one-half picocurie per liter.

SUBPART B - Standards for Cleanup of Land and Buildings Contaminated with Residual Radioactive Materials from Inactive Uranium Processing Sites

192.12 Standards

Remedial actions shall be conducted so as to provide reasonable assurance that, as a result of residual radioactive materials from any designated processing site:

(a) The concentration of radium-226 in land averaged over any area of 100 square meters shall not exceed the background level by more than -

(1) 5 pCi/g, averaged over the first 15 cm of soil below the surface, and
(2) 15 pCi/g, averaged over 15 cm thick layers of soil more than 15 cm below the surface.

(b) In any occupied or habitable building -

(1) The objective of remedial action shall be, and reasonable effort shall be made to achieve, an annual average (or equivalent) radon decay product concentration (including background) not to exceed 0.02 WL. In any case, the radon decay product concentration (including background) shall not exceed 0.03 WL, and
(2) The level of gamma radiation shall not exceed the background level by more than 20 microroentgens per hour.

SUBPART C - Implementation (condensed)

192.20 Guidance for Implementation

Remedial action will be performed with the "concurrence of the Nuclear Regulatory Commission and the full participation of any state that pays part of the cost" and in consultation as appropriate with other government agencies.

192.21 Criteria for Applying Supplemental Standards

The implementing agencies may apply standards in lieu of the standards of Subparts A or B if certain circumstances exist, as defined in 192.21.

192.22 Supplemental Standards

"Federal agencies implementing Subparts A and B may in lieu thereof proceed pursuant to this section with respect to generic or individual situations meeting the eligibility requirements of 192.21."

(a) ". . .the implementing agencies shall select and perform remedial actions that come as close to meeting the otherwise applicable standards as is reasonable under the circumstances."

(b) ". . .remedial actions shall, in addition to satisfying the standards of Subparts A and B, reduce other residual radioactivity to levels that are as low as is reasonably achievable."

(c) "The implementing agencies may make general determinations concerning remedial actions under this Section that will apply to all locations with specified characteristics, or they may make a determination for a specific location. When remedial actions are proposed under this Section for a specific location, the Department of Energy shall inform any private owners and occupants of the affected location and solicit their comments. The Department of Energy shall provide any such comments to the other implementing agencies [and] shall also periodically inform the Environmental Protection Agency of both general and individual determinations under the provisions of this section."

Ref: Federal Register, Volume 48, No. 3, January 5, 1983, 40 CFR Part 192.

TABLE 1.1 EPA STANDARDS

Table 1.1 EPA Standards

2.0 DESIGN METHODOLOGIES

Before scheduled one can design to the standards established by the 200 to EPA, one must first decide on the methodologies that are available to solve the problem. In the case of the UMTRA sites the criteria to use a design life of 1,000 years without ~~planned~~ maintenance presents a unique problem. In the design of earthen retention structures, it is generally assumed that routine maintenance will assure the continued operation of the facility. Relating this to probability means that the stability period of concern is only one year and to withstand a 100 year event, the probability of occurrence is 0.01. Why only 1 year?

Now, when one calculates the recurrence interval for a structure using a design life of 200 ~~1,000~~ years, using a level of risk corresponding to a probability of 0.10 ~~0.01~~, the recurrence interval is approximately ~~100~~,000 years. Clearly, there are no records available to define the methodologies needed to design for such a large recurrence interval. Also, there is no known way to extrapolate to 100,000 years based on only 50 or 100 years of record. Therefore, design methods must be adopted which incorporate conservatisms into the design. The design criteria for the stability of the UMTRA Project tailings piles due to erosive forces resulting from rainfall runoff across the top and down the sides of the stabilized embankment are based on the runoff from the localized Probable Maximum Precipitation (PMP). For flow occurring as a result of rainfall on the watershed above the stabilized embankment, the pile is designed to resist the runoff from the Probable Maximum Flood (PMF) as a result of the PMP. The PMP is the worst possible event that could occur as a result of a combination of the most severe meteorological conditions occurring over a watershed at the same time. Although no recurrence interval can be assigned to this event, it is felt by most experts that the recurrence interval is in excess of 100,000 years.

re-write as per ~~revise~~ SRP comme

2.1 Erosion protection design methodology

Erosion protection of cover material at the UMTRA project tailings reclamation sites is controlled by the erosive forces associated with the runoff both on and adjacent to the tailings pile. Based on the PMP/PMF for a particular site, the cover material can range from a coarse sand and gravel to large boulder riprap, depending on the size of storm, the size of the drainage basin and the velocities associated with the runoff. Procedures investigated for calculating the required mean rock size needed to provide a stable rock slope during the PMF have included:

- Bureau of Public Roads (Searcy, 1967).
- U.S. Army Corps of Engineers Waterways Experiment Station (U.S. Army, 1970).
- California Division of Highways (Cal. DPW, 1970).
- ASCE Task Committee on Preparation of Sedimentation Manual (ASCE, 1972).
- Bureau of Reclamation (Bur. Rec., 1958).

- o Lane's Method (Lane).
- o Campbell's Method (U.S. Army, 1966).
- o Wyoming State Highway Department (Safety Factors Method)(Stevens, 1976).
- o Rockfill Hydrolic Engineering (Stephenson, 1979)

All of these procedures use one of four basic approaches: 1) critical velocity, 2) lift and drag force mechanisms, 3) critical shear stress, or 4) empirical solutions.

The critical velocity equations consider the impact of flowing water on the particles such that the material of a given size and weight is just able to move. Inherent in this approach is the lack of good definition of the bottom velocity and the difficulty in accurately measuring or predicting this velocity. Another difficulty in using this approach is determining the relationship between bottom velocity and average velocity.

The lift force mechanism approach accounts for a pressure differential caused by the gradient of the velocity. The pressure differences occur because of steep velocity gradient, where the velocity at the top surface of a particle at rest on a channel bottom is greater than zero, while the velocity at the bottom surface is zero. This method although important, has been difficult to develop. Additionally, the critical shear stress and critical velocity methods implicitly include the lift force effects.

The critical shear stress equations consider frictional drag of the flow on the particles and considers the fluid shear stress on a rock layer to the mean flow velocity. This approach is the best for the UMTRA project because mean cross-sectional velocities, depths and flow can be easily obtained assuming sheet flow characteristics. sides & top only, not ditches.

The design method which is most applicable to the design of a rock blanket for erosion protection is the "Riprap Design with Safety Factors Method" developed for the Wyoming State Highway Department by Stevens et al. (1976). The theory and formulation of this method are not discussed as part of this paper since they are well documented in the published paper.* This method is based on the theory of critical shear stress and allows more flexibility in design because the designer is able to choose the factor of safety needed for the design of a particular site and work through the analysis to determine the required rock size. This flexibility is particularly important when considering the conservatism associated with using the PMF as the design storm. As will be discussed in more detail in later sections the best technique available for shallow flow on steep slopes is the method proposed by Stephenson (1979), which is based on critical shear stress and empirical solutions and takes into account interstitial flux. ? Stephenson says not to be used for stable packed rock channel lining.

Additionally since the design of rock covers for the UMTRA Project are based on the PMF, which is the worst possible flood that could occur over a particular site, a factor of safety of 1.0 against the PMF is used. This means that the factor of safety against all other flood events is

* some aspects of the procedure should be amplified as in the MKE Design Manual.

higher than 1.0. This criteria is supported by the CSU Civil Engineering group (NRC, 1983) and is presented here as one of the project design criteria.

2.2 Riprap design parameters

Information needed to design riprap by the Safety Factors Method are:

- o The angle of repose of rock to be used.
- o The specific gravity of rock to be used.
- o The slope of the bed or sideslope over which the rock will be placed.
- o The velocity of flow over the rock to be used.
- o The depth of flow over the rock to be used.

The angle of repose of a rock is dependent on the angularity and diameter of the rock and routinely varies from about 32 degrees to 42 degrees, with most naturally occurring rocks falling in the range of 34 to 37 degrees. This factor has a small effect on the final mean rock size and wherever data ~~is~~ are not available a conservative estimate of 35 degrees will be assumed.

The specific gravity of a rock is dependent on the mineralogy of the rock and can vary from 2.5 to 2.8. Where data ~~is~~ are not available a conservative estimate of 2.60 will be assumed.

The slope of the bed, side, and topslope will vary and will be part of the design. Typically, the topslope will be 2 to 5 percent and the sideslope will be 20 percent or less. The bedslope will be dependent on the topography.

The velocity (V) and depth of flow (Y) are related to the quantity of flow (Q) which occurs either on or adjacent to a stabilized tailing pile. The method used to calculate this quantity is somewhat controversial for rainfall on the pile itself. This controversy is centered on whether the flow across the top of the pile and down the sideslopes is in the form of sheet flow or whether there are flow concentrations which cause the flow per unit area to increase. The design concepts which have been implemented by the UMTRA Technical Assistance Contractor (TAC) will result in a configuration that, in the case of a relocated, recompacted pile, be placed and graded in such a way that sheet flow over the pile will occur. At piles where excessive differential settlements are predicted to occur, an increase in the flow due to some flow concentration will be calculated based on the area of flow and the area of differential settlement which would contribute to the increased flow.

Why not overbuild to compensate?

Another point which has lead to some confusion is the method of calculating the shear stress. The most common method adopted is to use equation 1.

$$\tau = \gamma_w RS \ldots\ldots (1)$$

where

τ = Average shear stress acting on the wetted perimeter.
γ_w = Total unit weight of water (normally 62.4).
R = Hydraulic radius (depth of flow, y, for sheet flow)
S = Slope of bed

Another method for calculating the sheer stress is to use equation 2, which takes into account the average velocity and the ratio of the depth of flow to mean rock size. This formulation includes the equation for Manning's "n" and one must be careful when using equations 2 and 3 that Manning's "n" is checked and reiterations performed until close agreement is reached.

$$\tau = .3 \times V^2 / (3.4/\ln(12.21(Y/K)))^2 \ldots\ldots (2)$$

where

V = Mean velocity
Y = Depth of flow
K = Mean rock size

If equation (1) is used to calculate the shear stress, the mean rock size is then calculated by equation 3.

$$K = 21 \times \tau / [(G_s - 1) \times 62.4 \times N] \ldots\ldots (3)$$

If equation (2) is used to calculate the sheer stress, the mean rock size is calculated by equation 4.

$$K = \tau / (N \times (G_s - 1) \times 32.2) \ldots\ldots (4)$$

where

N = Stability number calculated from the formulations in the Safety Factor's Method.

G_s = Specific gravity of the rock

This is also an iterative process because one has to first assume a value of K to solve for the calculated K. If they are not equal, the iteration for 'n' and k continues until they are equal. If equation (2) is used to calculate the shear stress, the formulation for "n", equation 11, has been incorporated into the equation, necessitating the need to iterate for both n and k.

One number which is critical to the solution for the mean rock size is the depth of flow over the rock, which results in a shear stress being exerted on the rocks. In order to calculate the depth of flow, one must first determine the quantity of flow.

