MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON-KNUDSEN COMPANY
UMTRA PROJECT
180 HOWARD ST. SAN FRANCISCO, CA 9410_

4005-GEN-L-01-00943-00

10 September 1985

WM Record File

WM Project 39
Docket No.
PDR ✓
LPDR

Distribution:
Ted Johnson w/encl, GGnugnoli
LBH DEM DSmith w/encl, DGillen
(Return to WM, 623-SS)

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

Subject: UMTRA PROJECT - GEN
Review Comments on "Standard Review Plan for Surface Water Hydrology and Erosion Protection" and "Design of Rock Armor for Uranium Mill Tailings Embankments"

Dear Mr. D'Antonio:

Enclosed are my comments marked on copies of the subject draft documents, which were distributed by Ted Johnson at the 24 July 1985 meeting of Working Group 1 of the DOE, NRC, TAC and RAC in Denver, Colorado.

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

Sincerely,

G. R. Thiers

Gerald R. Thiers
Principal Engineer

WM DOCKET CONTROL CENTER
'85 SEP 16 P4:33

GRT:kfb

cc w/enclosures:
Berg Keshian
Jim Oldham
Ted Johnson

Enclosures: 1. "Standard Review Plan for Surface Water Hydrology and Erosion Protection", NRC Draft Document, dated 8/29/85.

2. "Design of Rock Armor for Uranium Mill Tailings Embankments", NRC Draft Document, dated 2/19/85.

DRAFT

STANDARD REVIEW PLAN
SURFACE WATER HYDROLOGY
AND EROSION PROTECTION

I. HYDROLOGIC DESCRIPTION OF SITE

A. Areas of Review

The areas of review under this plan include

(1) identification of the relationships of the site to surface water features in the site area, and

(2) identification of mechanisms such as floods and dam failures that may require special design features to be implemented.

The review requires identification of the hydrologic characteristics of streams, lakes (e.g., location, size, shape, drainage area, etc.) and existing or proposed water control structures influencing flooding which may adversely affect the site design.

B. Acceptance Criteria

Acceptance of the information presented is based on a qualitative evaluation of the completeness and quality of information, data, and maps. The description and elevations of structures, facilities, and erosion protection designs should be sufficiently complete to allow independent evaluation of the impact of flooding and intense rainfall. Site topographic maps should be of good quality and of sufficient scale to allow independent analysis of pre- and post-construction drainage patterns.

Put USGS Map in Calc. Summary

The information presented forms the basis for subsequent hydrologic engineering analysis. Therefore, completeness and clarity of data are very important. Maps must be legible and adequate in coverage to substantiate applicable data. The descriptions of the hydrologic characteristics of surface water features should be detailed and correspond to those of the United States Geologic Survey (USGS), National Oceanic and Atmospheric Administration (NOAA), Soil Conservation Service (SCS), Corps of Engineers, or appropriate state and river basin agencies. Descriptions of all existing or proposed reservoirs and dams (both upstream and downstream) that could influence conditions at the site should be provided. Descriptions may be obtained from reports of the

USGS, United States Bureau of Reclamation (USBR), Corps of Engineers, and others. (Generally, reservoir descriptions of a quality similar to those contained in pertinent data sheets of a standard Corps of Engineers Hydrology Design Memorandum are adequate). Tabulations of drainage areas, types of structures, appurtenances, ownership, seismic and spillway design criteria, elevation-storage relationships, and short- and long-term storage allocations should be provided.

See Ref. 6 for adequate descriptions.

C. Review Procedures

The information normally presented is not generally amenable to independent verification, except through cross-checks with available publications relating to hydrologic characteristics of the site region and by site visits. The review procedure consists of evaluating the completeness of the information and data by sequential comparison with information available from references. Based on the description of the hydrosphere (e.g., geographic location and regional hydrologic features), potential site flood mechanisms are identified.

D. References

Because of the geographic diversity of sites and the large number of hydrologic references, no specific tabulation is given here. In general, maps and charts by the USGS, NOAA, Army Map Service (AMS), and Federal Aviation Administration (FAA); water-supply papers of the USGS; River Basin Reports of the Corps of Engineers; and other publications of state, federal, and other regulatory bodies, describing hydrologic characteristics in the site vicinity and region, are used. Other specific review areas, as given below, contain references that may be used in evaluating the specific hydrologic features of the site.

II. FLOODING DETERMINATIONS

A. Areas of Review

The flooding potential in the site area is reviewed to determine the extent of flood protection required to meet EPA standards. The areas of review include the precipitation potential, precipitation losses, the runoff response characteristics of the watershed, the accumulation of flood runoff through river channels and reservoirs, the estimate of the probable maximum flood (PMF) and other lesser floods at the site, and the determination of critical water levels and velocity conditions at the site. Included is a review of site drainage and a review of the probable maximum precipitation (PMP) potential and resulting runoff for site

drainage and for drainage areas adjacent to the site. The analyses involve modeling of physical rainfall and runoff processes to estimate possible flood conditions at the site.

Because uranium mill tailings and by-product materials may have potential detrimental effects on public health and safety, these hazardous materials must be contained in accordance with EPA guidelines (40 CFR 192). These regulations prescribe criteria for the design of protective covers to prevent wind and water erosion of tailings and ensure the sustained functioning of the tailings disposal system. Because little information exists on the long-term performance of protective covers, it is necessary to provide engineering designs which provide overall site stability for long time periods with no need for planned routine maintenance.

In providing such engineering designs for long-term performance, the selection of the design flood event is very critical, since the most disruptive natural phenomena affecting long-term stability are likely to be wind and water erosion. The selection of the flood event for the design of the protective cover should usually not be based on the statistical extrapolation of limited data bases, due to the unreliability of such estimates (Ref 3). Rather, the NRC staff concludes that, because the probable maximum flood (PMF) and the probable maximum precipitation (PMP) are based on site-specific physical meteorological limitations that eliminate the uncertainties associated with extensive extrapolation of limited data bases, it is reasonable and prudent to use these phenomena for the long-term design of reclamation covers. It is recognized, however, that many existing uranium mill sites are poorly sited; some are located immediately adjacent to streams with a high potential for extensive erosion. For these sites, the PMF forces may be so large as to preclude economical long-term stabilization. In these cases, the NRC staff has concluded that a flood smaller than the PMF may be considered if it is documented that implementation of a design to protect against the PMF is clearly impractical. In such cases, the staff may also consider that certain conservatisms normally present in the determination of design-basis floods or flood velocities may be relaxed (such as factors of safety, hydrograph peaking, etc.) and that such reductions may not significantly affect the overall safety and stability of the site. Any alternate approach selected using a flood smaller than the PMF should consider increased levels of maintenance, ~~repair,~~ ~~and environmental damage,~~ The staff will determine on a case-by-case basis whether there is reasonable assurance that the site stabilization program, as designed, will be effective for a minimum of 200 years and thus meet EPA regulations. — out

may result.

the ~~EPA~~ RAP should ~~be informed of~~ include the reasons for adopting the less severe flood, and of the fact that

B. Acceptance Criteria

The probable maximum flood as defined in ANSI N 170 (Ref. 4) has been adopted as one of the conditions to be evaluated in establishing the applicable stream and river flooding design basis. PMF estimates are needed for all adjacent streams, rivers, and site drainage channels. Two conditions may exist at the site under review, as follows:

1. The elevation and velocity attained by flooding on a large adjacent stream establishes a required protection level and the necessary flood protection.

2. The elevation and velocity attained by flooding onsite and in onsite drainage channels establishes the design basis flood protection.

as discussed under "A" above.

If it can be documented that implementation of a design to protect against the PMF is clearly impractical, a flood smaller than the PNF may be PMF adopted. The staff will estimate the flood level as described below. The estimate may be made independently from basic data, by detailed review and checking of the applicant's analyses, or by comparison with estimates made by others which have been reviewed in detail. Acceptance is based on general agreement of the staff and applicant estimates of static flood level and peak discharges. The evaluation of the adequacy of the flood estimates is generally a matter of engineering judgement, and is based on the confidence in the flood level estimate, the degree of conservatism in each parameter used in the estimate, and the relative sensitivity of each parameter as it affects the flood level or flood velocity. x

C. Review Procedures

The evaluation of flooding potential is, for review purposes, separated into two parts; flooding on large adjacent streams, and flooding on local drainage channels and protective features. The basis for the selection of the PMF as the design flood event is presented in Reference 3. The review procedure for evaluating a PMF on a large stream is outlined in Reference 4. The review procedure for evaluating a local PMP/PMF event is outlined in Part IV, below. PMF estimates approved by the Chief of Engineers, Corps of Engineers, and contained in published or unpublished reports of that agency, or generalized estimates such as those found in Ref. 8. may be used in lieu of independent staff-developed estimates. In the absence of such estimates, the staff will use both large and small basin techniques of the World Meteorological Organization in conjunction with Corps of Engineers' runoff, impoundment, and river routing models to estimate PMF discharge and water levels at the x

site. When detailed independent estimates are necessary, the applicant will be requested to provide ~~any~~ all necessary basic data.

In th~~e~~ose cases where it can be documented that it is clearly impractical to design erosion protection features for an occurrence of the PMF, the ~~documentation~~ evaluation will consist~~s~~ of the following:

X

(1) Review of several proposed designs (of varying slopes, configurations, alignments, drainage areas, etc) to ~~determine~~ document the difficulties in providing a reasonable design at a given site

(2) Review of erosion protection requirements associated with each of the above designs

(3) Review of costs (including transportation) associated with each design

(4) Review of analyses and logic which justify the reduction in flood criteria, both qualitative and quantitative

(5) Review of the bases for the ~~flood~~ flood selected design ~~bases~~ and the design of protective features with respect to the ability of the design to satisfy the EPA stability requirement of 200 years

(6) Review of the ability of readily available materials to satisfy design requirements (as a percentage of the PMF, e.g.,)

In general, the proposed design must provide reasonable assurance of meeting the EPA stability requirement of 200 years. While flood events and precipitation events should be treated on a site-specific basis, a generally acceptable minimum design (for meeting the 200-year stability requirement) would provide protection for ~~a~~ the largest flood or precipitation event known to have ~~that has~~ occurred in the site region. Enveloped historic floods (on a cfs/mi^2) basis), such as defined in Reference 24, establish acceptable minimum flood design bases. In addition, a flood or rainfall event of approximately 75-80% of the PMF/PMP may be acceptable where historic flood data ~~is~~ are not available. out The ability of the design to resist such flood events ~~is~~ ~~will be~~ are independently checked and evaluated by the staff to assure that minimum EPA standards are met.

~~What if this is clearly impractical?~~

III. DAM FAILURES

A. Areas of Review

or hydrologic

Peak water levels, routing procedures, and velocities are reviewed to determine potential hazards due to the failure of upstream water control structures from either seismic or hydrologic causes. When data are provided to show that seismic events will not cause failures of upstream dams that could produce the governing flood at the site, the areas of review will include items necessary to justify such conclusions. Where analyses are provided in support of either a conclusion that a dam failure flood due to a PMF is the design basis flood for a stream, or that a postulated or arbitrarily assumed seismically-induced flood is the design basis flood for a stream, the areas of review consist of the following:

1. Conservatism of modes of assumed dam failure (breach configuration, duration of flow, etc)

2. Consideration of storage capacity of flood control reservoirs

3. Conservatism of downstream flow rates and levels

4. Flood wave attenuation to downstream dams, or to the site

5. Potential for multiple upstream dam failures and resultant flood wave effects.

B. Acceptance Criteria

or hydrologic

The staff will review the applicant's analyses or independently estimate the coincident river flows at the site and at the dams being analyzed. The acceptable "worst conditions" to be postulated for analysis of upstream failures in lieu of substantiation of seismic resistance capability are: (1) a 25-year flood ~~in a full~~ on normal operating pool in a reservoir coincident with the dam-site equivalent of the ~~maximum credible~~ project design earthquake ~~(MCE)~~ (PDE); (2) a standard project flood (a flood about half the severity of PMF) on ~~a full~~ normal operating pool in reservoir coincident with the dam site equivalent of 1/2 of the ~~MCE~~ (PDE); and (3) a PMF on a reservoir which is not designed to safely store or pass ~~or breaching~~ such a flood.

The location of dams and potentially "likely" or severe modes of failure should be identified. The potential for multiple dam failures (of closely spaced dams) and the domino failure of a series of dams, should be discussed. Analytical hydraulic failure models will require complete model description and documentation. A determination of the peak flow rate and water level at the site for the worst possible combination of dam failures and a summary analysis (that substantiates the condition as the critical permutation) should be presented, along with a description of all

coefficients and methods used. Computations, coefficients, and methods used to establish the water level at the site for the most critical dam failures should be summarized. Comparison with steady or unsteady flow models with adequate site-related coefficients, serves as a basis for acceptance.

As stated in II.A, above, a flood less severe than the PMF may be acceptable in those cases where it can be documented that it is clearly impractical to design for a PMF. Additionally, if it can be documented that the reservoir has been or will be designed for the ~~MCE~~ PDE and the PMF, no dam failure and flooding analyses need to be performed.

C. Review Procedures

To be expanded w/specific refs.

In general, the conservatism of the applicant's estimate of flood potential and water levels from dam failures is analyzed and when required, an analysis is performed using simplified, conservative procedures (such as instantaneous failure, minimal flood wave attenuation, and extrapolated site discharge-rating curves). Techniques for such analyses are identified in standard hydraulic design references and text books. If the simplified analysis indicates a potential flooding problem, the analysis may be repeated using more refined techniques, and additional information and data are requested from the applicant, if necessary. Detailed failure models, such as those of the Corps of Engineers, National Weather Service, and the Tennessee Valley Authority, are utilized to identify the outflows and various failure modes and resultant water level at the site

If floods less than a PMF can be justified, the review procedures outlined in II.C, above, are employed to determine the impracticality of designing for a PMF and to determine the acceptability of the flood used.

IV. EROSION PROTECTION DESIGN

A. Areas of Review

In this section, the following erosion protection designs are reviewed:

1. Erosion protection to be placed to provide protection ~~due to~~ against the effects of flooding from nearby large streams.

2. Erosion protection to protect drainage channels.

3. Erosion protection to protect the top and side slopes of the remediated pile.

4. Durability of the erosion protection to be provided.

The staff review assesses the peak discharge rates, water levels, water velocities, and associated riprap requirements needed to provide protection to meet EPA long-term stability criteria.

As stated in II.A, above, a flood less severe than the PMF may be adopted if it can be documented that it clearly impractical to design for a PMF.

B. Acceptance Criteria

The erosion protection designs must be capable of meeting the long-term stability requirements of 40 CFR 192. In general, erosion protection that is designed to resist on occurrence of the PMP or PMF, or less severe flood as described in II.A above, provides an acceptable design. Additional details and acceptable methods of analysis may be found in Regulatory Guide 4.xx. (Ref. 23)

C. Review Procedures

The staff will check applicant analyses or perform independent analyses in accordance with the guidelines provided in Regulatory Guide 4.xx. If the design assumptions and calculations are reasonable, accurate, and/or compare favorable with independent staff estimates, the designs are found acceptable.

If floods less than a PMF can be justified, the review procedures outlined in II.C, above, are employed to determine the impracticality of designing for a PMF and to determine the acceptability of the flood selected.

References

1. Reports of Corps of Engineers, Department of the Army:

a. EM 1110-2-1411, "Standard Project Flood Determinations", 26 March 1952 (rev. March 1965).

b. EC 1110-2-27, "Policies and Procedures Pertaining to Determination of Spillway Capacities and Freeboard Allowances for Dams," 19 February 1968.

c. EM 1110-2-1405, "Flood Hydrograph Analysis and Computations," 31 August 1959.

d. EM 1110-2-1408, "Routing of Floods Through River Channels," 1 March 1960.

e. EM 1110-2-1406, "Runoff From Snowmelt," 5 January 1980.

f. EM 1110-2-1603, "Hydraulic Design of Spillways," 31 March 1965.

g. EM 1110-2-1409, "Backwater Curves in River Channels," 7 December 1959.

h. EM 1110-2-1601, "Hydraulic Design of Flood Control Channels," 1 July 1970.

i. CE 1308, "Stone Protection," January 1948.

j. EM 1110-2-1410, "Interior Drainage of Leveed Urban Areas: Hydrology," 3 May 1965.

k. Waterways Experiment Station, "Hydraulic Design Criteria," continuously updated.

l. ETL 1110-2-120, "Additional Guidance for Riprap Channel Protection," May 1971.

2. Hydrometeorological Reports of the U.S. Weather Bureau (now U.S. Weather service, NOAA), Hydrometeorological Branch: Nos 43, 49, 55

3. Johnson, T.L., "Design of Rock Covers for Reclaimed Uranium Mill Tailings Impondments: A Regulatory Prospective" Proceedings of Seventh Symposium on Management of Mill Tailings, Low-Level Waste and Hazardous Waste, February 6-8, 1985.

4. American National Standard Institute, Standards for Determining Design Basis Flooding at Power Reactor Sites, ANSI N 170, November, 1976.

