URANIUM MILL TAILINGS REMEDIAL ACTION (UMTRA)
DESIGN PROCEDURES MANUAL

Prepared by
Morrison-Knudsen Engineers, Inc.

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REVISION 4 TO THE MKE UMTRA DESIGN PROCEDURES

Chapter	Title	Revision	Description of Revision
4	Site Drainage	3	Added requirements for retention basin on Page 4-3.
5	Erosion Protection	3	Added limitations on interstitial flow, requirements for riprap at top of embankment side slope, limits on Stephenson's Method, details of Safety Factors Method for side slopes, correction to equation for Safety Factors parameter, equations relating gradation diameters, limits on criteria for bedding and filters, and an addition to the methodology for assessing rock sources. These additions are on Pages 5-1 through 5-4, 5-5 through 5-16, and 5-18.
6	Radon Barrier	1	Corrected Equation 6.1 and added a third procedure for estimating residual moisture content. These revisions are on Pages 6-2 through 6-3.
8	Site Seismicity Evaluation and Development	2	Deleted Appendix 8A.
14	Radon Barrier	1	Corrected misprint on Page 14-2.

FOREWORD

This manual provides MKE engineers with guidelines for the design of remedial actions at UMTRA sites and standardizes our procedures for designing those elements that are critical to the DOE and NRC. The manual is intended to provide guidance and procedures only for those elements of design that are peculiar to the UMTRA Project, and that are expected to be common to most UMTRA sites. Routine design is not included herein. Engineers are encouraged to be creative in all design work.

The procedures included in this manual are based on state-of-the-art design methods and current criteria. The manual will be changed as required to ensure that the methods remain current. Suggestions for improvement are invited from all users.

Application of the methods presented herein will result in designs for remedial action requiring minimum maintenance, and will provide assurance that the control measures will meet the design life requirements of at least 200 years and, to the extent reasonably achievable, up to 1,000 years.

MKE UMTRA DESIGN PROCEDURES

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MKE UMTRA DESIGN PROCEDURES

CHAPTER 4

SITE DRAINAGE

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CHAPTER 4

SITE DRAINAGE

4.1 INTRODUCTION

This chapter includes:

- o the types of drainage facilities to be designed for UMTRA sites;
- o The types of data required for design of drainage facilities; and,
- o 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 nation 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 contours) [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 4-15)
      (2) Site-specific (e.g., State or COE Requirements) [see RAP and coordinate with permitting task.]
   3. Probable Maximum Precipitation (PMP) intensity and rainfall distribution [see Processing Site Characterization Report (PSCR), and check PMP using appropriate Hydrometeorological Report by NOAA (See Fig. 4-1, Ref's. 4-2 through 4-7, Table 4-1, and Chapter 5, Sec. 5.1.C1.)]
   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)
   7. Soil type and vegetation cover in drainage area.

B. Construction Drainage Facilities
   1. Design criteria (to be presented in DBM)
      a. UMTRA general (see Refs. 4-1 and 4-15)
      b. Site-specific (e.g., State or COE requirements) [see RAP and coordinate with permitting task.]
   2. Existing topography (contours)
   3. Required construction facilities and interim and final grading plans (with contours) (see RAP and coordinate with final design)
   4. Site design storms (Ref. 4-17)
      When duration of a storm is specified, it is important to define the site-specific rainfall distribution.

a. 10-year, 24-hour precipitation - minimum retention basin criteria (Ref. 4-1) - Also check site specific requirements regarding run-off, infiltration, evaporation, water treatment, and other considerations. R-3
b. 10-year precipitation intensity versus duration, for events in the range of anticipated times of concentration. 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, 24-hour, 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, direct precipitation, runoff and snowmelt (preferably monthly averages), and soil type in basin area. 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 one of the following methods to calculate peak runoff:

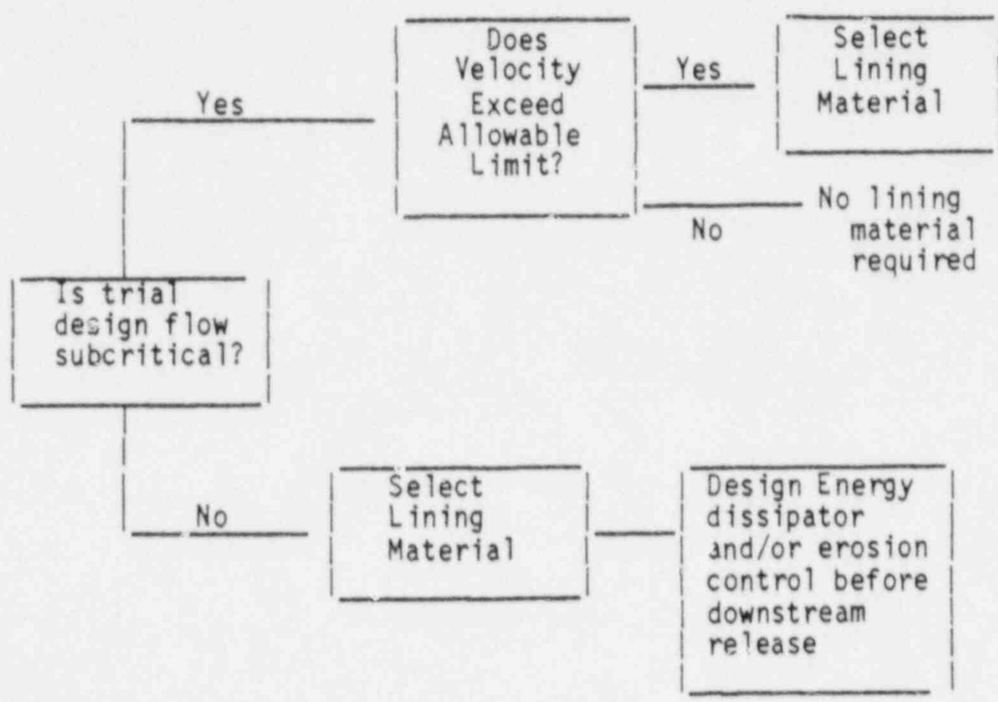
Drainage Area	Run-off Calculation Procedure
Up to 200 acres	Rational Method
Up to 500 acres	Santa Barbara Method (Ref. 4-16)
Up to 20 sq. mi.	SCS Method (Tabular, Graphical, or Unit Hydrograph Method (Ref. 4-11)
Any drainage area	HEC-1 (Ref. 4-18)

Tables and formulas similar to those shown in Table 4-2 (Rational Method) should be used.

The HEC-1 program uses the following methods for synthesizing unit Hydrographs:

Method	Drainage Area
SCS	up to 100 sq. mi.
Clark	up to 100 sq. mi.
Snyder	10 - 10,000 sq. mi.

2. Determine cross-section shape (triangular or trapezoidal). If discharge (Q)   10 cfs, use triangular ditch, otherwise use trapezodial ditch with initial bottom width = 10'.
3. Using trial side slope(s) and Manning's formula (e.g., see Ref. 4-8), design ditch for subcritical flow where practical by adjusting channel slope. Also, may optimize ditch dimensions for site-specific conditions. Use site-specific information (e.g. soil type, vegetation, and channel slope) to select allowable channel velocities. Allowable velocity guidance is available in References 4-8 and 4-20.
4. Determine final cross-section dimensions and lining, as follows:
   a. General - Proceed as follows:



   b. Riprap-lined - Perform iterative calculations relating depth of flow, riprap size and Manning's n, as outlined in Chapter 5.

4. Check superelevation 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

Retention Basins associated with Temporary Drainage Facilities:

1. Basic criteria - minimum storage of 10-year, 24-hour storm or minimum retention time (24 hr.) for runoff plus 3-year sediment storage or storage of sediment from a 10-year, 24-hour storm.
2. Runoff volume - Perform hydrologic calculations to determine volume required to store runoff, using the Santa Barbara Urban Hydrograph Computer Model (Ref. 4-9) as modified by T. J. Ward (Ref. 4-16) (available in MKE computer program library)*, the SCS method (Ref. 4-11), or HEC-1 (Ref. 4-18).
   Note: The MKE program assumes initial obstruction loss equal to zero to account for possibility that antecedent conditions have saturated the obstruction capacity.
   a. Input for Santa Barbara Method
      (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. 4-10) and SCS Velocity Method (Ref. 4-11).
      (5) Time increment (minutes) and rainfall depth (inches). 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. 4-21.
      (7) Soil suction head (inches) should be based on (a) USDA textural classification and information in Ref. 4-12, (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. 4-12.
      (9) Number of rainfall increments (as needed to accurately model rainfall distribution with time, up to a maximum of 100 increments). The time increment for the rain should be approximately equal to 1/5 of time of concentration.

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* See Appendix 4-1 for comments on the use of the Santa Barbara Method.

(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.)

3. 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 Universal Soil Loss Equation (USLE) or modified form given in Ref. 4-13 to estimate volume of sediment.

4. Spillway Capacity - Spillway design flood depends on regulatory agency-requirements and should not be less than 25-year, 24-hour. Perform hydrologic and hydraulic calculations to determine the required size of emergency spillway using the modified version of Santa Barbara Model (see 1. above), with the reservoir routing routine.
   a. Input
      (1) Retention basin area capacity curve.*
      (2) Rating curve for trial design dimensions.
   b. Output
      (1) Maximum depth of water in the retention basin (feet).
      (2) Maximum depth of flow in V - ditch (feet), or depth of flow across weir (feet).

4.5 WATER QUALITY

Provide monitoring systems for surface and ground water if required.

*Model currently assumes vertical side slopes; to be modified to include actual side slopes.

4.6 REFERENCES

4-1 U. S. Department of Energy, Design Criteria for Stabilization of Inactive Uranium Mill Tailings Sites, UMTRA-DOE/AL-050424.0049, June 1984.

4-2 U. S. Weather Bureau, Probable Maximum Precipitation, Northwest States, Hydrometeorological Report No. 43, NOAA, U.S. Dept. of Commerce, April, 1981.

4-3 Hansen, E. M., Schwarz, F. K., and Riedel, J. T., Probable Maximum Precipitation Estimates, Colorado River and Great Basin Drainages, Hydrometeorological Report No. 49, National Weather Service, NOAA, U.S. Dept. of Commerce, 1977.

4-4 Schreiner, L. G. and Riedel, J. T., Probable Maximum Precipitation Estimates, United States East of the 105th Meridian, Hydrometeorological Report No. 51, National Weather Service, NOAA, U.S. Dept. of Commerce, 1978.

4-5 Hansen, E. M., et. al., Probable Maximum Precipitation Estimates, United States East of the 105th Meridian (Application Report), Hydrometeorological Report No. 52, National Weather Service, NOAA, U.S. Dept. of Commerce, 1982.

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4-7 Miller, J. F., et. al., Probable Maximum Precipitation Estimates, U.S. Between Continental Divide and 103rd Meridian, Hydrometeorological Report No. 55, National Weather Service, NOAA, U.S. Dept. of Commerce, 1984.

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4-13 Israelsen, C. E., Clyde, C. G., Fletcher, J. E., Israelsen, E. K., Haws, F. W., Packer, P. E., and Farmer, E. E., Erosion Control During Highway Construction, National Cooperative Highway Research Program Report 221, National Academy of Sciences, Washington, D. C., April 1980.

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4-17 U. S. Department of Commerce, "Precipitation Frequency Atlas of The Western United States," U. S. Government Printing Office, Washington, D. C., (specific atlas and date depend on area covered).

4-18 U.S. Army Corps of Engineers, "HEC-1, Flood Hydrograph Package Users' Manual," The Hydraulic Engineering Center, Davis, California, 95616, Revised January 1985.

4-19 Brater, E. F. and King, H. W., Handbook of Hydraulics, Sixth Edition, McGraw-Hill, 1976.

4-20 U.S. Army Corps of Engineers, 1 July 1970, "Hydraulic Design of Flood Control Channels", Changes 1 through 4 included, EM 1110-2-1601, Washington, D.C.

4-21 Israelsen, O.W. and Hanson, V.E., Irrigation Principles and Practices, 1962, John Wiley and Son, Inc., P. 168

APPENDIX 4 - 1

COMMENTS ON THE USE OF THE SANTA BARBARA URBAN HYDROGRAPH METHOD

- Limitation of the method

  The Santa Barbara Urban Hydrograph method is subject to the following restrictions in application:

  o For watershed less that 500 Acres
  o For time of concentration less than 1 hour
  o For rainfall duration less than about 10 hours

  Violation of any of the above requirements may render the results unreliable.

- Limitation on time increment, t, for rainfall and hydrograph during simulation.

  The value of t should be restricted to either (a) or (b) as follows, whichever is smaller.

  (a) t should not be more than one fifth of the time of concentration.

  (b) In the case of modeling infiltration for pervious area, t should be reduced to Dt when the hourly rainfall intensity is equal to or greater than:

  $$\frac{k}{2} \times \frac{Dt}{60} \times \frac{1}{FRAC}$$

  Where k = hydraulic conductivity (in/hr)
  Dt = time duration with highest known rainfall intensity (min)
  FRAC = Fraction of hourly rainfall during Dt when the most intensive rainfall occurs

- Vegetation Cover is not considered in the runoff-infiltration relationship

- Green and Ampt Infiltration

  (a) The Green and Ampt infiltration equation used in the program assumes homogeneous condition for the soil. If non-homogeneity occurs close to the ground surface, the equation should be modified to account for a composite hydraulic conductivity for the layered system.

(b) The porosity and effective porosity values listed in Table 2 of Rawls, et.al.[1], are not reliable and should be used with caution. Other values seem to be reasonable and may be used in modeling for values of porosity and effective porosity for different textural soils. Other references (e.g., Israelsen & Hansen[2]) should be consulted.

- Reservoir Routing

  (a) The computer program assumes the water surface area in the retention basin is always constant. This will be conservative if the area selected is the minimum during the routing period. To model accurately the rise of water in the basin during a storm, the area-capacity curve should be used to iterate the stage in the reservoir during each time step until a water balance is achieved, consistent with the spillway rating curve (conventional modified Puls method).

  (b) In general, when the spillway channel has a mild slope, Manning's equation may be used to describe the flow. However, when the channel slope is increased to or beyond the critical slope, a control will be created at the outlet of the reservoir in which case weir-flow will be controlling. Thus, specifying the channel slope in the program without checking critical flow may introduce error in the outflow. A spillway rating curve should be derived after checking the control and considering the velocity of approach. This, then, would be in the form usable for reservoir routing in the computer program. This check can be done by hand calculation.

Recommendation

The computer program SHUHYD should be used as suggested above, especially in the reservoir routing routine.

When use of the Santa Barbara Method is not appropriate, the SCS Unitgraph Method should be used.

---

1. Rawls, W.J. Brakensiek, D.L. and Miller, N., "Green-Ampt Infiltration Parameters for Soil Data" J. of the Hydraulics Division, Vol. 109, No. 1, ASCE, New York, January 1983.

2. Israelsen, O.W. & V.E. Hansen, "Irrigation Principles and Practices", 1962, John Wiley & Son, Inc., P. 168

TABLE 4-1

Generalized PMP Studies for Conterminous United States, Ref. 4-7, Page 2 2

Hydrometeorological Report	Geographical Region	Scope
No. 36 (U.S. Weather Bureau 1961 Revision, U.S. Weather Bureau 1969)	Pacific coast drainage of California	General-storm PMP; areas to to 5,000 mi^2, 6 to 72 hr., seasonal values October through April
No. 43 (U.S. Weather Bureau 1966 addendum 1981)	Columbia River and coastal drainages of Oregon and Washington	General-storm PMP, areas up to 5,000 mi^2, west of Cascades Ridge, areas up to 1,000 mi^2 east of Cascades Ridge, 6 to 72 hr., seasonal values October through June. Local-storm PMP east of Cascades Ridge, areas up to 500 mi^2, durations to 6 hr., seasonal values May through September.
No. 49 (Hansen et al. 1977)	Colorado River and Great Basin drainage. Also provides local storm for all of California	General-storm PMP, areas up to 5,000 mi^2, 6 to 72 hr., monthly values. Local-storm PMP, areas up to 500 mi^2, durations up to 6 hr., all season values.
No. 51 (Schreiner and Riedel 1978)	U.S. east of 105th meridian*	PMP from 10 to 20,000 mi^2, 6 to 72 hr., all season values.
No. 52 (Hansen et al. 1982)	U.S. east of 105th meridian*	PMP from 10 to 20,000 mi^2, durations $\leq$ 6 hr. all season values (Application report).
No. 53 (Ho and Riedel 1980)	U.S. east of 103rd meridian*	PMP for 10 mi^2, 6 to 72 hr., monthly values.
No. 55 (Miller et al. 1984)	U.S. between Continental Divide and 103rd meridian	General-storm PMP, areas 10 to 20,000 mi^2 in nonorographic regions and 10 to 5,000 mi^2 in orographic regions, 1 to 72 hr., all-season values. Local-storm PMP, for selected portions of study region, up to 500 mi^2, durations <6 hr., all season values.

*Reports 51, 52, and 53 originally provided PMP for the U.S. east of the 105th meridian, PMP between the 103rd and 105th meridian from these reports are now superseded by HMR 55. Application portion of HMR 52 is valid for Eastern U.S. out to the 105th meridian.

TABLE 4-2

REQUIRED DITCH CAPACITIES BY RATIONAL METHOD

(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 Q = CIA, (cfs)
	Subarea No. j	Area A_j (ac)	Runoff Coefficient[1] C_j	Flow Path Data			Method 1	Method 2				
				Segment No.	Length (ft)	Elev. Diff. (ft)						

NOTES:

1. Use Table 3.3, Ref. 4-14 for temporary ditches. Assume C = 1 for permanent ditches.

2. For time of concentration for temporary ditches, Method 1 = Figure 3C, Method C, Ref. 4-10; Method 2 = SCS Velocity Method (Ref. 4-11). For permanent ditches, See Chapter 5, Section 5.1.C.4.

3. Use rainfall intensity for duration equal to smallest values for T_c for each ditch.

4. $C = \frac{A_j C_j}{A_j}$ (i.e., area-weighted average C).

TABLE 4-3

SUMMARY OF DITCH DESIGN RESULTS

(EXAMPLE FORMAT)

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

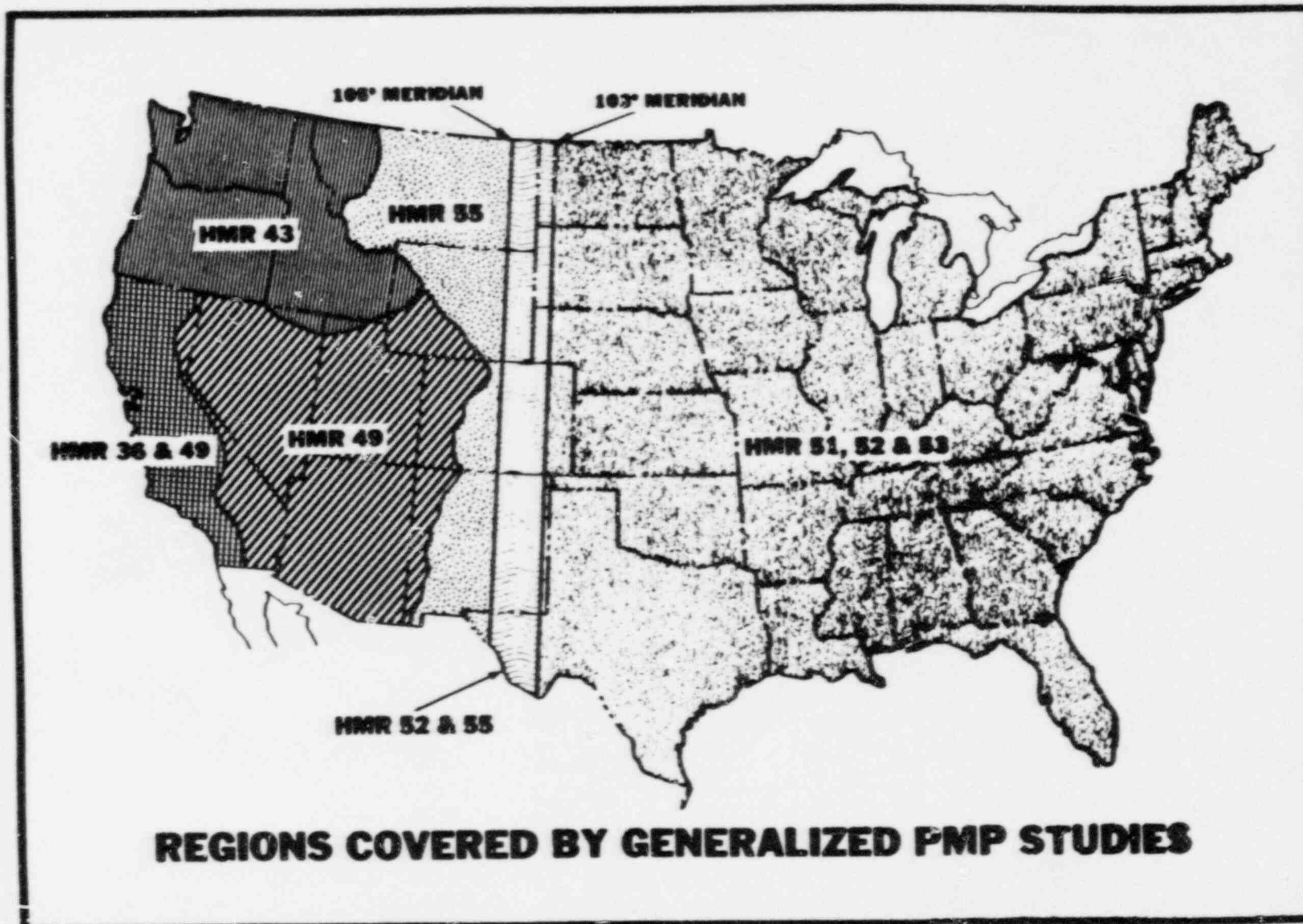

Figure 4-1.—Regions of the conterminous United States for which PMP estimates are provided in indicated Hydrometeorological Reports. See Table 4-1 for description of geographical region covered and scope of each report. (Ref. 4-7, Page 3).