For the case of sheet flow, the quantity of flow (Q_T)(cfs) is calculated by equation 5 which is derived from the equation Q = CIA:

$$Q_T = \frac{I \times L}{43,560} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

where

A = Area (L x W)
I = The ~~maximum 1 hour~~ rainfall intensity (inches) corresponding to t_c ×
L = The length of flow (feet)
C = Constant, Assumed to equal 1.0
W = Width (usually W = 1.0)

The maximum 1-hour intensity is calculated by first determining the local PMP for the pile locations and then determining the storm rainfall distribution using the appropriate Hydrometeorological Report based on geographical locations. For most* of the UMTRA Project sites, Hydrometeorological Report (HMR) No. 49 (Hansen et al., 1977) is the most appropriate. Next, the time of concentration (t_c) is determined. This can be estimated by determining the largest length of flow (L), dividing that length by 2, and then dividing that length by the estimated flow velocity (V_{est}). Once t_c is determined, use this number as the most intense period of time for the PMP. Then using Table 1 which is an extrapolation of the values in HMR 4c 49? choose the appropriate ratio for the calculated time of concentration to calculate the intensity by equation 6.

Table 1 Incremental rainfall duration percentages

Rainfall duration (min)	Percentage of 1-hr PMP
5	30
10	48
15	60
20	71
30	84
40	93
50	99
60	100
120	114
240	125
360	130

These values depend on the ratio of 6-hr. to 1-hr. which varies w/site, even w/in HMR 49. Why give values which may only ~~apply~~ to a few sites?

$\% = RD/(7.39 \times 10^{-3} \times RD + 2.25 \times 10^{-3})$

$$I = PMP_{(t_c)} \times \frac{60}{t_c} \text{ inches/hour} \quad \ldots\ldots\ldots\ldots\ldots (6)$$

*Why ~~not~~ give something that does not apply to all sites.

where

$PMP_{(t_c)}$ = the incremental rainfall amount for the time of concentration

t_c = time of concentration

Next, one must calculate the interstitial flow in the rock layer. This is that portion of the flow that occurs within the ro layer. To calculate this quantity, one must first calculate the velocity of flow. One such equation to calculate the velocity is the one suggested by Leps (1972), which is shown by equation 7.

$$V = WM^{0.5} i^{0.54} \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

where

V = Average velocity of water in the soils of the rock layer
W = Empirical constant (33 for crushed gravel to 46 for polished marble)
M = Hydraulic mean radius
i = Hydraulic gradient (slope ft/ft)

Using W equal to 33 to be conservative and assuming ~~x~~ M onosized rock Table 1 presents the factors that should be used for various rock sizes.

Table 2 Empirical correlations for flow in rock

Rock size (inches)	$Wm^{0.5}$ (in/sec)
2	16
6	28
8	32
24	8
48	84

This equation should only be used if the grading of the rock shows that less than 30 percent of the rock is ≤ 1 inch. If more than 30 percent of the rock is ≤ 1 inch then the rock layer should be ~~tested~~ analyzed ? as an earthfill.

Once the velocity is calculated, the quantity of flow per foot can be calculated by equation 8.

$$R_i = \frac{V}{12} \times \text{Thickness of layer} \quad \ldots\ldots\ldots\ldots (8)$$

Then once the interstitial flow (Q_i) is calculated the flow on top of the rock layer 2 can be calculated by subtracting the Q_i from the total flow.

The depth of flow over the rock is then calculated using Manning's equation for sheet flow as shown in equation 9.

$$Y = \left(\frac{n \times Q}{1.486 \times S^{1/2}} \right)^{3/5} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

where

n = Manning's friction factor
Q = Quantity of flow (cfs)
S = Slope of bed top or sideslope (feet/feet)
Y = Depth of flow (ft.)

The velocity of the flow is calculated from the results of equations 4 and 9 by equation 10:

$$V = Q/A \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

where

V = velocity (fps)
A = Y x unit width ($ft.^2$)

This solution is iterative in that one must first estimate a velocity to find t_c and then in the end a new velocity is calculated. If the two velocities are within ten percent then there is no need to iterate; if not the iterative process is continued, replacing the estimated velocity with the calculated velocity until the two velocities are within ten percent.

One number which is critical to making the calculations is Manning's friction factor (n). This number is very subjective and is usually based on previous experience. Tables have been published which give values of 'n' for various types of vegetation. Most of these published values are for river channels or overland flow and are difficult to apply to a tailings pile covered with rock and sparse vegetation.

One method of calculating 'n' is by a formulation developed by the Corps of Engineers (1970) which is related to the depth of flow and size of rock as shown by equation 11.

$$n = \frac{Y^{1/6}}{23.85 + 21.95 \log_{10}(Y/k)} \qquad \ldots\ldots\ldots\ldots\ldots (11)$$

where

Y = depth of flow (ft.)
k = mean diameter of rock (ft.)

For some combinations of flow depth and rock size this equation gives the value of 'n' that may be either too conservative or not conservative enough. Therefore, when using equation 11, a lower bound of .02 and an upper bound of 0.06 for "n" should be used, based on ________. ?

2.3 Design sequence

Once all of the design parameters have been calculated, they can be input into the equations for determining the mean diameter rock size. The Safety Factors Method has four (4) sets of equations depending on the type of flow. These flow conditions are:

1. Non-horizontal flow on a sideslope
2. Horizontal flow on a sideslope
3. Flow on a plane sloping bed
4. Flow on a horizontal bed

Flow conditions 1, 2, and 4 are used when flood flow from the associated drainage area flows adjacent to the pile. Flow condition 3 is used for flow which occurs due to rainfall which falls on the pile and flows across the top and down the sideslopes.

Once the flow condition is determined it is a simple calculation to determine the mean rock size (D_{50}) that will be required to protect against the PMF.

In order to make the calculation of the mean rock size for various design conditions and changes, a computer program has been developed for the Safety Factors Method. This program is user oriented, requires minimal input, and will run on most microcomputers. A listing of the program (RIP-RAP) is shown in Appendix A, and example outputs are shown in Appendix B.

As was mentioned in Section 2.1, we feel that the best method available for shallow flow on slopes in excess of 10 percent is the method proposed by Stephenson (1979) in the book "Rockfill Hydraulic Engineering."* This method, which is based on flume studies by Oliver (1967), is an empirical solution to a series of tests in which different size rock layers and different slope angles (2 to 20 percent) were tested to evaluate failure conditions of various size rock and slope conditions and what factors were involved. ** See note on Stephenson's method on pg. 4.

Based on this work, a formulation for which any slope would not fail was derived.

*The 10 percent limit is not "mentioned in Section 2.1", nor is there anything in Stevens' paper on the Safety Factors method regarding such a limit. What is the basis for such a limit?

When the situation of steep slopes and shallow flow occurs the method proposed by Stephenson (1979) is proposed. The input parameters for this method are as follows and the formulation is shown as equation 12:

- o Quantity of flow (cfs)(Q).
- o Angle of slope ( ).
- o Constant (C)(varies from .22 to .27).
- o Specific gravity of rock to be used (G_s).
- o Angle of repose of rock to be used ( ).
- o Porosity of rock fill (related to density)(p).

$$k = \frac{Q(\tan\ )^{7/6} p^{1/6}}{Cg^{1/2}\,[(1-p)(G_s-1)\,\mathrm{Cos}\ \ (\tan\ - \tan\ )]^{5/3}}^{2/3} \quad \ldots\ldots (12)$$

where

k = Mean rock size, D_{50}.

When this method is used, the D_{50} size rock is conservative and includes a safety factor on the order of 1.2 to 1.8.

When Stephenson's method is used, the computer program Riprap incorporates this method as one of its options (Appendix A).

*What about the case of a soil cover over rock protection, where gully formation is likely?

**What about erosion protection in ditches?

[Both of these subjects are covered in the MKE Design Manual.]

***What about a maximum velocity for riprap to even be considered? Say 20 feet per second?

3.0 ROCK DURABILITY

Durability of rock is defined as the ability of the material under consideration to withstand the forces of weathering. These forces may be either chemical or physical. Therefore, the durability of rock riprap is a major concern in the design and long term stability of erosion barriers. Long term records (200 to 1000 years) of the weathering of rocks are usually not available.

Factors which affect the durability of rocks are the chemistry of the water that comes in contact with rocks; the temperature of the water; raindrop impact; ~~and~~ the amount and velocity of windblown sand which impact on the rock; ~~Another~~ and ~~important factor is~~ the effects of wetting, ~~and~~ drying ~~together with~~ and temperature changes. Th~~is~~ese effects become~~s~~ more serious in climates that experience large changes in temperature and especially those climates with frequent freeze-thaw cycles.

Of importance to the UMTRA Project is a study by Colman (1981) on weathering rates as a function of time for the Colorado front range and San Juan Mountains. These studies showed that there is a decrease in the rate of weathering with time due to a build up of residue on the surface of the weathering rock. The implication of this study is that laboratory tests are conservative and do not truly represent the weathering or durability of a particular rock.* Can specific rates of weathering per 200 years be stated?

However, in order to get a qualitative answer to the question of durability, the following section discusses the method of selection and testing required to determine the relative durability of a rock proposed for use.

3.1 Material selection, testing and placement

The sources from which the rock riprap will be obtained should be selected well in advance of the time when the stone will be required for placement. The acceptability of the rock should be determined by review of previous uses and testing and/or suitable tests. If testing is required, suitable samples of rock should be taken using Standard Practices for Sampling Aggregate (ASTM D75), at least 60 days prior to the start of construction. Additionally, the approval of some rock from a borrow area should not be construed as constituting the approval of all rock taken from the borrow area.

If historical records of service are not available* or do not exist, resistance to disintegration from the type of exposure to which the stone will be subjected should be determined by any or all of the following tests depending on the rock to be used and the site climatic conditions:

1. One of the parameters needed in the design of the size of rock required for erosion is the specific gravity.** Additionally, the specific gravity and absorption (ASTM C127) can be used to evaluate the durability of a rock. The specific gravity of a rock is an indicator of its strength. The higher the specific gravity

*Give more emphasis to collecting historical data. Should be No.1.

**Petrographic analysis is the next best (No.2) indicator.

the better the quality of the rock. The specific gravity is also a good indicator of a rock's ability to withstand cycles of freezing and thawing.

4. Absorption by itself is not a good indicator of a rock's freeze-thaw characteristics. However, a low absorption is a desirable property to prevent the rapid disintegration by salt action and mineral hydration.

It is recommended that a suitable rock be one that has a specific gravity greater than 2.6* and an absorption less than 1 percent.

7. ~~2.~~ When riprap must withstand abrasive action from material transported by streams, or large flow on or adjacent to the pile, the Los Angeles Abrasion Test should be used. When the abrasion test in the Los Angeles Machine (ASTM C131 or C535) is used, the stone shall not have a percentage loss of more than 40 percent after testing. Additionally, the ratio of the loss after 100 revolutions to the loss after 500 revolutions for ASTM C131 and after 200 resolutions to the loss after 1000 revolutions for ASTM C535 should not exceed 20 percent for material of uniform hardness.

6. ~~3.~~ In locations subject to freezing or where the stone is exposed to salt water, the Sulfate Soundness Test (ASTM C88) should be used. Stones should not have a loss after five cycles exceeding 10 percent if sodium sulfate is used and a loss not exceeding 15% when magnesium sulfate is used.

5. ~~4.~~ A better guide to weathering which may be used in place of 3. above is AASHTO Test 103 for ledge rock, Procedure A. From this test the stone should not have a loss exceeding 10 percent after 25 cycles of freezing and thawing. This test should only be used when a material is questionable since the time and cost of this test is expensive.

3. ~~5.~~ Another method which can be used to evaluate durability of a particular rock is the hardness test as determined by the Point Load Test; or the Schmidt Rebound Hammer. If the point load test is used a value >300 is acceptable. If the Schmidt Hammer is used a value of >40 is acceptable.