5. Bureau of Reclamation, "Design of Small Dams," Second Edition, U.S. Department of the Interior (1973).

6. NRC, Regulatory Guide 1.59, "Design Basis Floods for Nuclear Power Plants."

7. "Flood Hydrograph Package," HEC-1, Corps of Engineers Hydrologic Engineering Center. Davis, California, continuously updated.

8. "Water Surface Profiles," HEC-2, Corps of Engineers Hydrologic Engineering Center, Davis, California, continuously updated.

9. "Reservoir System Operation for Flood Control," HEC-5, Corps of Engineers Hydrologic Engineering Center, Davis, California.

10. "Routing of Floods Through River Channels," EM 1110-2-1408, Corps of Engineers, March 1960.

11. Hunter Rouse, ed., "Engineering Hydraulics," John Wiley & Sons, Inc., New York (1950).

12. Ven Te Chow, "Open-Channel Hydraulics," McGraw-Hill Book Co., New York (1964).

13. J.M. Garrison, J.P. Granju, and J.T. Price, "Unsteady Flow Simulation in Rivers and Reservoirs," Jour. Hydraulics Division, Proc. Am. Soc. of Civil Engineers Vol. 95, No. HY5, pp. 1559-1576 (1969).

14. "Gradually Varied Unsteady Flow Profiles," 723-62-L2450, Corps of Engineers Hydrologic Engineering Center Davis, California, March 1969.

15. R.A. Baltzer and C. Lai, "Computer Simulation of Unsteady Flows in Waterways," Hydraulics Division, Proc. Am. Soc. of Civil Engineers, Vol. 94, No. HY4, pp. 1083-1117 (1968).

16. J.J. Stoker, "Numerical Solution of Flood Prediction and River Regulation Problems," Reports I and II, New York Univ. (1953-54).

17. V.L. Streeter and E.B. Wylie, "Hydraulic Transients," McGraw-Hill Book Co., New York, pp. 239-259 (1967).

18. W.A. Thomas, "A Method for Analyzing Effects of Dam Failures in Design Studies," Corps of Engineers Hydrologic Engineering Center, Davis, California, (for presentation at the ASCE Hydraulics Division Specialty Conference, Cornell University, August 1972).

19. "Flow Through a Breached Dam," Military Hydrology Bulletin No. 9, Corps of Engineers (1957).

20. "Floods Resulting from Suddenly Breached Dams, Conditions of High Resistance," Misc. Paper No. 2-374, Report 2, Corps of Engineers (1961).

21. Bureau of Reclamation, "Flood Routing," Chapter 6/0 in "Flood Hydrology," Part 6 in "Water Studies," Volume IV, U.S. Department of the Interior (1947).

22. National Weather Service - DAMBRK Model

23. Regulatory Guide 4.XX, Design of Long-Term Erosion Protection Covers for Reclamaton of Uranium Mill Sites.

? Conterminous

24. Crippen, J.R. and Bye, C.D., "Maximum Floodflows in the Contaminous United States," USGS Water Supply Paper #1887, 1977.

How does this compare to NWS Hydro 33, "Greatest Known Areal Storm Rainfall Depths for the Contiguous United States," by Albert P. Shipe and John T. Riedel, Dec., 1976 (NOAA)? In Hydromets? Send to Ted.

Also See CSU Rpt. Ch. 2 p. 8 & 9.

Document Name:
SRP/TJ/85/06/13/0

Requestor's ID:
DEBW

Author's Name:
Johnson

Document Comments:
Standard Review Plan Surface Water Hydro & Erosion Protect.

Johnson
8/23/85

4005-GEN-L-09-00891-00

Borg / Jerry

Here's the latest version of the SRP for your review and comment. This will serve as a starting point for our meeting on 9/5.

I've tried to incorporate all the discussion points and agreements that were reached at the last meeting. Notice that much of the criteria that we discussed will be incorporated into the Reg. Guide that will be finished later

Ted

RECEIVED - IECO
AUG 29 1985
UMTRA - S. F.

Draft 2/19/85 GRT

DESIGN OF ROCK ARMOR FOR URANIUM MILL TAILINGS EMBANKMENTS

by Richard Codell, Hydrology Section, WMGT, NMSS, USNRC

A. Introduction

EPA requires that uranium mill tailings embankments be stabilized and protected from natural phenomena for a period of 200 to 1000 years (40 CFR 192). The staff of the Hydrology Section, WMGT, interprets this rule to mean protection from the local Probable Maximum Precipitation (PMP) on the slopes. The direct impact of the falling water, runoff from the slopes and flow in the drainage channels at the base of the embankment must be taken into account. The present report is a summary of methods currently used to design the rock armor for the local PMP and the preliminary development of a method which will be used by the staff to check the design. This report considers only runoff and the design of armor on the slopes and does not consider either the design of drainage ditches or floods from rivers adjacent to the site. The Shiprock NM tailings embankment (Ref.3) will be used throughout this report as an illustrative example of the methods and models described.

DRAFT

Intense rain falling onto the embankments may pond on the ground surface, although part of this water will infiltrate the soil. The remainder will run off the slope into the drainage ditches and be carried away.

Rock armor is designed to resist the hydraulic forces of the flowing water, and to protect the soil from erosion. This is usually accomplished by placing a layer of relatively fine, well graded gravel directly on the soil to act as a "filter" layer, onto which much larger rock is placed. Larger rock is usually more expensive so the armor is generally designed to accommodate the smallest rock size suitable for the slope protection.

Tailings embankments will generally have two distinct slopes as illustrated in Fig. 1. The top slope is usually gentle, typically 2% to 5% grade, while the side slope is usually steep, typically 20% grade. Runoff from the top will be greatest at the bottom edge of the top area. Similarly, the flow will generally be greatest at the base of the steep slope, except in some cases where there could be appreciable flow over top of the rock layer on the top slope. Peak runoff is calculated using the rainfall intensity chosen from the local Probable Maximum Precipitation (PMP) at the site. The PMP is the greatest rainfall which is likely to be encountered at a given place. The PMP is a function of location (i.e., latitude and longitude) an the size of the drainage basin. The intensity of the rainfall at a particular point will be

DRAFT

greater for a small drainage basin than a large basin, and greater for a shorter duration PMP than a longer duration PMP. Intensity of rainfall alone however, does not determine the runoff from the slopes, since the dynamics of water flow through and over the rock armor must be taken into account in the calculation.

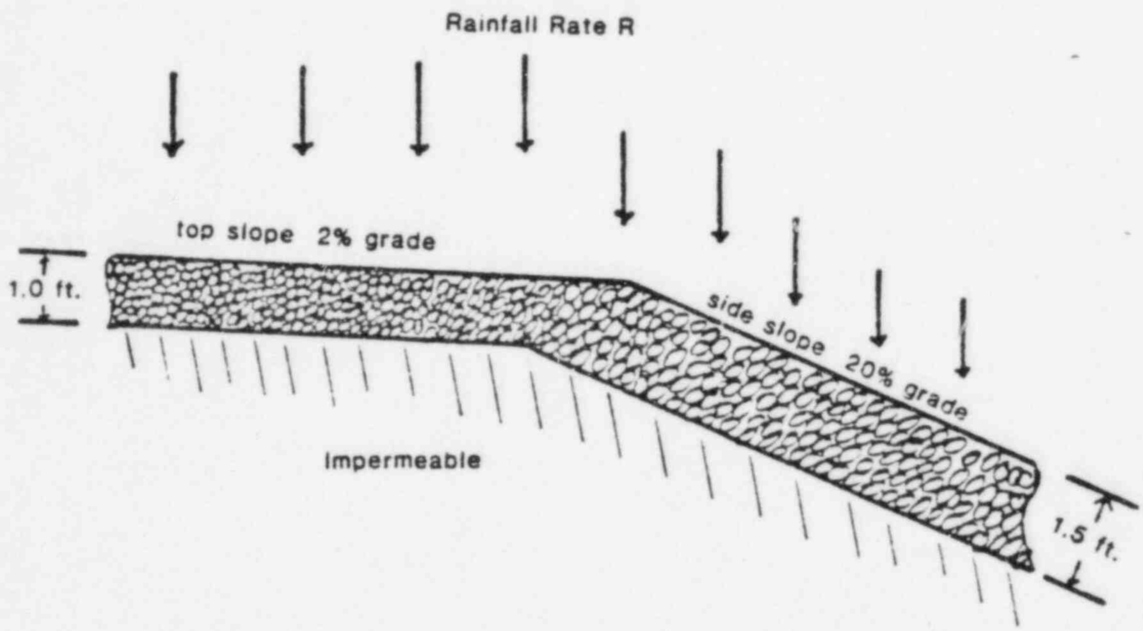


Figure 1 - Tailings Embankment in Profile

Once the peak runoff has been determined, the rock size needed to resist the maximum velocity of flowing water is determined.

The analysis of the embankment design for the Shiprock site performed for DOE will now be described.

B. DOE Analysis

The design of slope armor for the Shiprock New Mexico case as performed by for DOE is presented below. Parameters of the Shiprock case are described in Table 1.

Runoff Calculations

The DOE analysis for Shiprock estimated runoff from the slopes, Q using the Rational Formula (Ref.1):

$$Q = C\,I\,A \qquad (1)$$

$$\frac{\text{inches}}{\text{hour}} \times \text{acres} = \frac{\text{ft} \times 12}{\frac{\text{sec}}{3600}} \times \frac{\text{sq ft}}{43{,}560} = 0.9917\ \frac{\text{ft}^3}{\text{sec}}$$

$$\times\, 1.008 = \underline{\underline{1.0000}}$$

where C is the Runoff coefficient,

I is the rainfall intensity, ft/sec

A is the surface area of the drainage basin., sq ft.

Q is the runoff (peak) in ft^3/sec.

DRAFT

Table I - Base Parameters of Shiprock Example

Void fraction of rock, n (porosity) = 0.35

Thickness of rock layer = 1.0 ft. top layer, 1.5 ft. side layer

Length of slope = 440 ft top, 260 ft. side

Width of base of embankment (2-d model only) = 1200 ft.

Grade of slope = 2% top, 20% side

Rock diameter - top slope: d_{50} = 1.5" d_{100} = 8.0"

Side slope: d_{30} = 3" d_{50} = 4.5" d_{100} = 12"

Rock diameters used in models define

Top slope - d^{*} = 0.1 ft, d_{84} = 0.32 ft

Side slope - d^{*} = 0.3 ft, d_{84} = 0.75 ft

Friction factor for flow through rock, K = 2.0 explain

Finite difference grid spacing = 20 ft

DRAFT

The runoff coefficient C was conservatively chosen to be unity, i.e., there was total runoff of the precipitation with no loss from infiltration to the ground. Rainfall intensity was chosen from an intensity-duration curve for the Probable Maximum Precipitation in "Design of Small Dams" (Ref.2), using a 5 minute time of concentration calculated from a nomograph for overland flow (Ref.3). Peak runoffs from the top and bottom slopes were calculated to be 0.28 and 0.33 CFS/(~~sec.~~ ft), respectively (Ref.3).

misstatement — see OSM/TR-82/2 Table 3.3

superseded by data in HMR # 49

Determination of Rock Size

The analysis for the Shiprock site employed two methods for sizing rock. The rock on the top slope was sized using the Safety Factor method of Simons and Senturk (Ref.4). The analysis assumed that all flow was over the top of the rock. The depth of water at the edge of the top area, was determined by the Manning formula:

$$h = (Q\ m/1.49\ S_y^{1/2})^{0.6} \qquad (2)$$

DRAFT

where m = Manning's coefficient,

S_y = the slope of the face.

For flowing water at steady state, the tractive force on the rock surface, τ_s, must just balance the force of gravity:

$$\tau_s = \gamma \, h \, S_y \qquad (3)$$

where γ = density of water (62.4 lb./ft^3)

The diameter of the stable rock, d, can be determined if the tractive force on the rock is balanced against the natural tendency of the rock to remain in place because of gravity:

$$d = 21 \, \tau_s \, /(S_s - 1)\gamma \, \eta \qquad (4)$$

$$\text{where } \eta = \cos \alpha \, (1/SF - \tan \alpha/\tan \phi) \qquad (5)$$

for flow over plane sloping bed

S_s = specific gravity of rock

α = angle of grade = $\tan^{-1} S_y$

ϕ = angle of repose of dumped rock

SF = safety factor

DRAFT

The angle of repose is an empirical relationship shown in Fig.2, which is a function of the average rock diameter, d_{50}, and rock angularity (i.e., crushed, angular or very round). For a peak flow of 0.28 ft^3/sec ft on the top, a top slope of 2%, a Manning's coefficient of 0.1, and a safety factor of 1.5, DOE determined that the d_{50} rock diameter should be 1.4 inches, which was rounded to 1.5 inches.

too high (our experience at Shiprock using the Corps of Engr's formula: $n \approx 0.025$ to 0.03 for embankment top).

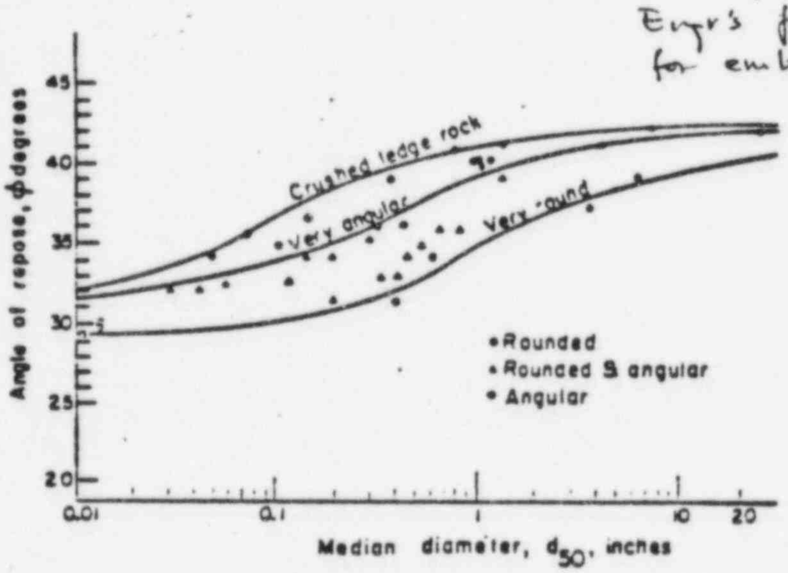


Figure 2 - Angle of Repose for Rock (Ref. 12)

DRAFT

DOE's analysis of side slope stability is based on Stephenson's method, which is for the calculation of the stability of ~~or~~ rock-fill in flowing water ~~dams in rivers~~ (Ref.5). ? The rock diameter which would just begin to move under the influence of flowing water has been determined to be:

$$d = \left\{ \frac{q\, S_y^{7/6}\, n^{1/6}}{C\, g^{\frac{1}{2}}\, ((1-n)\,(S_s - 1)\cos\alpha\,(\tan\phi\, S_y))^{1.67}} \right\}^{2/3} \qquad (6)$$

Where q = the flowrate on the embankment

g = acceleration ~~of~~ due to gravity

ϕ = angle of repose of the rock

n = ~~rock space fraction (i.e.,~~ porosity ~~),~~ of rock fill

S_s = specific gravity of rock,

and C is a factor which accounts for the angularity of the rock (determined to range from 0.22 for gravel to 0.27 for crushed granite).

The diameter determined from Eq. 6 is for the "threshold" ~~flowrate, which~~ condition, that is for the given flowrate ~~at which~~ the rock will just start to move. At slightly higher flowrates ~~just above the threshold,~~ the rock will rearrange to a more stable configuration. At much higher ~~velocities~~ rates, collapse of the structure will occur. Olivier

DRAFT

reports that the flowrate for collapse is from 120% of threshold flow for gravel to 180% ~~of~~ for crushed ledge rock (Ref.6).

DOE employed Eq. 6 in the Shiprock analyses, with the flowrate determined from Eq. 1 to calculate a d_{30} rock diameter, and assummed that $d_{50} = 1.5d_{30}$. For a flowrate of 0.33 CFS/ft., C = 0.22 (for gravel), n = 0.4 (conservatively large), $S_s = 2.65$ and S = 20%, they determined a d_{30} rock diameter of 3 inches. Angle of repose ϕ was determined from the emperical relationship shown in Fig. 3 for rounded gravel. The d_{50} and d_{100} diameters were chosen to be 4.5 inches, and 12 inches respectively, based on the diameter distributions for the gravel to be used.

rock pile porosity

Appraisal of DOE Design for Shiprock

C = 1?

Runoff was determined by the Rational formula with total runoff (R = 1), using the Probable Maximum Precipitation (PMP) for less than one hour from "Design of Small Dams" (Ref. 2). The correct PMP should have been taken from "Hydromet 49" (Ref. 7), which gives significantly greater precipitation for durations less than 1 hr. This is partially offset by the conservative runoff model employed. The safety factor method, used for the top slope, and the Stephenson method, used for the side slope, generally employed conservative coefficients.

Yes.

NRC did not take a stand on SF. method vs. Stephenson's for riprap sizing.

DRAFT

see also page 41 bottom

A major weakness of the DOE design methodology is that it fails to account for flow concentrations caused by poor design or slope failure.

see also top of page 42.

state-of-knowledge on flow concentrations is highly speculative.