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  The value of Δt should be restricted to either (a) or (b) as follows, whichever is smaller.

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  $$\frac{k}{2} \times \frac{Dt}{60} \times \frac{1}{FRAC}$$

  Where k = hydraulic conductivity (in/hr)
  Dt = time duration with highest known rainfall intensity (min)
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  (a) The Green and Ampt infiltration equation used in the program assumes homogeneous condition for the soil. If non-homogeneity occurs close to the ground surface, the equation should be modified to account for a composite hydraulic conductivity for the layered system.

(b) The porosity and effective porosity values listed in Table 2 of Rawls, et.al.[1], are not reliable and should be used with caution. Other values seem to be reasonable and may be used in modeling for values of porosity and effective porosity for different textural soils. Other references (e.g., Israelsen & Hansen[2]) should be consulted.

- Reservoir Routing

(a) The computer program assumes the water surface area in the retention basin is always constant. This will be conservative if the area selected is the minimum during the routing period. To model accurately the rise of water in the basin during a storm, the area-capacity curve should be used to iterate the stage in the reservoir during each time step until a water balance is achieved, consistent with the spillway rating curve (conventional modified Puls method).

(b) In general, when the spillway channel has a mild slope, Manning's equation may be used to describe the flow. However, when the channel slope is increased to or beyond the critical slope, a control will be created at the outlet of the reservoir in which case weir-flow will be controlling. Thus, specifying the channel slope in the program without checking critical flow may introduce error in the outflow. A spillway rating curve should be derived after checking the control and considering the velocity of approach. This, then, would be in the form usable for reservoir routing in the computer program. This check can be done by hand calculation.

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1. Rawls, W.J. Brakensiek, D.L. and Miller, N., "Green-Ampt Infiltration Parameters for Soil Data" J. of the Hydraulics Division, Vol. 109, No. 1, ASCE, New York, January 1983.

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CHAPTER 5
EROSION PROTECTION
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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 The top and sides of the covered tailings pile.
- o Drainage swales and ditches.

2. Design Methods - The design methods presented herein are:

   a. The Stephenson's Method (Ref. 5-9), used for slopes equal to or steeper than 10 percent subjected to sheet flow [top and sides of embankment where no topsoil is provided (topsoil will develop gulleys, and sheet flow assumption will no longer apply)],

   b. The Safety Factors Method (Ref. 5-10), used for ditches, swales and gulleys, including gulleys formed on top or sides of embankment when topsoil is provided, and for sheet flow conditions for slopes flatter than 10 percent.

The Stephenson's Method referred to herein is a method by D. Stephenson for stability of stones on the downstream face of a rockfill embankment subjected to overflow (Ref. 5-9). It has been shown to be satisfactory for sheet flow on slopes of 10% or greater and mean rock sizes of 1.5 inches or greater (Ref. 5-14 and 5-20). Interstitial flow is included in the Stephenson formula, and should not be subtracted from total flow. However, where insterstitial flow is expected, this flow is subtracted from total flow to obtain runoff flow rate for calculating appropriate time of concentration. Manning's formula is used to calculate velocity and the Corps of Engineers formula is used to calculate Manning's n (as outlined below for the Safety Factor Method) to calculate the time of concentration for determining total flow rate to be used with Stephenson's formula.Key details for applying the Safety Factors Method are as follows: R-3

- o The riprap is to have a minimum safety factor of 1.0 against flows due to the design storm.
- o The safety factor is determined as the available shear resistance divided by the peak local shear stress due to runoff from the design storm.
- 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 Peak local shear stress for triangular ditches with 5H:1V side slopes is computed from

$$\tau = 0.9\ \gamma_w yS,$$

where γ_w = unit weight of water,

y = depth of flow, and

S = slope.

- o Peak local shear stress for trapezoidal ditches is computed from:

$$\tau = C_{Rs}\ \gamma_w\ RS \text{ for sides, and:}$$

$$\tau = C_{Rb}\ \gamma_w\ RS \text{ for bottom,}$$

where C_{Rs} and C_{Rb} are functions of side slope and width/depth ratio, and R is the hydraulic radius.

- o Rill and gully formation is assumed on the top and sides of the pile when topsoil is provided over the riprap.

The design methods give 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). A minimum mean rock size of 1.5 inches has been adopted for protection against effects of rainsplash impact, wind concentrations, and irregularities in the finished surface. At locations where the embankment slope increases significantly (usually from 4 percent on the top to 20 percent on the sides of an embankment) the riprap on the side slope should be extended 10 feet up the top slope to protect against potential effects on non-uniform flow at the transition. At any slope transition, increases or decreases in insterstitial flow may occur. When interstitial flow is subtracted from total flow to determine adequate rock size at such transitions, the lesser interstitial flow for either slope should be subtracted from total flow to determine stable rock size on both slopes. In all cases, the maximum flow rate (either total flow rate or runoff flow rate, depending on the design method) should be used to determine stable rock size. The potential exists for the maximum flow rate for design to occur at locations other than at the end of a slope or ditch R-3

section of constant slope. This may occur due to 1) change in interstitial flow at slope transitions or 2) where calculated time of concentration at an upslope location results in a greater precipitation intensity sufficient to result in a greater flow rate than the flow rate due to a greater time of concentration and increased drainage area at a downslope location. R-3

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 in contributory drainage areas and erodibility of ground beyond outlets.
- 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 (ϕ) of riprap. Design values of ϕ can be obtained as a function of rock size (Ref. 5-2, Fig. 16). A value greater than 40 degrees, as suggested by the USCE (Ref. 5-11, p. 41) should not be used without site-specific data indicating a larger value.
- o Porosity (p) of the in-place riprap layer. The degree to which porosity can affect rock stability and throughflow determinations can vary depending on the situation. A value of 0.3 has been used, based on data for rockfill (Ref. 5-22) and relatively uniform-sized, clean gravel (Ref. 5-23).

C. Methodology

1. Precipitation Intensity

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

(1) Plot location of site on Figure 4-1.

(2) Use appropriate Hydrometeorological Report (HMR) by NOAA (see Figure 4-1 and Table 4-1) to determine precipitation intensity vs. time of concentration. For sites near boundaries shown in Figure 4-1, compare PMP estimates from HMR's applicable to areas near site, and use more conservative HMR results.

(3) Obtain design rainfall intensity compatible with time of concentration by interation as described below.

b. Time of Concentration - Time of concentration (Tc) estimates are based on hydraulic flow computations using Manning's formula for riprap-covered slopes and ditches. At some sites, significant overland flow from natural ground contributes to design flows. Time of concentration for such flow should be estimated by more than one method, since such estimates can vary significantly and be limited in their applicability. One estimate should be obtained by the Soil Conservation Service (SCS) Velocity Chart method (Figure 5-5). This method provides very good estimates for Tc for overland flow as long as flow paths exceed approximately 300-400 feet (Ref. 5-21). Other methods are available (e.g. several are given in Ref. 5-4 and Ref. 4-9), but their applicability to the design case under study should be considered in selecting a design Tc value.

2. Top and Sides of Pile With Slopes of 10 Percent or Steeper and No Topsoil (Stephenson's Method) R-3

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

b. Assume a trial mean rock size, D_{50}.

c. Compute q_t = q at critical (design) section from:

$$q_t = \frac{K^{3/2} C g^{1/2} [(1-p)(G_s-1)\cos\theta(\tan\phi-\tan\theta)]^{5/3}}{(\tan\theta)^{7/6}\,(p)^{1/6}}$$

Where $K = D_{50}$
C = 0.22 for gravel and pebbles, 0.27 for crushed granite

g = gravitational constant = 32.2 ft/sec^2 = 9.81 m/s^2

p = porosity

G_s = specific gravity

θ = embankment slope

and ϕ = angle of internal friction.

d. The intensity of rainfall for a strip 1-foot wide by length L_t, in inches, is:

$$I_{PMP} = \frac{43{,}560\ q_t}{L_t} \quad \text{(Ref. 5.15, page 15).}$$

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

$$q_v = pA\ v_v$$

where p = porosity,

A = cross-sectional area,

= $t \times 1$ for 1-foot wide strip,

t = $1.9D_{50}$ or minimum allowable thickness, whichever is larger,

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 $Wm^{0.5}$.

f. Compute $q_{tn} = q_t$ at end of each segment n, and $L_n = L$ for each segment n, where the total number of segements n_t is such that L_n is no greater than about 100 feet, as follows:

$$q_{tn} = \frac{n\ q_t}{n_t}$$

$$L_n = \frac{L_t}{n_t}$$

g. Compute $q_{rn} = q_{runoff}$ for each segment n as $q_{rn} = q_{tn} - q_v$.

h. Select trial value of depth of flow, y, at end of segment n.

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:

$$y = (q_{rn}\, n/1.486)^{0.6}/(\tan \theta)^{0.3}$$

(based on Ref. 5-15, pp. 15-16).

k. Repeat Steps h through j until resolution is achieved for y for each segment.

l. Compute $v_{rn} = q_{rn}/y$.

m. Compute $t_{rn} = t_{runoff}$ for segment n from

$$t_{rn} = \frac{2\, L_n}{V_{r(n-1)} + V_{rn}}$$

n. Compute T_r = sum of t_{rn}.

o. Repeat Steps b through n until resolution is achieved for I_{PMP} from T_r (using Tc versus i curve for site) and I_{PMP} from q_t and L_t (step d). Resolve I_{PMP} for each design section (e.g. ends of top slope and side slope).

3. <u>Top and Sides of Pile with Slopes Flatter than 10 Percent and No Topsoil (Safety Factor Method)</u>

a. The sheet flow approach is used, with calculations performed on a 1-foot-wide strip of slope length, L.

b. Assign a safety factor against rock movement, which is usually 1.0 (incipient motion under PMP condition). Alternatively higher values of SF may also be assigned, at designer's discretion. R-3

c. With the assigned value of SF calculate the rock stability number n_s on a plane sloping bed from the equation:

$$n_s = \cos\theta\left(\frac{1}{SF} - \frac{\tan\theta}{\tan\phi}\right)$$

(Equation 36 of Ref. 5.10 for flow on plane sloping bed.)

d. Assume a trial mean rock size, D_{50}.

e. Calculate shear stress τ_s that can be sustained on the riprap by solving the equation:

$$\eta_s = 21\tau_s/(G_s - 1)\gamma_w D_{50}$$

(Equation 17 of Ref. 5.10)

f. Solve for flow depth Y from the equation:

$$\tau_s = \gamma_w YS$$

Note that under sheet flow conditions across a very wide plane (flow depth Y << flow width), the hydraulic radius R = Y and the shear stress across the top of the riprap is uniformly distributed.

g. Calculate Manning's n and the flow rate per unit width on top of the riprap, q_r, from the following equations:

$$n = Y^{1/6}/[23.85 + 21.95 \log_{10}(Y/D_{50})]$$

$$q_r = (\frac{1.486}{n} Y^{2/3} S^{1/2})Y$$ R-3

h. The q_r so calculated is at the end of the sloped segment of length, L. Subdivide L into N subsegments each having a length of 100 feet or less.

$$\Delta L = L/N$$

and allocate q_r to each of these subsegments.

$$q_{ri} = i(q_r/N), \quad i = 1, 2, 3.....N$$

i. Solve Manning's equation to obtain flow depth Y_i at the end of each subsegment.

$$q_{ri} = \frac{1.486}{n_i}(Y_i)(Y_i)^{2/3}S^{1/2} \qquad i = 1, 2,.....N$$

where n_i = Manning's n at the end of ith subsegment.

$$= \frac{(Y_i)^{1/6}}{23.85 + 21.95 \log (\frac{Y_i}{D_{50}})} \quad i = 1, 2, \ldots N$$

j. Calculate the velocity at end of each subsegment, V_i, the incremental time of concentration, ΔT_i, and the total time of concentration, T, as follows:

$$V_i = \frac{q_{ri}}{Y_i}, \; i = 1, 2 \ldots N$$

$$\Delta T_1 = \frac{\Delta L}{V_1/2}$$

$$\Delta T_i = \frac{\Delta L}{(V_i + V_{i-1})/2}, \; i = 1, 2 \ldots N$$

and $T = \sum \Delta T_i$, $i = 1, 2 \ldots N$

k. The rainfall intensity I corresponding to this time of concentration T can be obtained from the intensity duration curve developed for the site. R-3

l. Compare this rainfall intensity I to the rainfall intensity I' computed from the rational formula.

$$I' = \frac{q_t \times 43{,}560}{C(L \times 1)}$$

where q_t = total flow per unit width
$= q_r + q_v$

where q_v = through-flow in the riprap voids per unit width

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

where p = porosity of riprap
y_{th} = thickness of the riprap layer.

q_v can be neglected for slopes of 10 percent or less since the through-flow velocity will be small.

m. If I' = I, the assumed D_{50} rock will be at the point of incipient motion under PMP condition, but stable under rainfalls of lesser magnitude.

If I' > I, the assumed D_{50} rock will be stable under PMP.

If I' < I, the assumed D_{50} rock will be unstable. In this case increase the D_{50} rock size and perform calculation steps e through m as many times as necessary to obtain a stable rock size D_{50}. R-3

n. Continue calculations for D_{50} for any additional sloped segments downslope from the one considered above. Use procedure C.3 above for slopes flatter than 10 percent; use procedure C.2 for slopes of 10 percent or steeper. In these subsequent calculations use the already established value of D_{50} to calculate Manning's n, velocity and time of concentration on the upslope segment.

4. <u>Top and Sides of Pile With Topsoil Provided</u>:

a. The gully development approach is used, with the Safety Factors Method.

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) X 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 the Rational Formula:

$$Q = C\ i\ A,$$

where $C = 1.0$,
i = PMP intensity

and A = drainage area.

f. Estimate time of concentration T_c. Determine i from PMP versus time curve for site and calculate Q.

g. Assume a trial mean rock size, D_{50}.

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

i. Compute flow in the riprap voids from:

$$Q_v = pA_v v_v$$

where p = porosity,

$A_v = Bt$, and

$t = 1.9D_{50}$ or minimum allowable thickness, whichever is larger, 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.

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

k. Select trial value of flow depth, y.

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

m. Compute Manning's coefficient, n, from:

$$n = R^{1/6}/[23.85 + 21.95 \log_{10}(R/D_{50})]$$ R-3

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

o. Repeat steps j through n until resolution is achieved for y and Q_{net}.

p. Compute peak local shear stress on riprap. The riprap is at the base of a trapezoidal channel. The peak local shear stress on the base, $\tau_b)_{max}$, is determined from:

$$\tau_b)_{max} = C_{Rb}\ \gamma_w RS,$$

where C_{Rb} = factor given by Figure 5-3 (Ref. 5-2, p. 13).

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

r. Compute $SF = \cos\theta\tan\phi/(n_s \tan\phi + \sin\theta)$ (Ref. 5.10, p. 643).

s. If $SF \neq 1.0$, repeat Steps f through q until resolution is achieved; i.e., $SF = 1.0$ for trial D_{50}.

t. Calculate T_c by averaging 1) time of concentration

assuming overland flow for entire slope and 2) time of concentration assuming flow velocity in gully for entire slope. Compare estimated T_c (step f) with calculated T_c.

u. Repeat steps f through t until resolution is achieved for PMP intensity, i, and T_c.

5. Swales and Ditches R-3

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

$$Q = C\ i\ A$$

where $C = 1.0$
i = intensity (PMP)
and A = drainage area.

b. Estimate time of concentration T_c. Determine i from PMP versus time (duration) curve for site and calculate Q.

c. Assume a trial mean rock size D_{50}. Determine thickness of riprap layer, t, for throughflow calculations by:

$t = 1.9D_{50}$ or minimum allowable thickness, whichever is larger.

d. Except for the cases described in Subsection 9d, compute throughflow from R-3

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

where p = porosity,
A_{th} = area of throughflow,

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

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

and S = slope.

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

f. Compute area of flow, A, wetted perimeter, P, and hydraulic radius, $R = A/P$ for a trial value of y.

g. Compute Manning's coefficient, n, from

$$n = R^{1/6}/[23.85+21.95\ \log_{10}(R/D_{50})]$$

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

i. Repeat steps e through h until resolution is achieved for y and Q_{net}.

j. Compute peak local shear stress as follows:

For triangular ditch with 5H:1V side slopes,

$\tau_{max} = 0.9\gamma\ yS$ (Ref. 5-2, p. 12, Fig. 10).

For trapezoidal ditch,

$\tau_{max.} = C_{Rs}\ \gamma_w\ RS$ (for sides)

$\tau_{max.} = C_{Rb}\ \gamma_w\ RS$ (for bottom)

where C_{Rs} and C_{Rb} are given in Ref. 5-2, (Fig. 11 and 12, respectively).

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

l. Compute SF from:

$$SF = \frac{\cos\theta\ \tan\phi}{0.5\ [1+\sin(\lambda + B)]\ n_s\tan\phi + \sin\theta\ \cos B},$$

where $B = \tan^{-1} \dfrac{\cos\lambda}{[2\ \sin\theta/(n_s\tan\phi)] + \sin\lambda}$

$\lambda = \sin^{-1}(\sin\alpha/\sin\theta)$, R-3

θ = embankment slope, and

$\alpha = \tan^{-1}S$.

m. If SF $\neq$ 1.0, repeat Steps b through l until resolution is achieved, where SF = 1 for trial D_{50}.

n. Calculate T_c and compare to estimated T_c, if necessary. Incremental times of concentration T_{cn} for flow in ditches are obtained from ditch flow velocity V_n as follows:

$T_{cn} = L_n/V_n$.

o. Repeat steps b through n until resolution is achieved for intensity i and T_c.

6. Gradation R-3

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

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

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

$$D_{100})_{min} = [(6 \times 2\pi G_s\gamma_w D^3_{50\,min}/6)/(\pi G_s\gamma_w)]^{1/3} = (2)^{1/3}\ D_{50})_{min}$$

$$= 1.26\ D_{50})_{min}$$

$$D_{100})_{max} = (5)^{1/3}\ D_{50})_{min} = 1.71\ D_{50})_{min}$$ R-3

$$D_{25})_{min} = D_{100})_{max}/(16)^{1/3} = 0.68\ D_{50})_{min}$$

d. Plot upper and lower bound gradation curves, adjust if necessary to utilize gradations already produced locally, and determine ranges for a minimum number of commonly-used sieve sizes that will ensure minimum and maximum size requirements. The following format should be used:

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

7. Thickness - The minimum thickness of a riprap layer, T_{min}, should be the greater thickness as determined by the following: R-3

a. $T_{min} \geq 1.9\ D_{50})_{min}$

b. $T_{min} \geq 1.5\ D_{50})_{max}$

c. $T_{min} \geq 12$ inches

These requirements are based on USCE recommendations (Ref. 5-11). The first requirement is derived from the USCE minimum as follows:

$D_{100})_{max} = (5)^{1/3}\ D_{50})_{min}$ (after Ref. 5-11)

$T_{min} \geq 1.1\ D_{100})_{max} = 1.9\ D_{50})_{min}$

8. Bedding Material

a. Bedding layers beneath riprap protect subgrade materials from erosion. Governing sizes should be determined as follows (after Ref. 5-18, p. 10 and 11): R-3

Subgrade Soil Group No.	Subgrade Soil Percent Fines*	Design Criteria***
1	40 - 100	$D_{15})_f \leq 0.7$ mm, except where base soil is dispersive or cohesionless, for which special study is required
2	0 - 15	$D_{15})_f/D_{85})_b \leq 4$ where base material is radon barrier, or ≤ 7.5 otherwise+
3	15 - 40	Interpolate linearly with percent fines between sizes required by criteria for groups 1 and 2

Notes:

* By weight, smaller than No. 200 sieve.