It must be recognized that considerable judgement is required during site evaluation and evaluation of the laboratory test data. The laboratory or index tests are dependent on the availability of equipment and the contract testing laboratory. There is sufficient interaction among the various tests described to provide a basis for the judgement of the durability of a rock source with a minimum amount of testing. The greatest number of tests should be run on rock types that have been judged to be marginal during site investigations.

?

Should any of the rock being evaluated for use as erosion protection not meet the recommended standards for acceptable rock durability, a new rock source should be evaluated if one is available within a reasonable distance from the site.

*A specific gravity of 2.6 would be quite low for a dense rock such as gabbro, basalt or gneiss.

If an alternative rock source of better durability cannot be found, the size of the rock should be increased to take into account the degradation of the rock with time. The increase in size is subjective, but it is proposed that the rock size be increased by a percentage based on the criteria that the rock fails and whether the rock fail all or only one or two of the criteria and which ones they are.*

When determining the durability of a rock, one should also consider the geologic setting and age of the rock and make estimates of the erosion with time that may have taken place.

When placing the rock, each load of riprap should be reasonably well-graded from the smallest to the maximum size specified. Gradation can be controlled by visual inspection. If any differences of opinion occur between the engineer and the contractor, the difference shall be resolved by dumping and checking the gradation of any two random truckloads of rock. Alternatively, if the rock size is not greater than 3 inches, the rock can be physically tested, using U.S. standard sieves of the appropriate sizes.

*Because there is no basis for increasing the rock size in direct proportion to the percentage by which the rock fails to meet a criteria this approach could provide larger rock than required in the long run (thus wasting funds). ~~or I~~ If the rock does disintegrate it can be replaced, but it should not be "replaced" before any rock is seen to disintegrate.

4.0 FILTER REQUIREMENTS AND DESIGN

When designing the cover system, one must evaluate the need for a filter layer between the radon barrier and the erosion protection layer. Most of the research into the needs and design criteria is over 20 years old and has varied somewhat. The most widely accepted design criteria for filters are shown in equations 13, 14, and 15:

References?

$$\frac{D_{15}\text{ filter}}{D_{85}\text{ soil}} < 5 \quad \text{------ Stability Criterion} \ldots\ldots\ldots\ldots (13)$$

$$5 < \frac{D_{15}\text{ filter}}{D_{15}\text{ soil}} < 40 \quad \text{--- Permeability Criterion} \ldots\ldots\ldots\ldots (14)$$

$$\frac{D_{50}\text{ filter}}{D_{50}\text{ soil}} \leq 25 \quad \text{------ Separation Criterion} \ldots\ldots\ldots\ldots (15)$$

? Ref. ?

It has also been suggested at times that equation 9 be modified as shown in equation 13.

?

$$12 < \frac{D_{50}\text{ filter}}{D_{50}\text{ soil}} < 58 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

?

Recent studies by Sherard et al. (1984) have shown that equation 13 is conservative and has a built-in factor of safety between 1.5 and 2.0 and is supported by experimental data.

by ______ (Ref.) required for

Results of experiments ~~from the research~~ have shown that the D_{15} size of the soil (equation 14) has no significant influence on the properties ~~of~~ a filter. Additionally, experiments on silts and clays as the base soils show that the D_{15}/D_{15} ratio commonly exceeded 1000 for successful tests.

The criteria shown in equations 15, and 16 have also been found to be unsupported by theory or experimental results. The research by Sherard et al. recommends that equation 10 continue to be used and that equations 14, 15, and 16 be abandoned.

?

?

Therefore it is recommended that equation 10 be used as the criteria for all filters. This criteria can be relaxed in some instances for a clay with a ? high plasticity or if there are fairly low flow gradients. In addition to the above criteria, the following requirements for graded filter should be met:

?

- The filter material should pass the 3-inch sieve for minimizing particle segregation and bridging during placement. Smaller maximum particle sizes may be specified if practical. Also, filters must not have more than 5 percent ~~minus~~ the No. 200 mesh sieve, to prevent excessive movement of fines in the filter.

passing

?

- o The gradation curves of the filter and the base material should be approximately parallel in the range of the finer sizes, because the stability and proper function of protective filters depends upon skewness of the gradation curve of the filter towards the fines, giving support to the fines in the base material. Additionally, the material should be reasonably well graded throughout the in-place layer thickness.

- o The minimum thickness of the layer should be 6 inches in order to facilitate ease of construction during placement.

When a rock blanket is to be used over a filter, the rock used should be essentially equidimensional, well-graded in size, with a maximum size equal to about one-tenth of the blanket thickness. The rock blanket should also meet the filter criteria of equation 13 so that the filter material does not migrate through the voids in the rock. The thickness of the rock layer should not be less than the spherical diameter of the upper limit of D_{100} rock or less than 1.5 times the spherical diameter of the upper limit of D_{50} rock, whichever is greater. (Ref.)

Too strict

When a rock blanket is to be used on a sideslope and flow will be horizontal to the slope, scour at the toe will most likely occur. To prevent the scour, which could cause failure of the rock blanket, it is recommended that a shallow trench be excavated at the toe of the slope and that the trench be backfilled with the same rock blanket material. As rule of thumb, the depth of the trench should be at least two times the thickness of the blanket and the width of the trench should be twice the depth.

5.0 DESIGN SEQUENCE

The following steps are essential for the design of an adequate cover system for tailings embankments.

1. From the borrow investigation make a composite plot of all the grain size distributions from the radon barrier cover borrow site. From this plot determine the practical upper and lower bounds of what the ~~material~~ grain size distribution will be after ~~it~~ -the material is placed.

2. Determine the velocities characteristic of the flood flow on and adjacent to the pile after reclamation. Flood flow on and adjacent to a pile can occur due to a storm occurring in the watershed above a pile, resulting in a flood flow adjacent to a pile and by a storm occurring on the pile resulting in sheet flow across the top of and down the slopes of a pile. Velocities and depths and flows adjacent to a pile are normally calculated with the use of the HEC-2 computer program, which routes the storm through the watershed.

   Velocities and depths of flow for a storm occurring on a pile are more difficult to calculate. When a pile is designed with a rock cover and no topsoil for vegetation, sheet flow hydraulics should be used to calculate the total flow. The local PMP should be calculated using the appropriate HRM report. Next, the time of concentration should be calculated. This is an iterative process based on the velocity across the pile, Manning's friction factor, the size of rock and the depth of flow. Then using the time of concentration compute the maximum one hour intensity (equation 6). Once the maximum one hour intensity is calculated the total flow can be calculated using equation ~~6~~. 5 ?

3. Determine the mean rock size needed to resist erosion of the stabilized pile from the calculated velocities and depths of flow. For flow adjacent to the pile and across the topslopes the Safety Factors Method using the computer program RIPRAP should be used, inputting the data described in Section 2.0. For flow down the sideslopes the Safety Factors Method should also be used. However, for certain combinations of velocity, depth of flow and sideslope the Safety Factors Method is not appropriate. When this occurs Stephenson's method should be used, also using the computer program RIPRAP. When determining the rock size gradation, make sure that the grain size distribution meets the criteria in Section 4.0.

4. Review the data on the rock borrow source to determine its durability, if it meets the criteria, no adjustment in the rock size will be needed. If the rock does not meet the criteria, increase the rock sizes proportionally to the percent that the rock failed the tests. (Basis ? for this)

5. Calculate the filter requirements for the rock cover and the radon barrier to determine if there is a need for a filter between the two layers. If a filter or gravel layer is needed, again make sure that filter requirements are met for all cover materials with the most flexibility being in the central filter.

?

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APPENDIX A

RIPRAP DESIGN COMPUTER PROGRAM

"RIPRAP"

APPENDIX B

EXAMPLE COMPUTER RUNS

FOR PROGRAM "RIPRAP"

4005-GEN-R-04-00655-00

Marked Copy
Marked by: Jim Kam
Reviewed by: G. Thiers
7/12/85

SURFACE WATER HYDROLOGY

1.0 INTRODUCTION

It is noted that the following discussion concerning the methodologies and analysis of surface hydrology summarizes the design approach presently used by the TAC to ensure longevity of the final design. This discussion in no way concerns itself with impacts of surface water drainage during construction. This area is the responsibility of the RAC in implementing the final design and in no way affects the longevity of the final design.

Surface water hydrology at UMTRA sites will be assessed in order to provide adequate protection for long term stability of the tailings piles. Adequate protection will prevent dispersal of tailings by natural forces such as wind, rain, and flood waters. This discussion will concentrate on any upland or upstream surface water drainage that impacts upon tailings piles. These floodwaters can be divided into two types of impacts. The first is that which occurs from small watersheds directly upland from the tailings pile itself. This type of drainage is usually diverted to control its flow around and away from the pile itself. The second and most severe impact is that which occurs due to proximity to a major drainage tributary. Flows of this type cannot be diverted and adequate protection must be afforded if stabilization at the present location is chosen. Stabilization of sites that are located in or near flood plains presents unique problems in design for long-term performance. To accomplish the objective of long-term flood protection, the standard design approach is to determine the magnitude and potential impacts resulting from a Probable Maximum Flood (PMF)* event. If a design for this event is not practical, then alternative design events or solutions are assessed.

* The consequences of damage due to exceedance of the design flood do not justify adoption of the PMF as the design flood. The design floods should be the floods for which the probability of exceedance in 200 and 1000 yrs. is 10%. This gives 90% assurance of non-exceedance. The choice of design for 200 yr. or 1000 yr. should be based on practicality on a case-by-case basis. Should substitute "design flood" for "PMF".

2.0 METHODS

2.1 DESIGN EVENT SELECTION

The use of the PMF* as the design flood event to achieve long-term control of uranium tailings is not clearly defined. The Environmental Protection Agency's (EPA) standards require that control of the uranium tailings must be effective for 1,000 years (to the extent reasonably achievable) and, in any case, for at least 200 years. The standards do not specifically state that a PMF* event must be used for design in order to achieve the stated containment life. An analysis of exceedence probabilities for various events with respect to the containment life (Junge and Dezman, 1983) suggests that design events with a very long return period (e.g., 10,000 years) must be used to meet a long-term containment objective. However, the limited statistical data that are available cannot be extrapolated accurately to such long return periods. The generally accepted alternative, therefore, is to use maximum credible events, such as the PMF*, for design.

By definition, a PMF is based on the most severe combination of critical meteorologic and hydrologic conditions for a particular area, and has a very small chance of being exceeded. Therefore, a tailings disposal system designed to withstand a PMF* would have a very small risk of failure and thus, adequately meets both the intent and containment objective of the EPA standards.

* See footnote pg. 1

2.2 PMP DETERMINATION

Prior to determining the runoff response from the design drainage basin, the analysis requires determination of the Probable Maximum Precipitation (PMP)* amounts and hydrographs for the various regions in the drainage basin. Techniques for the PMP and hydrograph determinations have been developed for the entire United States primarily by the National Oceanographic and Atmospheric Administration (NOAA, 1977, 1978, 1984) in the form of hydrometeorological reports for specific regions. These techniques are commonly accepted and provide straight-forward procedures with a minimum amount of variability.

2.3 SMALL UPLAND WATERSHED

As stated previously, the impact of drainage from small watersheds upland of tailings piles is controlled by diversion of such drainage. Diversion channels must be designed to ensure adequate capacity and erosion protection for the runoff due to a PMP.* These channels must also have sufficient gradient to ensure that smaller storm events will wash out sediment buildup and maintain the proper capacity.