C. NRC Design Methodology

The NRC staff has developed mathematical models based on physical pricipals, to account for some of the weaknesses in the more conventional approaches employed by the DOE contractors. The NRC approach is developed below and in Appendix A.

C.1 Runoff Models

One and two dimensional finite difference models for flow on a sloped, rock covered surface have been developed for the purpose of estimating the peak flowrates resulting from intense local precipitation. Details of the runoff model are given in Appendix A. ~~The~~ A one dimensional flow model was ~~exercised~~ used for the embankment slope~~s~~ typical of the Shiprock NM site.

Rates of precipitation ~~for all runs.~~ were generated using the one hour PMP of 8.0 inches, and intensity factors for periods less than one hour from "Hydromet 49" (Ref. 7) For periods shorter than 15 minutes, the intensities were scaled from the 15 minute values using factors from "Design of Small Dams" (Ref 2). The 2.5 minute PMP was estimated by straight line extrapolation from the 5 and

DRAFT

10 minute values. Figure 3 shows intensity versus duration for the "Hydromet 49" and "Design of Small Dams" curves. Values of the world record local PMP are also plotted on Fig. 3 for perspective (Ref.8). Although speculative, the PMP extrapolated for periods shorter than 15 minutes does not appear to be greatly out of line with reality.

Several experiments run with the model illustrate the hydrologic behavior of the slope under conditions of steady, intense precipitation, with the rate and duration determined from Fig. 3. The flows are maximized for storms of duration approximately 15 minutes, much longer and of lower intensity than the 5 minute PMP used in the DOE analysis. The main explanation for this difference is that the rockfill has a considerable ability to store water in the spaces between rocks, and most of the water flowing down slope would remain below the surface of the rock. Longer storms could saturate the rock and allow some water to travel over the top of the rockfill layer, particularly on the less steep top slope. Shorter but more intense storms would not cause flow over the top of the rock layer for the stated conditions. Friction for flow over the top of the rock layer would be considerably smaller than for flows restricted to the rock layer only.

DRAFT

K•E LOGARITHMIC 2 X 3 CYCLES
KEUFFEL & ESSER CO. MADE IN U.S.A.

46 7323

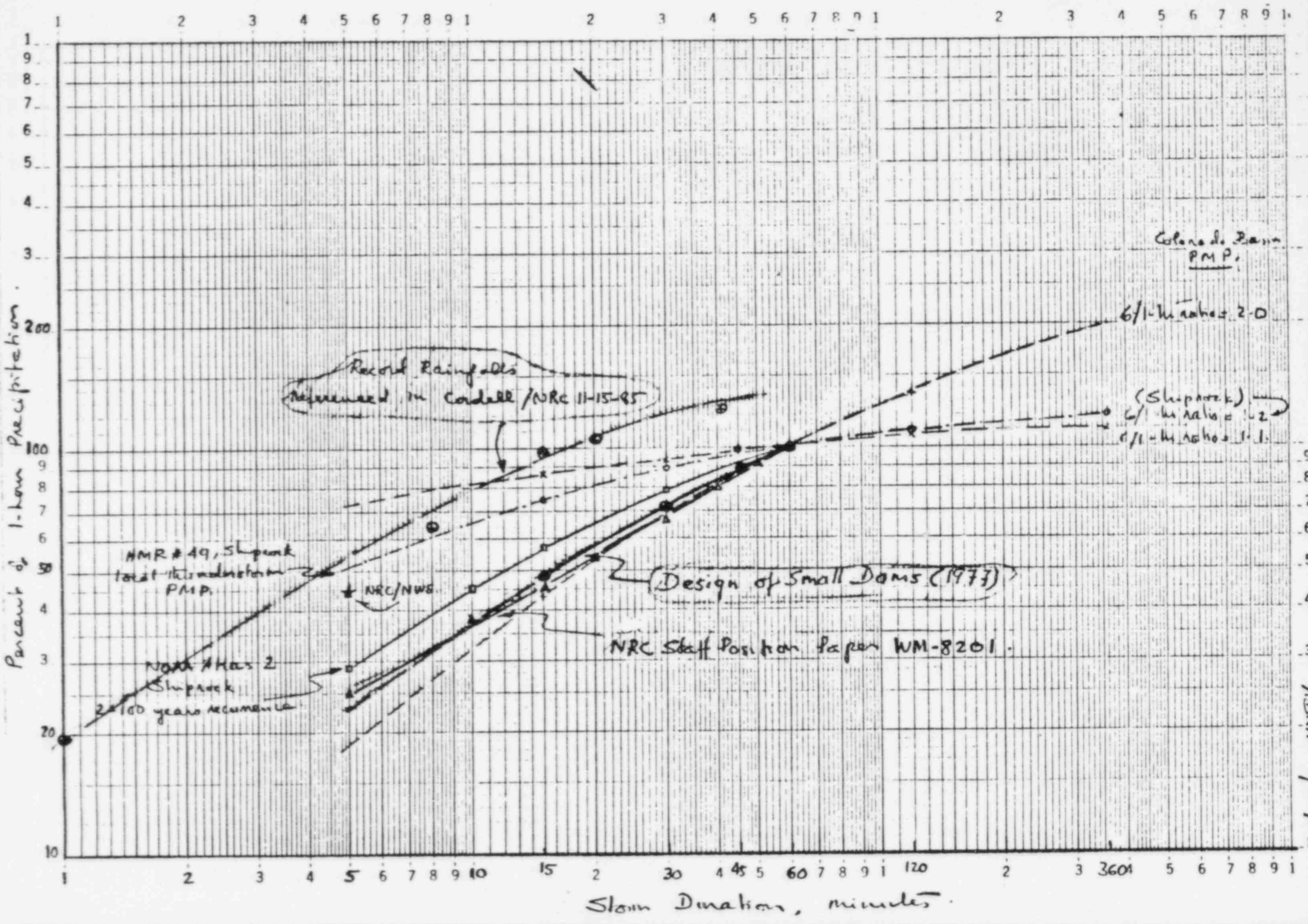


TABLE 2 - SUMMARY OF SHIPROCK RUNOFF CALCULATIONS

MODEL	EXPERIMENT	FIGURE	RUNOFF, CFS/FT TOP SLOPE	SIDE SLOPE	COMMENTS
1-d	Benchmark	4	0.25	0.32	Benchmark, Table 1 parameters
2-d	Benchmark run	8	0.165	0.33	Benchmark, table, parameters
2-d	½% slump	9	0.49	0.38	Considerable flow concentration Runoff from top slope dominates
2-d	1% slump	10	1.01	0.53	as above
2-d	2½% trench	11	1.24	0.74	as above
1-d	halve d *	12	0.26	0.27	increases runoff from top, decreases runoff from side
1-d	double d *	12	0.20	0.39	decresase runoff from top, increases runoff from side
2-d	double d * 1% slump	13	0.82	0.57	decreases runoff from top, increases runoff from side, but top dominates

DRAFT

TABLE 2 - SUMMARY OF SHIPROCK RUNOFF CALCULATIONS

MODEL	EXPERIMENT	FIGURE	RUNOFF, CFS/FT TOP SLOPE	RUNOFF, CFS/FT SIDE SLOPE	COMMENTS
1-d	halve layer thickness	14	0.43	0.43	increases runoff from top and side
1-d	½ grid size ½ layer thickness	-	0.41	0.43	effect of 10 ft grid size small, 20 ft. is credible
2-d	infinite layer thick	10	0.2	0.44	largely eliminates the effects of flow concentration for this case, largest rock thickness needed are 3.1 ft. top, 1.3 ft. side

DRAFT

ABH Agree? ADH ?

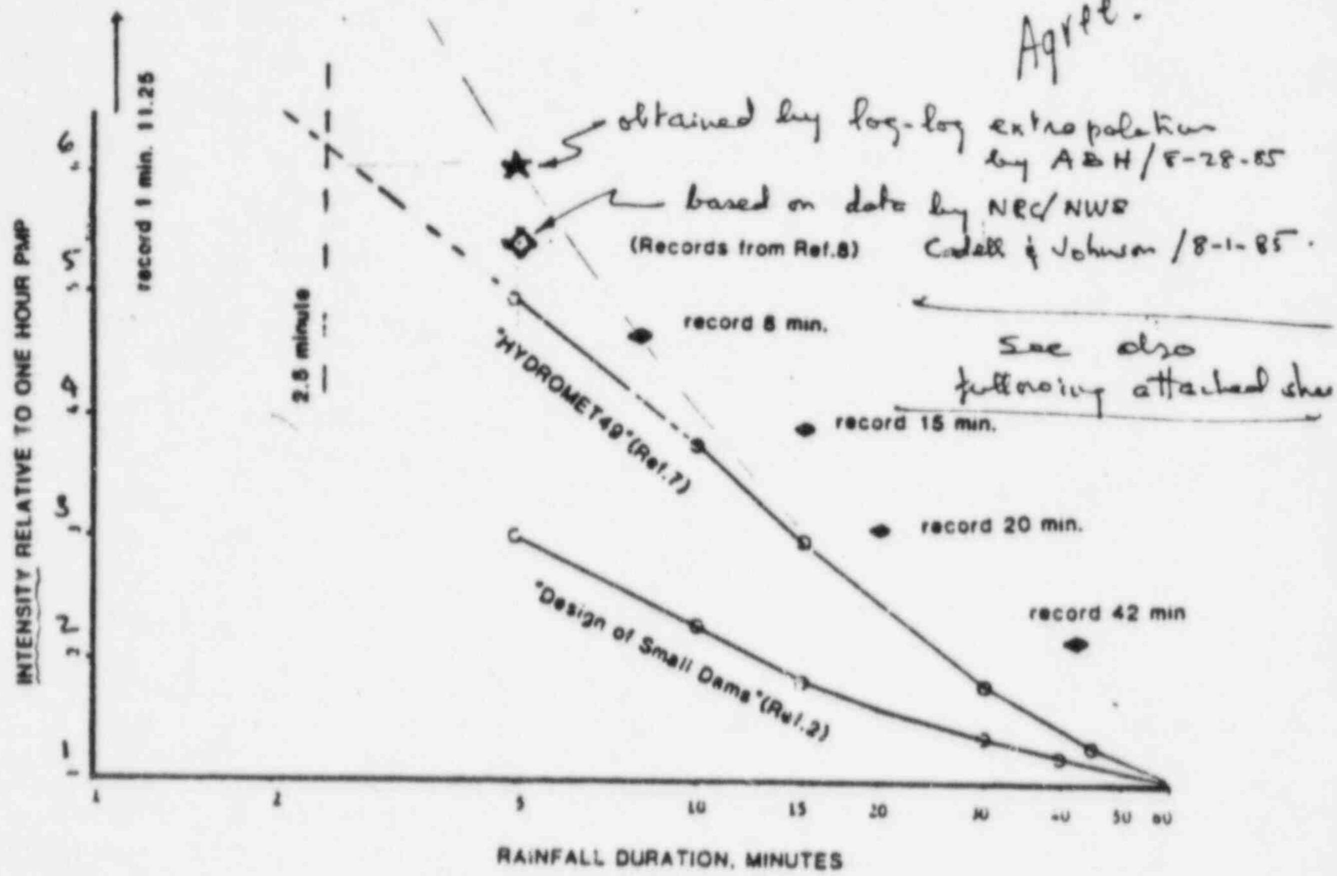


Figure 3 - Intensity vs. Duration for Local PMP

Recognizing that conditions which saturate the rock layer are likely to produce the greatest flows, the design basis hyetograph (time rate of precipitation) for the slope was formulated so that there was an increasing intensity of precipitation within the one-hour PMP, with the last 2.5 minutes of the hour being the most intense. Total precipitation for the first hour was 8 inches. Precipitation beyond one hour was arbitrarily held at 5.5 inches per hour,

DRAFT

which is the 5 minute intensity for the second hour of a 2 hour PMP. The hyetograph is presented in Fig. 4 along with the hydrograph for flow off the top and side slopes. The peak flow for the side slope was about 0.32 CFS/ft. Peak flow off of the edge of the top slope was about 0.25 CFS/ft. These values are close to the original DOE estimates of 0.33 and 0.28 CFS for the edges of the top and side slopes, respectively, using the storm intensity from "Design of Small Dams" (Ref.2) and a 5 minute time of concentration. The Rational formula, Eq.1, would have yielded much higher flows if the Hydromet 49 (Ref.7) storm intensity were used, estimated as 0.65 CFS/ft. and 0.44 CFS/ft. for the side and top slopes, respectively.

DRAFT

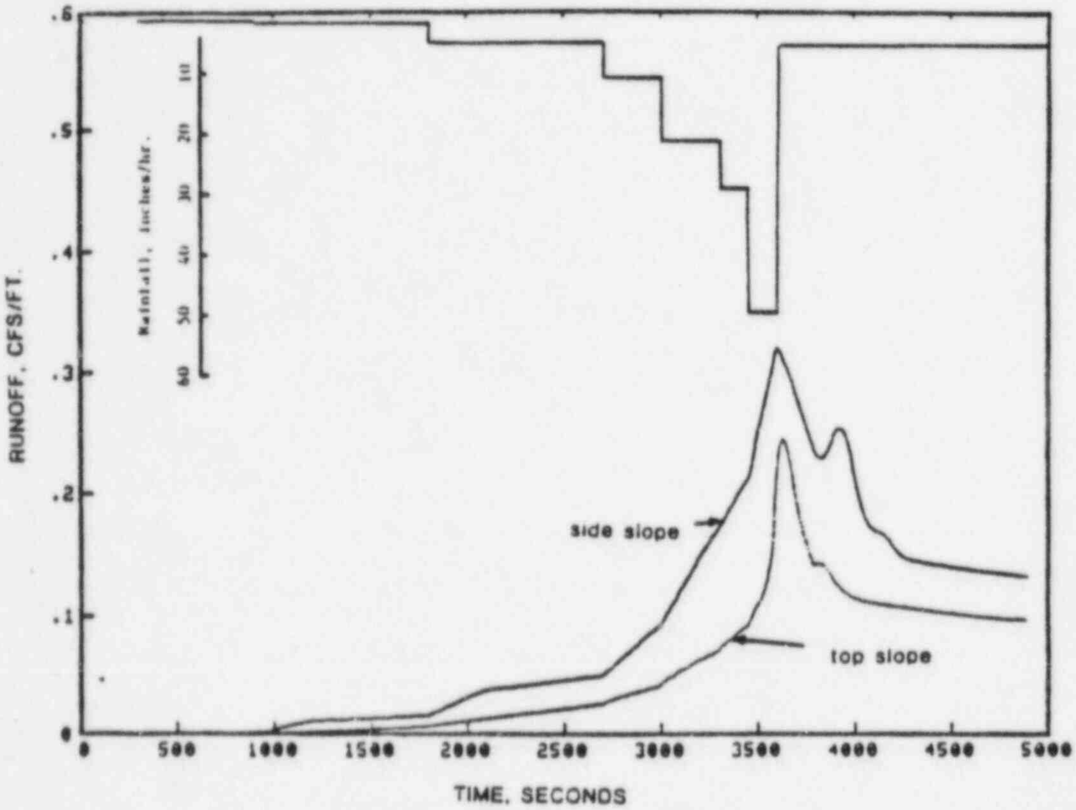


Figure 4 - Runoff for One-Dimensional Benchmark Case

C.1.a - Flow Concentration

The one dimensional analysis of runoff presented above is for infinitely-wide, flat surfaces with uniform slopes. Construction practices on the embankment earthworks will presumably strive to maintain flat surfaces and uniform rock

DRAFT

layer placement as illustrated in Fig. 5a. Nonuniformity of the slopes, however, could lead to concentration of runoff, causing higher flowrates than would otherwise be predicted. Conditions which could lead to flow concentration include:

1. Non-uniform slope grading.
2. Uneven placement of rock.
3. Gullying caused by erosion.
4. Slumping of earthwork.

The smaller grade and lower water carrying ability on the top slope would accentuate the effects of settlement on flow concentration, especially since part of the flow during the PMP would probably be above the top of the rock, and would be sensitive to changes in gradient because of the relatively low friction of the surface flow compared to the flow through the rock layer.

Soil erosion will be inhibited by the protection of the rock armor and filter layers, so gullying will be less severe than it would be on an unprotected natural slope. Observation of erosion patterns on natural unprotected slopes will not, therefore be highly relevant.

DRAFT

The most likely cause of flow concentration, given that good grading and rock placement practices are followed, will be a failure or differential settlement of the earthwork with subsequent slumping. Such a failure could create a depression toward which water running off the slope would collect. The nature of such a failure is highly speculative.

There are at least two compensating factors tending to resist flow concentration:

1. If the rock layers are thick enough, flow will occur beneath the rock layer surface, and the uniformity of the layer should be less important.

2. Tailings piles are often narrow at the top and wide at the bottom. This condition leads to a natural hydraulic gradient out from the centerline of the slope, tending to dispense rather than concentrate flow.