** D_{15} and D_{85} are sizes for which 15 percent and 85 percent of the particles are smaller, respectively "f" denotes filter and "b" denotes base.

\+ Page 10 of the reference recommends $D_{15})_f/D_{85})_b \leq 4$, but p. 14 states that this results in a factor of safety of about 2, whereas $D_{15})_f/D_{85})_b \leq 7.5$ still results in a factor of safety ≥ 1.

b. If conditions require provision for adequate flow capacity, in addition to prevention of base material migration, the following criterion applies:

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

c. Filter $D_{max} \leq 3$ inch (Ref. 5-12, p. 236).

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

e. 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.

f. 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	___ ___
No. 4	___ to ___
No. 16	___ to ___
No. 30	___ to ___
No. 50	___ to ___
No. 100	___ to ___
No. 200	___ to ___

g. The minimum thickness of a filter or bedding layer should be $1.1 \times D_{100)max}$ or 6 inches, whichever is larger.

9. Filters - Criteria are the same as for bedding, except $D_{15)f}/D_{85)b} \leq 4$ for percent fines 0 to 15. R-3

10. Special Problems

a. Ditch Bends: Increased shear at the outside of ditch bends should be accounted for by USCE guidelines for maximum shear in channel bends for rough channels (Ref. 5-11, Plate 34). Bends with longer radii will not require as large an increase in rock size, so longer radii should be used where practicable.

b. Slope Decrease Transitions: Turbulence and increased shear stress results when slopes change from steeper to flatter, due to non-uniform flow in the transition area (Ref. 5-24). Rock sizes should be increased in such

transition areas above rock sizes required for uniform flow. Increased rock sizes should be determined by several methods and the results compared before a final determination of design rock size is made. The following methods give rock sizes which are larger than those computed from methods for uniform flow:

1. USCE shear stress method [Ref. 5-11 with increased shear factor of 1.5 (Ref. 5-24)].

2. USCE method for design of riprap in turbulent areas beyond stilling basins. See Ref. 5-25, p. 44 and Ref. 5-26, Sheet 712-1 for guidelines for this method.

3. Stephenson's method for design of stones in flowing water (Ref. 5-9, p. 41).

4. Safety Factor Method with design shear stress greater than shear stress for uniform flow.

Considerable judgment must be used in applying these methods and selecting appropriate rock sizes, due to insufficiency of information on reliability for particular design situations.

c. Emergent Throughflow: When the throughflow capacity of a downstream layer is less than the upstream layer capacity, some throughflow will emerge above the section with the reduced capacity. This emergent throughflow should be included as runoff in flow rate determinations (for methods that subtract throughflow to obtain runoff).

d. Clogging of Voids in Riprap: Throughflow shall be assumed to be zero where ditches drain areas unprotected by riprap, where embankment slopes are adjacent to unprotected areas, or where ditches end in a key with no outlet for interstitial flow. R-3

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 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 one through five 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 one through four 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.

- 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 more than 50 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. This will ensure that acceptable rock continues to be supplied from the source.

- 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. If "good" quality rock is not available use NUREG/CR-4620, with modifications, to oversize available rock. R-3

- 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. Petrographic analysis provides an important basis in forming a judgment.

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:

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

Tests	Designation	Requirements
Specific Gravity and Absorption	ASTM C127	S.G. (SSD) not less than ___ Absorption not more than ___%
Soundness (sodium-sulfate method)	ASTM C88-76	Maximum weight loss ___%
Abrasion	ASTM C131 or ASTM C535	Not more than ___% loss of weight after ___ revolutions. Ratio of loss after ___ revolutions to loss after ___ revolutions shall not exceed ___ percent

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

Tests	Designation	Requirements
Specific Gravity (SSD):	ASTM C127 or ASTM C128	Greater than _____
Soundness (sodium-sulfate method):	ASTM C88	Less than __ percent loss of weight after 5 cycles
Abrasion	ASTM C131 or ASTM C535	Not more than ___ percent loss of weight after ___ revolutions. Ratio of weight loss after ___ revolutions to loss after ___ revolutions shall not exceed ___ percent.

5.3 REFERENCES

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5-3 Ecker, R. M., Effects of Rock Riprap Design Parameters on Flood Protection Costs for Uranium Tailings Impoundments, U.S. Nuclear Regulatory Commission, NUREG/CR-3751, July 1984.

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5-16 Walters, W. H., Rock Riprap Design Methods and Their Applicability to Long-Term Protection of Uranium Mill Tailings Impoundments, U.S. Nuclear Regulatory Commission, NUREG/CR-2684, August 1982.

5-17 U.S. Navy, Design Manual, Soil Mechanics, Foundations and Earth Structures, NAVFAC DM-7, March 1971.

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5-19 Lindsey, C.G., Long, L.W., and Begej, C.W. (1982), Long-Term Survivability of Riprap for Armoring Uranium Mill Tailings and Covers: A Literature Review, U.S. Nuclear Regulatory Commission, NUREG/CR-2642.

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5-23 Winterkorn, H.F., and Fan, H-Y, (Editors), (1975), Foundation Engineering Handbook, Van Nostrand Rheinhold Co., New York.

5-24 U.S. Army Corps of Engineers (USCE), (14 May 1971), "Engineering and Design: Additional Guidance for Riprap Channel Protection", Engineering Technical Letter (ETL) No. 1110-2-120, Washington, D.C.

5-25 U.S. Army Corps of Engineers (USCE), (31 March 1965), Hydraulic Design of Spillways, EM 1110-2-1603, Washington, D.C. (Reprint with Change 1, 21 April 1969).

5-26 U.S. Army Corps of Engineers (USCE), 1959 (Revised), Hydraulic Design Criteria, Vol. 2, Washington, D.C.

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	Bedding or Filter	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)

		Yes	No
8.	Review of Conceptual Design in RAP		
	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	Fair	Good
1. Bulk specific gravity	2.5	2.5 to 2.65	2.65
2. Absorption, %	1.0	0.5 to 1.0	0.5
3. Na_2SO_4 weight loss, %	10	5 to 10	5
4. Los Angeles abrasion loss, %[b]	10	5 to 10	5
5. Freeze-thaw weight loss, %[a]	5	0.5	0 to 0.5
6. Ultrasonic cavitation rating	0 to 5	5 to 7	7 to 10
7. Schmidt impact hammer	40	40 to 60	60
8. Scleroscope	30	30 to 50	50
9. Coefficient of restitution[c]	0.5	0.5 to 0.7	0.7
10. Tensile strength, psi	500	500 to 1,000	1,000
11. Compressive strength, psi	15,000	15,000 to 20,000	20,000
12. 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

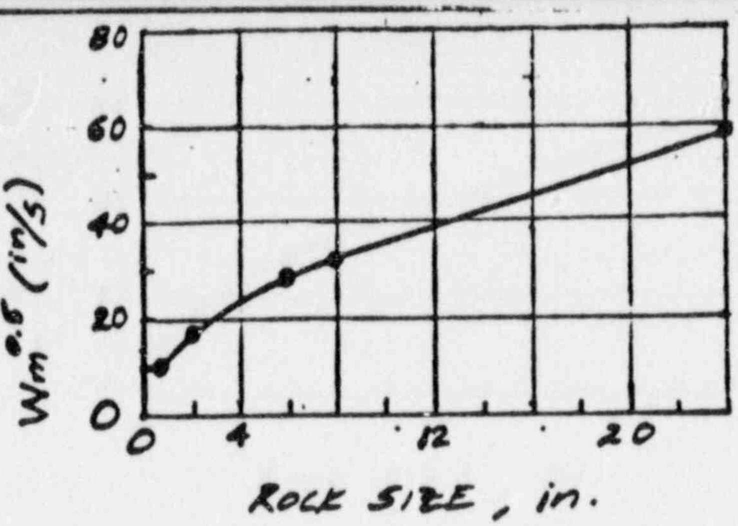


FIGURE 5-1 Mean hydraulic radius function versus rock size for the computation of turbulent flow velocity (After Ref. 5-5, Table 2, page 90)

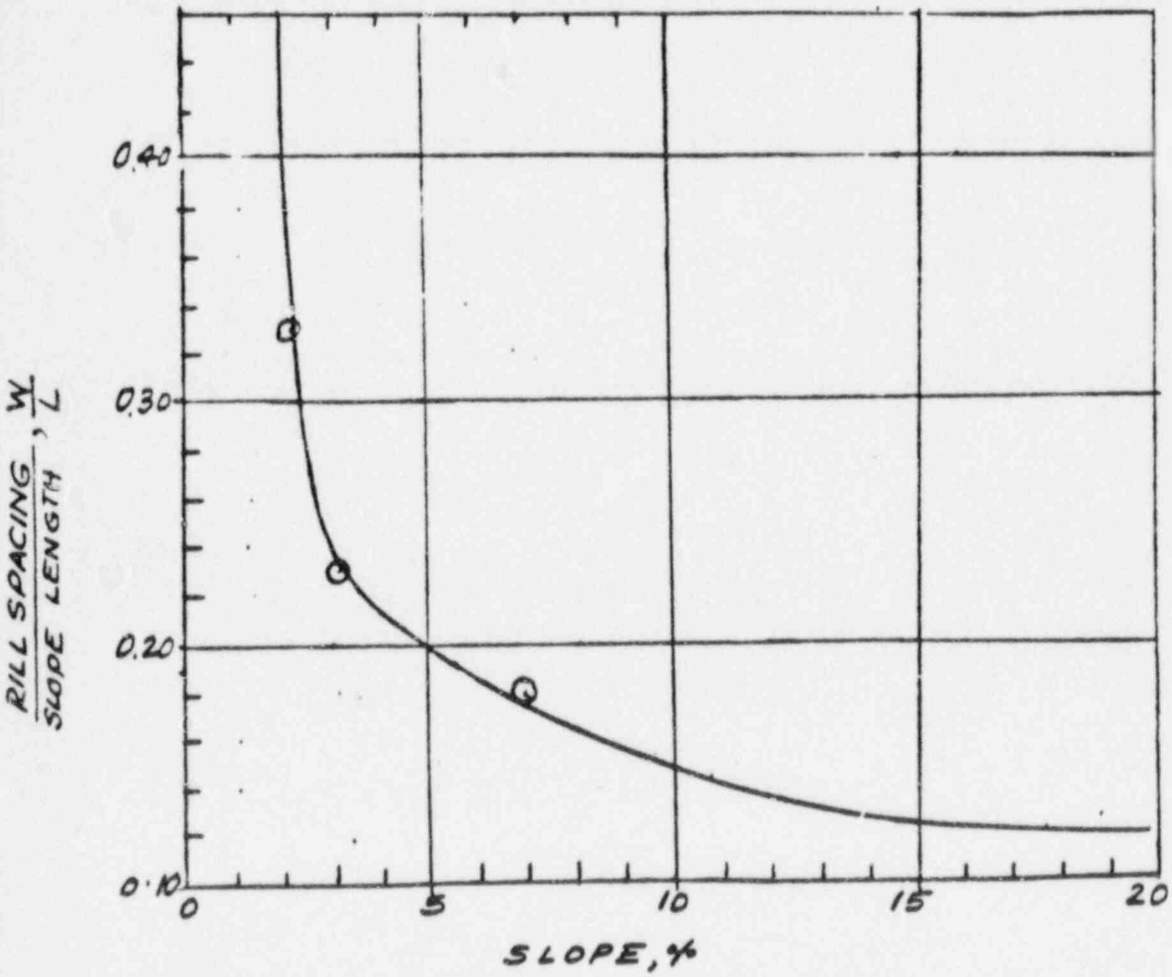


FIGURE 5-2 Rill spacing/slope length versus slope inclination (Based on data from Ref. 5-7, page 75)

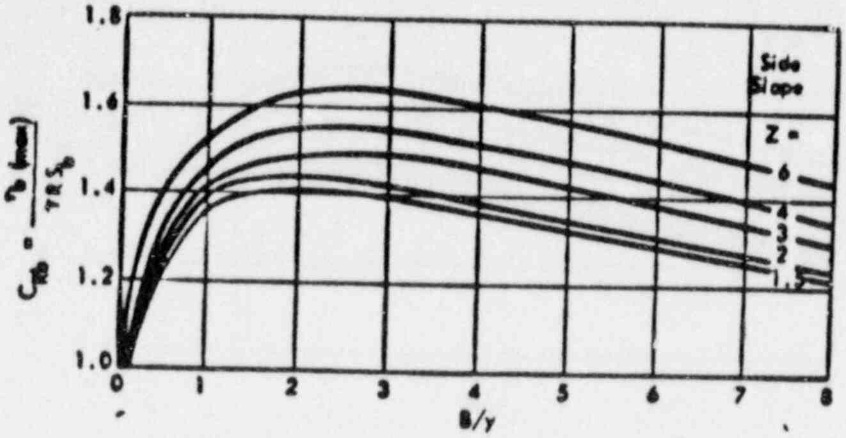


FIGURE 5-3 Maximum boundary shear stress on bottom of trapezoidal channels. (Ref. 5-2, page 13)

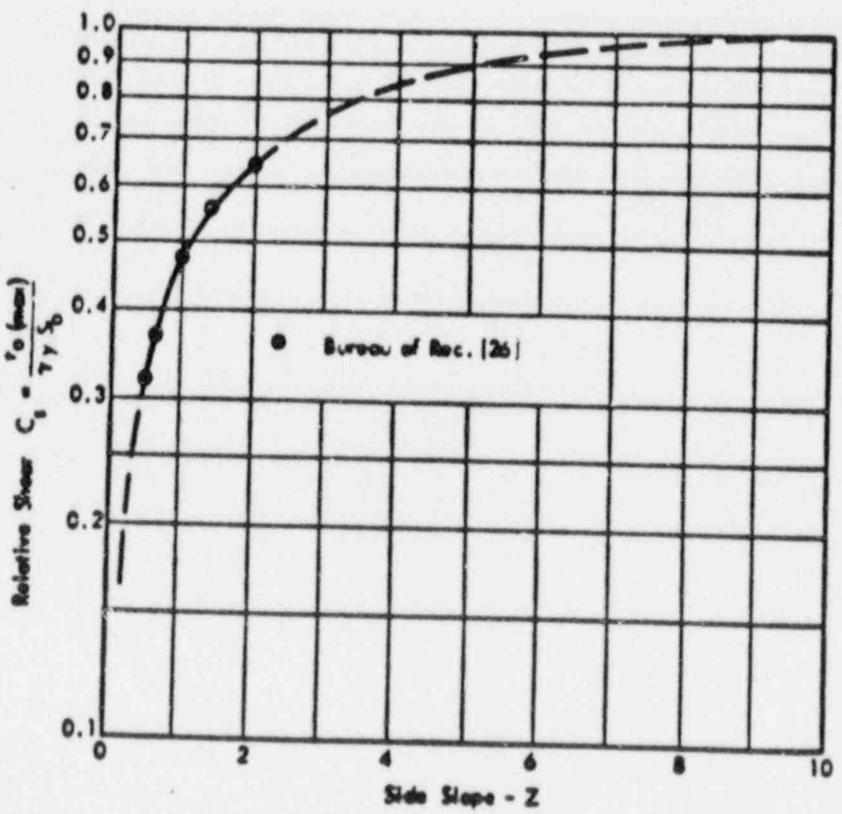


FIGURE 5-4 Maximum boundary shear stress on sides of triangular channels. (Ref. 5-2, page 12)

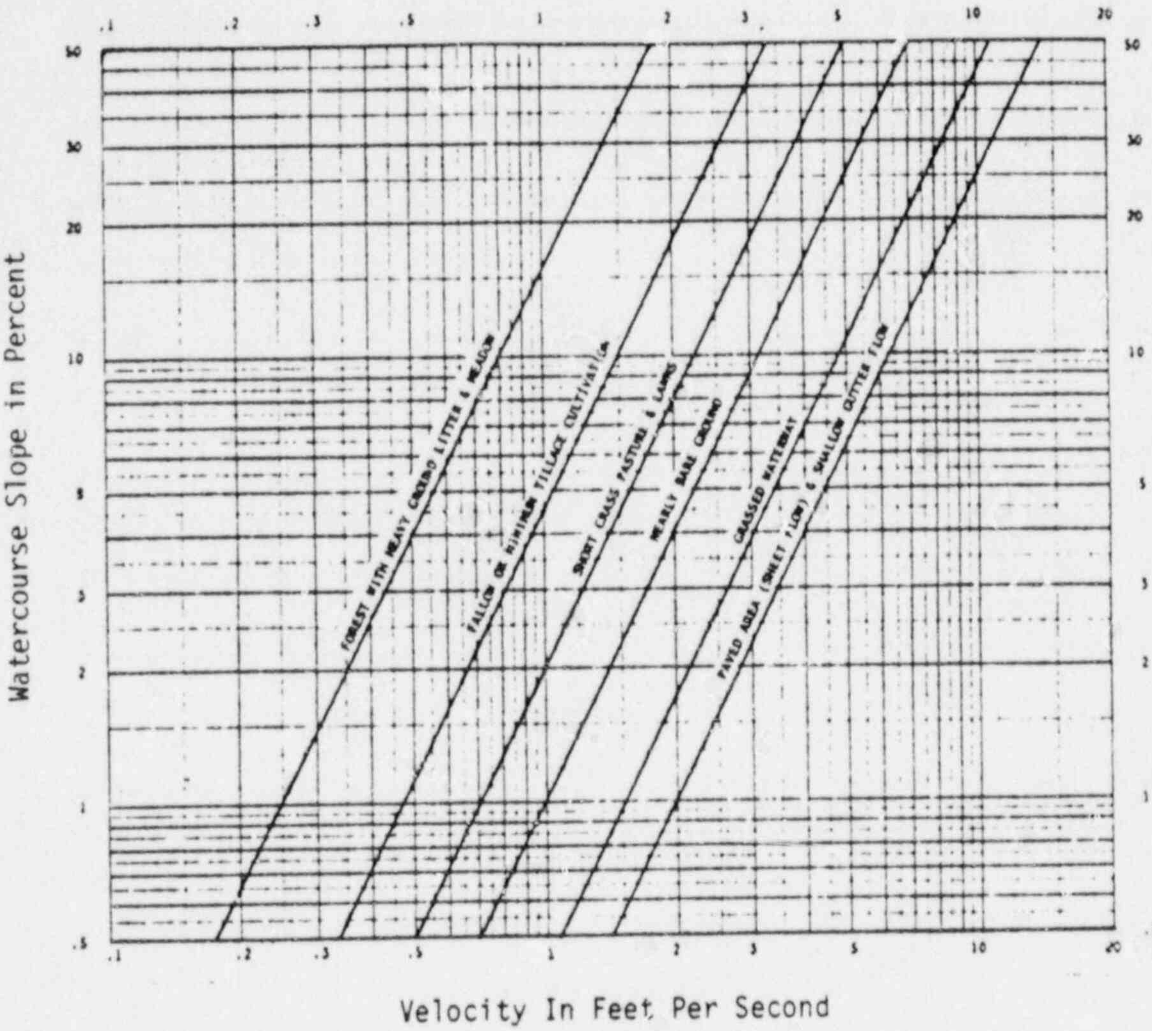


Figure 5-5 -- Average Velocities for Estimating Travel Time for Overland Flow. (Soil Conservation Service Method, Ref. 4-6)

MKE UMTRA DESIGN PROCEDURES

CHAPTER 6

RADON BARRIER

REVISION 1

SEPTEMBER, 1987

CONTENTS

TABLES

FIGURES

APPENDICES

CHAPTER 6
RADON BARRIER

6.1 INTRODUCTION

In most cases, the radon barrier thickness will have been determined during the conceptual design phase. The basis for, and adequacy of, this design should be reviewed, as outlined in Chapter 2 of this manual, and the reasons for proceeding with an additional design should be listed as required therein.

As discussed in Chapter 1, the primary purpose of the radon barrier is to limit the average radon exhalation to 20 pCi/m^2/sec. or less, throughout the 200-year to 1000-year design life of the remedial action (Ref. 6.1). This requirement is met by providing a layer or layers of compacted soil of specified thickness. Presently, the minimum acceptable thickness is determined by using the RAECOM model (Ref. 6.2).

Because in most cases the radon barrier provides significant resistance to infiltration of precipitation, this resistance is included in the analysis of infiltration and potential migration of contaminants as described in Chapter 9. If greater infiltration resistance is required, more than one alternative cover design may have to be developed. Modifying the radon barrier soil with the addition of bentonite or inclusion of liner materials (Ref. 6.4, pg. 90) should also be considered, if special site conditions dictate, to control infiltration as outlined in Chapter 9. However, the primary focus of the design will be to utilize natural soil, thus simplifying construction and affording a greater level of confidence in the long-term performance of the cover.