2.3.1 Hydrologic modeling

For small watersheds two methods are employed to determine the design runoff discharge:

* see footnote on pg. 1

- The Linear Reservoir Routing Technique (Stubacher, 1975) using Green-Ampt infiltration parameters (Rawls and Brakensiek, 1983).

The Rational Method can be used for watersheds of less than 200 acres while the Linear Reservoir Routing Technique is good up to 500 acres.

2.3.2 Hydraulic modeling

Once the design discharge is determined, the capacity of the diversion channel can be checked using Manning's formula (COE, 1970). Ditch velocities are also determined by this formula. Riprap design techniques, as discussed under erosion protection design, are then employed to determine the proper rock protection requirements.

Note: Manning's formula is generally applied to uniform flow.

2.4 DRAINAGE TRIBUTARY

As stated previously flow impacts of this type cannot be diverted. Typically, those UMTRA sites located in major floodplains have the potential flood impact of flows several feet up the side of the embankment or

flows completely surrounding the tailings pile. If the tailings pile is to be stabilized in the floodplain adequate protection must be provided for longevity.

2.4.1 Hydrologic modeling

The most common and widely used technique for computing the magnitude of the PMF* involves the use of the U.S. Army Corps of Engineers HEC-1 model (COE, 1981). The HEC-1 model is designed to simulate the runoff response of a river basin to precipitation by representing the basin as an interconnected system of hydrologic and hydraulic components. This model can be used for those watersheds greater than 50 square miles and even as large as several thousands square miles. The HEC-1 model can also be used for watershed less than 50 sq. mi. e.g. using SCS triangular unit hydrograph for 100 Ac. of drainage area.

2.4.2 Hydraulic modeling

Once the HEC-1 modeling is completed, the analysis involves the consecutive use of the U.S. Army Corps of Engineers HEC-2 (COE, 1982) model. The HEC-2 model is useful in determining stream hydraulics resulting in water surface elevations and velocity gradients at the tailings site.

*

See footnote on pg. 1

2.4.3 Geomorphic considerations

When dealing with the EPA longevity requirement, a design that considers flood encroachment only for existing conditions is not sufficient. The design philosophy for long-term control must also consider that geomorphic changes could have a profound effect on the hydraulic conditions at a site and create a condition in total variance to existing conditions.

The primary geomorphic concern with long-term stabilization of sites located in flood plains is the potential for lateral movement of a stream channel causing undermining or erosion of the tailings impoundment. Stream channel migration can occur gradually during the design life of the containment. A more severe situation that can occur, however, is a rapid channel shift in response to a major flood event. The potential for channel migration must be carefully evaluated on a site-specific basis using all available geomorphic data.

3.0 ANALYSIS

3.1 RATIONAL METHOD, Q = CiA

Where Q = Peak rate of runoff, in cfs

C = Weighted runoff coefficient, value of 1.0 is used for the PMP condition

i = Average intensity of rainfall, in inches per hour; see erosion protection design for determination of this parameter

A = Drainage area, in acres

3.2 LINEAR RESERVOIR ROUTING TECHNIQUE

Under this method, rainfall is subjected to infiltration losses, the magnitude of which depend on antecedent rainfall. The resulting rainfall excess is multiplied by the watershed area to obtain an instantaneous hydrograph which is routed through an imaginary linear reservoir with a routing constant equivalent to the time of concentration to obtain the final hydrograph.

All rainfall losses are compared to precipitation such that losses are less than or equal to precipitation. Soil infiltration is accumulated after each specified time step; therefore, the Green-Ampt infiltration equation performs a soil moisture accounting and the resulting infiltration rate is a function of the existing soil moisture conditions. In order to obtain conservative discharge estimates it is assumed that the watershed soils are initially close to saturation.

3.3 HEC-1 MODEL

There are four major input parameters that are estimated in order to model the basin

- o Amount and temporal distribution of the PMP *
- o Lag times of runoff within the basin
- o Computation interval for the hydrograph
- o ~~Less~~ ~~less~~ rate ~~or~~ of loss of precipitation within the basin.
- • Unit hydrograph

3.3.1 PMP* amount and temporal distribution

Determination is according to procedures outlined in the appropriate hydrometeorological reports as discussed in Section 2.2.

See footnote on pg. 1

3.3.2 Lagtimes

Lag times for subbasins throughout a watershed are typically computed using the lag time empirical relationship contained in Design of Small Dams (DOI, 1977). Experience has indicated that this relationsnip generally gives longer, less conservative lag times than what might actually occur during a PMF.* The approach of the UMTRA Project team is to initially calculate lag times with the above stated relationship. The resulting peak flows for each subbasin hydrograph are then used with Manning's equation to determine a better estimate of channel velocities. The velocities are then used to recalculate the routing lag times.

3.3.3 Hydrograph computation interval

The selection of a computation interval "T" must meet the criteria that it be less than 0.25 x T_{peak} (COE, 1981).

where $1.7 \times T_{peak} = T + T_c$

T_{peak} = time to hydrograph peak

T_c = time of concentration

~~A computation interval of one hour usually meets this criteria. However, based upon recommendations by the Hydrologic~~

This statement becomes reduntant because Δt depends on T_c. The relationship of Δt and T_c has been defined as above in 3.3.3

* See footnote on pg. 1

Engineering Center, Davis, California, a time interval of less than one hour would be too detailed for a PMF analysis.

3.3.4 ~~Loss~~ rate of loss of precipitation

Soils in any watershed are classified by the Soil Conservation Services (SCS) into four hydrologic soil groups. Each soil group has recommended ranges of minimum infiltration rates after saturation. Uniform loss rates can then be selected from this information. This selection can be greatly aided with recommendations from the Corps of Engineers for specific drainage basins.

3.4 HEC-2 MODEL

The primary input parameters for this model are as follows:

- Boundary geometry of the floodplain is specified in terms of ground surface profiles (cross-sections) and the measured distances between them (reach lengths).

- Values of roughness coefficient (Manning's "n") are specified for the channel and overbank areas.

3.4.1 Boundary geometry

Cross-sections are located at intervals along a stream to characterize the flow-carrying capability of the stream and its adjacent floodplains. They should extend across the entire floodplain and should be perpendicular to the anticipated flow lines. Occasionally it is necessary to lay out cross-sections in a curved or dog-leg alignment to meet this requirement. Every effort should be made to obtain cross-sections that accurately represent the stream and floodplain geometry (COE, 1982). Ideally 2 foot contour maps are necessary but not always available. In lieu of this USGS 15 minute Quad maps in addition to field observation must be relied upon.

3.4.2 Roughness coefficient

Values of Manning's roughness coefficients "n" are varied to account for conveyance differences and to impose constrictions where topography alone would not adequately define flow paths. The basis of determination of n values is 0.020 to 0.025 for clear channels and 0.050 to 0.100 for flood plain including mid-channel islands. It is assumed that normal floodplains would be covered at great depth and vegetation, buildings and other topographic features would be submerged or removed, greatly reducing in values. As a rule an n value of 0.025 is used where depths are greater than five feet and 0.05 where less than five feet; although some

variance from this rule occurs in order to achieve reasonable conveyance values for differing topographic conditions along the profile.

3.4.3 Design considerations

Due to the assumptions that are required in this analysis, uncertainties exist in both the hydrologic and hydraulic modeling. As discussed in Section 2.4.3, geomorphic conditions can produce unpredictable results.

In order to compensate for the uncertainties the following items are implemented for the design.

- Flow characteristics are determined for flow discharges greater and less than the design PMF* flow rate. This checks the sensitivity of the system to changes in hydrologic characteristics to ensure a conservative design.

- It is assumed that during a PMF* deposition and scour will occur such that mean channel depth and velocity is approached across the entire flow width.

- Mean channel depths and velocities are used for the erosion protection design.

* See footnote on pg. 1

o Riprap protection is provided to bedrock or to depth ~~or~~ of scour; or provided as a thickened apron with sufficient quantity to extend to bedrock or scour depth if undercutting occurs.

o Riprap provided as a thickened apron is designed for the mean channel depth and velocity using an assumed 2:1 sideslope for the potential undercut condition.

o Sensitivity of the required rock size is also performed by varying the energy gradient to ensure a conservative design.

o The riprap design is based on the riprap design with-safety factors method as discussed in the erosion protection design section.

Note: the following should be added:

- Different structures or different portions of a structure for a site may be designed according to different design criteria depending on the consequences of failure of the
- A 50 or 100 yr. storm may be used as a design criterion for temporary water diversion during construction.
- Sediment yield and sediment retention facilities should be considered for short and long terms, SEE
- For ditch design, see MKE "UMTRA DESIGN PROCEDURES" Manual, Chapter 4 (attached)

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7/15/85

EMBANKMENT DESIGN CONSIDERATIONS

RECEIVED - IECO

JUL 5 1985

UMTRA - S. F.

1.0 INTRODUCTION

In order to meet the EPA standards at each UMTRA site, many different areas of engineering design are involved. This process becomes quite complex when trying to account for all areas into one cohesive design. The following discussion summarizes many of the design considerations taken into account for the three types of stabilization.

- Stabilization in place
- Stabilization on site
- Relocation to an alternate site for stabilization.

2.0 GROUND WATER PROTECTION

Stabilization in place is possible ~~only~~ if there is no degradation of the ground water regime anticipated ~~occurring~~ due to actual contact with the tailings.* If this is the case then the entire tailings pile must be raised above the ground water for stabilization on site to be viable.* This is performed by excavating the tailings and placing fill to at or above the ground water surface. A capillary break layer and clay liner are then laid down prior to replacing the tailings on top of the prepared foundation.**

If relocation to an alternate site becomes the chosen alternative then it is first assumed that the chosen site has no ground water near the surface. Partial below grade disposal is usually the prime choice ~~in order~~ if it is possible to make use of in-situ materials ~~as~~ for the radon cover or site grading. Generally the excavated ground is reconstituted to act as a partial liner but additional protection with a capillary break or clay liner are deemed unnecessary. Upon further investigation, the ground water may be nearer to the surface than originally assumed. If this occurs than partial below grade disposal may not be possible or a capillary break and/or a clay liner may be necessary.**

*A slurry wall could isolate the ground below the tailings from the groundwater, making stabilization in place possible.

**A capillary break may not be needed in all cases. A clay liner will not be needed if the tailings are placed above flood level and above maximum ground-water surface ~~and~~ covered with a clay cover of sufficiently low permeability.

3.0 PILE CONFIGURATION

Under stabilization in-place an attempt is made to reconfigure the tailings pile to stable slopes with a minimum movement of tailings. In conflict with minimum tailings movement, however, is the goal to minimize the final pile area that will remain restricted. A balance should be achieved between ~~Although constrained by~~ drainage and erosion considerations, and the concept that sideslopes ~~are always~~ should be maximized and topslopes minimized for greater volume to pile area ratio.

Under stabilization on site and relocation, there is more flexibility on final pile shape since the entire pile is being picked up and moved anyway. the final pile area can be minimized greatly. Based on ____________, A general width to height restriction is maintained by attempting to keep the ratio greater than 20 to 1. In general the higher the pile becomes the less economical the volume increase is per foot of height increase.*

Possibly less, but not generally significantly less.

* Is this true in all cases? [Please demonstrate.]

For stabilization on site the final pile location is usually determined by the flattest open area with minimal contamination that part of the foundation can be easily prepared with minimal rehandling of contaminated materials. The remainder of the work would continue in sequence.

Confusing sentence.