Settlement of from one to several feet might be possible on the sites (Ref.9). The effect of settlement can be overcome by good engineering practice, and it is not certain that slope failures would occur at all from these mechanisms. Nevertheless, several scenarios of embankment failure have been postulated and studied with the two dimensional numerical runoff model. The effects of embankment failure on flow concentration will be illustrated for several

DRAFT

postulated scenarios. A uniform inward slope of the embankment toward the centerline of the slope as shown in Fig. 5b would cause flow to concentrate. Multiple failures as illustrated in Fig. 5c would probably cause less severe flow concentration, because the drainage area for each sub-basin is smaller than the single basin case. Another case which would lead to flow concentration would be a single steep gully not extending over the entire width of the slope as shown in Fig. 5d.

The modeled embankment shown in Fig.6 is typical of the largest embankments at the Shiprock site. The slope is assumed to be triangular and symetrical around the vertical centerline in order to economize in the numerical solution. The modeled embankment is 700 feet long from the top to the drainage channel. The total slope is assummed to be 1200 feet wide at the bottom. The embankment is the same in profile as the one dimensional case shown in Fig.1 , with a 2% grade on the top slope and a 20% grade on the lower slope. Other parameters used in the model study are given in Table 1.

DRAFT

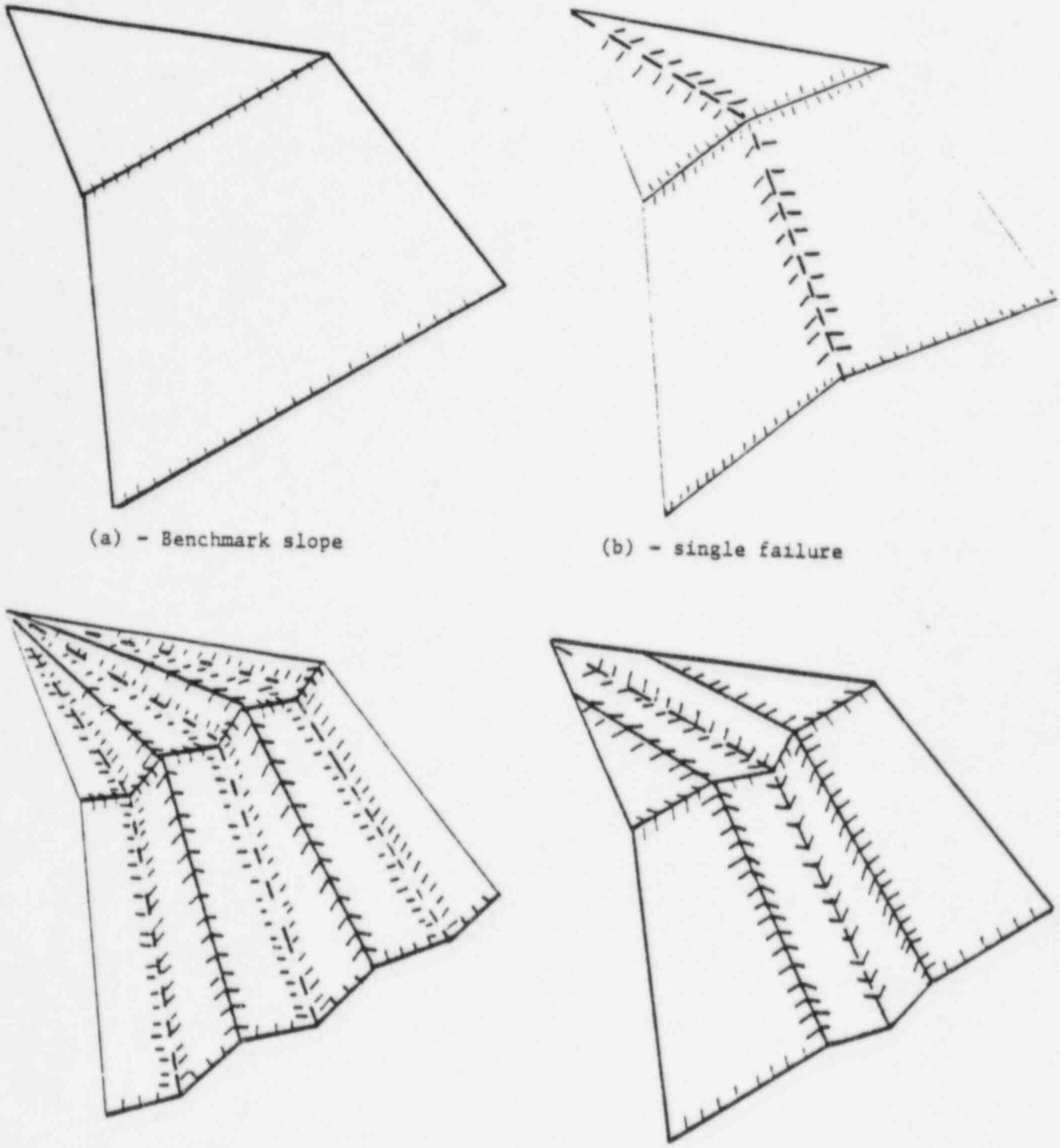


FIGURE 5 - FAILURE SCENARIOS FOR MILL TAILINGS EMBANKMENTS

DRAFT

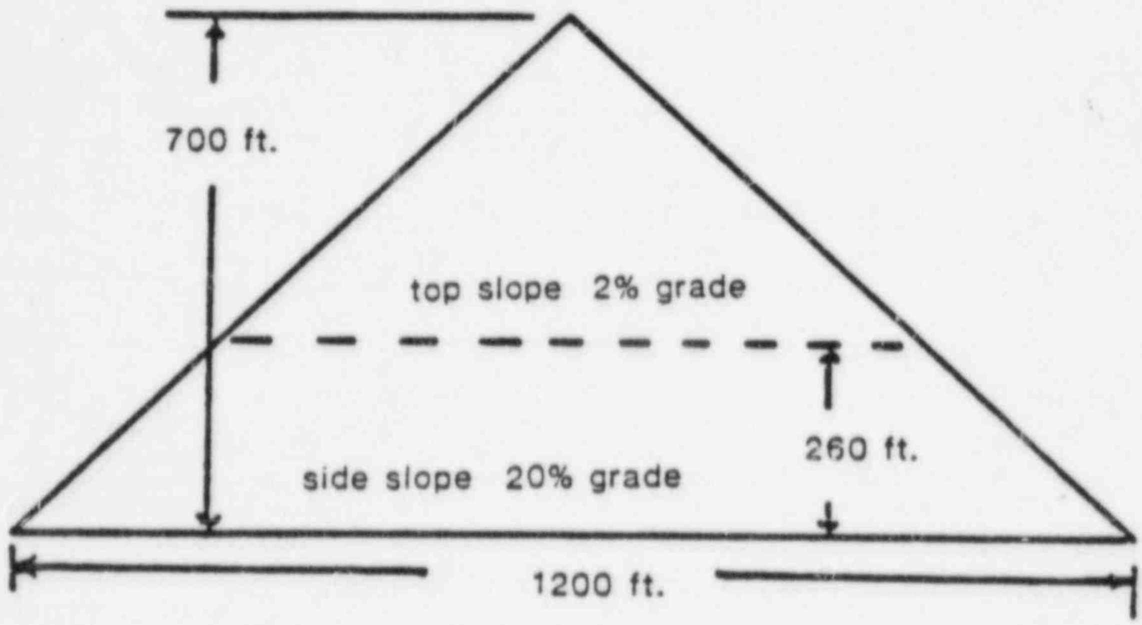


Figure 6 - Representative Two-Dimensional Slope for Example

Four scenarios were run for the two dimensional case in order to estimate the effects of embankment failure on flow concentration.:

1. 2-D Benchmark case, no slumping (Fig. 5a);

2. Uniform inward slope of ½% toward centerline (Fig.5b);

DRAFT

3. Uniform inward slope of 1% toward centerline (Fig.5b);

4. No inward slope, except for a 200 foot wide triangular channel centered on the centerline, slopeing 2½% toward the centerline (Fig. 5d)

C.1.b - Results of Two-Dimensional Scenarios

Steady-State Cases

Flow concentrations for Cases 1, 2 and 3 resulting from steady rainfall are presented in Fig. 7 as the ratio of runoff at the slope centerline from the 2-D model to runoff from the 1-D model. Flow concentration for the Benchmark due to spreading 2-D case are less than unity, especially on the top slope, and are relatively insensitive to the rate of precipitation. The low runoff from the top slope is attributed to the triangular shape of the embankment of the top, which leads to a gradient away from the centerline. The relatively gentle slope of the top slope accentuates the effect of this phenomenon. The side slope is both wider and steeper, so this phenomenon is less important to runoff there.

DRAFT

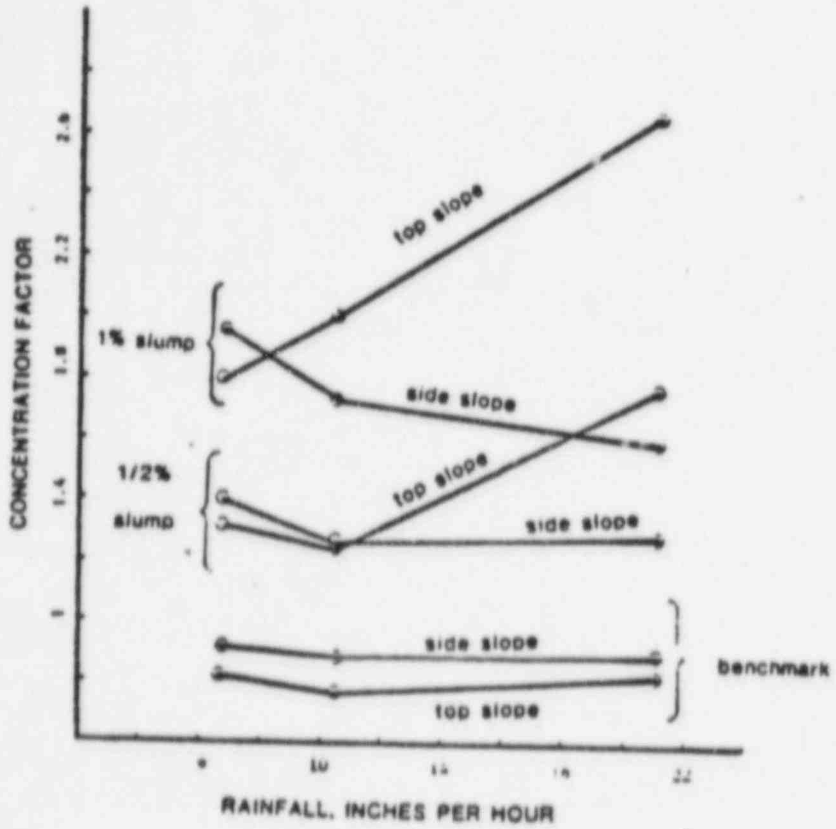


Figure 7 - Flow Concentration at Steady State, 2-d Model

In-gathering takes place

Flow concentration for the ½% and 1% inward slope scenarios are all greater than unity, and depend on the rainfall intensity. The high degree of flow concentration from the top slope is explained largely by the saturation and

DRAFT

overtopping of the rock layer. Resistance to flow is greatly reduced once overtopping occurs. In addition, the inward slope in each case is a significant fraction of the 2% gradient of the original slope. There is significantly less flow concentration on the steep side slopes. Overtopping would occur only at points close to the top. Flow rates are attenuated within the rock layer of the side slope because of the higher flow resistance. There was a persistant oscillation around the steady-state value of runoff at the highest precipitation rate. This oscillation may be either a hydrodynamic phenomenon or an artifact of the numerical method used to solve the equations. One of the recommendations for future work is to reprogram the 2-d model with a semi-implicit algorithm, which would tend to eliminate some numerical instabilitites.

Transient Cases

Runoff from the top and side slopes for Cases 1 through 4 resulting from the local PMP is presented in Figs. 8, 9, 10 and 11. Peak flowrates are also summarized in Table 2. Figure 8 shows runoff from Case 1, which is the Benchmark case. Peak runoff from the top slope is 0.165 CFS/ft, which is considerably lower than the 0.25 CFS/ft predicted from the one-dimensional Benchmark case shown in Fig.4. Peak runoff from the toe of the side slope is only slightly lower than that predicted by the one-dimensional model.