6.2 INPUT DATA AND PARAMETERS

A. Radium Content of Tailings

The radium content changes with time, as the radium present decays into radon and the thorium present decays into radium. The governing equation (Ref. 6.3) is:

$$Ra(t) = \frac{\lambda_2 (Th)_o (e^{-\lambda_1 t} - e^{-\lambda_2 t})}{\lambda_2 - \lambda_1} + (Ra)_o e^{-\lambda_2 t} \qquad \text{(EQUATION 6.1)}$$ R-1

where λ_1 = decay constant for Th = 8.63×10^{-6} year^{-1}
λ_2 = decay constant for Ra = 4.32×10^{-4} year^{-1}
$(Th)_o$ = initial content of Thorium-230
$(Ra)_o$ = initial content of Radium-226

e = Napierian base of logarithms

and t = time = design life.

To determine input values, $(Th)_0$ and $(Ra)_0$, it is necessary to obtain average values for the tailings. The concentrations may vary significantly from point to point. It may be necessary to sub-divide the tailings into layers and sub-areas. The concentration in the upper 10 feet is especially important, as this zone has a dominant influence on radon barrier requirements.

The number of sample points needed to develop meaningful average values of $(Th)_0$ and $(Ra)_0$ for a given site will depend on the variability of these parameters at the site. The values determined at each depth in each boring should be shown on soil profiles to aid in deciding whether sufficient data are available.

If sufficient Th and Ra readings are available, the merits of dividing the tailings into layers and sub-areas can be studied. If the average Ra content in a large sub-area or layer of significant thickness differs from that of another area or layer by a factor exceeding 1.5, separate average values should be computed for each sub-area and layer. A similar evaluation should be made with respect to Th content. For sites which exhibit wide spatial variation in radon content, some statistical frequency analyses may be necessary for developing the modeled section and the geometry of the tailing piles.

B. <u>Long-Term (Residual) Moisture Contents of Tailings and Radon Barrier</u>

Soil moisture plays a dominant role in the control of radon diffusion (Ref. 6.4, page 21). One procedure for estimating the average residual (design life) moisture content for a given soil layer (tailings layer or radon barrier), is to use the following equation (Ref. 6.2, pg. xiv): R-1

$$m_r = (0.124\ P_p^{0.5} - 0.001E_v - 0.04 + 0.156\ f_{cm}) \qquad \text{(EQUATION 6.2)}$$

where m_r = residual fraction of moisture saturation
P_p = annual precipitation (inches)
E_v = annual lake evaporation (inches)
f_{cm} = fraction of soil passing No. 200 sieve

Parameters P_p and E_v are fixed for a given site but, f_{cm} must be known for each layer of tailings and radon barrier.

A second procedure (Ref. 6.5, pg. 274) is to use the wilting point (15-bar moisture content) as the long-term moisture content. This parameter is determined by ASTM D2325 (coarse-grained soils) or ASTM 3152 (fine-grained soils). A third procedure is to use an empirical R-1

relationship such as that given in the NRC SRP, or that developed by the SCS (Ref. 6.6).

The 15-bar moisture content is determined and compared to the results of the other methods for reasonableness. Unless the 15-bar value is unreasonable, it is used. Other situations are treated on a case-by-case basis. R-1

C. <u>Radon Diffusion Coefficients of Tailings and Radon Barrier</u>

Estimated values of the radon diffusion coefficient (D) can be obtained as follows (Ref. 6.2, pg. xiii):

$$D = 0.07\, e^{-4(m_r - m_r p^2 + m_r^5)} \qquad \text{(EQUATION 6.3)}$$

where D = diffusion coefficient in cm^2/sec.

m_r = residual fraction of moisture saturation,

and p = porosity.

This equation can be used for preliminary design but measured values should be used for final design. Measurements should be accomplished at the minimum density required by the specifications and at water contents above and below the expected long-term value so that D versus m_r curves can be constructed for the cover and the tailings. In this way design diffusion coefficients can be determined from the D versus m_r curves for the selected design residual moisture contents (m_r) for the cover and the tailings. If more than one borrow source is considered, or an additive is to be mixed with the radon barrier soil, a D versus m_r curve will be required for each additional material.

D. <u>Radon Emanation Coefficient for Tailings</u>

The emanating power (E) of a soil containing radium is the fraction of the radon generated that is free to diffuse from the soil. E varies from <u>0.02 to 0.42</u> depending upon moisture content and other tailings characteristics (Ref. 6.2, pg. 5-2). E should be determined by laboratory testing of representative tailings samples (Ref. 6-1, p. 13). E values are not needed for layers which do not produce radon.

E. <u>Summary of Parameters for Tailings and Radon Barrier</u>

The parameters required for the various sub-areas and layers, and the site and soil characteristics needed to develop these parameters, should be summarized in tables similar to Table 6-1 for each sub-area. The sources of all the basic and computed data also should be clearly identified.

6.3 METHODOLOGY

A. Calculation of Maximum Radium Content

As discussed in Section 6.2.A, the total radium concentration in the tailings at any time (t) will be given by Equation 6.1. The radium concentration may reach a maximum during or at the end of the design life. Lawrence Livermore Laboratory has a computer code for Equation 6.1. The code prints out radium concentrations at equal time periods within each logarithmic interval, along with a plot of radium concentration vs. time. (An example calculation is shown in Appendix 6-A.) This code was used for design calculations for Burrell site. A similar computer code is being developed and introduced into the in-house (MKE) system.

The average values of $(Th)_0$ and $(Ra)_0$ are entered into the computer code, along with λ_1 and λ_2 (see Sec. 6.2 for definitions), and the print-out covering 1000 years is obtained. The maximum value of Ra-226 concentration is selected from the output and used for radon barrier thickness calculations.

B. Calculation of Long-Term Degree of Saturation Values

Annual precipitation, P, annual lake evaporation, E_v, and fraction of soil passing the No. 200 sieve, f_{cm}, are used with Equation 6.2 to calculate the long-term degree of saturation for each layer of tailings and the radon barrier (hand calculation).

C. Determination of Diffusion Coefficients

The long-term degree of saturation (m_r) and the expected average porosity (p) for each material are used with Equation 6.3 to calculate the corresponding diffusion coefficient (D).

The predicted long-term degree of saturation of cover (m_r) and average porosity, p, for each material are used with the corresponding respective best fit curve established from the experimentally-determined plots of D vs. m_r for that material.

D. Determination of Radon Barrier Thickness

As stated in the introduction, the minimum acceptable thickness for a given tailings sub-area is determined using the RAECOM model (Ref. 6.4). The flow chart, showing the major components of the model, is presented in Figure 6-1. The required input parameters are listed in Table 6-2, and the modeled soil profile is shown in Figure 6-2. The Radon Attenuation Handbook (Ref. 6.2) should be reviewed before using this model.

The model code automatically computes the minimum barrier thickness corresponding to the allowable flux of 20 pCi/m^2/sec. An example input and output record from the RAECOM code is included as Appendix

6-8. The computed minimum thickness should be increased by at least 0.2 feet to allow for construction accuracy, and to the next whole inch for construction control simplicity. To accommodate degradation and various other uncertainties associated with the cover design, which is in the process of evolution, the need for multiplying the computed cover thickness by a factor (> 1) to arrive at the design thickness is being debated and discussed. If this change is approved, an addendum will be issued.

6.4 DESIGN SUMMARY

The Site Design Engineer shall prepare a cover thickness design summary for different sub-areas, using a format similar to that shown in Table 6-3 and Figure 6-2. The minimum computed thickness and the design thickness should be clearly indicated. It also must be clearly indicated that the design thicknesses shown in Table 6-3 do not include the thickness of the cover protection materials. The prospective borrow source(s) considered for cover design also should be identified. The modeled section developed should be based on either a single layer or multilayer cover system, as appropriate. An example of a multilayer system is shown in Figure 6-2; a single-layer system is shown in Figure 6-3.

6.5 REFERENCES

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

6.2 Rogers, V. C., Nielson, K. K. and Kalkwarf, D. R., Radon Attenuation Handbook for Uranium Mill Tailings Cover Design, U.S. Nuclear Regulatory Commission, NUREG/CR-3533, 1984.

6.3 Lapp, R. E., and Andrews, H. L., Nuclear Radation Physics, Second Edition, 1954.

6.4 U.S. Department of Energy, Technical Summary of the UMTRA Project Technology Development Program, (1980-1984), UMTRA-DOE/AL 200125.0000, January, 1985.

6.5 Gee, G. W., Nielson, K. K. and Rogers, V. C., "Predicting Long-Term Moisture in Earthen Covers", Proceedings, Sixth Symposium on Management of Uranium Mill Tailings, Low-Level Waste, and Hazardous Waste, Fort Collins, Colorado, pages 267-276, 1984.

6.6 U.S. Soil Conservation Service, Soil Water Retention Data Use System (SWRDAT), 1982. R-1

TABLE 6-1. PARAMETERS FOR RADON BARRIER DESIGN

Sub-Area No.	Material Layer & No.	Layer Thickness cm	Layer Thickness ft.	Avg. Conc. (pCi/gm) Ra-226	Avg. Conc. (pCi/gm) Th-230	Fines Content (-No.200 Frac.), f_{cm}	Moisture Fraction, m_r g/cm^3	Dry Density, γ_d (pcf) g/cm^3	Sp. Gr. G	Porosity P	Source Term, $pCi/cm^3/sec.$	Diffusion Coeff., D $cm^2/sec.$	Moisture Content, M %	Std Proctor Compaction %
1	Tailings-1													
	Tailings-2													
	Tailings-3													
	Tailings-4													
	Tailings-5													
	Tailings-6													
	Tailings-7													
	Radon Barrier													

Average Precipitation, P_p = ________ inches/yr.

Lake Evaporation, E_v = ________ inches/yr.

Emanating Power, E_p = 0.2 for all layers

Thorium decay constant, λ_1 = ________

Radium decay constant, λ_2 = ________

Background Radon Concentration = ________ pCi/l

TABLE 6-2. INPUT DATA FOR RAECOM

(USER INFORMATION FOR RAECOM [1])

HEADING FOR THE RAECOM RUN		MODELED SECTION	LAYER NOS.
NUMBER OF LAYERS, TAILINGS AND COVER			1
*INITIAL FLUX INTO LAYER 1 (pCi/m^2-s)	0.		2
*AMBIENT AIR RADON CONCENTRATION (pCi/l)	Background		3
LAYER TO BE OPTIMIZED	(Layer No. or 0) [2]		
*FLUX LIMIT AT SURFACE (pCi/m^2-s)	20. or 0. [2]		
*ACCURACY NEEDED (.1 TO .0001)	0.001		

Layer No. N	Lowest Layer 1	2	3	4	5	6	7
*THICKNESS OF LAYER N (cm)							
*DIFFUSION COEFF. OF LAYER N (cm^2/s)							
*POROSITY OF LAYER N (FRACTION)							
*PROJECTED RA-226 CONC. OF LAYER N (pCi/g)							
*EMANATING FRACTION OF LAYER N (FRACTION)							
*BULK DENSITY OF LAYER N (g/cm^3)							
*MOISTURE CONTENT OF LAYER N (% DRY WT)							

Notes:
[1] On MKE Harris system RAECOM will read and input file (see User's Manual). File is created by input of all data above in order presented. (All data for each layer is input first.)
[2] First value will cause minimum required thickness of optimized layer to be calculated. Second value will cause flux to be calculated for all layers.
* Indicates a decimal point is required.

TABLE 6-3

RADON BARRIER THICKNESS DESIGN SUMMARY

		1	2	3	4
1.	Sub Area Number	1	2	3	4
2.	Projected Ra-226 (pCi/gm)				
3.	Diffusion Coefficient (cm^2/sec.)				
4.	Minimum Computed Thickness of Barrier without protective cover				
5.	Design Thickness of Barrier without protective cover				
6.	Single or Multilayer Cover System				
7.	Prospective Cover Material Source				

8. Comments:

1. See Modeled Section in Figure 6-2.

2.

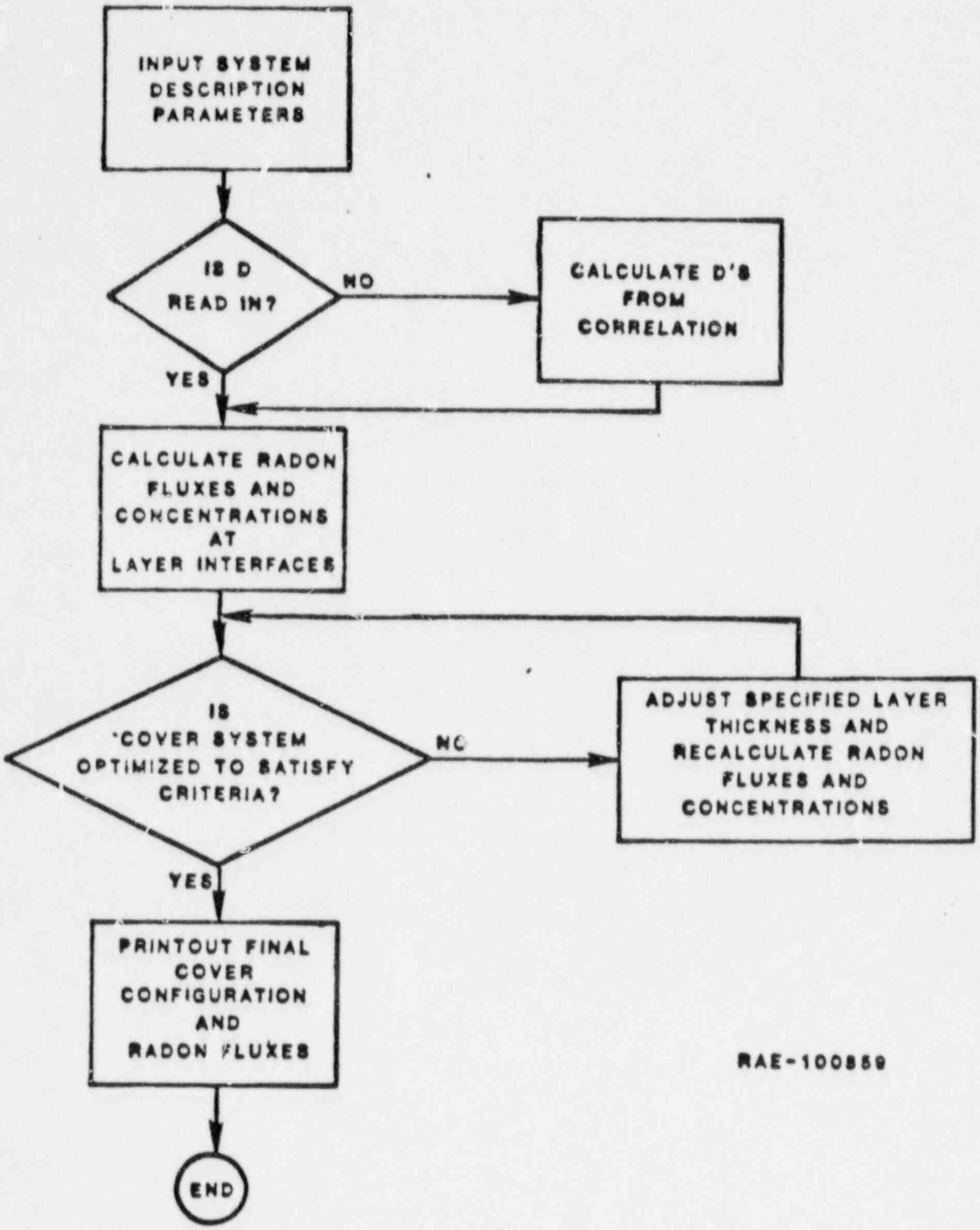


FIGURE 6-1. FLOW CHART SHOWING COMPONENTS OF RAECOM MODEL. (REF. 6-2)

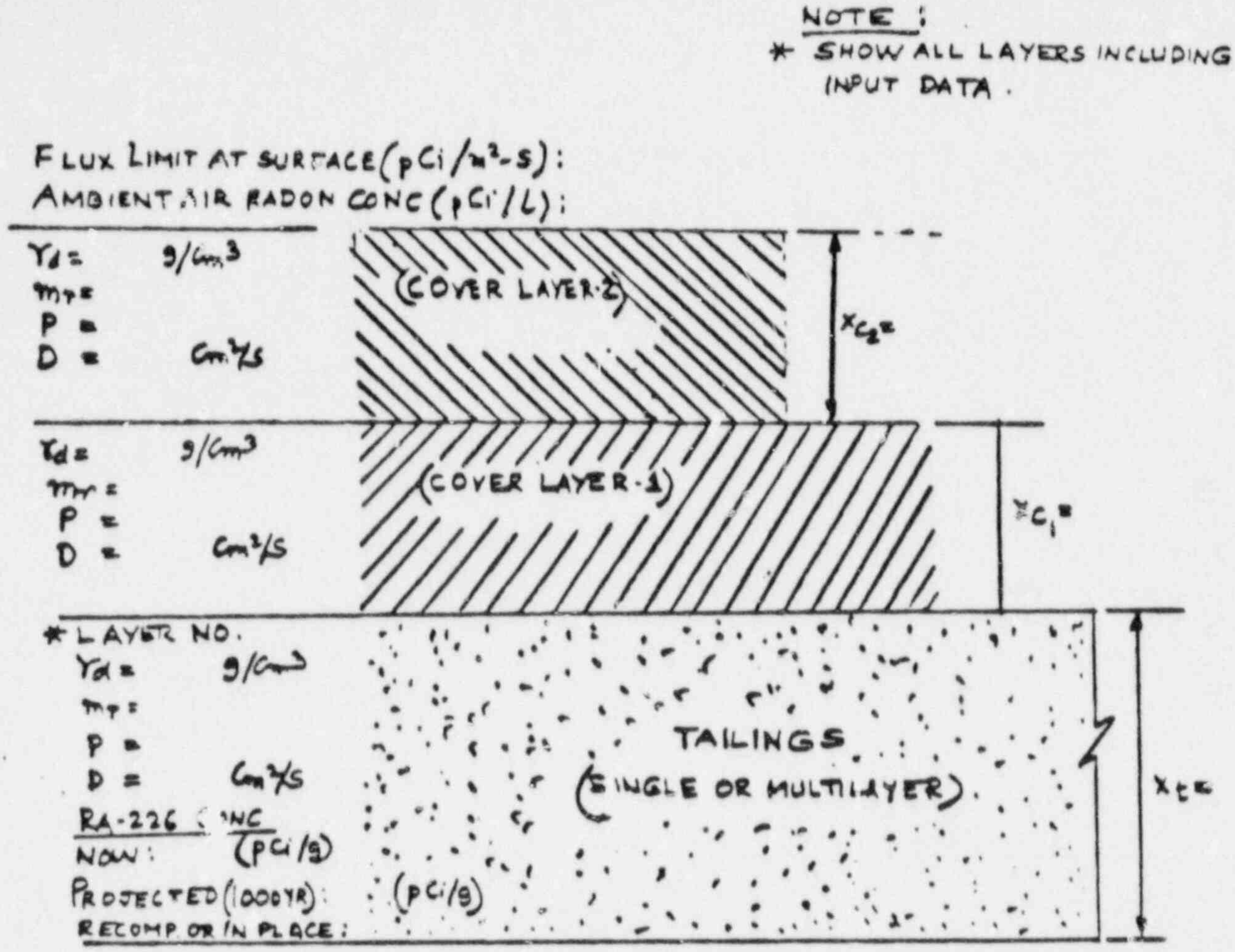


FIGURE 6-2. MODELED SECTION WITH MULTI-LAYER COVER

NOTE

* SHOW ALL LAYERS INCLUDING INPUT DATA

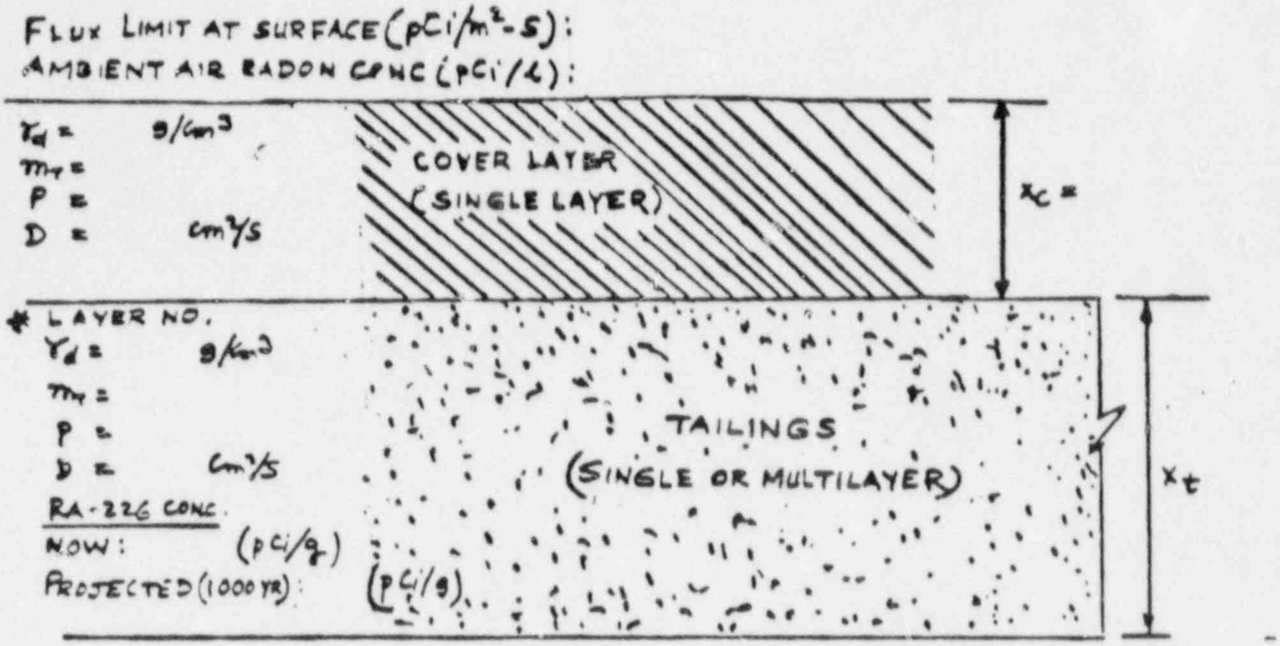


FIGURE 6-3. MODELED SECTION WITH SINGLE LAYER COVER

APPENDIX 6A

EXAMPLE CALCULATION OF DECAY OF THORIUM INTO RADIUM
FOR DETERMINATION OF MAXIMUM RADIUM CONTENT

APPENDIX 6A

EXAMPLE CALCULATION OF DECAY OF THORIUM INTO RADIUM FOR DETERMINATION OF MAXIMUM RADIUM CONTENT

The enclosed calculation was produced by substituting the values given for time, t, in the first column, into Equation 6.1, with the values for λ_1 and λ_2 shown on page 6-2, and the following values for the other parameters:

$(Th)_o$ = 3,080 pCi/g,
and $(Ra)_o$ = 640 pCi/g.