~~① Minimizing final embankment relief above surrounding grades significantly improves the stabilitity with respect to erosion, i.e., the embankment should not be constructed as high as possible to minimize the area. A balance of all factors must be sought.~~

4.0 SLOPE STABILITY/ SETTLEMENT

Slope stability and minimizing settlement effects can be difficult to ensure ~~are difficult to control~~ with the stabilization in place option. Since the goal with this option is minimum tailings movement, large concentrations of slimes may still exist. If the slime concentration is large enough, slope stability may be difficult to achieve and larger settlements ~~will~~ may cause problems, e.g. create unpredictable drainage flow concentrations. In severe cases, preloading of the entire tailings pile may be necessary, or ~~movement~~ excavation and ~~reconstitutions~~ recompaction of large portions of the pile.

Ideally, stabilization on site and relocation provide the greatest control. These options allow for improving the tailings settlement and stability properties, e.g., by complete mixing of slimes with sand ~~and~~ or proper compaction of the entire tailings pile.

5.0 RADON COVER

The pile configuration and type of stabilization can greatly affect the required amount of radon cover. If the pile ~~size~~ area is minimized then ~~the~~ radon emissions ~~are~~ will be concentrated over a smaller area.* ~~However, if the~~ On the other hand, the contaminated materials ~~are~~ can be layered with the lesser contaminated material on top, ~~then~~ ~~resulting~~ lesser tending to reduce radon cover ~~is required.~~ thickness requirements.

Stabilization in place provides the least amount of control over the location and layering of higher contaminated materials. Also, more data must be obtained regarding the of the existing pile contamination in order to properly design the radon cover. At best under this option, any windblown materials of low level contamination should be spread evenly over the tailings pile in order to reduce the necessary radon cover.

Stabilization on site and relocation result in complete mixing of the tailings. This aids and simplifies the design by allowing averaging of concentrated slime areas over the entire depth of tailings. Windblown materials of low level contamination should still be ~~retained separately for even placement~~ placed over the top of the tailings.

*Is this really important? Cover requirements are mostly dependent on the radioactive level of the top 10 feet of tailings, not what is below the top 10 feet. (See Summary of UMTRA Technology, 1980-1984)

6.0 SURFACE WATER HYDROLOGY

Under stabilization in place or on site there are greater restrictions with regard to improving surface water drainage conditions. Upland watershed drainage is generally diverted around the stabilized pile but in severe cases relocation may become necessary. If a tailings pile is to be stabilized in a major floodplain, then the design becomes very difficult and a major point of contention. Relocation to an alternate site ~~can become~~ will generally be a preferred alternative ~~quite easily~~ for severe situations.

Under the relocation option, the primary goal is to locate an alternative site that is not only outside of any floodplain but is also at or near the head end of a drainage area. and is stable against erosion. ~~However, the site relief above surrounding grades should be as low as practicable to improve stability against erosion (e.g., gullying in surrounding areas could reach and disrupt the embankment.)~~

Under any of the options, the pile configuration is very important in affecting the on pile surface runoff. Consideration must be given to splitting up the topslope drainage to avoid concentration of large drainage areas down one sideslope. Drainage of the topslope can also be pitched away from one side to avoid drainage down a longer sideslope.

7.0 EROSION PROTECTION

Erosion protection requirements are ~~primarily determined~~ based on the surface water hydrology. ~~conditions.~~

As discussed in Section 6.0, location of a tailings pile below a large upland watershed or ~~of~~ within a major floodplain can become extremely critical in terms of erosion protection.

With regard to the on-pile surface runoff, the erosion protection is primarily affected by slope, drainage length and flow concentrations.

The steeper the slope the greater the impact of surface drainage and the larger the rock protection ~~is~~ required. Five to one sideslopes have been chosen as the maximum slopes ~~for~~ to facilitate volume containment and slope stability, and will generally require reasonably sized rock protection.* ~~However,~~ For those sites ~~that~~ where durable rock is not reasonably available in the area, flatter ~~more gentle~~ ~~lower~~ slopes may have to be considered to reduce erosion protection requirements.

With regard to flow concentrations, these can occur if settlement occurs within the pile and is not compensated for by overbuilding. Flow concentrations are difficult to predict and can greatly increase the size of the rock protection. Flow concentrations can become severe with long drainage length off the pile. If at all possible, the pile should be configured such that drainage lengths are minimized. ~~especially away from longer sideslopes.~~

* Five to one slopes may not be appropriate in some cases. ~~(e.g., short, low slopes)~~

8.0 SUMMARY

Although many different design considerations can be standardized with general guidelines, it is still quite obvious from this discussion that the process is quite complex. Many different design combinations can occur from the variety of considerations. The design must be site specific in order to account for all the design considerations.

[Recommend that a matrix of all considerations and desired design goals be placed here.]

MKE UMTRA DESIGN PROCEDURES

CHAPTER 4
SITE DRAINAGE

CONTENTS

TABLES

CHAPTER 4

SITE DRAINAGE

4.1 INTRODUCTION

This chapter includes:

- the types of drainage facilities to be designed for UMTRA sites;
- The types of data required for design of drainage facilities; and,
- Details of the procedures to be used for design of ditches and retention basins.

Procedures are presented for determining capacity requirements for ditches, retention basins, and emergency spillways. Site drainage includes all necessary facilities to 1) control runoff and construction wastewater at the site and 2) divert off-site flows away from the site. Primary considerations are to control all runoff and wastewater that may be contaminated, to limit erosion and prevent sediment transport to offsite locations.

Design criteria are expected to vary from site to site, because different federal, state and Indian tribal regulations will apply. An important source of site drainage criteria will be the permit guidance documents provided by the permitting agencies. Therefore, site drainage design should be coordinated with the permitting task. Minimum criteria for all sites have been established by the DOE (Ref. 4-1). The type and number of facilities required at each site also will vary, depending on site conditions and design criteria.

4.2 FACILITIES

Drawings and specifications will be required for one or more of the following types of drainage facilities:

A. Permanent Drainage Facilities
   1. Ditches (sections, locations, grades ...)
   2. Outlet to natural drainage course
   3. Crossings for maintenance vehicles
   4. Fence or barrier crossings
   5. Permanent diversion facilities

B. Construction Drainage Facilities
   1. Ditches (contaminated water directed to retention basin, uncontaminated water diverted away from site)
   2. Silt fences
   3. Sumps and pumps (where gravity drainage is infeasible)

4. Dewatering facilities for excavations
5. Retention basin
   a. Inlet
   b. Basin
   c. Emergency spillway
   d. Outlet to natural drainage course
6. Wastewater treatment facilities
7. Flood control berms
8. Vehicle crossings (e.g., culverts)
9. Fence crossings

4.3 DATA REQUIREMENTS

The following data will be needed for the design of drainage facilities:

A. Permanent Drainage Facilities
   1. Conceptual design site plan and grading plan (with countours) [see Remedial Action Plan (RAP) and coordinate with final design of site plan and grading plan.]
   2. Design criteria [to be presented in Design Basis Memorandum (DBM)]
      (1) UMTRA general (see Refs. 4-1 and 4-10)
      (2) Site-specific (e.g., State or COE Requirements) [see RAP and coordinate with permitting task.]
   3. Probable Maximum Precipitation (PMP) intensity [see Processing Site Characterization Report (PSCR), and check PMP using sources and procedures recommended by the NRC. (Ref. 4-2)]
   4. Location of outlet(s) to natural drainage course (see PSCR and RAP)
   5. Locations of drainage courses with a potential for the Probable Maximum Flood (PMF) and PMF data for site (average velocity, water surface elevation, mean channel slope). (See PSCR and RAP)
   6. Proposed embankment cover material (i.e., rock or grass) and cover type in areas outside of embankment (usually native grasses)

B. Construction Drainage Facilities
   1. Design criteria (to be presented in DBM)
      a. UMTRA general (see Refs. 4-1 and 4-10)
      b. Site-specific (e.g., State or COE requirements) [see RAP and coordinate with permitting task.]
   2. Existing topography (contours)
   3. Conceptual design layout of construction facilities and interim and final grading plans (with contours) (see RAP and coordinate with final design)
   4. Site design storms (Ref. 4-12)
      a. 10-year, 24-hour precipitation - minimum retention basin criteria (Ref. 4-1) - Check site specific requirements also.

b. 10-year precipitation intensity versus duration, for events in the range of anticipated times of concentration (can be obtained from 24-hour intensity). Use for ditch capacity determinations.
c. 25-year storm data, use for minimum retention basin spillway capacity, (Ref. 4-1). Also, check site specific requirements.
d. Other storms required by any site-specific requirements.

5. Design floods (water surface elevations which could affect the site).
   a. 10-year flood - minimum requirement for construction protection (Ref. 4-1).
   b. Other site-specific requirements.
6. Dewatering data (if excavation dewatering is anticipated).
   a. Excavation plan
   b. Ground-water levels (see PSCR)
   c. Subsurface and material data (e.g., material types, structures, pump test results)
7. Construction schedule (see RAP and coordinate with final design). Use for estimating volume of sediment that will accumulate in retention basin
8. Construction facilities and layout requirements (for coordination of all facilities; i.e., access may be restricted to one side of site, certain areas may require excavation of contaminated material, certain areas could be used as borrow sources, etc.).
9. Evaporation data (preferably monthly averages). Possibly necessary in sizing retention basin and determining feasibility of pumping basin dry between storms. (See PSCR)

4.4 DESIGN PROCEDURES FOR DITCHES AND RETENTION BASINS

Procedures to be used for design of ditches and retention basins are:

A. Ditches
   1. Calculate required ditch capacities using the methods, tables and formulae listed in Table 4-1, and following a format similar to that presented.
   2. Determine cross-section shape (triangular or trapezoidal) and side slope(s).
   3. Determine cross-section dimensions and lining.
      a. Grass - or synthetic-lined (temporary condition only) - Use Manning's formula (e.g., see Ref. 4-3). If velocity exceeds allowable velocity for the lining, use riprap.
      b. Riprap-lined - Perform iterative calculations relating depth of flow, riprap size and Manning's n, as outlined in Chapter 5.
   4. Check superelvation and riprap size requirements where ditches change direction and at ditch intersections.
   5. Summarize results in a format similar to that shown in Table 4-3.
   6. Draw typical sections for construction drawings.

B. Retention Basins

1. Runoff volume - Perform hydrologic calculations to determine volume required to store runoff, using the Santa Barbara Urban Hydrograph Computer Model (Ref. 4-4) as modified by T. J. Ward (Ref. 4-11) (available in MKE computer program library).
   a. Input
      (1) Name of the site
      (2) Total drainage area (acres)
      (3) Portion of total area assumed to be impervious (acres)
      (4) Time of concentration (t_c) (minutes). Suggested methods for determining t_c are Figure 30, Method C (Ref. 4-5) and SCS Velocity Method (Ref. 4-6).
      (5) Time increment (~~hours~~) and rainfall depth (inches). minutes
      Time increment duration and number of increments [See (9) below] are chosen as necessary to accurately define rainfall distribution with time.
      (6) Initial and saturated volumetric soil moisture contents, each expressed as a fraction. (Volumetric moisture content is defined as volume of water divided by total volume.) The volumetric soil moisture contents should be determined from data given in the site specific documents, if possible, or from information presented in Ref. 4-7.
      (7) Soil suction head (inches) should be based on (a) USDA textural classification and information in Ref. 4-7, (b) data in the site specific documents, or (c) both.
      (8) The effective hydraulic conductivity, which is taken as one-half the saturated hydraulic conductivity. Site specific hydraulic conductivity data should be used, if possible, or hydraulic conductivity can be based on the USDA textural classification and information in Ref. 4-7.
      (9) Number of rainfall increments (as needed to accurately model rainfall distribution with time, up to a maximum of 100 increments).
      (10) Number of output steps (as needed to accurately model the outflow hydrograph, up to a maximum of 200 steps).
   b. Output
      (1) The outflow hydrograph (flowrate versus time).
      (2) The total volume of outflow (acre-ft.)
2. Sediment storage volume
   a. Sufficient volume should be provided to store total sediment to be collected during entire construction period, if feasible, to avoid need for cleaning.
   b. Use method given in Ref. 4-8 to estimate volume of sediment.
3. Spillway Capacity - Perform hydrologic and hydraulic calculations to determine the required size of emergency spillway using the modified version of Santa Barbara Model (see 1. above).

a. Input
    (1) Retention basin surface area (acres) and side slopes.*
    (2) Emergency overflow ditch (i.e. spillway) side slopes.
    (3) Emergency overflow ditch grade (fraction)
    (4) Manning's n value (e.g. see Ref. 4-3)
b. Output
    (1) Maximum depth of water in the retention basin (feet).
    (2) Maximum depth of flow in the emergency overflow ditch (feet).