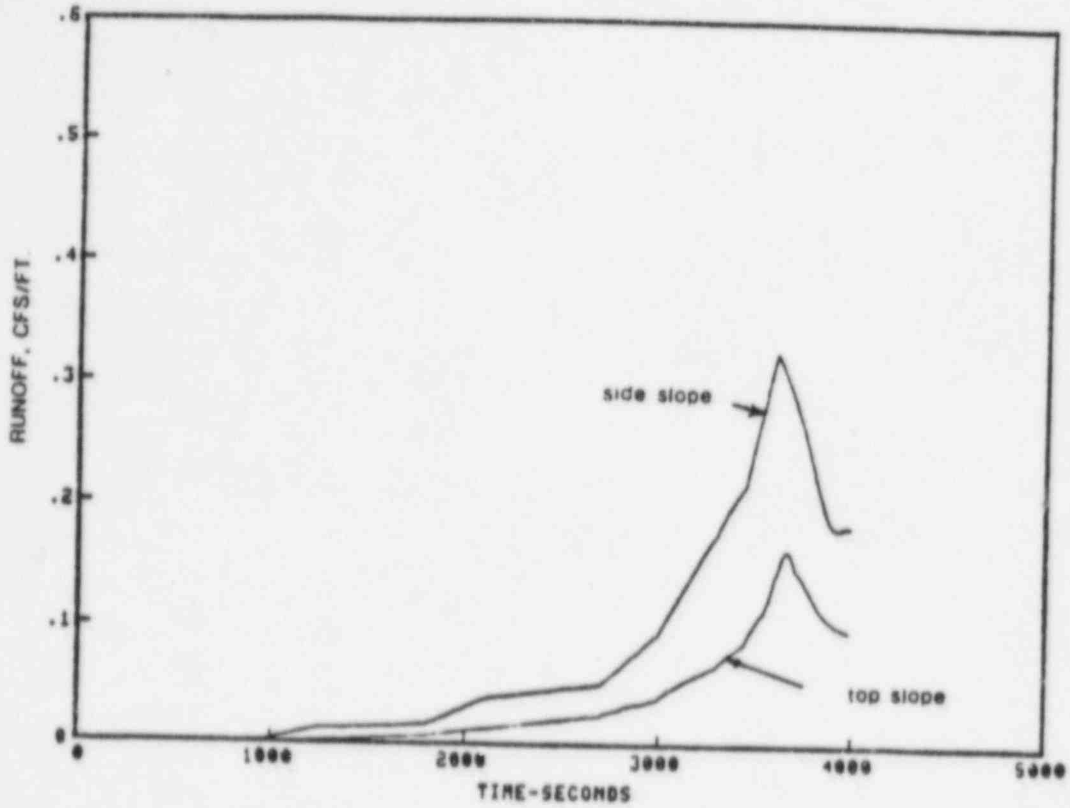


Figure 8 - Two-dimensional benchmark case

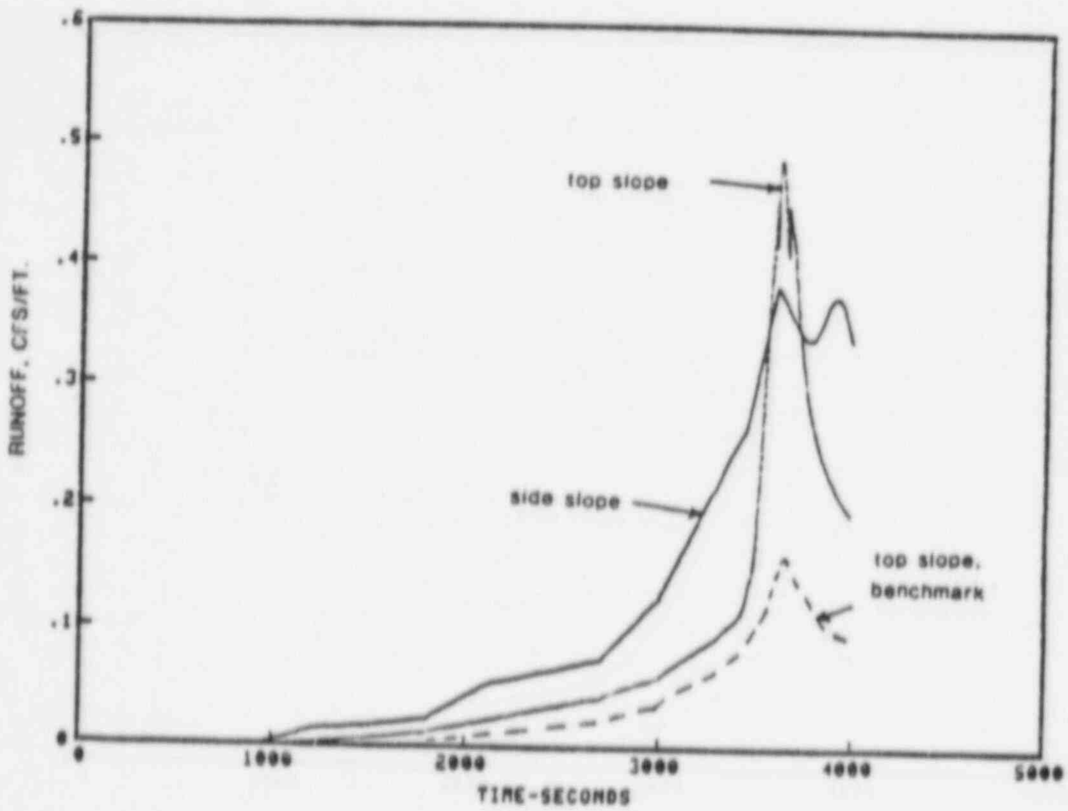


Figure 9 - Two-dimensional case, 1/2% slump

DRAFT

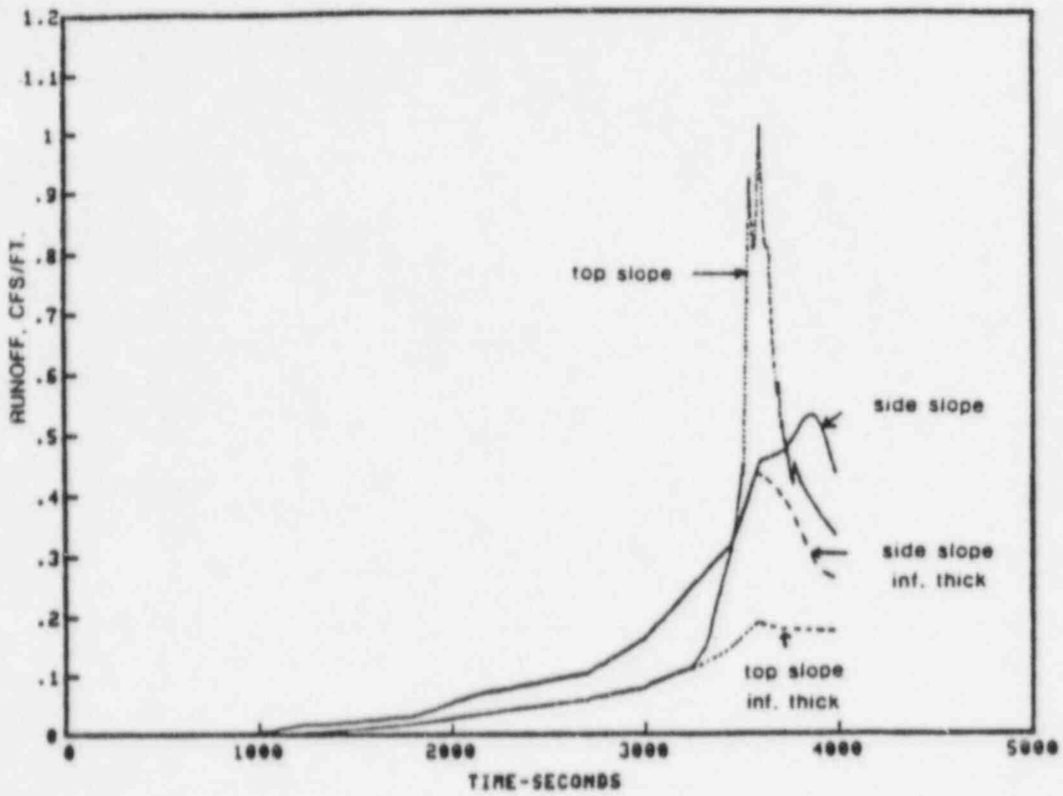


Figure 10 - Two-dimensional case, 1% slump

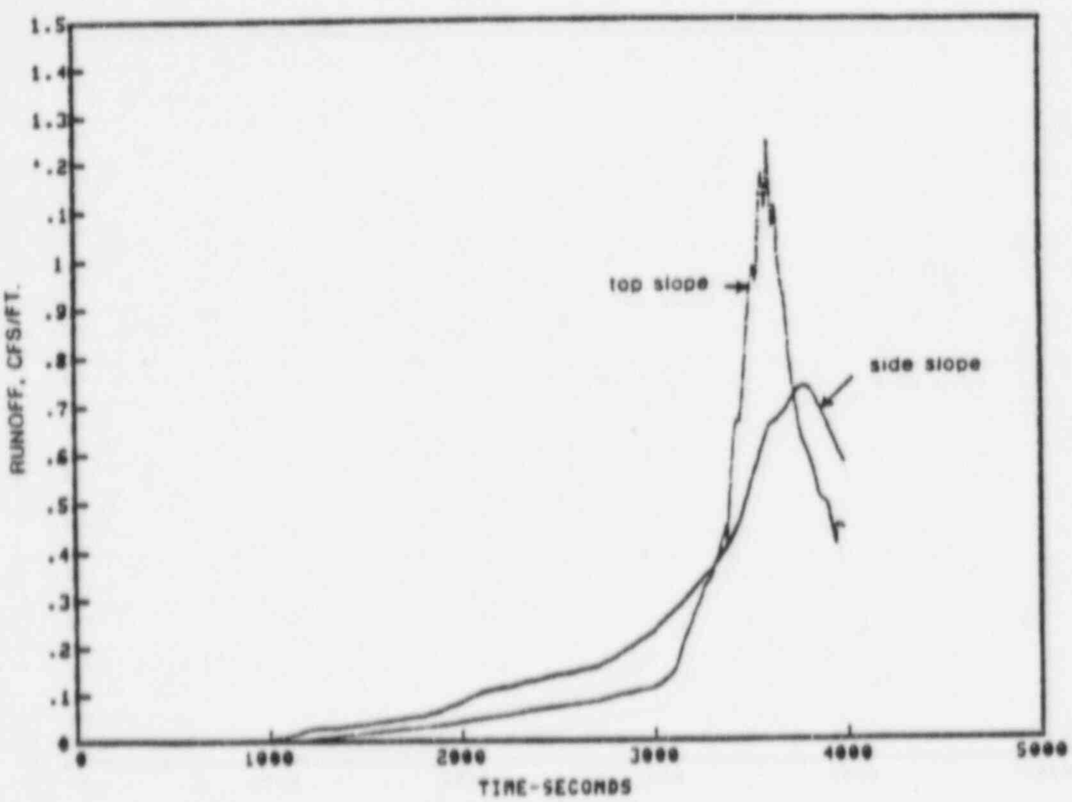


Figure 11 - Two-dimensional case - trench failure

DRAFT

Figures 9 and 10 show runoff from Cases 2 and 3, the ½% and 1% inward slope scenarios respectively. There is a considerable degree of flow concentration

in each case, particularly on the top slopes. The peak runoffs occur from the top slope rather than the side slope, even though the drainage area of the top slope is considerably smaller.

Figure 11 shows runoff for Case 4, the single trench failure. Flow concentration effects are greater in this case than for the other failure scenarios. Once again, peak runoff occurs from the top rather than the side slopes.

The transient runs for Cases 2, 3 and 4 all exibited some apparent instabilities, but it is not clear if this instability is a manifestation of a real hydrodynamic phenomenon or an artifact of the numerical solution. If the latter is the case, the peak runoffs may have been slightly overestimated.

An interesting observation from the four cases run with the two-dimensional model is that peak runoff may occur at the edge of the top slope rather than at the toe of the side slope. The design of the rock layer on the side slope may therefore be controlled by runoff from the top slope.

DRAFT

C.1.c - Sensitivity Experiments

High quality data on flow resistance through and over rock layers are generally scarce. Reported data often lack specific background information such as rock size grading, roughness, void fraction, density and rock layer thickness. Several test runs of the models were made in order to determine sesitivity of peak runoffs to errors in parameter estimation. The results of these runs are reported below and summarized in Table 2.

Sensitivity to Rock Diameter

One of the areas of uncertainty is the effective diameter of the rock, d^*, which is necessary to calculate resistance for flow through the rock. Leps (Ref.10) suggests that the d_{50} rock diameter is suitable for use in the resistance equations, providing that the percentage of fine material in the rock was not great. Stephenson (Ref.5) suggests that the harmonic mean diameter d^r be used, but the correlations of the experimental data he presents generally are in terms of the d_{50} diameter. Differences between d_{50} and the d^r can be large. The ratio d_{50}/d^r calculated for anticipated rock grades specified for the Shiprock example in Ref.3 is about 1.3. Ratios calculated on typical grades of crushed rock are in the range d_{50}/d^r = 2 to 3, which is much larger than that calculated for the reported Shiprock gradations. The harmonic

mean diameter places heavier emphasis on the smaller rock sizes. Only the d_{30}, d_{50} and d_{100} rock diameters were reported for Shiprock, so the harmonic mean calculated may not be an accurate reflection of the finer rock grades.

Friction relationships for flow over the top of the rock layer appear to be better known than the relationships for flow through the rock layer. The relationship used for flow resistance in the overtop layer is based on the d_{84} rock diameter (Ref.11).

The sensitivity of peak runoff to the estimate of mean rock diameter d^*, used for flow through the rock layer is demonstrated for several cases using the one and two dimensional models of the Shiprock example. Figure 12 shows the sensitivity of runoff from the top and side slopes calculated with the 1-d model. Sensitivity to these changes is also summarized in Table 2. Increasing the diameter has the effect of decreasing the runoff from the top slope and increasing the runoff from the side slope. Increasing the diameter lowers the internal friction, allowing more of the flow to travel beneath the rock layer on the top slope. Overall, <u>increasing</u> d^* <u>increases</u> flow resistance on the top slope.

Most if not all of the water is transported within the rock layer on the side slopes anyway, so an increase in d^* will lead to a decrease in overall

DRAFT

friction, with a subsequent increase in the calculated peak runoff from the side slope. Since runoff from the top slope contributes to the side slope runoff, it is not always clear that the observed sensitivity would be characteristic of all cases. Note that the d_{84} diameter, which is used for the flow resistance of the overtop layer in these runs has not been changed, although increasing the estimate of d_{84} would increase the overall resistance for those flows which overtop the rock.

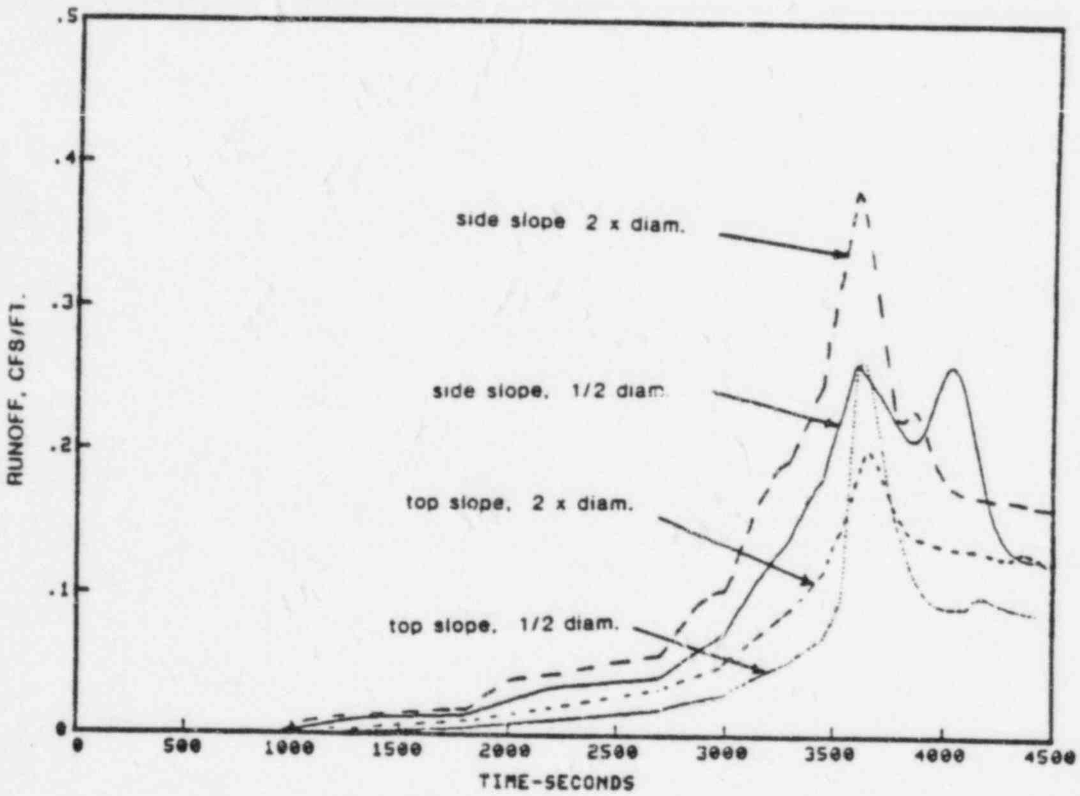


Figure 12 - Sensitivity of Runoff to d^*, One-Dimensional Model

DRAFT

Figure 13 shows the results of doubling the d^* rock diameter for runoff from the top and side slopes in the case of the 1% inward slope failure scenario. The effect of increasing d^* is the same as observed for the 1-d case; decreased peak runoff from the top slope and increased peak runoff from the side slope. Note however that the design peak flowrate for the scenarios indicating flow concentration is dictated by runoff from the top slope rather than runoff from the side slope, which increases the significance of the observed effect to the design of the rock layers.

DRAFT

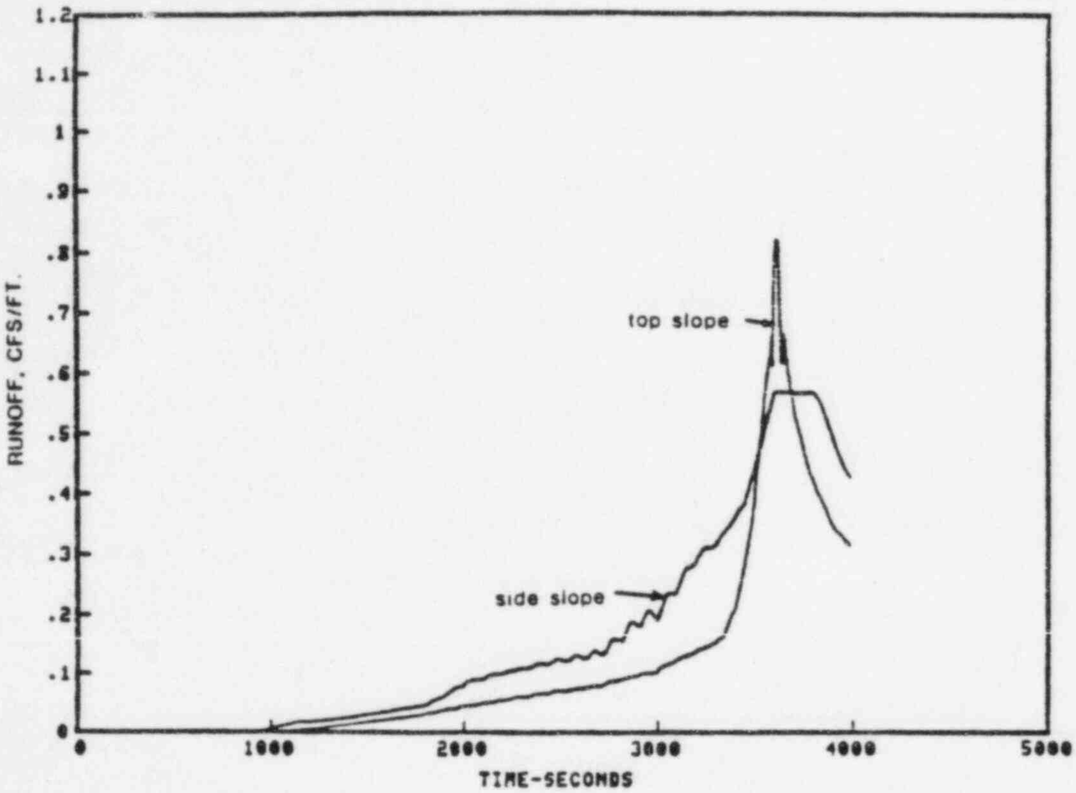


Figure 13 - Runoff for 1% Slump Case with Double Rock Diameter

Sensitivity to Layer Thickness

Peak runoff is sensitive to the ability of the flow to remain confined to the rock layer rather than overtop it. The ability of the rock layer to store and transport most of the runoff is a critical factor in the atteuation of peak flow from the slope. This effect will be diminished however if the rock layer

DRAFT

is too thin, its friction too great, or its porosity too small. This somewhat counter-intuitive result is demonstrated in Fig. 14, which is for the same conditions as the one dimensional Benchmark case, but with the thickness of the rock layer halved to 0.5 feet and 0.75 feet on the top and side slopes respectively. Peak flows for this case were calculated to be 0.44 and 0.42 CFS/ft. and 0.42 CFS for the toe and top respectively, as opposed to the 0.32 and 0.25 CFS/ft runoff for the case of the full layer thickness presented in Fig. 5.