The $(Ra)_o$ value is given for Canonsburg in Ref. 6A-1, page F.2-11, and the $(Th)_o$ value was computed from the average Th/Ra ratio given for Canonsburg Area A in Ref. 6A-2, page 464.

The results of the calculations, as shown in Columns 2 and 3, are the values of Th-230 and Ra-226 corresponding to each value of t. Ra-226 increases continually throughout the 1,000-year design period, as shown in the plot at the end of the calculations, so that the maximum value is reached at the end of the design period. From the tabulation of values, $(Ra)_{max} = (Ra)_{1000}$ = 1500 pCi/g. Thus the ratio $(Ra)_{1000}/(Ra)_o$ = 1500/640 = 2.34. This is essentially the same value as that reported for this situation in Ref. 6A-2, page 465.

References for Appendix 6A

6A-1 Baker, K. R., D. E. Mohr, and R. L. Hillman, "Radiological Aspects - Canonsburg, Pennsylvania UMTRA Site," Sixth Symposium on Management of Uranium Mill Tailings, Low-Level Waste, and Hazardous Waste, Fort Collins, Colorado, February 1984, pp 463-472.

6A-2 U.S. Department of Energy, Remedial Action Plan for Stabilization of the Inactive Uranium Mill Tailings Site at Canonsburg, Pennsylvania, UMTRA-DOE/AL-140, October 1983.

RADIOACTIVE DECAY CHAIN
FROM (T1) = 1.00E+03 TO (T2) = 1.00E+07

INITIAL INPUT

			HALF-LIFE		CONC
ISOTOPE 1	=	TH230	8.03E+04	Y	3.08E+03
ISOTOPE 2	=	RA226	1.60E+03	Y	6.40E+02
ISOTOPE 3	=	BK101	1.00E+10	S	0.
ISOTOPE 4	=	BK102	1.10E+10	S	0.
ISOTOPE 5	=	BK103	1.20E+10	S	0.
ISOTOPE 6	=	BK104	1.30E+10	S	0.
ISOTOPE 7	=	BK105	1.40E+10	S	0.

TIME IS IN HOURS, ACTIVITY IS IN PCI/G

TIME	TH 230	RA 226
1.00E+03	3.08E+03	6.40E+02
1.20E+03	3.08E+03	6.40E+02
1.40E+03	3.08E+03	6.40E+02
1.60E+03	3.08E+03	6.40E+02
1.80E+03	3.08E+03	6.40E+02
2.00E+03	3.08E+03	6.40E+02
2.20E+03	3.08E+03	6.40E+02
2.40E+03	3.08E+03	6.40E+02
2.60E+03	3.08E+03	6.40E+02
2.80E+03	3.08E+03	6.40E+02
3.00E+03	3.08E+03	6.40E+02
3.20E+03	3.08E+03	6.40E+02
3.40E+03	3.08E+03	6.40E+02
3.60E+03	3.08E+03	6.40E+02
3.80E+03	3.08E+03	6.40E+02
4.00E+03	3.08E+03	6.40E+02
4.20E+03	3.08E+03	6.41E+02
4.40E+03	3.08E+03	6.41E+02
4.60E+03	3.08E+03	6.41E+02
4.80E+03	3.08E+03	6.41E+02
5.00E+03	3.08E+03	6.41E+02
5.20E+03	3.08E+03	6.41E+02
5.40E+03	3.08E+03	6.41E+02
5.60E+03	3.08E+03	6.41E+02
5.80E+03	3.08E+03	6.41E+02
[illegible]	3.08E+03	6.41E+02
6.20E+03	3.08E+03	6.41E+02
6.40E+03	3.08E+03	6.41E+02
6.60E+03	3.08E+03	6.41E+02
6.80E+03	3.08E+03	6.41E+02
7.00E+03	3.08E+03	6.41E+02
7.20E+03	3.08E+03	6.41E+02
7.40E+03	3.08E+03	6.41E+02
7.60E+03	3.08E+03	6.41E+02
7.80E+03	3.08E+03	6.41E+02
8.00E+03	3.08E+03	6.41E+02
8.20E+03	3.08E+03	6.41E+02
8.40E+03	3.08E+03	6.41E+02
8.60E+03	3.08E+03	6.41E+02
8.80E+03	3.08E+03	6.41E+02
9.00E+03	3.08E+03	6.41E+02
9.20E+03	3.08E+03	6.41E+02
9.40E+03	3.08E+03	6.41E+02
9.60E+03	3.08E+03	6.41E+02

TIME	TH 230	RA 226
0.00E+00	1.08E+03	6.41E+02
1.00E+04	1.08E+03	6.41E+02
1.20E+04	1.08E+03	6.41E+02
1.40E+04	1.08E+03	6.42E+02
1.60E+04	1.08E+03	6.42E+02
1.80E+04	1.08E+03	6.42E+02
2.00E+04	1.08E+03	6.42E+02
2.20E+04	1.08E+03	6.43E+02
2.40E+04	1.08E+03	6.43E+02
2.60E+04	1.08E+03	6.43E+02
2.80E+04	1.08E+03	6.43E+02
3.00E+04	1.08E+03	6.44E+02
3.20E+04	1.08E+03	6.44E+02
3.40E+04	1.08E+03	6.44E+02
3.60E+04	1.08E+03	6.44E+02
3.80E+04	1.08E+03	6.45E+02
4.00E+04	1.08E+03	6.45E+02
4.20E+04	1.08E+03	6.45E+02
4.40E+04	1.08E+03	6.45E+02
4.60E+04	1.08E+03	6.46E+02
4.80E+04	1.08E+03	6.46E+02
5.00E+04	1.08E+03	6.46E+02
5.20E+04	1.08E+03	6.46E+02
5.40E+04	1.08E+03	6.47E+02
5.60E+04	1.08E+03	6.47E+02
5.80E+04	1.08E+03	6.47E+02
6.00E+04	1.08E+03	6.47E+02
6.20E+04	1.08E+03	6.47E+02
6.40E+04	1.08E+03	6.48E+02
6.60E+04	1.08E+03	6.48E+02
6.80E+04	1.08E+03	6.48E+02
7.00E+04	1.08E+03	6.48E+02
7.20E+04	1.08E+03	6.49E+02
7.40E+04	1.08E+03	6.49E+02
7.60E+04	1.08E+03	6.49E+02
7.80E+04	1.08E+03	6.49E+02
8.00E+04	1.08E+03	6.50E+02
8.20E+04	1.08E+03	6.50E+02
8.40E+04	1.08E+03	6.50E+02
8.60E+04	1.08E+03	6.50E+02
8.80E+04	1.08E+03	6.51E+02
9.00E+04	1.08E+03	6.51E+02
9.20E+04	1.08E+03	6.51E+02
9.40E+04	1.08E+03	6.51E+02
9.60E+04	1.08E+03	6.52E+02
9.80E+04	1.08E+03	6.52E+02
1.00E+05	1.08E+03	6.53E+02
1.20E+05	1.08E+03	6.54E+02
1.40E+05	1.08E+03	6.57E+02
1.60E+05	1.08E+03	6.59E+02
1.80E+05	1.08E+03	6.62E+02
2.00E+05	1.08E+03	6.64E+02

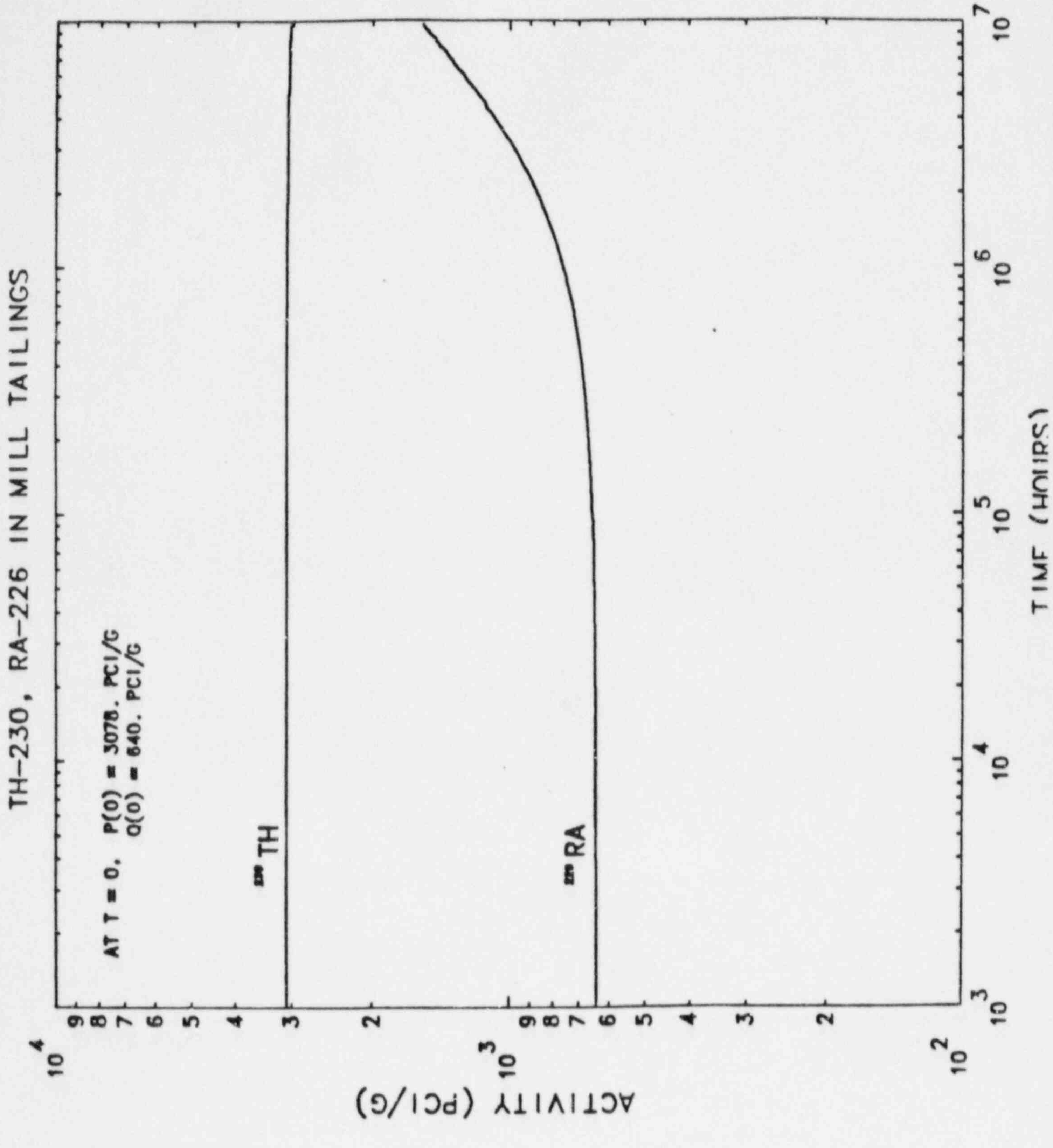
TIME	TH 230	RA 226
2.20E+05	3.08E+03	6.66E+02
2.40E+05	3.08E+03	6.69E+02
2.60E+05	3.08E+03	6.71E+02
2.80E+05	3.08E+03	6.74E+02
3.00E+05	3.08E+03	6.76E+02
3.20E+05	3.08E+03	6.78E+02
3.40E+05	3.08E+03	6.81E+02
3.60E+05	3.08E+03	6.83E+02
3.80E+05	3.08E+03	6.85E+02
4.00E+05	3.08E+03	6.88E+02
4.20E+05	3.08E+03	6.90E+02
4.40E+05	3.08E+03	6.93E+02
4.60E+05	3.08E+03	6.95E+02
4.80E+05	3.08E+03	6.97E+02
5.00E+05	3.08E+03	7.00E+02
5.20E+05	3.08E+03	7.02E+02
5.40E+05	3.08E+03	7.04E+02
5.60E+05	3.08E+03	7.07E+02
5.80E+05	3.08E+03	7.09E+02
6.00E+05	3.08E+03	7.11E+02
6.20E+05	3.08E+03	7.14E+02
6.40E+05	3.08E+03	7.16E+02
6.60E+05	3.08E+03	7.18E+02
6.80E+05	3.08E+03	7.21E+02
7.00E+05	3.08E+03	7.23E+02
7.20E+05	3.08E+03	7.25E+02
7.40E+05	3.08E+03	7.28E+02
7.60E+05	3.08E+03	7.30E+02
7.80E+05	3.08E+03	7.32E+02
8.00E+05	3.08E+03	7.35E+02
8.20E+05	3.08E+03	7.37E+02
8.40E+05	3.08E+03	7.39E+02
8.60E+05	3.08E+03	7.42E+02
8.80E+05	3.08E+03	7.44E+02
9.00E+05	3.08E+03	7.46E+02
9.20E+05	3.08E+03	7.48E+02
9.40E+05	3.08E+03	7.51E+02
9.60E+05	3.08E+03	7.53E+02
9.80E+05	3.08E+03	7.55E+02
1.00E+06	3.08E+03	7.58E+02
1.20E+06	3.08E+03	7.80E+02
1.40E+06	3.08E+03	8.03E+02
1.60E+06	3.08E+03	8.25E+02
1.80E+06	3.07E+03	8.48E+02
2.00E+06	3.07E+03	8.69E+02
2.20E+06	3.07E+03	8.91E+02
2.40E+06	3.07E+03	9.13E+02
2.60E+06	3.07E+03	9.34E+02
2.80E+06	3.07E+03	9.55E+02
3.00E+06	3.07E+03	9.76E+02
3.20E+06	3.07E+03	9.96E+02
3.40E+06	3.07E+03	1.02E+03

TIME	TH 230	RA 226
3.60E+06	3.07E+03	1.04E+03
3.80E+06	3.07E+03	1.06E+03
4.00E+06	3.07E+03	1.08E+03
4.20E-06	3.07E+03	1.10E+03
4.40E+06	3.07E+03	1.12E+03
4.60E+06	3.07E+03	1.13E+03
4.80E+06	3.07E+03	1.15E+03
5.00E+06	3.06E+03	1.17E+03
5.20E+06	3.06E+03	1.19E+03
5.40E+06	3.06E+03	1.21E+03
5.60E+06	3.06E+03	1.23E+03
5.80E+06	3.06E+03	1.25E+03
6.00E+06	3.06E+03	1.26E+03
6.20E+06	3.06E+03	1.28E+03
6.40E+06	3.06E+03	1.30E+03
6.60E+06	3.06E+03	1.32E+03
6.80E+06	3.06E+03	1.33E+03
7.00E+06	3.06E+03	1.35E+03
7.20E+06	3.06E+03	1.37E+03
7.40E+06	3.06E+03	1.38E+03
7.60E+06	3.06E+03	1.40E+03
7.80E+06	3.06E+03	1.42E+03
8.00E+06	3.06E+03	1.43E+03
8.20E+06	3.06E+03	1.45E+03
8.40E+06	3.05E+03	1.46E+03
8.60E+06	3.05E+03	1.48E+03
8.80E+06	3.05E+03	1.50E+03
9.00E+06	3.05E+03	1.51E+03
9.20E+06	3.05E+03	1.53E+03
9.40E+06	3.05E+03	1.54E+03
9.60E+06	3.05E+03	1.56E+03

4.210

1,000 Yrs. = 8.8 X 10^6 Hrs

$$\frac{Ra_{1000\ Yrs}}{Ra_0} = \frac{1{,}500\ pCi/gm}{640\ pCi/gm} = 2.34$$



TH-230, RA-226 IN MILL TAILINGS
AT T = 0. P(0) = 3078. PCI/G
Q(0) = 640. PCI/G
TH
RA
ACTIVITY (PCI/G)
TIME (HOURS)

APPENDIX 6B

EXAMPLE CALCULATION OF RADON BARRIER THICKNESS

APPENDIX 6B

EXAMPLE CALCULATION OF RADON BARRIER THICKNESS

This calculation uses the computer code RAECOM as explained in Ref. 6-2. The use of RAECOM at MKE starts with an intermediate program INRAEC, which takes the input information, computes the source term,

$$Q = D \times p,$$

and enters the data into RAECOM. RAECOM can be used to compute 1) the minimum thickness of radon barrier (or of one layer of a multi-layered cover) required to limit radon exhalation to 20 $pCi/cm^2/sec$; or 2) the radon exhalation corresponding to a specific cover system or for uncovered tailings.

The example is arranged as follows:

Sheet 1: Input to INRAEC to compute Q.

Sheet 2: Input to RAECOM.

Sheet 3: Output from RAECOM. Repeats input for quality assurance check. Presents required thickness for Layer 3, the layer being adjusted.

The following is an example of input for INRAEC.

HEADING FOR THE RAECO2 RUN--> TEST RUN

NUMBER OF LAYERS, TAILINGS & COVER--> 2

INITIAL FLUX INTO LAYER 1 (pCi/m^2-s)--> 0. *

AMBIENT AIR RADON CONCENTRATION (pCi/1)--> 0.18 *

LAYER TO BE OPTIMIZED--> 2 0--> optimization
2--> optimization of second layer

FLUX LIMIT AT SURFACE (pCi/m^2-s)--> 20. *

ACCURACY NEEDED (.1 TO .0001)--> .001 *

THICKNESS OF LAYER 1 (cm)--> 548.6 *

DIFFUSION COEFF. OF LAYER 1 (cm^2/s)--> .000326 *

POROSITY OF LAYER 1 (FRACTION)--> .466 *

RA-226 CONC. OF LAYER 1 (pCi/g)--> 2200. *

EMANATING FRACTION OF LAYER 1 (FRACTION)--> .2 *

BULK DENSITY OF LAYER 1 (g/cm^3)--> 1.44 *

MOISTURE CONTENT OF LAYER 1 (% DRY WT)--> 29.1 *

THICKNESS OF LAYER 2 (cm)--> 91.4 initial thickness *

DIFFUSION COEFF. OF LAYER 2 (cm^2/s)--> .00193 *

POROSITY OF LAYER 2 (FRACTION)--> .326 *

RA-226 CONC. OF LAYER 2 (pCi/g)--> 0. *

EMANATING FRACTION OF LAYER 2 (FRACTION)--> 0. *

BULK DENSITY OF LAYER 2 (g/cm3)--> 1.82 *

MOISTURE CONTENT OF LAYER 2 (% DRY WT)--> 15. *

* Note that a decimal point is required for these entries.