---

* Model is to be modified to include retention basin side slopes.

4.5 REFERENCES

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4-9 U. S. Department of the Interior, Surface Mining Water Diversion Manual, OSM/TR-82/2, Office of Surface Mining, U. S. Government Printing Office, Washington, D. C., September 1982.

4-10 U. S. Department of Energy, Plan for Implementing EPA Standards for UMTRA Sites, UMTRA-DOE/AL-163, January 1984.

4-11 Ward, T. J., "Modifications to the Santa Barbara Urban Hydrograph Method," Proceedings, Symposium on Watershed Management in the Eighties, ASCE, 1985.

4-12 U. S. Department of Commerce, "Precipitation Frequency Atlas of ______________ Area," U. S. Government Printing Office, Washington, D. C., (date depends on area covered).

TABLE 4-1

REQUIRED DITCH CAPACITIES

(EXAMPLE FORMAT FOR CALCULATIONS)

Ditch No.	Drainage Area Characteristics						Time of Concentration[2] (min)		Rainfall Intensity[3] (in/hr) i	Average Runoff Coefficient[4] C	Total Area A (ac)	Required Ditch Capacity[5] Q = CiA, (cfs)
	Subarea No. j	Area A_j (ac)	Runoff Coefficient[1] C_j	Flow Path Data								
				Segment No.	Length (ft)	Elev. Diff. (ft)	Method 1	Method 2				

NOTES:

1. Use Table 3.3, Ref. 4-9 for temporary ditches. Assume C = 1 for permanent ditches.
2. Method 1 = Figure 30, Method C, Ref. 4-5; Method 2 = SCS Velocity Method (Ref. 4-6).
3. Use rainfall intensity for duration equal to smallest values for T_C for each ditch.
4. $C = \frac{\sum A_j C_j}{\sum A_j}$ (i.e., area-weighted average C).

TABLE 4-2

SUMMARY OF DITCH DESIGN RESULTS

(EXAMPLE FORMAT)

Ditch No.	Slope	Required Ditch Capacity cu.ft/sec	Flow Depth (ft)	Ditch Depth (ft)	Lining Type

MKE UMTRA DESIGN PROCEDURES

CHAPTER 5

EROSION PROTECTION

CONTENTS

TABLES

FIGURES

APPENDIX

CHAPTER 5

EROSION PROTECTION

5.1 RIPRAP DESIGN

A. Introduction

1. Use of Riprap - Riprap is required for erosion protection of the following site features:

- o On the top and sides of the covered tailings pile.
- o In drainage swales and ditches.

2. Design Method - The design method presented herein is the Safety Factors Method (Ref. 5-10). The riprap is to have a minimum safety factor of 1.0 for each of the following conditions:

- o Available shear resistance divided by average shear stress due to runoff from Probable Maximum Precipitation (PMP), and
- o Available shear resistance divided by peak shear stress due to runoff from 200-year storm.

Key details for applying the Safety Factors Method are as follows:

- o Flow in the riprap voids is deducted from the total runoff to give the flow used to size the riprap.
- o Manning's formula is used to calculate the average velocity.
- o The Corps of Engineer's formula (Ref. 5.11) is used to compute Manning's n. This formula accounts for depth of flow and average rock size so that iteration involving these parameters is required.
- o Average shear stress is computed from

  $\tau = \gamma_w RS$, where γ_w = unit weight of water,
  R = hydraulic radius, and
  S = slope.

- o Peak shear stress is computed from

  $\tau = \gamma_w yS$, where y = depth of flow.

- o Rill and gully formation is assumed on the top and sides of the pile when topsoil is provided over the riprap.
- o Sheet flow is assumed on the top and sides of the pile when no topsoil is provided.

- o The Safety Factors Method gives minimum D_{50} for a given condition. The remaining gradation limits, $D_{100})_{min}$, $D_{100})_{max}$, $D_{25})_{min}$ and layer thickness are then determined using the Corps of Engineer's method (Ref. 5-11).

B. Input Data and Parameters

1. Runoff Calculations - The following input is required to calculate design runoff parameters:

- o Plan view of tailings pile, adequately dimensioned or to a scale that is satisfactory for determining areas and lengths, with contours, and elevations of key points.
- o Plan view of ditches, swales, and area contributing to runoff, meeting the same requirements as specified for the plan of the tailings pile.
- o Cross-sections of tailings pile, cover, swales and ditches.
- o Ground cover and topographic characteristics in sufficient detail for determination of roughness coefficients.
- o Location of the site (latitude and longitude).

2. Riprap Size Calculations - The following input is required to determine optimum riprap sizes:

- o Specific gravity (G_s) of available stone of adequate durability.
- o Angle of internal friction (Ø) of riprap.

C. Methodology

1. Precipitation Intensity

a. Probable Maximum Precipitation (PMP) - The PMP is determined as follows:

(1) Plot location of site on appropriate figure (Figures 15, 17, or 20) given in Chapter 3, Section B in Design of Small Dams (Ref. 5-12) and also included as Appendix 5A.

(2) Read probable maximum precipitation of 1-hour or 6-hour duration, whichever governs, from Figure used in (1) above.

(3) Obtain incremental rainfall amounts, q_r, in inches/hour using time of concentration from below, and NRC Staff Technical Position WM-8201 (Ref. 5-14, page 6).

b. Two Hundred-Year Storm - The 200-year, 30-minute precipitation is determined as follows:

(1) Plot the 2-, 5-, 10-, 25-, 50-, and 100-year, 30-minute precipitation values from Ref. 5-13 on log normal or extreme-value probability paper (Ref. 5-13, page 6).

(2) Draw a straight line, fitting the above values, and extrapolate to 200 years.

(3) Using Ref. 5-1, page 141, Table 4-1, adjust the extrapolated value to determine the precipitation corresponding to the time of concentration determined below.

c. Time of Concentration - Obtain initial value of time of concentration, T_c. For top slope $T_c = 0.0078\ L^{0.77}S^{-0.385}$ (Ref. 5-4, page 8.5) or T_c = 5 minutes, whichever is larger, where L = length of travel in feet and S = slope. Design T_c will be a function of calculated velocity as shown in step h below. For side slope T_c is the sum of $T_c)_{top} + T_c)_{side}$.

2. Top and Sides of Pile With No Topsoil Provided

a. The sheet flow approach is used; i.e., calculations are performed for a 1-foot wide strip of slope length, L.

b. The intensity of runoff for a strip 1-foot wide by length L, in cfs, is: $q_1 = q_r L/43{,}560$ (Ref. 5.15, page 15), where q_r corresponds to T_c.

c. Assume a trial D_{50}, thickness of riprap layer, $t = 1.5\ D_{50}$ or 12 inches, whichever is greater, and porosity, p.

d. Compute through-flow in the riprap voids, q_v, using

$$q_v = pA\ v_v$$

where p = porosity,
A = cross-sectional area,
= t x 1 for 1-foot wide strip,

and v_v = velocity in voids,
$= Wm^{0.5} S^{0.54}$ (Ref. 5-5 p. 90),
where W = empirical constant,
m = mean hydraulic radius,
and S = slope.

Figure 5-1 gives values of $Wm^{0.5}$.

e. Compute net flow $q_{net} = q_1 - q_v$.

f. Select trial value of depth of flow, y.

g. Compute $v = q_{net}/y$ and design $T_c = L/v$ for the top of the pile, and $(L/v)_{top} + (L/v)_{side}$ for the side of the pile.

h. The factor $(L/v)_{top}$ in this equation should correspond to the portion of the top slope directly above the side slope design section. Compare design T_c to initial value of T_c. If design T_c = initial T_c, o.k. Otherwise obtain new q_r and repeat steps b through h until resolution is achieved.

i. Compute trial value of Manning's Coefficient, n, from:
$n = y^{1/6}/[23.85 + 21.95 \log_{10} (y/D_{50})]$ (Ref. 5-11, p. III-7)

j. Check depth of flow assumed using:
$y = (q_{net}\, n/1.486)^{0.6}/(\tan \alpha)^{0.3}$
(based on Ref. 5-15, pp. 15-16).

k. Compute stability factor, n_s, from:
$n_s = \cos\ (1 - \tan \alpha/\tan \emptyset)$
where α = slope angle (Ref. 5-10, p. 643).

l. Compute D_{50} from:

$D_{50} = 21yS/[n_s(G_s - 1)]$,

which is based on $n_s = 21\ \tau_s/[(G_s - 1)\ \gamma_w\ D_{50}]$ (Ref. 5-9, p. 641)

and $\tau_s = \gamma_s\ yS$

m. Compute $t = 1.5\ D_{50}$.

n. Compare t, y and D_{50} to assumed values. Repeat Steps b through m until resolution is achieved.

3. Top and Sides of Pile With Topsoil Provided:

a. The gully development approach is used.

b. The length of a given slope, L, defines the maximum length of potential gully for that slope.

c. The ratio W/L, where W = spacing of potential gullies on the slope, is determined from Figure 5-2. Then $W = (W/L) \times L$.

d. The drainage area for a top slope gully is W X L. Side slope gullies may have a different spacing. The worst case for a side will be a gully which crosses the top and extends down the side, having a total drainage area = $W_t\ L_t + W_s\ L_s$ (t and s indicate top and sides, respectively).

e. The design flow rate, Q, is given by:

$Q = C\ i\ A$,

where $C = 1.0$,

i = precipitation intensity (compute for both 200-year and PMP)

and A = drainage area.

f. Assume a trial D_{50}, thickness of riprap layer, $t = 1.5 \times D_{50}$ or 12 inches, whichever is greater, and porosity, p.

g. Assume gully base width $B = 2 \times D_{50}$, side slopes 2 horizontal to 1 vertical.

h. Compute flow in the riprap voids from:

$$Q_v = pA_v v_v$$

where p = porosity,

$A_v = Bt$, and

$v_v = Wm^{0.5} S^{0.54}$ (Ref. 5-5, p. 90)

where $Wm^{0.5}$ = factor from Figure 5-1,

and S = slope.

i. Compute net flow $Q_{net} = Q - Q_{th}$.

j. Select trial value of flow depth, y.

k. Compute area of flow, $A = 2y^2 + 0.5y$, and hydraulic radius, $R = (2y^2 + 0.5y)/4.5y + 0.5)$.

l. Compute Manning's coefficient, n, from:
$n = R^{1/6}/[23.85 + 21.95 \log 10 \, (R/D_{50})]$

m. Solve $Q = 1.486 \, AR^{0.67} \, S^{0.5}/n$ for y by trial.

n. Compute average shear stress on riprap. The riprap is the base of a trapezoidal channel. In most situations the ratio of base width, B, to flow depth, y, will be larger than 0.5, so that the average stress on the base can be conservatively approximated by the maximum shear stress on the boundary of the section, $\tau_b)_{max}$. $\tau_b)_{max} = C_{RB} \, \gamma_w RS$ where C_{RB} = factor given by Figure 5-3 (Ref. 5-2, p. 13) .

o. Compute $n_s = 21\ \tau_b)_{max}/[(G_s - 1)\ \gamma_w D_{50}]$ (Ref. 5.10, p. 641).

p. Compute $SF = \cos\alpha \tan\emptyset/(n_s \tan\emptyset + \sin\alpha)$ (Ref. 5.10, p. 643).

q. If $SF \neq 1.0$, repeat Steps f through p until resolution is achieved.

r. It is not necessary to repeat the calculation for peak stress, because only the riprap on the base of the trapezoidal cross-section is subjected to shear stress.