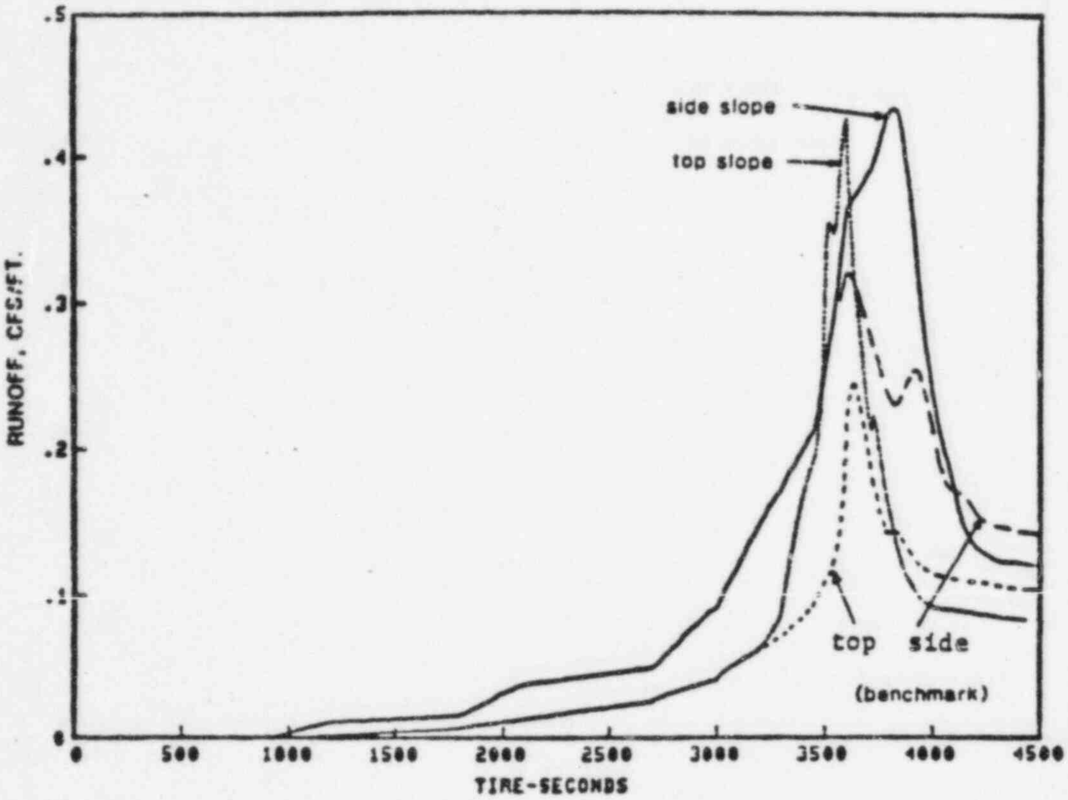


Figure 14 - Effect of Halving Layer Thickness, 1-d Model

why this?

The two-dimensional model was rerun for the case of the 1% slump, but with an essentially infinite layer thickness which eliminates the possibility of overtopping. The results of this run are shown in Fig. 10 along with the runoff for the normal rock layer thickness. Peak runoffs are dramatically lowered from the case in which the rock layer thicknesses were 1.0 ft and 1.5

DRAFT

ft on the top and side slopes respectively. Furthermore, the peak runoff occurs at the toe of the side slope and is no longer controlled by runoff from the top slope. The maximum rock layer thicknesses necessary for the Shiprock embankment to completely contain the peak flows are about 3.1 ft. on the lower end of the top slope and 1.3 ft. on the toe of the side slope. Smaller rock layer thicknesses would be adequate further up the slope from these points

see last paragraph on p. 22

Sensitivity of Runoff to Other Parameters

No specific numerical experiments have been conducted to measure the sensitivity of runoff to other paramenters of the model, but some inferrences can be drawn from groupings of terms which appear in the differential equations, Eqs. A1 and A2 in Appendix A. Friction within the rock layer is dependent on the grouping $Kv^2/gd^* n^2$, where K is the friction factor, v is the bulk velocity, g is the acceleration of gravity, d^* is the effective rock diameter, and n is the void fraction (porosity) between rocks. Reducing porosity n would both increase friction and reduce the water-carrying ability of the rock layer, forcing more flow to the surface. Uncertainty in n, however, is considerably smaller than uncertainty in the other parameters, so the sensitivity of runoff to n is probably less significant than the sensitivity to other parameters.

DRAFT

Friction factor K would affect peak runoff in the same way as an equivalent change in $1/d^*$. This factor depends largely on rock surface angularity; e.g., a layer of crushed rock will carry less water than a layer of rounded alluvial gravel, all other factors being equal.

C.2 - Design of Rock

The design of the rock diameter and layer thickness to resist runoff from the local PMP on the slope closely follows the methods employed by DOE. The calculations must generally proceed in an iterative manner, because peak runoff is also a ~~factor~~ function of ~~the~~ both rock diameter and layer thickness.

The Safety Factor method described in Section B.2 is recommended for the design of the top slope. The method is modified however to account for flow both through and over the rock layer. The thickness of the overtopping layer would be determined directly from the runoff model or from a rating curve such as Fig.15. The rationale for this modification is that the forces tending to dislodge rock resting on the top of the layer would not be a function of flows beneath the surface of the rock layer. The Modified Safety Factor method would apply only for cases where there was flow over the top of the rock layer.

DRAFT

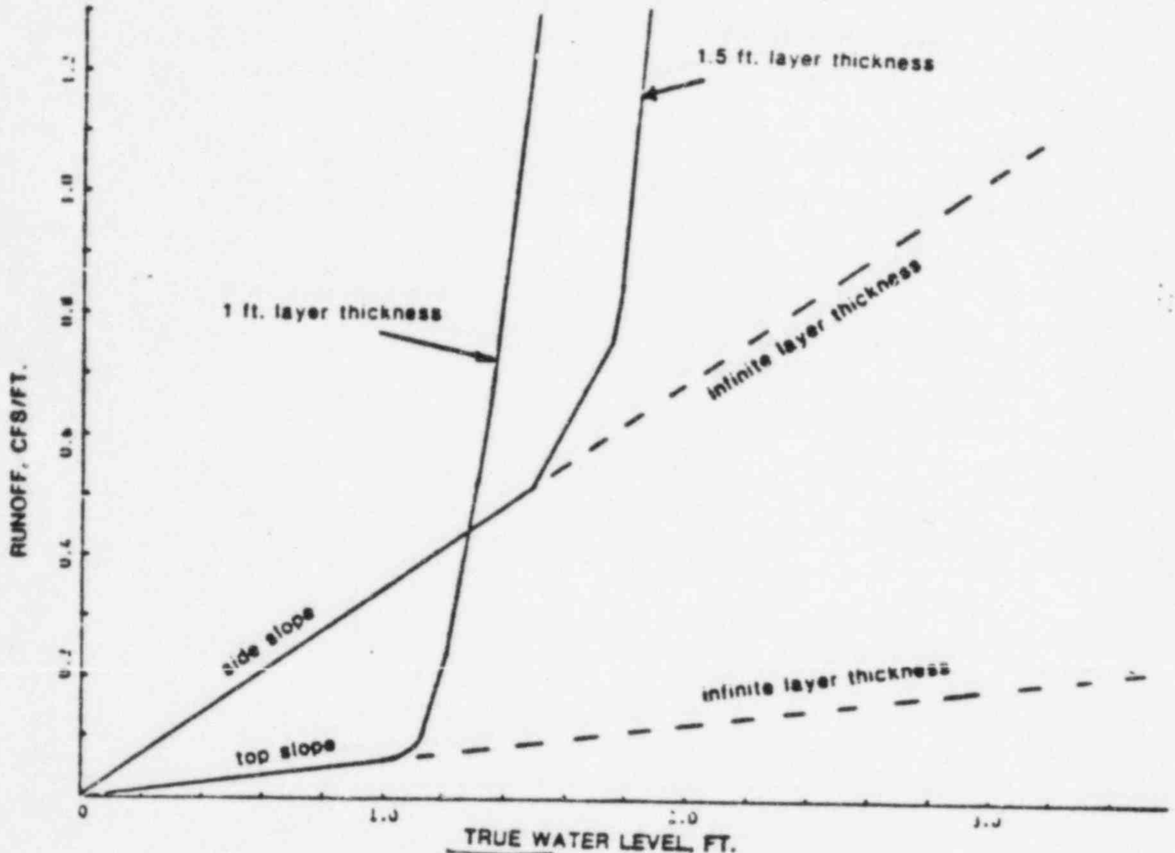


Figure 15 - Rating Curves for Example Problem

The Stephenson method described in Section B.2 is recommended for the design of rock on the side slopes. This formula accounts for flow both through and over the rock layer. It may also be used for the top layer, especially for those cases in which the runoff model predicts no overtopping.

is "overtopping" not the same as "flow over" the rock layer

DRAFT

Discussion

The two methods recommended for the design of the rock layer generally give considerably different answers. The Safety Factor method considers the stability of single rocks on an inclined plane. The diameter of the rock is determined by a balance of forces acting to overturn the rock and the forces of gravity and friction tending to keep the rock in place. The stability of a single rock however is smaller than of that of a rock in a layer of other rocks. Olivier (Ref.6) studied the stability of rockfill dams in a series of model flumes, and determined the flowrates at which the rocks began to move. He noted that the rockfill layers tended to collapse at flowrates which were 120% to 180% above the threshold flowrates for which the first motion of rocks within the layer was observed. Other factors which affect the stability of the rockfill structures included the rock size gradation and the slope. Olivier noted that rockfill layers constructed of rock with a uniform size tended to be more stable than rock layers constructed of poorly sorted rock. Also, rock layers constructed of larger rock placed on a layer of smaller gravel tended to be less stable than those cases where no gravel was present and the larger rock was placed on ______ instead. This observation might be a factor in considering the benefits of filter layers to prevent erosion.

DRAFT

The Safety Factors method appears to always predict design rock diameters which are considerably larger than the Stephenson method. The latter method should be applicable to the design of embankments, however, and is backed by some convincing experimental data. Olivier's experiments were conducted with relatively thin layers, usually 1 to 3 diameters of rock. Most of the flow therefore, must have occurred over the top of the rock layer rather than through it. Thicknesses of rock layers proposed for the tailings embankments are considerably greater than those used in Olivier's experiments. It is likely that the thicker layers would be more stable than the thinner layers because more flow would be contained under the the surface, which would reduce or eliminate stress on the surface of the rock layer.

false false

The determination of rock diameters and layer thicknesses for the top and side slopes will be demonstrated below, using ~~by means of~~ example. ~~for~~ the Shiprock case as an

C.3 - Example Calculations of Rock Armor For Shiprock

Assume for the Shiprock example that an independent geotechnical analysis has determined that the 1% inward slope scenario with the local PMP would be the design basis event. Other properties of the embankment are those given in

DRAFT

Table 1. Determine the adequacy of the rock to resist the calculated runoff.

See Fig. 10, p.

Solution

Top Slope

The design of the rock layer for the top slope will be illustrated using the Safety Factor method. The peak runoffs for the given case have been previously determined in Section C.1.b to be 1.01 CFS/ft. and 0.53 CFS/ft. on the top and side slopes, respectively. The 1.01 CFS/ft. runoff will therefore control the rock design for both the top and side slopes at the point where they meet. Note however that flowrates decrease both upstream and downstream of this point. It may therefore be acceptable to design the armor for lesser runoffs away from the slope break.

For the design basis flow, the depth of the water layer can be determined from the rating curve, Fig.15, to be 1.42 ft. The depth of flow over the top of the rock layer is therefore $\Delta h = (1.42 - 1.0) = 0.42$ ft. The shear stress from Eq.3 is:

$$\tau = 62.4 \times \Delta h \times S_y = 62.4 \times 0.42 \times 0.02 = 0.52 \text{ lb./ft}^2.$$

where S_y is the slope.

GRT:. Shiprock design change (following NRC comment, using HMR #49) shows peak flows (Weiland) to be 0.82 cfs/ft for top slope and 1.09 cfs/ft for side slope of embankment. Through flow was neglected in the current design and left as such in the proposed design change also.

DRAFT

ADH: Agree? [GRT] As soon as the water reaches the slide slope won't interstitial flow immediately drop & to <0.53 cfs/ft.?

see Fig. A-1. (Line sketch in pencil.)

The stability number η is calculated from Eq.4 :

$$\eta = \cos\alpha\ (1/SF - S_y/\tan\phi) = 1.0 \times (1/1.5 - 0.02/0.7) = 0.638$$

where α = the angle of the slope = $\tan^{-1} S_y$

SF = the safety factor = 1.5

ϕ = the angle of repose from Fig.3 = 35 degrees for smooth rock.

The d_{50} rock diameter is then determined from Eq. 5:

$$d_{50} = 21\ \tau/(S_s - 1)\ \gamma\ \eta = 21 \times 0.52/((2.65-1) \times 62.4 \times 0.638)$$
$$= 0.166 \text{ ft.} = 2 \text{ inches}$$

where S_s = the specific gravity of the rock = 2.65 gm/cc.

Using safety factor 1.0 we obtained d_{50} = 1.5 inches in the current design and is not changed under HME # 49.

Side Slope

The rock diameter for the side slope is determined by. Eq. 6:

$$d = \left\{ \frac{q\ S_{xy}^{7/6}\ n^{1/6}}{C\ g^{1/2} \left[(1-n)(S_s-1) \cos\alpha\ (\tan\phi - S_{xy}) \right]^{5/3}} \right\}^{2/3}$$

DRAFT

where C = the roughness coefficient which ranges from 0.22 for smooth gravel and pebbles to 0.27 for crushed rock. For this case C = 0.22.

If the angle of repose is assumed to be about 35 degrees, the rock diameter is determined to be:

$$d = \left\{ \frac{1.01\,(0.2)^{7/6}\,(0.35)^{1/6}}{0.22\,(32.2)^{1/2}\left[.65 \cdot 1.65 \cdot .981\,(.7 - .2)\right]^{5/3}} \right\}^{2/3} = 0.45 \text{ ft.} = 5.4''$$

Following the DOE example, the diameter calculated would correspond to the d_{30} rock size. If it is assumed that $d_{50} = 1.5d_{30}$, then $d_{50} = 8.1''$.

In our experience D50/D30 (side slopes) = 1.05 to 1.19 on the average with locally 1.90 maximum possible for similar size rock, using Corp of Engrs gradation

The stated d_{50} rock diameters for the top and side slopes were 1.5" and 4.5" respectively, which is somewhat less than the calculated rock diameters. If the calculated rock diameters of 2.0" and 8.1" were specified, it would not be necessary to repeat the runoff calculations, since it has been shown in Section C.1.c that increasing the rock size would have the effect of decreasing the peak runoff from the top layer, which is the design basis for both the top and side layers.

See following attached page

GRT

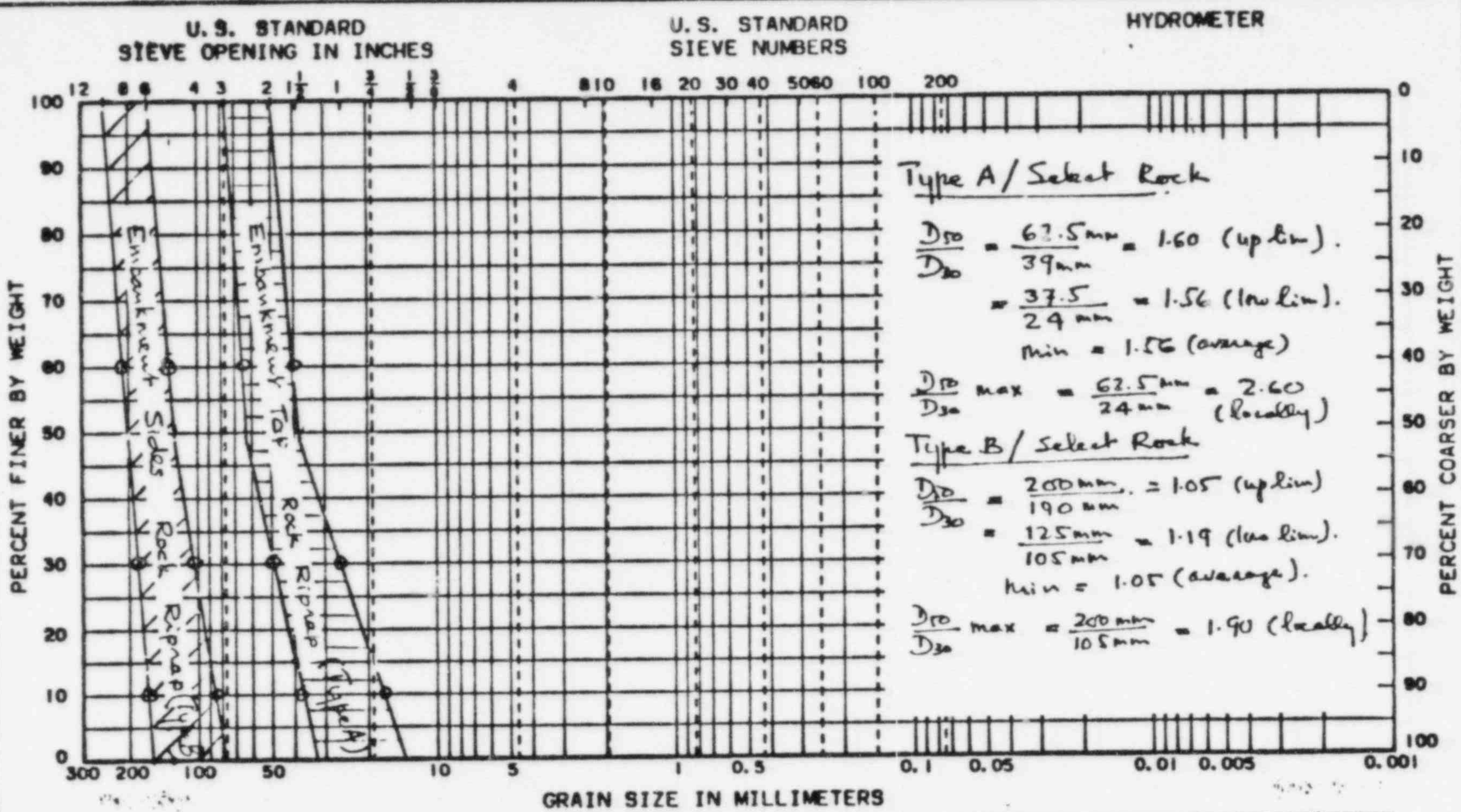
With regard to side slopes, our present design calls for D50 > 5 inches for the side slopes, and under HMR # 49 (proposed change) DRAFT, we will recommend D50 >8½ inch using the Safety Factor method.