SUMMARY OF RAECOM INPUT

)ING:	TEST RUN 8-7-84
LAYERS:	3
INITIAL FLUX:	0.000
AMBIENT RN:	0.000
OPTIMIZED LAYER:	3
SURFACE FLUX LIMIT:	20.000
PRECISION:	.0010

LAYER NO.	X_o THICKNESS CM	D_e, D_c DIFFUSION CM2-SEC	P_e, P_c POROSITY FRACTION	RA-226 PCI/G	EMANATING FRACTION	BULK DENSITY G/CM3	Q_+, Q_c SOURCE TERM PCI/CM3-SEC	MOISTURE % DRY WT
1	500.0	.01300	.440	400.4	.20	1.50	.0005733	11.7000
2	50.0	.00784	.300	0.0	0.00	1.60	0.0000000	6.3000
3	100.0	.02200	.370	0.0	0.00	1.60	0.0000000	5.4000

TEST RUN 8-7-84

********** INPUT PARAMETERS **********

NUMBER OF LAYERS :	3	
RADON FLUX INTO LAYER 1 :	0.00	pCi/m2/sec
SURFACE RADON CONCENTRATION :	0.00	pCi/liter

LAYER 3 ADJUSTED TO MEET Jcrit : 20.0 +/- .100E-02 pCi/m2/sec

RF(1) = (R(1)*P(1)/AB)*TANH(AB*DI(1))

RF(1)	0.00000000
R(1)	.573299958E-03
P(1)	.440000000
AB	.127097782E-01
DI(1)	500.000000

BARE SOURCE FLUX (Jo) FROM LAYER 1 : 0.000 pCi/m2/sec

LAYER	THICKNESS (cm)	DIFF COEFF (cm2/sec)	POROSITY	SOURCE (pCi/cm3/sec)	MOISTURE (dry wt. percent)
1	500.	1.3000E-02	.4400	5.7330E-04	11.70
2	50.	7.0000E-03	.3000	0.0000E+00	6.30
3	100.	2.2000E-02	.3700	0.0000E+00	5.40

***** RESULTS OF RADON DIFFUSION CALCULATION *****

LAYER	THICKNESS (cm)	EXIT FLUX (pCi/m2/sec)	EXIT CONC. (pCi/liter)	MIC
1	500.	7.6977E+01	1.6710E+05	.7025
2	50.	4.5307E+01	4.4224E+04	.7063
3	149.	2.0011E+01	0.0000E+00	.8163

MKE UMTRA DESIGN PROCEDURES

CHAPTER 8

SITE SEISMICITY EVALUATION AND DEVELOPMENT

OF SEISMIC DESIGN PARAMETERS

REVISION 2

SEPTEMBER 1987

CONTENTS

TABLES

FIGURES

CHAPTER 8

SITE SEISMICITY EVALUATION AND DEVELOPMENT OF SEISMIC DESIGN PARAMETERS

8.1 INTRODUCTION

This chapter presents the procedures for developing site-specific earthquake design parameters. This document is to be used in conjunction with the DOE's Technical Approach Document (Ref 8-1) and the NRC's Standard Review Plan (Ref. 8-2). If site seismicity and earthquake design parameters have been developed by others, the guidelines in this chapter will be used for reviewing such studies for adequacy and completeness.

All UMTRA sites, irrespective of their location on the seismic zone map of the USA (Figure 8-1), have to be investigated in sufficient detail for seismo-tectonic characterization of each site. This will involve review of literature and historic earthquake records (published and unpublished), office and field studies (aerial and over ground) for the identification of active faults and lineaments and the development of the design earthquake and the related parameters. The tasks performed and the end results achieved should provide reasonable assurance that compliance with the EPA standard 40 CFR Part 192 (Ref. 8-3) will not be jeopardized due to the direct or indirect effects of earthquakes.

8.2 DESIGN LIFE OF STRUCTURES

The EPA Standard (Ref. 8-3) specifies that the design life of structures at all UMTRA sites is to be 1,000 years to the extent reasonably achievable, and in any case at least 200 years. It was resolved in the Inter-Agency (DOE, NRC, TAC, RAC) Workgroup 2 meetings that a design life of 1000-years should be selected for evaluating the safety of UMTRA facilities against seismic hazards (Ref. 8-4).

8.3 SITE SEISMICITY AND DESIGN EARTHQUAKE

A. General

The EPA Standard (Ref. 8-3) does not provide specific criteria to be used in determining the seismic safety of structures at the UMTRA sites. Following the provisions of Section 192.2 of the above Standard (requirement of Section 108 of the Act), the DOE and the NRC adopted design earthquake criteria consistent with the intent of the standards.

DOE criteria (Ref. 8-1) indicate the design earthquake will be the Maximum Credible Earthquake (MCE). Extreme earthquake motion such as the MCE is generally recommended for critical structures, such as functionally needed nuclear power plants and dams, since their sudden failure provides little time for remedial action and therefore can cause serious environmental damage and loss of life. For comparatively less critical structures, a Safety Evaluation Earthquake (SEE)/Design Earthquake could be an event smaller than the MCE (Ref. 8-5). These alternatives were discussed in the Inter-Agency meetings, and because of the unusually long life (1000-years) of the UMTRA Project and the uncertainties associated with the predictability of long recurrence interval earthquakes, it was decided to select the MCE event as defined in 10 CFR 40, App. A (Ref. 8-6) as the design earthquake. This section defines important terminology such as the MCE, capable fault, floating earthquake, and describes the level of effort and various earthquake parameters required for evaluating the site seismicity and characterizing the design earthquake.

B. Definition of Important Terminology

1. Maximum Credible Earthquake

There is no single, well-accepted definition of the MCE. In determining the MCE, little regard is given to its probability of

recurrence, except that some sources state that "the recurrence interval shall be great enough to be of concern based on the currently known tectonic framework" (Ref. 8-7). Other sources consider the MCE to be the largest rationally conceivable event. Such an event may have a recurrence interval of several hundred years in some regions, but many tens of thousands of years in others (Refs. 8-8 and 8-9).

Even though somewhat different terminology may continue to be used in various UMTRA documents to describe the design earthquake motion, they should be considered analogous to the term MCE defined below:

"The term maximum credible earthquake means that earthquake which would cause the maximum vibratory ground motion based upon an evaluation of earthquake potential considering the regional and local geology and seismology and specific characteristics of local subsurface material." (Refs. 8-2 and 8-6)

2. Definition of Capable Fault

The capable fault, as defined in 10 CFR 100, App. A, III (g) (Ref. 8-10) and quoted below, has been accepted for the UMTRA project after discussion in the Workgroup 2 meetings:

o Capable fault. A capable fault is a fault which has exhibited one or more of the following characteristics:

   - Movement at or near the ground surface at least once within the past 35,000 years, or movement of a recurring nature within the past 500,000 years.

   - Macroseismicity, instrumentally determined, with records of sufficient precision to demonstrate a direct relationship with the fault.

- A structural relationship to a capable fault such that movement on one fault could be reasonably expected to cause movement on the other.

This definition is essentially the one adopted by the NRC for the siting of nuclear power plants and tailings impoundments. Since 40 CFR 192 (Ref. 8-3) does not include any active fault definition, this definition was also proposed by the NRC for the UMTRA project.

3. Floating Earthquake

For those UMTRA sites where the historic earthquakes are not associated with a known tectonic structure and no surface trace for a significant active fault is visible or well established, an earthquake called the 'floating earthquake' will be designated as the design earthquake. The 'floating earthquake' which will be selected as the design earthquake for a particular site is the largest earthquake which has either occurred or is potentially possible in the tectonic province in which the site is located, or in an adjoining province, and whose effect is most severe on the tailings pile.

The site to source distance of "floating earthquakes" should be 15km for 'floating earthquakes' in the tectonic province containing the site or in proximate tectonic provinces less than 15km from the site. For floating earthquakes in tectonic provinces more than 15km from the site, the actual distance of closest approach of these provinces to the site should be used as the site-to-source distance (Ref. 8-2).

C. Level of Effort

The level of effort for site seismicity evaluation and tectonic characterization of the UMTRA sites should include but not be restricted to the following:

- Collection and review of historic earthquake data available from NOAA earthquake data file and other sources covering a radius of at least 200 km from the site. Significant historic earthquake motions could be summarized in the format shown in Table 8-1.

- Search and review of literature relating to Quarternary geology, regional seismotectonic setting, mapped faults and lineaments, and aerial photos including stereo pairs for each site.

- Review of seismicity study reports for completed facilities in the area, such as dams, nuclear power plants, etc.

- Aerial and ground reconnaisance including trenching and carbon-14 dating to identify potentially active faults and their characteristics using state-of-the-art techniques.

- For sites where the historic earthquake data base is poor or data are unavailable, and where the surface expression of active faults is not well established, reliance must be placed on the work of previous investigators; at such locations supplementary work should be done to complete the data base.

- The size of the area to be investigated and the type of data to be collected and methodology adopted should be in broad compliance with that outlined in the T.A.D. and the SRP (Refs. 8-1 and 8-2).

D. Design Earthquake Characteristics

1. Richter Magnitude of Tentative MCE Events

There is some theoretical and empirical evidence that the highest intensities of ground shaking are correlated more closely with the fault offset, than with the fault rupture length. However, often reliable measurements of fault offsets are not easily available, so that reliance must be placed on fault rupture length in estimating the potential maximum earthquake magnitude which could be generated by a given capable fault.

Some of the empirical relationships which could be used for estimating the magnitude of a maximum earthquake which could be trigged by a capable fault are:

a. Normal-Slip Faults: $M_s = 0.809 + 1.341 \log_{10} (L)$ (Slemmons et al, 1982, Ref. 8-11).

b. Normal-Oblique Faults: $M_s = 0.875 + 1.348 \log_{10} (L)$ (Slemmons et al, 1982, Ref. 8-11), and

c. $\text{Log} (L) = 1.915 + 0.389M$ (Mark, 1977, Refs. 8-12 and 3-13),

Where: Ms = surface wave earthquake magnitude,
L = rupture length in meters.
M = Richter earthquake magnitude,
= M_L for earthquakes smaller than 6.75, and
= M_s for earthquakes greater than or equal to 6.75, and
M_L = local earthquake magnitude.

If the rupture length is not established from field measurements, it should be assumed as one-half of the well identified active fault segment. Thus, from the capable fault investigation data,

such as total fault length, the magnitudes of the tentative MCE events are established at this stage and presented in the format shown in Table 8-2. For sites where current earthquake activity is not related to any existing known active fault or faults, the MCE event for the seismo-tectonic province in which the site is located and that of the adjoining provinces will be established from historic earthquake data and current knowledge about the seismo-tectonics of the provinces. For sites where the magnitude of the historic earthquake events are not recorded, the Richter magnitude value should be computed using the relationship given by Gutenberg and Richter (Ref. 8-14) or another current well-accepted relationship.

2. <u>Peak Horizontal Acceleration of Potential MCE Events</u>

As decided in the Inter-Agency Workgroup 2 meetings, [Ref. 8-15], $(A_{max})_{84th\ percentile}$ will be computed using the following constrained relationship (after Campbell, 1981, Ref. 8-16):

$$PGA = 0.0185e^{1.28M}[R + 0.147e^{0.732M}]^{-1.75}$$

$$(A_{max})_{84th\ percentile} = [PGA] \times 1.47$$

Where PGA = Median peak ground acceleration in (g).
R = Shortest source to site distance in (km).
$M = M_S$ for magnitudes equal to or greater than 6.0.
$M = M_L$ for magnitudes less than 6.0.
1.47 = Multiplier for converting the median acceleration value to 84th percentile value (provided by Campbell).

For the purpose of comparative study, A_{max} values in rock will also be computed using the acceleration attenuation relationship of Seed and Idriss, presented in Figure 8-2 (Ref. 8-17), or similar relationships published in current literature.

The horizontal rock acceleration at the site, with a 90% probability of not being exceeded in a 1000-year exposure period should be determined by extrapolation by plotting the horizontal acceleration values at 10-years, 50-years and 250-years after Algermissen, et al (Ref. 8-13) in the format shown in Figure 8-3. This should be considered as the lower bound acceleration value, and if it is greater than the value obtained using Campbell's equation, the adequacy of the design earthquake investigation should be checked and verified.

3. Determination of Peak Ground Acceleration

For all practical purposes it is assumed that A_{max} values obtained using Campbell's equation are valid for rock without significant error. The A_{max} values for rock and the site conditions will be the basis for determining the response ground acceleration, using the computer program "SHAKE" (Ref. 8-18) or Figure 8-4 (Ref. 8-17). For most of the UMTRA sites, the simpler method , Figure 8-4, may be adequate.

4. Predominant Period of Earthquake Motion

The predominant period for given maximum acceleration in rock can be read from Figure 8-5 (Ref. 8-19).

5. Significant Duration of Earthquake

Significant duration can be computed by using various empirical equations, such as those suggested below:

a) $t_o = 11.5(M) - 53.0$

[after Ambraseys & Sarma (1967) Ref. 8-20]

where t_o = duration in seconds during which the acceleration will be equal to or greater than 0.03g.

or b) For rock sites: $D = 10^{(0.43M - 1.83)}$
[Dobry, et al (1978); Ref. 8-21]
where D = Duration in seconds

Alternatively, the significant duration can be determined by using Tables 8-3 and 8-4 or Figures 8-6 and 8-7. The choice of the equations, figures, and tables will depend partly on site conditions and partly on the individual designer's judgment. The significant duration should preferably be computed by different methods; from the computed range, the modal or the mean value should be selected as the significant duration of the earthquake.

6. Designation of the MCE/DE

Having completed steps (D.1) through (D.5) and presented all the data in Table 8-2, it will be easy to identify by inspection the earthquake motion whose effect will be most severe on the tailings impoundment; this event will be designated as the MCE and the design earthquake. Generally this will be the earthquake event with the largest peak acceleration with possible exceptions at some sites. An earthquake with a somewhat smaller A_{max} but longer significant duration may work out to be the MCE event in some cases; thus the selection of the MCE/DE event will depend not only on the earthquake characteristics but also on the method of analysis. It also should be ensured that the effect of the MCE/DE event at the site should not be less severe than any historic earthquake event recorded in the vicinity of the site.

8.4 SEISMIC DESIGN AND RELATED PARAMETERS

A. Design Efforts - Normal Risk Sites

From our current knowledge, excepting 2 or 3 UMTRA sites, most of the sites could be classified under this category. The factors which will be considered are some or all of the following: Low height of embankment (50'); unsaturated or very dense cohesionless foundation

material; absence of cohesionless foundation material; absence of groundwater; and low level of shaking.

For these sites, the seismic design effort will be confined to the following:

- o Pseudo-static slope stability analysis with appropriate horizontal seismic coefficients for horizontal forces (k_H) for short term and long term condition.

- o Simplified liquefaction analysis for critical locations.

The pseudo-static coefficient (k_H) will be equal to two-thirds of the corrected maximum acceleration computed for the site due to the MCE/DE for long term condition (Ref. 8-15). For the end of construction condition, either half of the long-term seismic coefficient or the horizontal acceleration (% of gravity) in rock with 90% probability of not being exceeded in a 50-year exposure time as provided by Algermissen et al (1982) (Refs. 8-1 and 8-13), whichever is larger should be used. However, the minimum values of K_H should not be less than 0.10 and 0.05 for the long-term and end of construction conditions, respectively.

B. <u>Design Effort for High Risk Sites</u>

A greater level of seismic design effort will be necessary for the high risk sites. In addition to the parameters and analyses described under Section 8.4.A, a dynamic slope stability analysis may be required for some of these sites. Site conditions, which may dictate dynamic slope stability analyses, are: (1) loose cohesionless foundation materials associated with a high ground water table, and (2) uncompacted uranium tailings embankment with height, H 50 feet. A design accelerogram must be developed for use in the dynamic slope stability analysis using state-of-the-art techniques (Refs. 8-22 and 8-23). For some sites, a simplified dynamic analysis may prove to be adequate (Refs. 8-24, 8-25

and 8-26). Necessary parameters for dynamic analyses will be developed from field and laboratory tests and literature search.

8.5 CONCLUSIONS AND RECOMMENDATIONS

The program outlined herein is considered to be the minimum program necessary to ensure seismic safety at UMTRA Project sites.

If the traces of active faults in the vicinity of a particular UMTRA site are poorly defined or unknown, the evaluation of site seismicity for that site will be based primarily on historical earthquake data and the geology and seismicity of the region. If adequate geologic and seismic information are not available, the seismic parameters must be inferred from studies of similar areas with more complete information. The results of the seismicity study will be summarized and presented in the format shown in Table 8-5. As a minimum, the study must provide all the information required to complete Table 8-5.

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TABLE 8-1

SUMMARY OF SIGNIFICANT HISTORIC EARTHQUAKE EVENTS
($M \geq 5.0$ or $I(MM) \geq$ VII) WITHIN 200-KM RADIUS
FROM UMTRA - ________ SITE

DATE	LOCATION		DEPTH KM	DISTANCE KM	RICHTER MAGNITUDE, $M^{a,b}$,	EPICENTRAL INTENSITY, I_0(MM)
	LATITUDE	LONGITUDE				

a For sites where recorded magnitude value is not available, the computed Richter magnitude of the earthquake events should be computed from the relationship by Gutenberg and Richter, 1956 (Ref. 8-14).

$$M = 1 + 2/3\ I_0$$

b Abbreviations to be Used

PAS = E.Q. Recorded at CIT Pasadena
BRK = E.Q. Recorded at U.C. Berkeley
M_b = Body-wave magnitude
M_L = Local Magnitude
M_S = Surface Wave Magnitude

TABLE 8-2
CHARACTERISTICS OF TENTATIVE MCE EVENTS/DESIGN EARTHQUAKE AND RELATED PARAMETERS
UMTRA - SITE, ______

Name of Fault or Fault System a	Tentative MCE Event							Peak Acce
	Fault Length b		Rupture Length km a (miles)	Richter Magnitude		Tentative MCE Magnitude, M* c	Epicentral Distance km/(miles)	Seed and Idriss (1982)
	Miles	Km		Slemmons et al(1982)	Mark (1977)			(Avg. value)
1	2	3	4	5	6	7	8	9

NOTE: From above identify the event whose effect is most critical at the site as the MCE and the Design Earthquake.
*M = Ms for 6.75 or greater; M = M_L for smaller earthquakes

a & b These basic data are obtained from the Seismicity Study Report, by others, or the RAP.

c Select higher magnitude value from Columns 5 and 6.

d The values in this column are same as in column 11. These values are chosen as the 84th percentile A_{max} values as obtained from the attenuation equation after Campbell (1981).

e These corrected peak acceleration values are obtained by correcting the values in Column 12 for local site conditions.

f The data provided in columns 15 and 16 may be required for selecting appropriate design accelerogram for the response analysis/dynamic analysis.

g The A_{max} value in column 17 should be used for the simplified liquefaction analysis as the crest acceleration, if the height (H) of tailings pile is less than or equal to 25 feet. For H 25 ft., determine the response peak acceleration at the top of the tailings, using computer program SHAKE with the Amax in column 17 as the input ground motion acceleration. Use also the A_{max} value in column 17 for the simplified dynamic analysis, if required.
If the minimum factor of safety is marginally below 1 in pseudo-static analysis, the adequate seismic stability of the slope should be established from a simplified permanent deformation analysis.

h These values (Fraction of Acceleration due to gravity) are 2/3 and 1/3 of the A_{max} values shown in Column 14, and should be used for the pseudo-static analyses for long-term and end of construction conditions, respectively. Use only values shown against the selected MCE/DE event.

eration, Amax Campbell (1981)		Peak Acceleration (Uncorrected) d	Site Condition (Soil or Rock).	Corrected Peak Acceleration e	Predominant Period of E.Q., Tp f	Significant E.Q. Duration, (Sces) f	Ground Accel. For Liq. Analysis g	Horizontal Seismic Coeff. for Pseudo-Static Analysis
Median (50)% Tile	(84)% Tile							
10	11	12	13	14	15	16	17	18 h

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TABLE 8-3

SIGNIFICANT DURATION OF EARTHQUAKES

Richter Magnitude, M	Significant Duration, Secs. After Ambraseys & Sarma (1967)[a]	Significant Duration, Secs. After Housner (1970)[b]	Significant Duration, Secs. Dobry, et al (1978)[c]	Comments
5.0	5	2	2	
5.5	10	6	4	
6.0	16	12	6	
6.5	22	18	9	
7.0	28	24	15	
7.5	33	30	25	
8.0	39	34	41	
8.5	45	37	67	

a) Ref. 8-20

b) Ref. 8-27

c) Ref. 8-21

TABLE 8-4

BRACKETED DURATION (sec) (Acc. 0.05 g; freq. 2 Hz) OF EARTHQUAKES
[After Bolt - Ref. 8-28]

Site to Source Distance (km)	Earthquake Magnitude, M						
	5.5	6.0	6.5	7.0	7.5	8.0	8.5
10	8	12	19	26	31	34	35
25	4	9	15	24	28	30	32
50	2	3	10	22	26	28	29
75	1	1	5	10	14	16	17
100	0	0	1	4	5	6	7
125	0	0	1	2	2	3	3
150	0	0	0	1	2	2	3
175	0	0	0	0	1	2	2
200	0	0	0	0	0	1	2

TABLE 8-5

SITE SEISMICITY AND EARTHQUAKE DESIGN PARAMETERS SUMMARY

1. Name of Site and State:

2. Approximate Coordinates:

3. Is this a Processing Site or a Disposal Site?

4. Seismic Risk Zone:

5. Largest Historic Earthquake within
(1) 15 km and
(2) 100 km distance.
(3) the seismo-tectonic province
(4) the adjoining seismo-tectonic province

	Date	Distance from Site (km)	Magnitude (Richter)	Intensity (MM)	A_{max}
(1)					
(2)					
(3)					
(4)					

6. Particulars of Active Fault, if any identified (Quote Ref.)

Name of Fault	Total Length (km)	Type of Fault	Age of Fault	Activity	Potential MCE	Dist. from Site (km)	A_{max}

7. Foundation Conditions at Site: Rock Site: [ ] Stiff Soil Site: [ ]

Soft Soil Site: [ ] Not Known: [ ]

8. Probabilistic Maximum Acceleration of Earthquake with 90% Probability of nonexceedence in 1000-yr design life: A_{max} = ______

If A_{max} value here is smaller than in Items 5 or 6, ignore this value.