4. Swales and Ditches

a. Design flow rate for a given swale is given by:

$$Q = C\ i\ A$$

where $C = 1.0$

i = intensity of 5-minute PMP

and A = drainage area.

b. Assume triangular cross-section with a trial D_{50}, depth of flow, y, thickness of riprap layer, $t = 1.5\ D_{50}$ or 12 inches, whichever is larger, and porosity p.

c. Compute through flow from

$$A_{th} = p\ A_{th} v_{th}$$

where p = porosity

$A_{th} = N\ (2yt + t^2)$ [Conservative approximation]

where N = side slope of ditch (N Horizontal: 1 Vertical)

and $v_{th} = Wm^{0.5}\ S^{0.54}$ (Ref. 5-5, pg 90)

where $Wm^{0.5}$ = factor from Table 5-1 or Figure 5-1

and S = slope.

d. Compute net flow $Q_{net} = Q-Q_{th}$.

e. Compute area of flow, $A = Ny^2$, and hydraulic radius, $R = Ny/[2\sqrt{1+N^2}]$.

f. Compute Manning's coefficient, n, from $n = R^{1/6}/[23.85+21.95 \log_{10} (R/D_{50})]$

g. Solve $Q_{net} = 1.486\ AR^{0.67}S^{0.5}/n$ for y by trial, and compute R.

h. Compute average shear stress $\tau_{avg.} = \gamma_w RS$ (Ref. 5-9, p. 40)

i. Compute $n_s = 21\ \tau_{avg}/[(Gs-1)\ \gamma_w D_{50}]$ (Ref. 5-8, p. 641).

j. For relatively flat invert grade (S __%*) compute SF from $SF = (S_m/2)[(E_2 + 4)^{0.5} - E]$
where $E = S_m\ n_s \sec \Theta$,
$S_m = \tan\phi/\tan\Theta$,
and Θ = side slope of swale or ditch.

k. If SF $\neq$ 1.0, repeat Steps b through j until resolution is achieved.

l. Repeat Steps a through k using rainfall intensity for a 200-year storm and peak shear stress $\tau_o)_{max} = C_s\ \gamma_w\ y\ S$, where C_s = factor given by Figure 5-3.

5. Gradation

a. Compute $W_{50})_{min} = \pi G_s\ \gamma_w D_{50}^3/6$ (assuming spherical rock pieces).

*Maximum slope for which this equaton applies is to be determined.

b. Compute $W_{100})_{min} = 2\ W_{50})_{min}$ (Ref. 5-11, p. 42).

$$W_{100})_{max} = 5\ W_{50})_{min}$$
$$W_{25})_{min} = W_{100})_{max}/16$$

c. Compute $D_{100})_{min}$, $D_{100})_{max}$ and $D_{25})_{min}$ from

$$D = [6W/(\pi G_s\ \gamma_w)]^{1/3}.$$

d. Plot upper and lower bound gradation curves, adjust if necessary to utilize gradations already produced locally, and determine ranges for the following sieve sizes:

Sieve Size	Percent Finer By Weight
$D_{100})_{max}$	100
sufficient sizes to	___ to ___
define curves, in even	___ to ___
inches	___ to ___
1 inch	___ to ___
1/2 inch	___ to ___
No. 4	___ to ___

6. Filters and Bedding Material

a. Compute governing filter sizes from:

(1) $D_{15})_f < 5\ D_{85})_b$ (Ref. 5-12, p. 235)

where b = base material (below filter)

(use gradation of portion passing No. 4 sieve) (Ref. 5-10, p. 236)

(2) $D_{50})_f < 25\ D_{50})_b$ (Ref. 5-17, p. 7-8-14)

(3) $D_{15})_f < 20\ D_{15})_b$ for uniform base material ($C_u < 4$) (Ref. 5-17, p. 7-8-14), or

$<40\ D_{15})_b$ for broadly graded base material ($C_u > 4$).

(4) $D_{15})_f > 5D_{15})_b$ (Ref. 5-16, p. 59)

(5) $D_{max} < 3$ inch (Ref. 5-12, p. 236).

(6) To avoid internal movement of fines, filter material should have no more than 5% passing No. 200 sieve (Ref. 5-12, p. 235).

(7) Based on recent studies (Ref. 5-18), strict compliance with filter criteria shown under a(1) will be required, but those shown under a(2) and a(3) could be relaxed.

b. Check need for second stage filter by considering first stage (from a.) as base and overlying riprap as filter in equations (1) through (4) above. Design 2nd stage filter, if required.

c. Plot upper and lower bound gradation curves for filter(s), adjust if necessary to utilize gradations already produced locally, and determine ranges for the following sieve sizes:

Sieve Size	Percent Finer By Weight
$D_{100})_{max}$	100
2 inches	___ to ___
1 inch	___ to ___
1/2 inch	___ to ___
No. 4	___ to ___
No. 4	___ to ___
No. 16	___ to ___
No. 30	___ to ___
No. 50	___ to ___
No. 100	___ to ___
No. 200	___ to ___

5.2 RIPRAP MATERIALS SELECTION

A. Introduction

Before completing specifications for erosion protection elements, the Site Design Engineer should review the potentially available quantities and the durability characteristics of local materials. It may be necessary to modify the computed gradation limits and thickness requirements in order to develop a balanced, practical design. Guidelines for the investigation of available materials are presented herein.

B. Input Data

In the beginning of the investigation, it is necessary to know:

1. Quantities and Sizes of Material - Approximate required quantities and sizes of erosion protection material can be estimated from thickness of rock protection required for similar sites and from rough calculations for the area of the site in question.

2. Riprap Durability - The data presented in Tables 5-2 and 5-3 are included as guidelines to judge the suitability of a rock source for use as riprap.

C. Methodology

The material selection process is divided into two phases: 1) Data Collection and 2) Data Analysis.

1. Data Collection - Much available data can be obtained via telephone and written correspondence with local quarry operators. Most operating local quarries have test results and quantity estimates on file. In addition, independent sources such as local highway departments, U.S. Army Corps of Engineers, and local governments may have valuable data for local sources. Generally, only results for tests two through six in Table 5-2, and petrographic analysis will be available. Communications regarding existing data should include inquiries into past local use of each type of rock in question, especially regarding length of exposure. Such data can be extremely valuable in judging long-term durability.

After an initial screening of the data available by telephone and mail, potential borrow sites should be visited. A site visit will permit visual examination, testing with a geologist's hammer, and independent selection of samples for gradation and rock quality testing. Tests two through six in Table 5-2 and petrographic analyses should be performed at a reputable laboratory. In addition, the search for cases where each rock type has been subjected to long-term exposure should be extended and these sites visited to evaluate the rock's performance. It is important to identify during the early stages of design any potential problem in obtaining suitable erosion protection materials within an economic hauling distance from the site.

2. Data Analysis - Gradation requirements determined, using the procedures presented in Section 5.1, should be plotted and compared with

gradations commercially produced. Without compromising performance, the gradation limits should be adjusted to make maximum use of available gradations. When necessary to ensure satisfactory results, layer thicknesses may be increased, if economically justified.

Presented below are guidelines for assessing the suitability of rock sources as erosion protection material.

- o Wherever possible, a particular source of riprap should be specified. Priority should be given to specific rock types that have been exposed for periods of 50 to 100 years and have suffered essentially no deterioration. As a quality control measure, the minimum specific gravity, maximum absorption, sulphate soundness weight loss, and abrasion loss values should be specified as acceptance criteria at each site.

- o Specifications for materials, for which observable evidence regarding long-term performance is not available, should be selected using the third column in Table 5-2 (good quality rock), and then modified to allow for unusual characteristics of particular rock types. For example, the specific gravity of certain rock types (gabbro, gneiss, etc.) is consistently greater than 2.9. Therefore gabbro with a value of 2.65, while acceptable according to Table 5-2, could perform in an unsatisfactory manner.

- o Because there are currently no physical tests that can predict the performance of rock or gravel after 200 to 1000 years exposure, some judgment will have to be used to supplement the test data in selecting acceptable quality rock.

3. Form of Specifications - The gradation limits developed in Section 5.1 should be included in the technical Specifications.

Rock quality criteria used for the acceptance of riprap, bedding, and filter materials may be prescribed in the following format:

- o Representative samples of riprap material shall meet the following requirements:

Tests	Designation	Requirements
Specific Gravity and Absorption	ASTM C127-81	S.G. (SSD) not less than ___ Absorption not more than ___%
Soundness (sodium-sulfate method)	ASTM C88-76	Maximum weight loss ___%
Abrasion	ASTM C131-81	Maximum loss of material finer than #12 sieve is ___%

- Samples of bedding and filter materials shall meet the following requirements:

Tests	Designation	Requirements
Specific Gravity (SSD):	ASTM C127-81 or ASTM C128-79	Greater than ____
Soundness (sodium-sulfate method):	ASTM C88-76	Less than __ percent loss of weight after 5 cycles
Abrasion	ASTM C131-81	Less than __ percent loss of weight after 500 revolutions

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TABLE 5-1

CHECK LIST - AREA DRAINAGE AND EROSION PROTECTION DESIGN
(Sheet 1 of 2)

1. Name of site, state:

2. Approximate Coordinates: Processing Site
Disposal Site

3. Basic Data:	Available	Not Available
a) Correct Base Map/Topo (1" 200')		
b) Hydro-Met Data (Precipitation, Evaporation, Temp, etc.)		
c) Information on Nature of Vegetation		
d) Stream Flow Data		
e) Drainage Area Topo (1" 200')		

4. Slope Protection Materials	Sand	Gravel	Riprap
a) Potential Sources			
b) Required Quantities (c.y)			
c) Available Quantities (c.y)			
d) Test Data Attached			

5. Reference Showing Contaminated Material Boundary

6. Reference Showing Geometry of Tailings Piles

7. Reference Showing Site Layout (Plan and X-Section)

TABLE 5-1
(Sheet 2 of 2)