ABH / 8-28-85

B-4

GRAIN SIZE ANALYSIS

INTERNATIONAL ENGINEERING COMPANY, INC.



Type A/ Select Rock

$\frac{D_{50}}{D_{30}} = \frac{62.5\,mm}{39\,mm} = 1.60$ (up lim).

$= \frac{37.5}{24\,mm} = 1.56$ (low lim).

min = 1.56 (average)

$\frac{D_{50}}{D_{30}}$ max $= \frac{62.5\,mm}{24\,mm} = 2.60$ (locally)

Type B/ Select Rock

$\frac{D_{50}}{D_{30}} = \frac{200\,mm}{190\,mm} = 1.05$ (up lim)

$= \frac{125\,mm}{105\,mm} = 1.19$ (low lim).

min = 1.05 (average).

$\frac{D_{50}}{D_{30}}$ max $= \frac{200\,mm}{105\,mm} = 1.90$ (locally)

COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

SAMPLE NO.	~~ELEV. OR~~ DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI
/ / / /	Rock Riprap	Embankment Side Slope				
====	Rock Riprap	Embankment Top				

PROJECT	UMTRA - Shiprock
JOB NO.	4005 Task 2.11
~~AREA~~	Embankment Final Design
HOLE NO.	Erosion Protection
DATE	2/25/85 ABH

ckd WYL 2/26/85

CAB110

It is interesting to note that the Stephenson method would have predicted a d_{50} rock diameter of only about 1 inch for the top slope. The Safety factor method would have predicted d_{50} of over 2 feet for the side slope.

See comment on bottom of preceding p. 37.

D. - Conclusions

The design of rock armor for embankments to resist the local PMP involves the calculation of runoff and the determination of the properties of the rock which can resist the calculated runoff. The staff has developed a set of mathematical models to calculate runoff from armored slopes. The models take into account the resistance to flow both through and on top of the armor layer. The models can be used to study the effects of various designs and failures of the embankments on the runoff caused by intense precipitation. Runoffs calculated from the models are subsequently employed in equations to determine the stability of rocks or rock layers to resist the erosive forces of the runoff.

Some of the conclusions which have been drawn from the results of the models for the case of the Shiprock tailings embankment are listed below:

DRAFT

1. The calculation of runoff must consider flow both through and over top of the armor layer.

2. Irregularities in the surface of the slopes may lead to large concentrations of flow along preferential paths, which would place more sever loads on the rock armor.

3. The peak runoff from the gentler top slope can frequently be more severe than the peak runoff from the steeper side slope, thereby controlling the design of the armor on both slopes. This condition may occur when the ability of the rock layer to carry the flow is inadequate, forcing the flow to overtop the rock layer. The most severe hydrologic stresses on the armor are likely to occur at the break between the top and side slopes for this situation.

4. Design factors which tend to diminish the peak runoff from the top slope include larger rock diameter and larger layer thickness. Degradation of the rock over the design lifetime of the embankments should be taken into consideration. The effects of flow concentration caused by geotechnical failure or slumping can be largely eliminated by having a large rock layer thickness.

DRAFT

5. There is no apparent basis for not including very intense, short duration rainfalls in the design basis hyetograph for the PMP. The staff included rates of precipitation as short as 2½ minutes for its studies, and based the 8 hour PMP on Hydromet 49 (Ref.7)

6. The DOE analysis for Shiprock did not consider the phenomena of flow through the rock layer or flow concentration. Furthermore, their analysis employed a less intense hyetograph than was used by the staff (Ref.2).

E. - Recommendations for Improvement

The usefulness of the methods presented in this report can be increased by investing the resources to resolve certain problems, and to document and clarify the methods so that they can be used by Applicants, Licensees and the NRC staff.

Several uncertainties were identified during the development of the NRC methods for the determination of runoff and design of the rock layer. Simplifying approximations had to be made in order to pursue the development of the models. Some of these uncertainties can be resolved through the collection of

experimental data and further literature review. The following areas should be addressed:

1. Collect data on flow resistance through and over the types of rockfill to be expected for tailings embankments. The experiments could be made in large flumes, properly scaled to assure turbulent conditions of flow. Data should be collected for flows which both overtop the rock layer and flows which are confined to the rock layer. Basic properties of the rock used, such as gradation, density and porosity should be determined for each experiment. Furthermore, the differences between carefully-controlled laboratory flumes and real slopes, caused by non-ideal construction practices in the field should be explored.

2. The applicability of Stephenson's method to tailings embankments should be determined. Specifically, research should address the question of why there is such a large discrepancy between the results predicted by the Safety Factor method and the Stephenson method. Furthermore, the proper application of these methods to graded rock, rather than rock of a single size, should be identified. The flume experiments for collecting data on flow resistance could possibly be used to determine flows at which the embankments would fail.

DRAFT

3. Mechanisms of slope failures leading to conditions for flow concentration should be explored. The scenarios in Section C.1.b are highly speculative.

4. The mathematical models should be verified and validated to the extent possible. Validation could be performed using data collected from the flume experiments discussed above. If possible, it should be determined whether oscillations noticed in some of the runs are manifestations of real hydrodynamic phenomena or are artifacts of the numerical solution. Sensitivity of the oscillations to varying grid spacing and the time step should be determined. A time centered numerical algorithm should be tried to replace the explicit algorithm presently being used in the two-dimensional model.

5. The computer programs which implement the mathematical models should be rewritten in FORTRAN and thoroughly documented. Clear users manuals should be prepared. The programs are presently written in BASIC.

6. Simplified methods for developing the slope armor should be developed if practical. The methods could be in the form of tables or nomograms developed from the numerical models, or simplified conservative models bounded by the results of the numerical models.

The principles developed in the NRC Methodology should be codified in Regulatory Guides, NUREG reports or both. Applicable portions of the research and model development should be published in the open scientific and engineering literature in order to disseminate the information and to seek peer review.

References

1. D.E. Overton, M.E. Meadows, Stormwater Modeling, Academic Press, New York, 1976

2. U.S. Bureau of Reclamation, Design of Small Dams, Second Edition, Dept. of Interior, Washington D.C., 1973

3. Jacobs Engineering, Analysis of Shiprock New Mexico Tailings Embankment, SHP-03-001, December 1983 (unpublished)
obsolete

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

5. D. Stephenson, Rockfill Hydraulic Engineering, Elsevier Scientific Publishing Company, Amsterdam, 1979

DRAFT

6. H. Olivier, "Through and Overflow Rockfill Dams - New Design Techniques", Proceedings, Institute of Civil Engineers, March 1967, pp 433 - 471

7. National Oceanic and Atmospheric Administration, "Hydrometeorological Report no. 49 - Probable Maximum Precipitation Estimates, Colorado River and Great Basin Drainages", U.S. Dept. of Commerce, Silver Spring MD, 1977

8. R.K. Linsley, M.A. Kohler, J.L.H. Paulis, Hydrology for Engineers, Second Edition, McGraw-Hill, New York, 1975

9. R.E. Wardwell, J.D. Nelson, S.R. Abt, and W.P. Staub, "Design Considerations for Long-Term Stabilization of Uranium Mill Tailings", in Mangement of Uranium Mill Tailings, Colorado State University, Fort Collins Co. 80523, Feb. 1-3, 1984

10. T.M. Leps, "Flow Through Rockfill" in Embankment Dam Engineering, Casagrande Volume, by R.C. Hirshfeld and S.J. Poulos (Editors), John Wiley and Sons, New York, pp 87-108, 1973

11. R.D. Hey, "Flow Resistance in Gravel Bed Rivers", Journal of the Hydraulics Division, ASCE, HY4, pp 365-379, April 1979

12. M.A. Stevens and D.B. Simons, "Stability Analysis for Coarse Granular Material on Slopes", Ch. 17 in River Mechanics, Edited and Published by H.W. Shen, P.O. Box 606, Fort Collins, CO 80521, 1971

DRAFT

APPENDIX (A)

APPENDIX A. THE RUNOFF MODEL

Introduction

Rain falling on an armored slope will largely flow downhill except for the fraction infiltrating the ground or evaporating. In flood routing studies for very small drainage basins it is customary to ignore infiltration losses, considering that the ground is totally saturated by an antecedent rainfall. The runoff will flow through the spaces between the rocks. If runoff is great enough, part of the runoff will overtop the rock layer.

Referring to Fig.A1, the flow of water on the slope may be described for a one-dimensional case (e.g., an infinitely wide plane) by a macroscopic mass and energy balance:

$$\frac{\partial(\xi v)}{\partial x} + n\frac{\partial \xi}{\partial t} = R \qquad \text{(A1)}$$

$$\frac{\partial \xi}{\partial x} + \frac{v}{g n^2}\frac{\partial v}{\partial x} + \frac{1}{g n}\frac{\partial v}{\partial t} + \frac{K v|v|}{n^2 d} - S_x = 0 \qquad \text{(A2)}$$

where ξ = water depth

APPENDIX A

A

APPENDIX A. THE RUNOFF MODEL

Introduction

Rain falling on an armored slope will largely flow downhill except for the fraction infiltrating the ground or evaporating. In flood routing studies for very small drainage basins it is customary to ignore infiltration losses, considering that the ground is totally saturated by an antecedent rainfall. The runoff will flow through the spaces between the rocks. If runoff is great enough, part of the runoff will overtop the rock layer.

Referring to Fig.A1, the flow of water on the slope may be described for a one-dimensional case (e.g., an infinitely wide plane) by a macroscopic mass and energy balance:

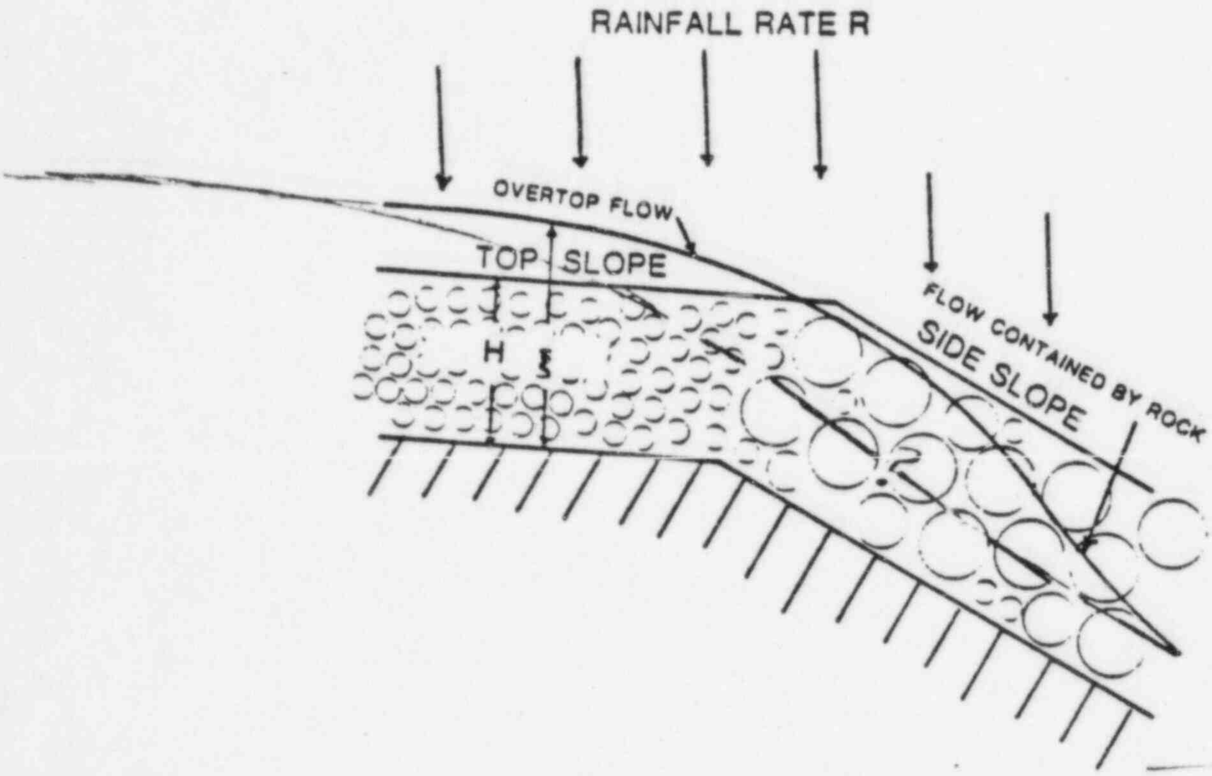
$$\frac{\partial(\xi v)}{\partial y} + n\frac{\partial \xi}{\partial t} = R \qquad \text{(A1)}$$

$$n\frac{\partial \xi}{\partial y} + \frac{v}{g n^2}\frac{\partial v}{\partial y} + \frac{1}{gn}\frac{\partial v}{\partial t} + \frac{K v|v|}{g n^2 d^*} - S_y = 0 \qquad \text{(A2)}$$

X

where ξ = water depth

DRAFT



See comment by GRT on bottom of p. 35

FIGURE A1 - RUNOFF FROM AN ARMORED SLOPE

DRAFT

Flow through rockfill is given by

$$i = \frac{KV^2}{gdn^2}$$

$$\text{or} \quad V = \sqrt{\frac{gidn^2}{K}}$$

where i = hydraulic gradient

and K = rock friction factor (ranges from 2 to 4),

n = rock void porosity (dimensionless)

t = time

R = rainfall rate

g = acceleration of gravity

K = friction factor (dimensionless).

d^* = representative rock diameter

ADH Follow?

These are know as the Saint Venant equations for shallow water waves, modified for flow through porous media (Ref. A1). For flow over the top of the rock layer, the depth ξ becomes a virtual depth; that is, the equivalent depth which the water would have to assume if the ~~rock layer were infinitely thick~~ - flow were totally through flow.

Resistance to Flow

Flow resistance through the rock layer is described by a quadratic function of velocity (Ref. A1):

$$\cancel{F = Kv|v|/n^2 d^*} \quad S_f = \frac{K v|v|}{g n^2 d^*} \qquad \text{(A3)}$$

where S_f = friction slope (resistance to flow).
= i (hydraulic gradient)

DRAFT

where K is a proportionality constant and d^* is the representative rock diameter. the proportionality coefficient K is a function of rock shape, roughness and Reynolds number Re:

Use $\vartheta = 10^{-5}$ ft²/sec for water

$$Re = V d^* / (n \vartheta) \tag{A4}$$

where ϑ is the kinematic viscosity. (For the large Reynolds numbers expected through the rock layers, K appears to be a function only of roughness and shape.) Stephanson (Ref. A1) empirically fitted available flume and field data, as shown in Fig. A2, and suggests the formula:

$$K = 800/Re + K_t \tag{A5}$$

where $K_t = 1$ for smooth polished spheres, 2 for semi-rounded rocks, and 4 for angular rocks. Stephanson suggests that the representative rock diameter d^* should be taken as an average based on the wetted area of the rock.? He suggests the harmonic mean diameter:

$$d^* = N / \sum_{i=1}^{N} 1/d_i = \frac{N}{\sum_N \left(\frac{1}{D_i}\right)} \tag{A6}$$

X

where d_i is the diameter of the rock in the i^{th} catagory.

better yet

$$d^* = \frac{\sum_N \omega_i}{\sum_N \left(\frac{\omega_i}{D_i}\right)} = \frac{1}{\sum_N \left(\frac{\alpha_i}{D_i}\right)}$$ ← weighted harmonic mean.

DRAFT

where $\alpha_i = \frac{\omega_i}{\sum_N \omega_i}$ = normalized weighting function for category i.