9. Design Earthquake: The MCE Event/Floating Earthquake with the following characteristics is selected as the Design Earthquake:

Richter Mag. : ______ Epicentral Distance: ______ km
Depth of Focus: ______ km Peak Acceleration : ______ g
Sig. Duration : ______ secs Predominant Period : ______ secs

10. Design Accelerogram: ____________________

11. Design Earthquake Parameters:

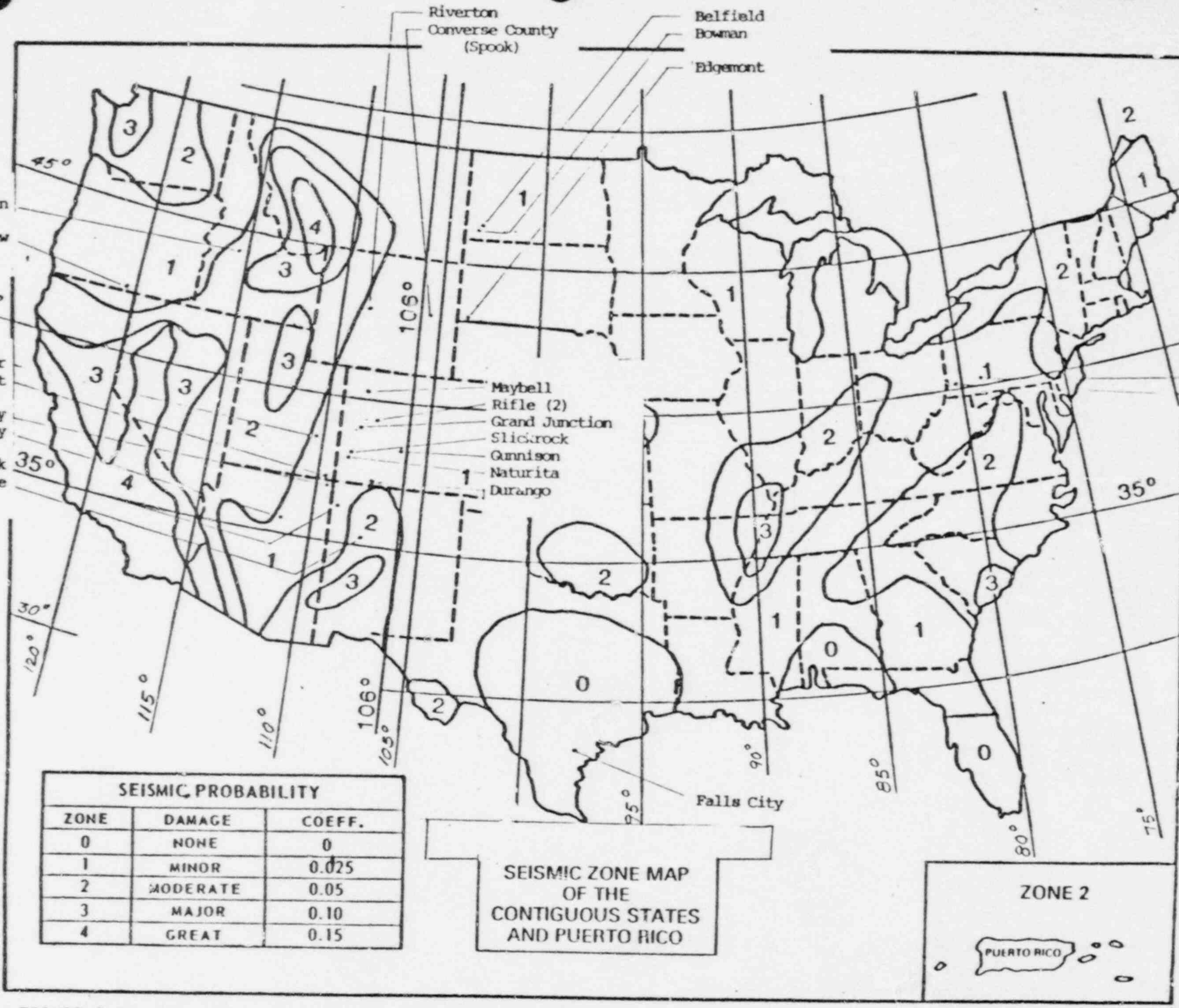
a. Base Liquefaction Analysis on Normalized STP Data (N_1).

b. For Peudo-static Slope Stability Analysis: Design Horiz. Seismic Coefficient: (i) Long Term Condition: K_H = ____g; (ii) E.O.C. Condition: K_H = ____g.

c. For shallow heights, use A_{max} from Item 9 above as Ground Acceleration.[a]

d. For Dynamic Stability Analysis[b] use computer programs (ISBILD, SHAKE, QUAD4, OR FLUSH) and accelerogram in Item 10 above.

Notes: (a) Item 11c: For large embankment height or deep soil deposit, determine ground acceleration from response analysis.
(b) Item 11d: Except at sites with very unusual site conditions, no dynamic analysis will be required at UMTRA sites.



SEISMIC PROBABILITY

ZONE	DAMAGE	COEFF.
0	NONE	0
1	MINOR	0.025
2	MODERATE	0.05
3	MAJOR	0.10
4	GREAT	0.15

FIGURE 8-1: Location of UMTRA sites on Seismic Zone Map of the Contiguous States and Puerto Rico.

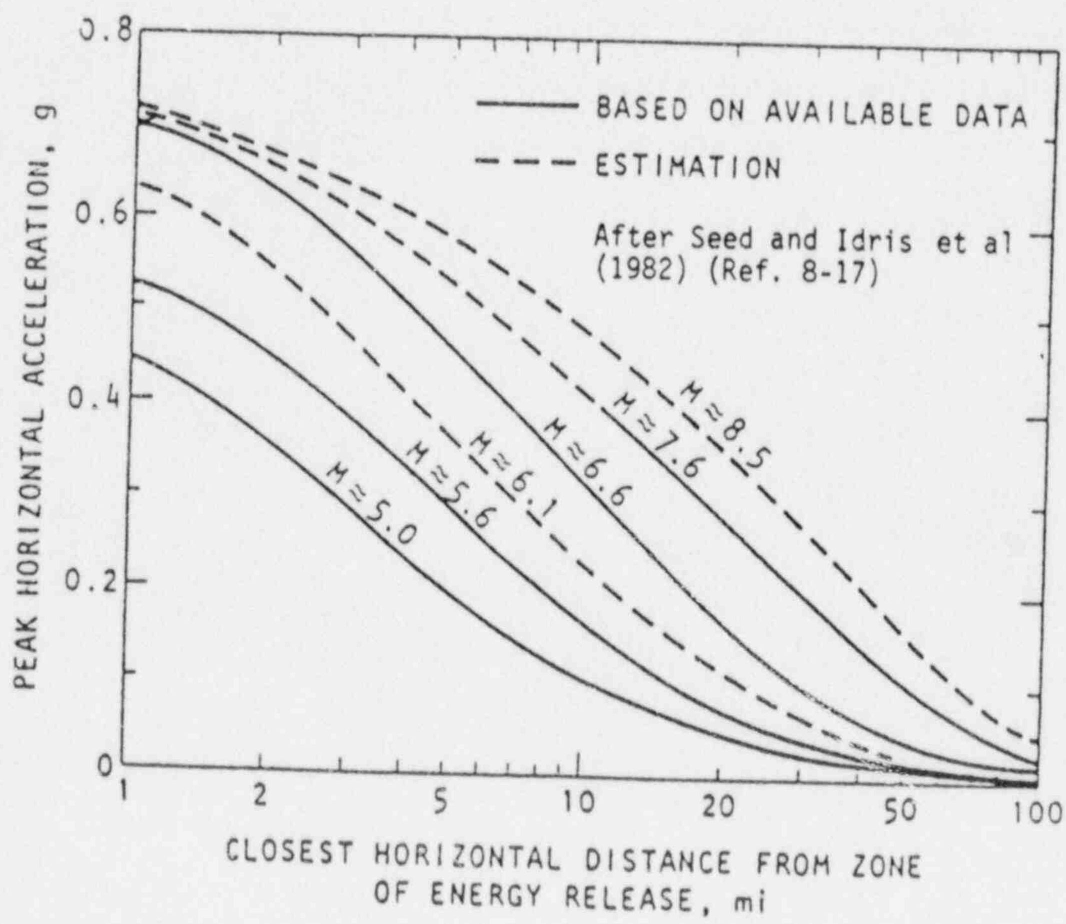


Figure 8-2: Average values of maximum accelerations in rock.

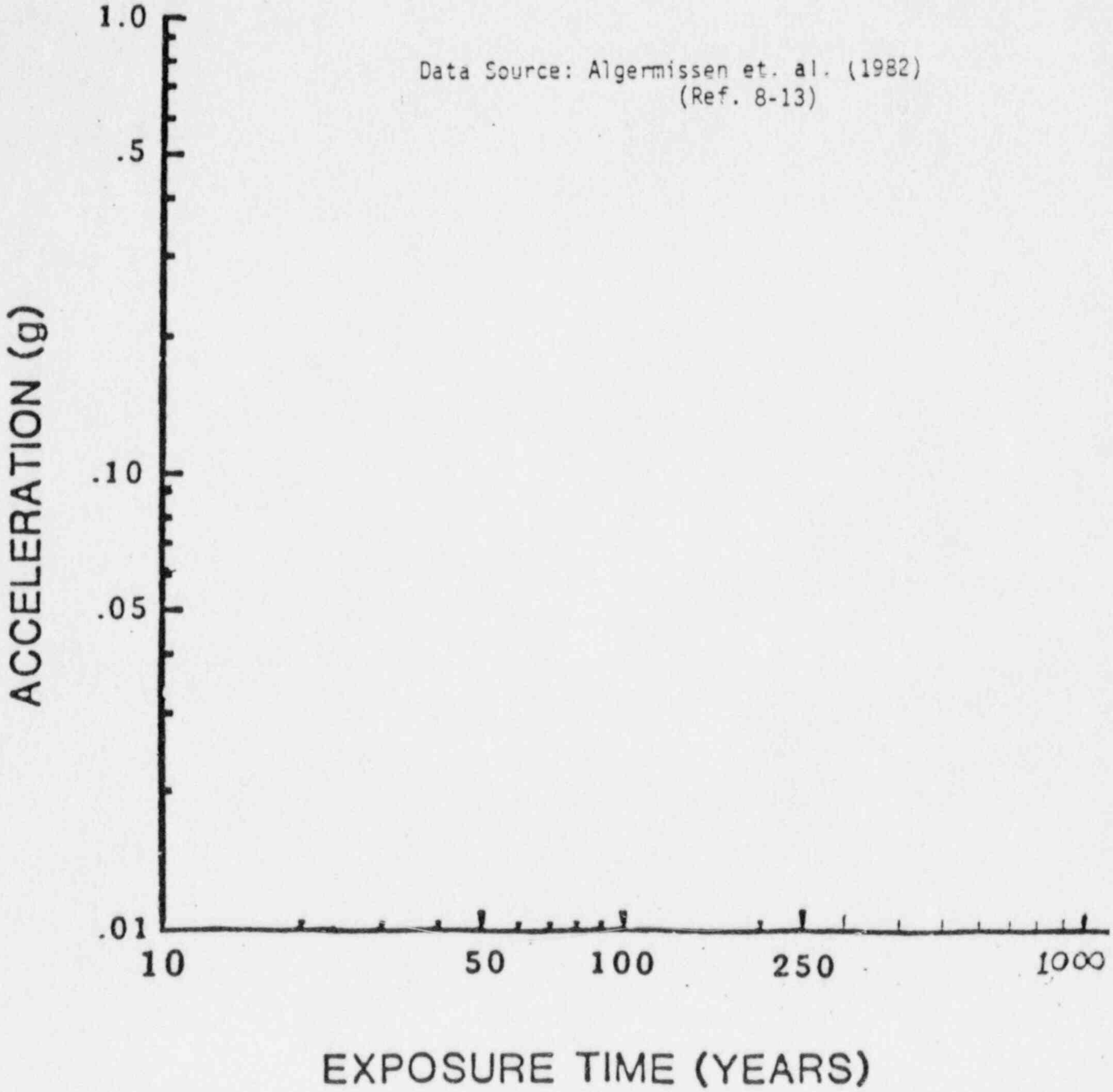
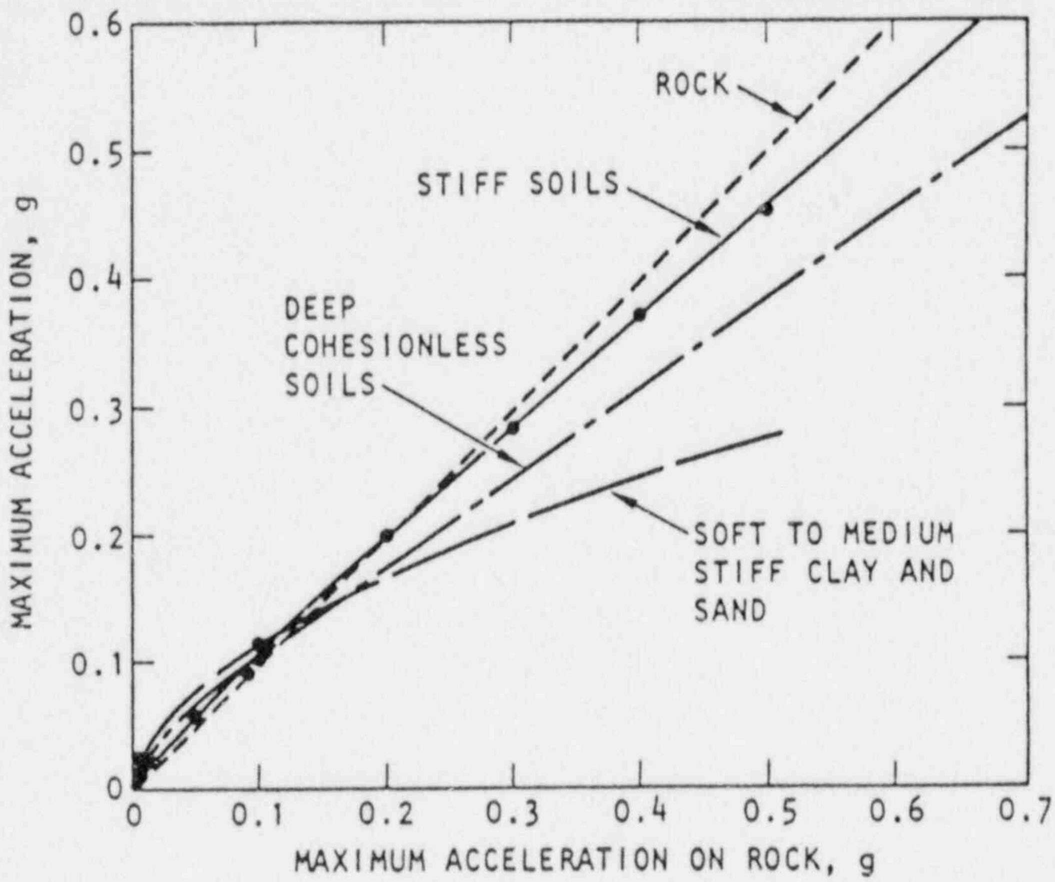
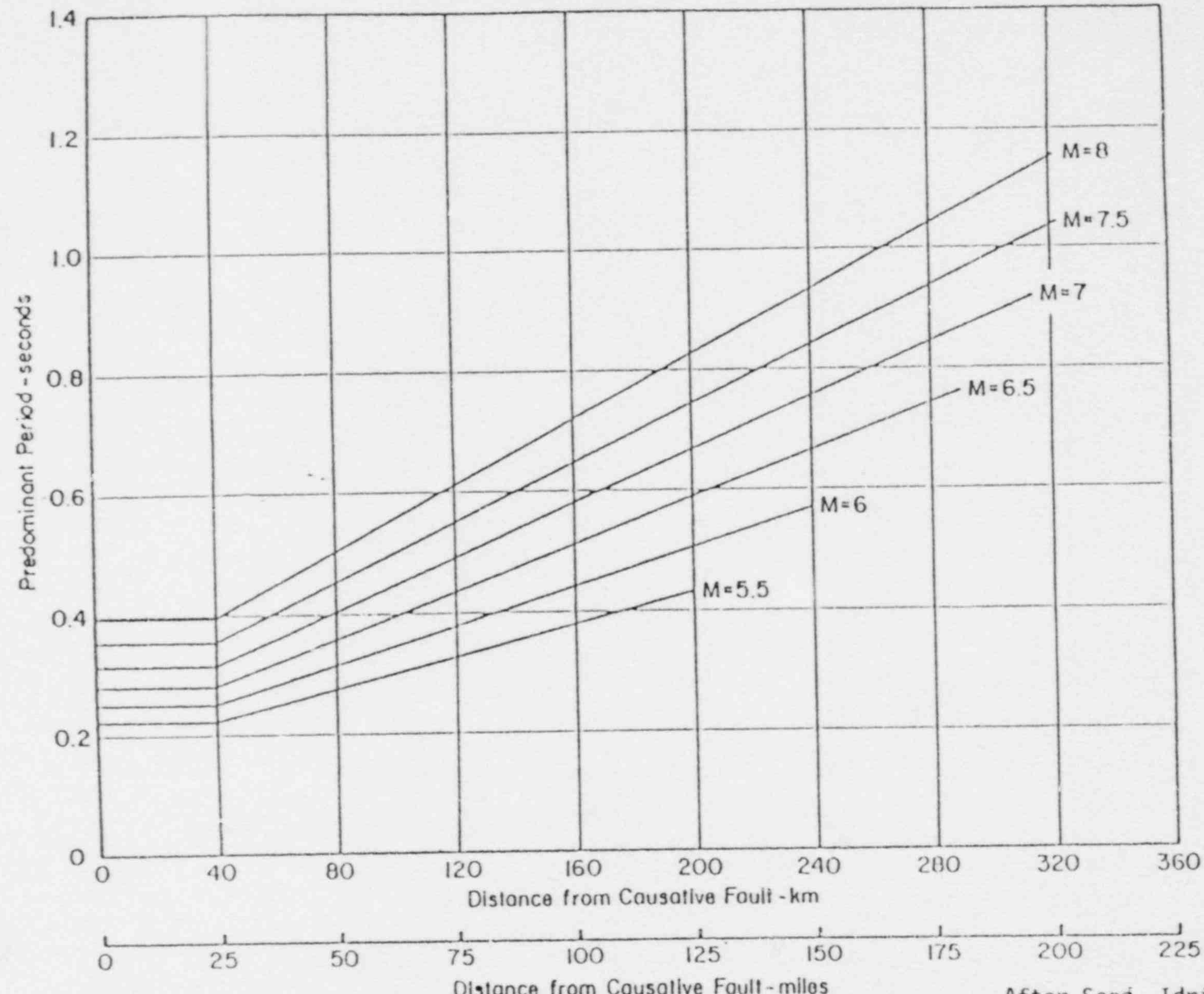


Figure 8-3: Probable maximum horizontal rock acceleration at the site, with 90 percent probability of not being exceeded in different exposure time.