8. Review of Conceptual Design in RAP — Yes — No

a) Design Sufficiently Detailed for Review Purposes

b) All Supportive Data & Docs. Available

c) Minor Change from Draft RAP, Need Not Redesign

d) Major Change from Draft RAP, Need to Redesign

9. a) If Redesigning, State Reasons ______________________________

____

b) Schedule for Redesign: Start __________ Finish __________

c) Proposed Method or Methods of Design ______________________________

____

d) Manual Computation/Computer Solution ______________________________

____

10. Other Data

11. Comments

TABLE 5-2

U.S. BUREAU OF RECLAMATION STANDARDS
FOR JUDGING RIPRAP DURABILITY
(REF. 5-19)

Test	Quality: Poor	Quality: Fair	Quality: Good
Ultrasonic cavitation rating	0 to 5	5 to 7	7 to 10
Bulk specific gravity	2.5	2.5 to 2.65	2.65
Absorption, %	1.0	0.5 to 1.0	0.5
Freeze-thaw weight loss, %[a]	5	0.5	0 to 0.5
Na_2SO_4 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 5-3

COMPRESSIVE STRENGTH OF VARIOUS ROCKS
(REF. 5-19)

Rock Type	Strength, psi
Diabase and some basalts and quartzites	Over 40,000
Fine-grained granite, diorite, basalt, quartzite, well-cemented sandstone and limestone	25,000 to 40,000
Average sandstone and limestone, coarse-grained granite and gneiss	10,000 to 25,000
Porous sandstone and limestone, shales	5,000 to 10,000
Tuff, talc, siltstone, very porous sandstone	Under 5,000

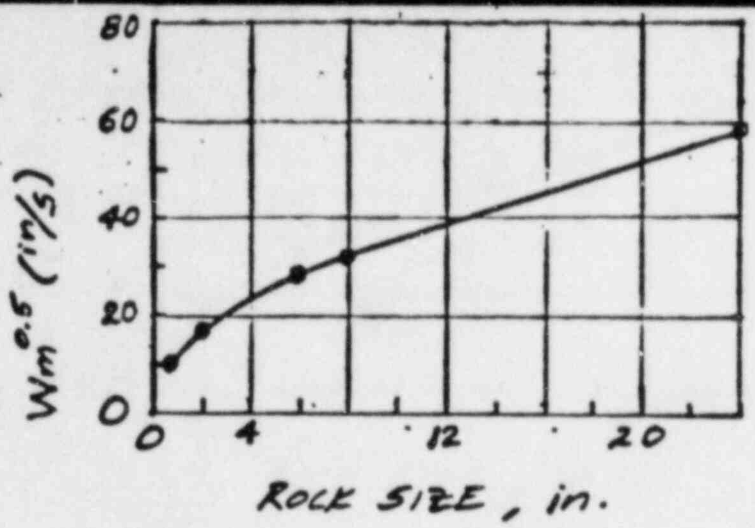

FIGURE 5-1 Mean hydraulic radius function versus rock size for the computation of turbulent flow velocity (After Ref. 5-5, Table 2, page 90)

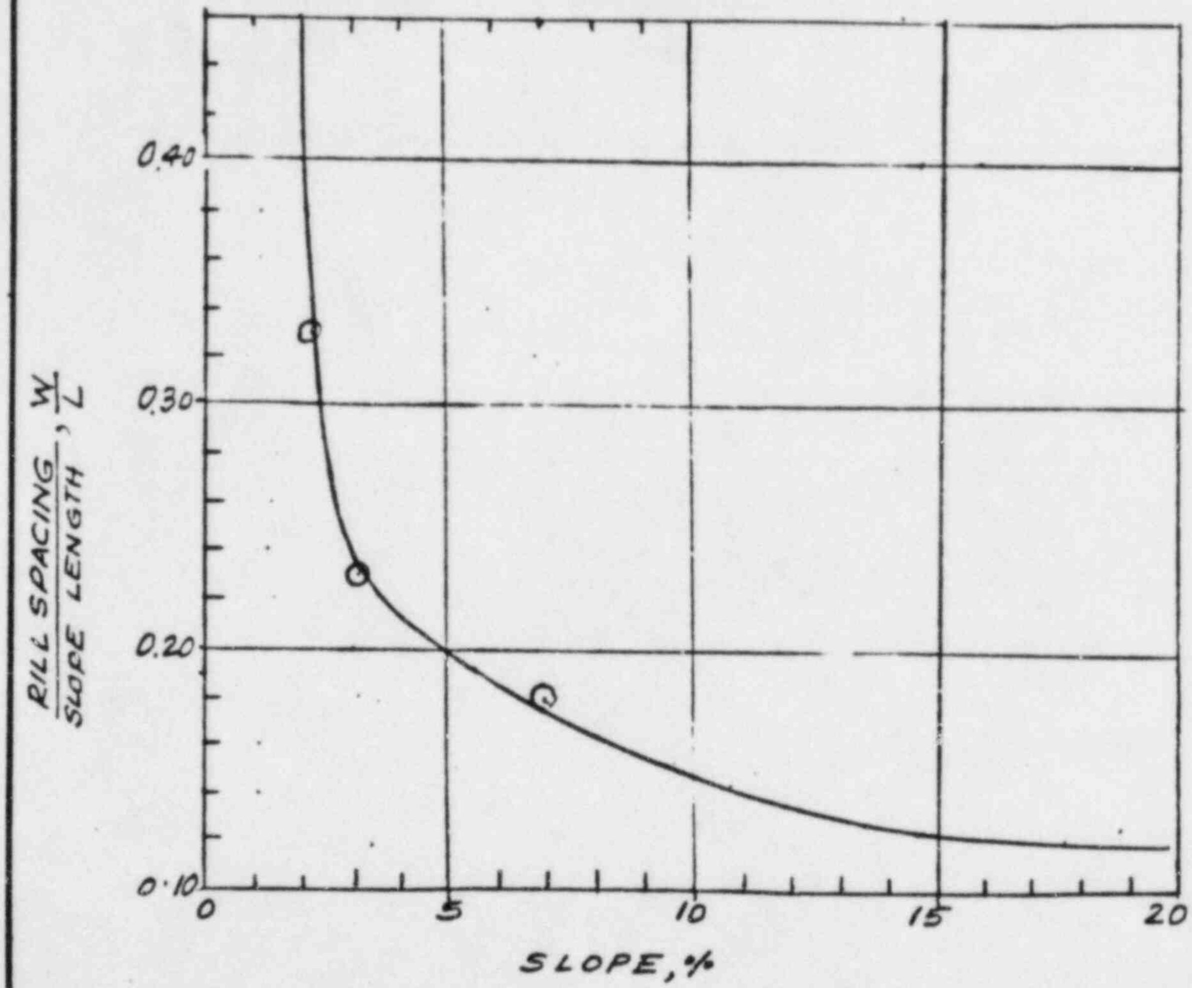

FIGURE 5-2 Rill spacing/slope length versus slope inclination (Based on data from Ref. 5-7, page 75)

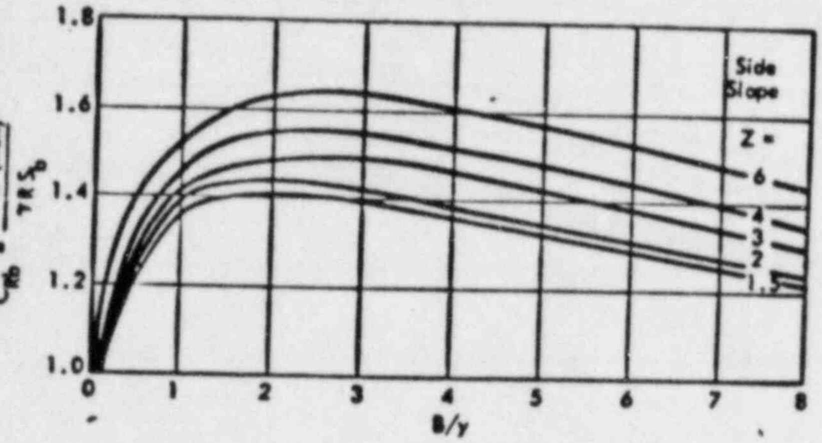

FIGURE 5-3 Maximum boundary shear stress on bottom of trapezoidal channels. (Ref. 5-2, page 13)

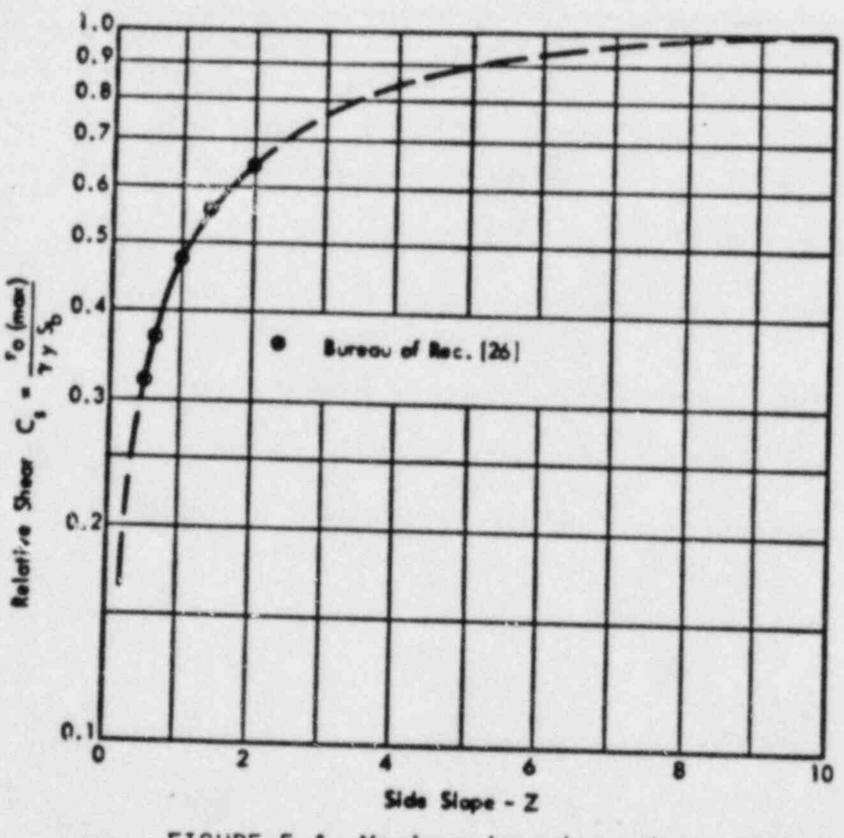

FIGURE 5-4 Maximum boundary shear stress on sides of triangular channels. (Ref. 5-2, page 12)

CONSULTING ENGINEERS
INTERNATIONAL ENGINEERING COMPANY, INC.
180 HOWARD STREET, SAN FRANCISCO, CALIFORNIA 94105

DESIGNED	DRAWN	CHECKED	RECOMMENDED	SHEET OF	REV
DATE		APPROVED		IECO NO	

APPENDIX 5A

PROBABLE MAXIMUM PRECIPITATION AND THUNDERSTORM MAPS OF THE UNITED STATES

Figure 5A-1 Probable maximum precipitation (inches) east of the 105° meridian for an area of 10 square miles and 6 hours' duration. 288-D-2449A, 288-D-2754, 288-D-2755.

(Ref. 5-12, Fig. 15, Pg. 48)

Figure 5A-2 Probable maximum 6-hour point values in inches for general-type storms west of the 105° meridian. 288-D-2756, 288-D-2757.

(Ref. 5-12, Fig. 17, Pg. 50)

Figure 5A3 Probable maximum thunderstorm 1-hour rainfall (point values in inches) for area west of 105° meridian. 288-D-2760, 288-D-2761.

(Ref. 5-12, Fig. 20, Pg. 53)