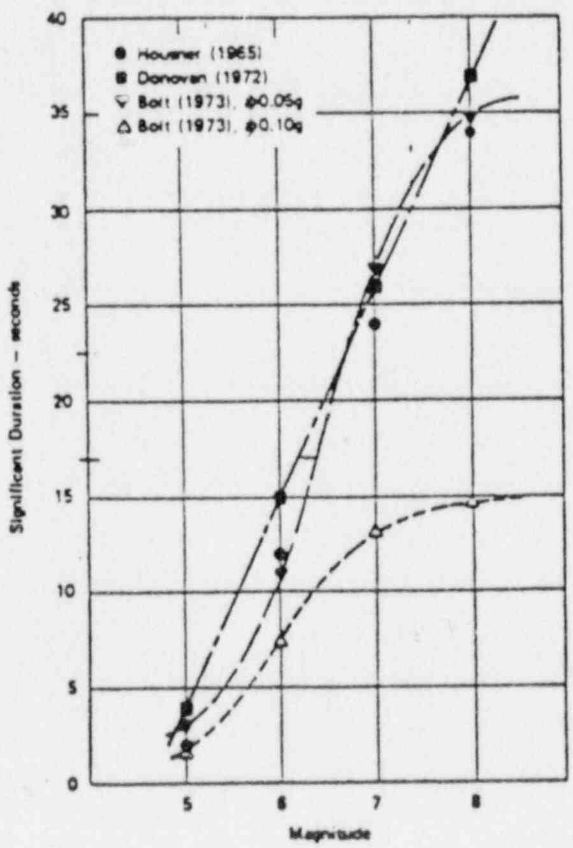
After Seed and Idriss
(1982) (Ref. 8-17)

Figure 8-4: Approximate relationships between maximum accelerations on rock and other local site conditions.



After Seed, Idriss and
Kiefer (1963) Ref. 8-19)

FIGURE 8-5: Predominant periods for maximum accelerations in rock.



After Dobry et al (1978)
(Ref. 8-21)

Figure 8-6: Typical available relations between magnitude and significant duration.

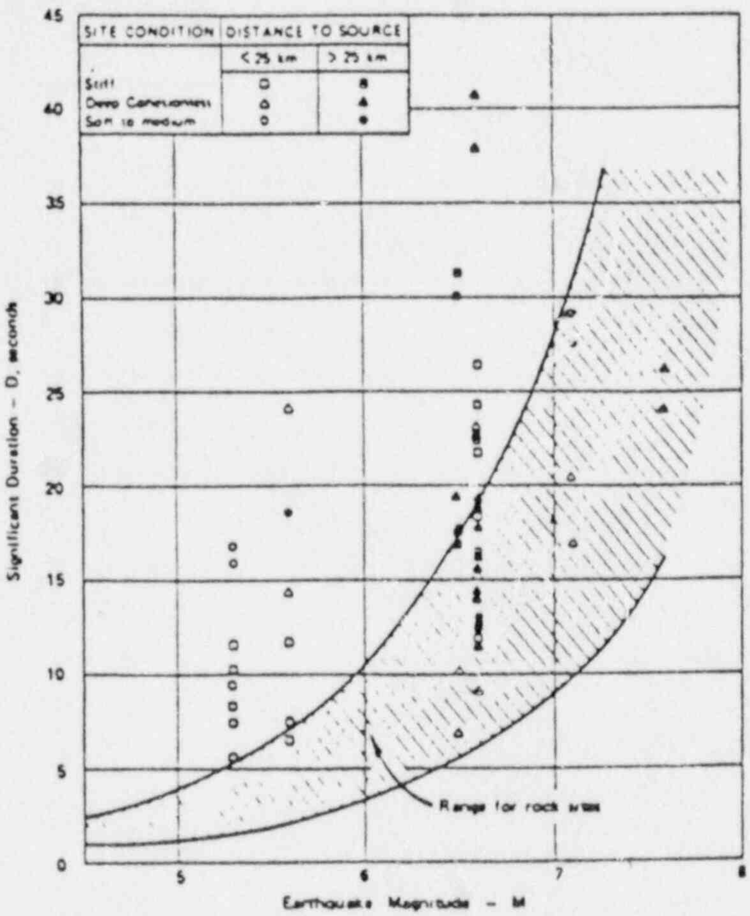


FIG. 15. *D* versus *M* for soil sites in western United States.

After Dobry et al (1978)
(Ref. 8-21)

Figure 8-7: D versus M for soil sites in Western United States.

MKE UMTRA DESIGN PROCEDURES

CHAPTER 14

RADON BARRIER CRACKING

REVISION 1

SEPTEMBER, 1987

CONTENTS

FIGURES

CHAPTER 14

RADON BARRIER CRACKING

14.1 INTRODUCTION

Radon barrier cracking can significantly affect the ability of the barrier to reduce radon release to acceptable levels. The two causes of cracking to be considered in radon barri ~ design are: 1) differential settlement, and 2) shrinkage fro, drying. The potential for cracking due to differential settlement can be evaluated as described below or by interpretation, using general guidelines for limiting differential settlements. The potential for shrinkage cracking can usually be evaluated based on available soil survey data and moisture content estimates. Finally, the effect of predicted cracking on average calculated site radon release must be considered in the design of the radon barrier.

14.2 DETERMINATION OF CRACKING POTENTIAL

A. Soils Data Required

The following radon barrier material data are required:

1. Soils data adequate to determine Unified Soil Classification System (USCS) classification.

2. Soil Conservation Service (SCS) reports including the description of shrink-swell potential, and COLE (coefficient of linear extensibility) test data, if available.

3. Compaction moisture content and estimated long-term moisture content.

B. Required Results from Previous Calculations

1. Settlement - Post-construction settlement of embankment (determined using procedures given in Chapter 7).

- o Settlement estimates must be adequate to accurately define the settlement profile at each location of concern. The settlement profile along a given reach is the ground surface profile which will exist along that reach after settlement has taken place.

- o Settlement profiles should be determined where the horizontal strain due to differential settlement will be the largest. Horizontal strain depends on the following factors:

1) Differential settlement (most important).

2) Thickness of relatively incompressible material (e.g. compacted tailings and radon barrier).

2. Radon Flux - Average radon flux for each sub-area of the site (determined using procedures given in Chapter 6).

- o Parameters used for radon flux calculations must be included to permit evaluation of potential effects of cracking.

- o Results for areas not subject to cracking will be included in computing the average flux for the site.

C. Cracking Due to Settlement

1. Settlement - Plot the settlement profile at each location of concern.

2. Horizontal Movement - Plot horizontal movement along the top surface of the radon barrier versus distance for each settlement profile determined under B.1 above. Horizontal movement can be calculated as follows (Ref. 14.1):

$$m = 2/3\ Hs, \text{ where}$$

m = horizontal movement
H = thickness of relatively incompressible material overlying compressible material (e.g. radon barrier plus compacted tailings thickness overlying in situ tailings plus foundation soils)
s = local slope of the settlement profile (expressed as a decimal fraction) - generally determined graphically.

3. Horizontal Strain - Determine the variation of horizontal strain along the top surface of the radon barrier. Horizontal strain at a given point is the slope of the horizontal movement profile at that point. This slope is generally determined graphically. Strains may be tensile or compressive. | R-1

4. Comparison of Strains - Compare calculated horizontal tensile strains with tensile strains that will cause cracking of the radon barrier. The strain to cause cracking is a function of the plasticity index (PI) and the moisture content of the soil. For soils compacted at moisture contents which are no drier than about 3% below optimum a lower bound for tensile strains causing failure is shown in Figure 14-1. Unless site specific failure strain data are available, this lower bound should be used to determine the area over which a potential for cracking exists.

5. Depth of Cracking - The depth of potential cracking can be estimated by assuming zero horizontal strain at the lower third point of the total thickness from the bottom of the compacted tailings and radon barrier (Ref. 14-1, p. 148), and a linear variation of strain with distance above that point.

D. Cracking Due to Shrinkage

1. Shrink-Swell Potential - If possible, only soils having a "low" shrink-swell potential, as defined in Table 12-1 should be used for radon barriers. If soils with "high" or "medium" shrink-swell potential are used, extra thickness should be provided, as indicated in the table.

2. Coefficient of Linear Extensibility (COLE) - COLE data can be used to estimate the area of open cracks relative to the total area (Ref. 14-2). In most cases this estimate generally will be conservative, because COLE data are determined by volume change from moist conditions (moisture content of soil at 1/3 bar tension) to the oven-dry state (Ref. 14-3). Therefore, determination of shrinkage crack effects by this method should be used only for conservative estimates, such as for marginal soils (i.e. soils having a "moderate" shrink-swell potential).

14.3 DETERMINATION OF POTENTIAL EFFECTS OF PREDICTED CRACKING ON RADON FLUX

An outline is presented below for determining the effects of cracking on radon flux.

A. Determine boundary and size of area of potential cracking.

B. Determine whether cracking is expected to extend through the entire thickness of the radon barrier.

C. Determine the potential for overburden stresses to prevent crack formation, especially in the lower portion of the radon barrier.

D. Determine potential effects of cracking on radon flux, using results of radon barrier flux calculations, as described below:

1. Estimate percentage of area that would actually be occupied by cracks. Use average estimated horizontal tensile strain as this percentage. Areas occupied by shrinkage cracks can be estimated using SCS soil survey data.

2. Reduce the effective cover thickness in areas of potential cracking, and recalculate the average site radon flux. Reductions in effective cover thickness in areas of potential cracking should be based on percentage of open area due to cracks, using Figure 14-2 (Ref. 14-4). The portion of the cover with reduced effectiveness will depend on the estimated depth of cracks.

EXAMPLE

Cracking potential was predicted for part of the radon barrier. Average tensile strain in that part of the area was estimated at 0.4%. Shrinkage cracking is not expected to be significant. Thus the percentage of area actually occupied by cracks is approximately 0.4%. Using the relationship on Figure 14-2, the cover effectiveness in that area will be reduced by 5%. Average (area-weighted) radon flux for the site should then be determined using only 95 percent of the cover thickness over the area of potential cracking, and the cover thickness adjusted to meet design criteria, if necessary.

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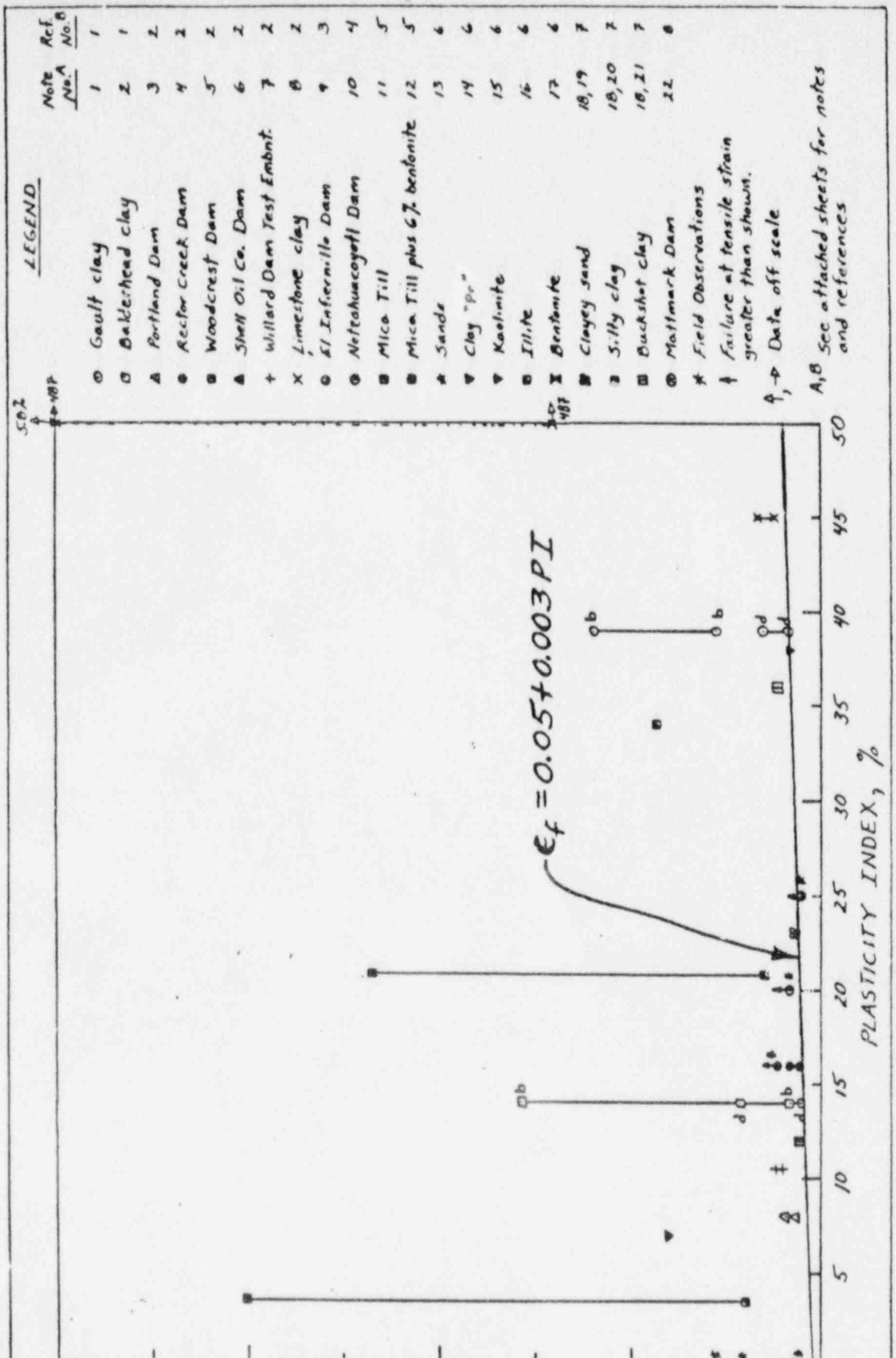

Figure 14-1: Tensile Failure Strain vs. Plasticity Index for Various Soils

(Sheet 1 of 5)

FIGURE 14 - 1
(Sheet 2 of 4)

NOTES FOR FIGURE 14 - 1

No.

1. Gault Clay: W_{opt} = 24.6%. Bending test results marked "b", W = 22.2-26.3%. Direct tension test results marked "d", W = 23.7-27.7%.

2. Balderhead Clay: W_{opt} = 13.0%; Bending test results marked "b", W = 11.5-15.2%. Direct tension test results marked "d", W = 13.2-16.2%.

3. Portland Dam: Range of long-term beam test results for W = W_{opt} = 16.3% and W = 13.5%.

4. Rector Creek Dam: Range of long-term beam test results for W = W_{opt} = 19.8% and W = 18.8%.

5. Woodcrest Dam: Range of long-term beam test results for W = W_{opt} = 10.2% and W = 7.2%.

6. Shell Oil Company Dam: Range of short-term beam test results for W = W_{opt} = 11.2% and W = 12.3% (no long-term tests).

7. Willard Dam Test Embankment: Range of long-term beam test results for W = W_{opt} = 16.4% and W = 12.5%.

8. Limestone Clay: Range of long-term beam test results for W = W_{opt} = 25.9% and W = 27.9%.

9. El Infiernillo Dam: Strain data averaged over long distance -- actual failure strain probably greater.

FIGURE 14 - 1
(Sheet 3 of 4)

10. Netzahuacoyotl Dam: No cracking observed for actual strains shown.

11. Mica Till: Indirect tensile test results for W = 7.2%-10.7%, W_{opt} = 9.2%. Rate of deformation was 0.005 in/min -- failure strains for slower deformation possibly less.

12. Mica Till plus 6% bentonite (by weight): Indirect tensile test results for W = 8.8-12.8%, W_{opt} = 10.8%. Rate of deformation was 0.005 in/min -- failure strains for slower deformation possibly less.

13. Sands: Results were near limit of sensitivity of apparatus.

14. Clay "Pr": W = 19.8%, W_{opt} and deformation rate not given.

15. Kaolinite: W = 37.6%, W_{opt} and deformation rate not given.

16. Illite: W = 31.5%, W_{opt} and deformation rate not given.

17. Bentonite: W = 93.5-109%, W_{opt} not given. Test durations between 5-425 minutes. PI = 487.

18. Failure strain calculated from reported results for failure stress and modulus (indirect tests).

19. Clayey Sand: Test result for W = 13.5%, W_{opt} = 14.0%.

20. Silty Clay: Test result for W = 15.9%, W_{opt} = 15.0%.

21. Buckshot Clay: Test result for W = 20.5%, W_{opt} = 22.0%.

22. Mattmark Dam: Data indicate post-construction measurements -- no cracking has been observed. Core material compacted near optimum water content according to AASHO standard.

FIGURE 14 - 1
(Sheet 4 of 4)

REFERENCES FOR FIGURE 14 - 1

No.

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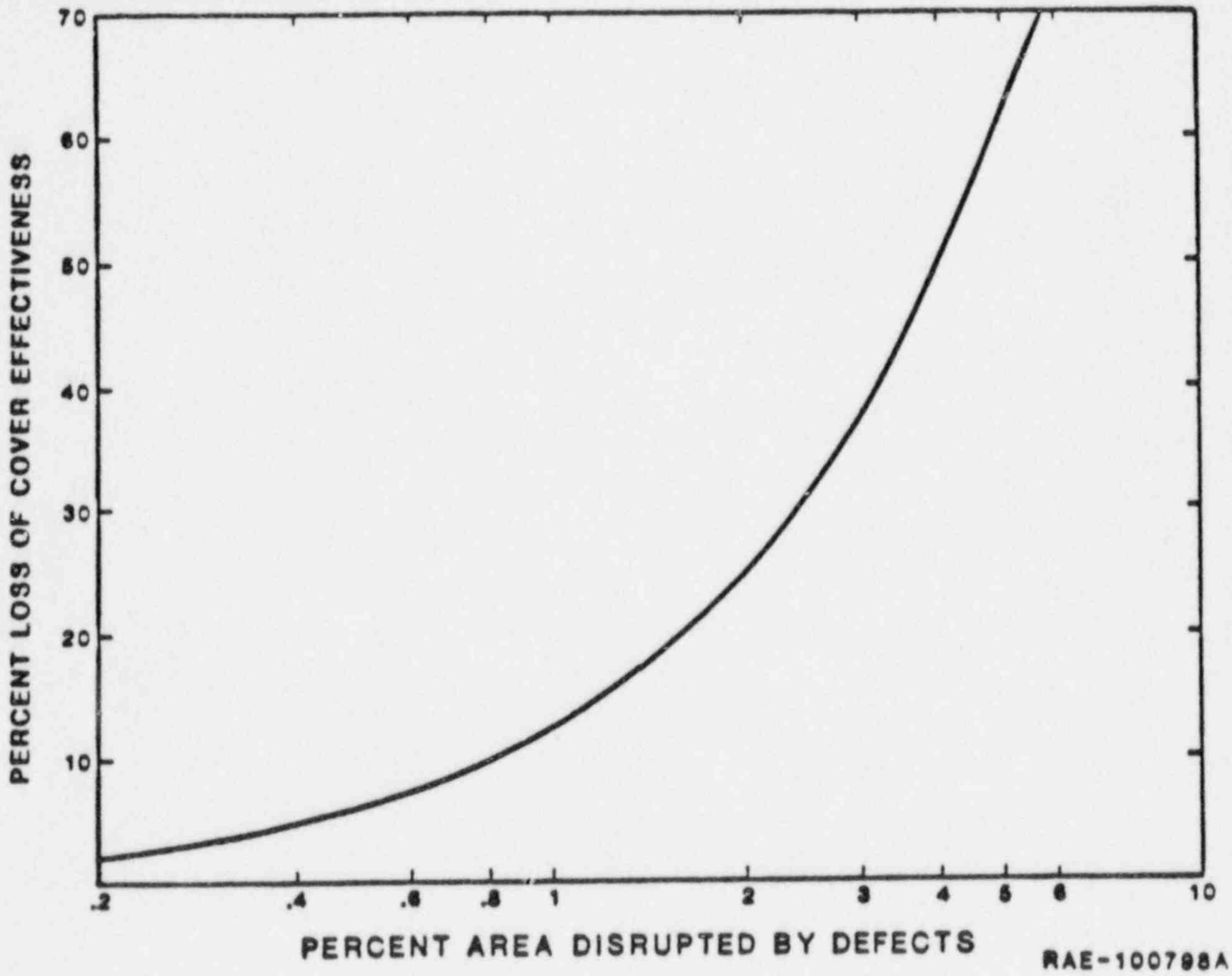

FIGURE 14-2 - LOSS IN RADON ATTENUATION EFFECTIVENESS
DUF TO CRACKS AND OTHER COVER DEFECTS
(Reference 14-4)

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