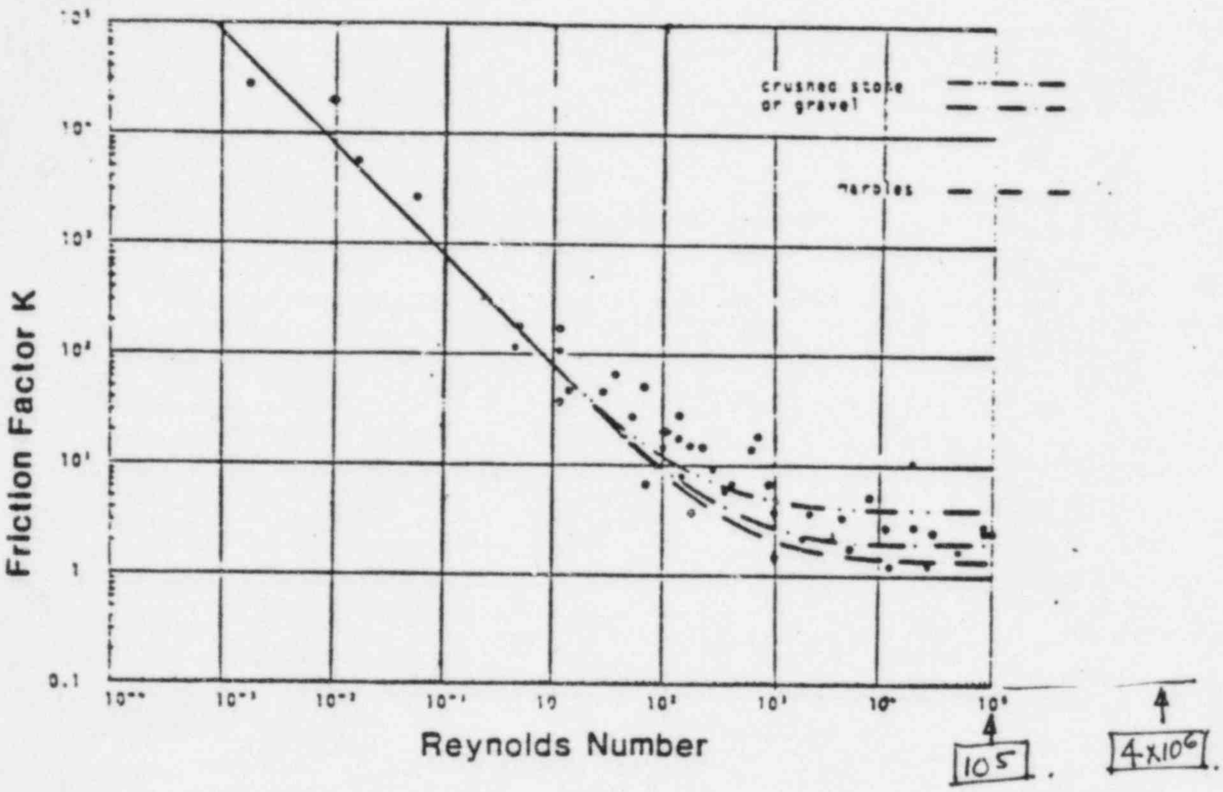


Figure A2 - Friction Factor for Rockfill

Ex. 1. $D^* = 12$ inches (1 foot)

$V = 10$ ft/sec

$n = 0.25$

$$Re = \frac{V \cdot D^*}{n \nu}$$

$$= \frac{10 \text{ ft/sec} \cdot 1 \text{ ft}}{0.25 \, (10^{-5} \text{ ft}^2/\text{sec})} = 4 \times 10^6$$

Ex. 2. $D^* = 8$ inches (0.67 ft)

$V = 0.70$ ft/sec

DRAFT

$n = 0.40$

$$Re = \frac{0.70 \text{ ft/sec} \times 0.67 \text{ ft}}{0.40 \, (10^{-5} \text{ ft}^2/\text{sec})} \cong 10^5$$

$$K = \frac{800}{Re} + K_t$$

$\simeq 2.0$ for subrounded rocks.

Darcy-Weisbach eqn for open channel flow states

$$f = \frac{8gRS}{U^2}$$

$$\text{or} \quad U = \sqrt{\frac{8RSg}{f}}$$

where f = Darcy-Weisbach friction factor

and is given by Colebrook-White formula

$$\frac{1}{\sqrt{f}} = 2.03 \log\left(\frac{13.46R}{3.5k}\right).$$

for turbulent flow.

Leps (Ref.2) however, suggests that d_{50} is a good representation of d^*, especially if the fraction of fines is not too great.

Well-documented experiments on flow resistance in rock layers are not readily available, and those data which are available have wide scatter. The sensitivity of the runoff results to changes in the rock friction relationships are demonstrated in Section C.1.c in the text.

Flow over top of rocks

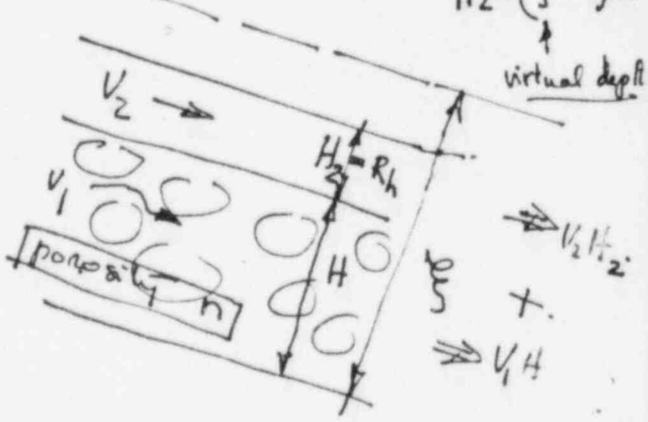
The resistance to flow through the rock layer is much higher than flow resistance for open channel or overland flow. Therefore, the resistance to flow will decrease once the water depth exceeds the thickness of the rock layer. The model accounts for this reduction in flow resistance by calculating an equivalent K based on the resistance to flow within and above the rock layer.

The total flow Q past a point on the slope is the sum of the flow ~~through~~ under the rock layer, Q_1 and ~~over~~ above the rock layer, Q_2:

$$Q = Q_1 + Q_2 = v_1 H + v_2 \overbrace{(\xi - H)}^{\text{water depth over top of rock}} n \qquad \text{(A7)}$$

$H_2 = (\xi - H) n$

virtual depth



$V_{1 void} \times$ porosity

where v_1 is the equivalent velocity in the rock layer,
v_2 is the actual velocity in the over-top layer, and
H is the thickness of the rock layer.

The velocity over the top of the rock layer is calculated using the Darcy-Weisbach equation for flow resistance in open channels (Ref.A4):

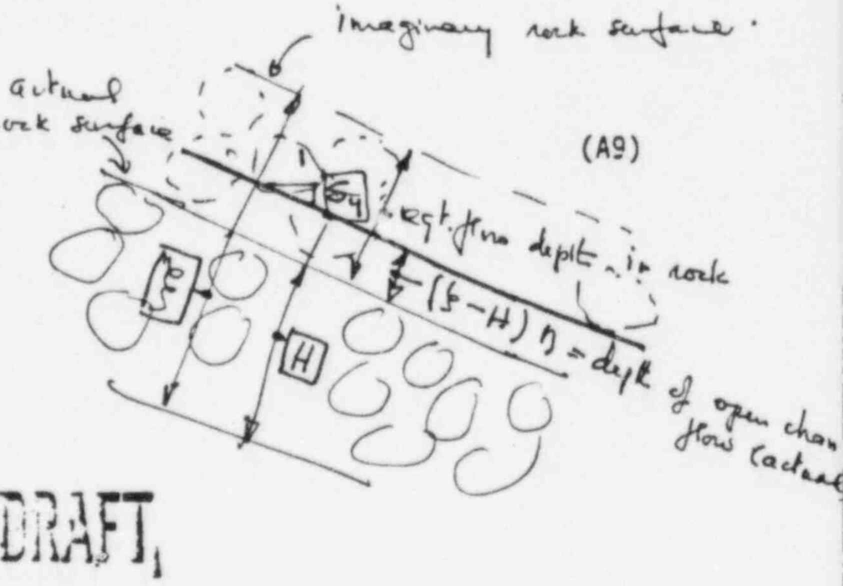
explain and elaborate with sketch if possible. non-uniform flow.

$$v_2 = \left[\frac{8 g R_h \left(S_y - n \frac{\partial \xi}{\partial y} \right)}{f} \right]^{1/2} \tag{A8}$$

where R_h is the hydraulic radius, and f is the Darcy-Weisbach friction factor. The water depth ξ and the gradient of the the water surface $\partial\xi/\partial y$ in Eq. A2 must be reduced by the porosity for flows above the top of the rock layer.

For a wide slope, the hydraulic radius may be approximated as the water depth over the top of rock:

$$R_h = n\,(\xi - H) \tag{A9}$$



DRAFT

The Darcy-Weisbach friction factor is dependent on the Reynolds number, rock diameter and roughness. For high Reynolds number as would be expected in the present case, the rock becomes hydraulically smooth, and only the form drag of the rock protruding into the flowing water will be important. Hey (Ref.A4) presents a correlation of f in terms of R and the d_{84} rock diameter (i.e., 84% of rock is finer) for riffle flow in gravel stream beds:

$$1/\sqrt{f} = 2.03 \log_{10}(13.46\ R_h\ /3.5\ d_{84}) \qquad \text{(A10)}$$

Agreement with field data is excellent as shown in Fig. A3.

for R/D_{84} (relative roughness) greater than 1.

for Shiprock embankment side slopes,
typically D_{50} = 5 to 8 inches.
D_{84} = 6 to 10 inches. (use $7\frac{1}{2}$ inches)
$R = \dfrac{0.12 \text{ to } 0.25 \text{ ft.}}{1\frac{1}{2} \text{ to } 3 \text{ inches}}$

then $R/D_{84} = \dfrac{1\frac{1}{2}}{7\frac{1}{2}}$ to $\dfrac{3}{7\frac{1}{2}}$
ie. 0.2 to ~0.5.

and $\dfrac{13.46\,R}{3.5\,D_{84}}$ = 0.77 to 1.9 (say 0.75 to 2).

but log 1 = 0.

DRAFT

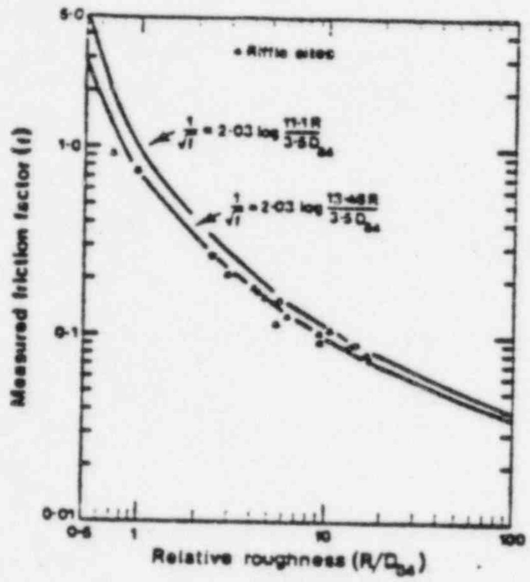


Figure A3 - Darcy-Weisbach Friction Factor for Riffle Flow (Ref. A4)

Equation A10 is based on natural riffle stream data for rounded alluvial rocks. It should be well-suited for the present case, since roughness of the rock surface has been found to be relatively unimportant at the high Reynolds numbers expected on the relatively-steep slopes (compared to stream beds) of the tailings embankments. Combining the expressions for the velocities within the rock layer (A3) and over the top of the rock layer (A8) with Eq. A10 (A7) gives the following expression:

roughness of rock (surface) relatively unimportant.

According to our experience at Shiprock, R/D_{84} ranged from 0.2 to 0.5 (typically) — well outside the range of plotted data. Also the Colebrook-White eqn (A10) has a singularity at R/D_{84} = 0.26.

i $\quad gi \quad S_y - n\frac{\partial \xi}{\partial y}$

$$\xi\left(\frac{g i d^* n^2}{K'}\right)^{1/2} = H\left(\frac{d^* n^2}{K}\right)^{1/2} + (\xi - H)n\left(\frac{8g(\xi-H)n}{f}\right)^{1/2} \quad \text{(A11)}$$

friction factor determined by Colebrook-White eqn. see (A10), page 6.

where K' is the effective resistance factor for the total layer. Solving for K' gives: (assuming $\underbrace{i = S_y}_{\text{saturated flow uniform flow in rock}}$ & $\underbrace{\left|\frac{\partial \xi}{\partial y}\right| << S_y}_{\text{gradually varied flow}}$)

$$K' = \frac{d^* \xi^2}{\left[H\left(\frac{d^*}{K}\right)^{1/2} + (\xi-H)\left(\frac{8(\xi-H)n}{f}\right)^{1/2}\right]^2} \quad \text{(A12)}$$

friction factor

Equations A1 and A2 are solved with the effective value, K' substituted for K when ξ is greater than the rock layer thickness H. Note, however that for small values of (ξ - H), K' may be greater than K, as shown in Fig. A4. This is not realistic, since friction is likely to only decrease as the flow overtops the rock layer. Therefore, the value of K' is limited to K: ?

$$K' \leq K \quad \text{(A13)}$$

At Shiprock, our estimates of through flow varies from 2 to 50% of peak runoff (PMP – Design of Small Dams) – with median estimate varying from 4 to 10 percent. For the high peak flows using HMR #49 and durations less than 5 min, the through flow estimates will be even less than the above values.

DRAFT

With above in mind, we can safely assume that effective resistance factor K' will be controlled by f (for surface flow) and hence previous objections to f (page 7) also apply to K' indirectly.

206/RC/2/8/84/4

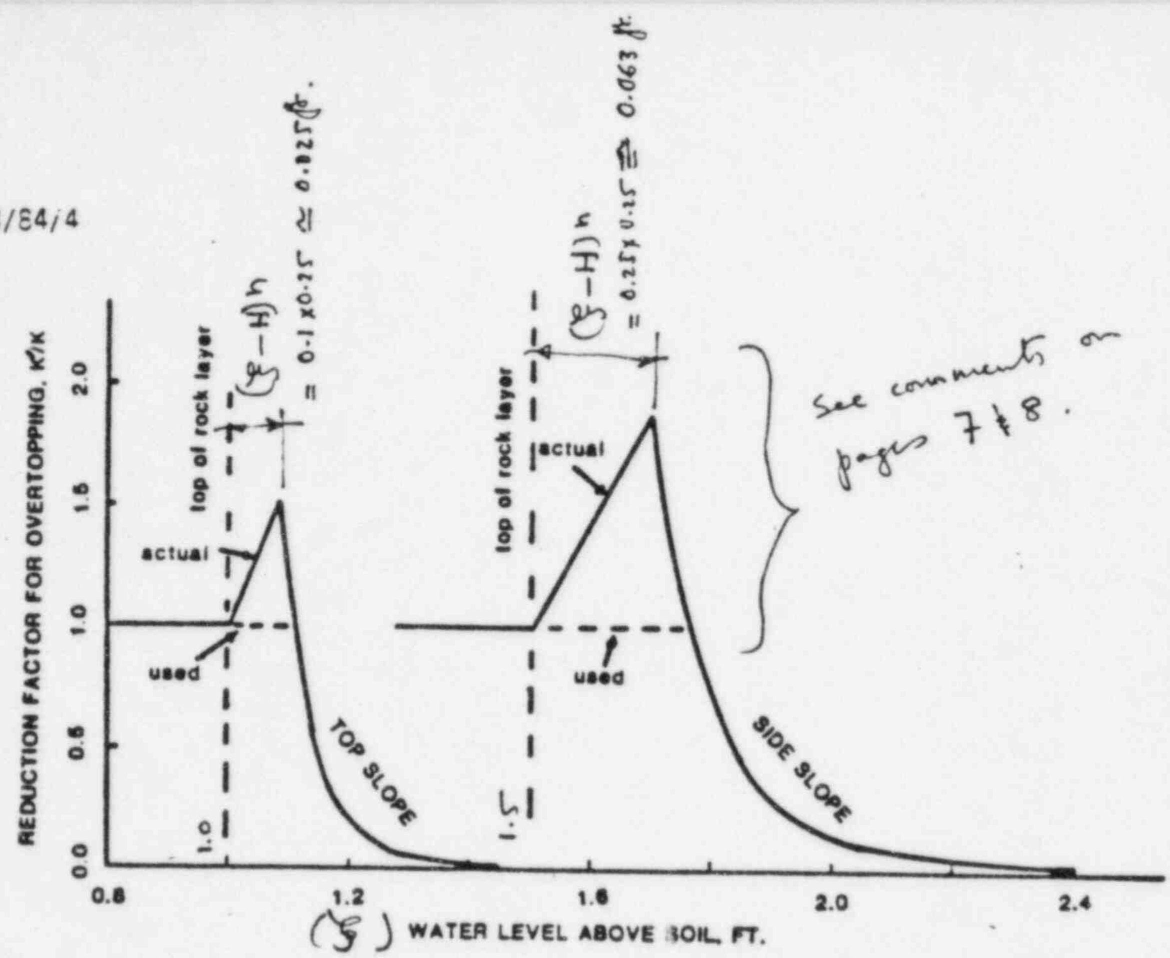


Figure A4 - Reduction in Flow Resistance caused by Overtopping

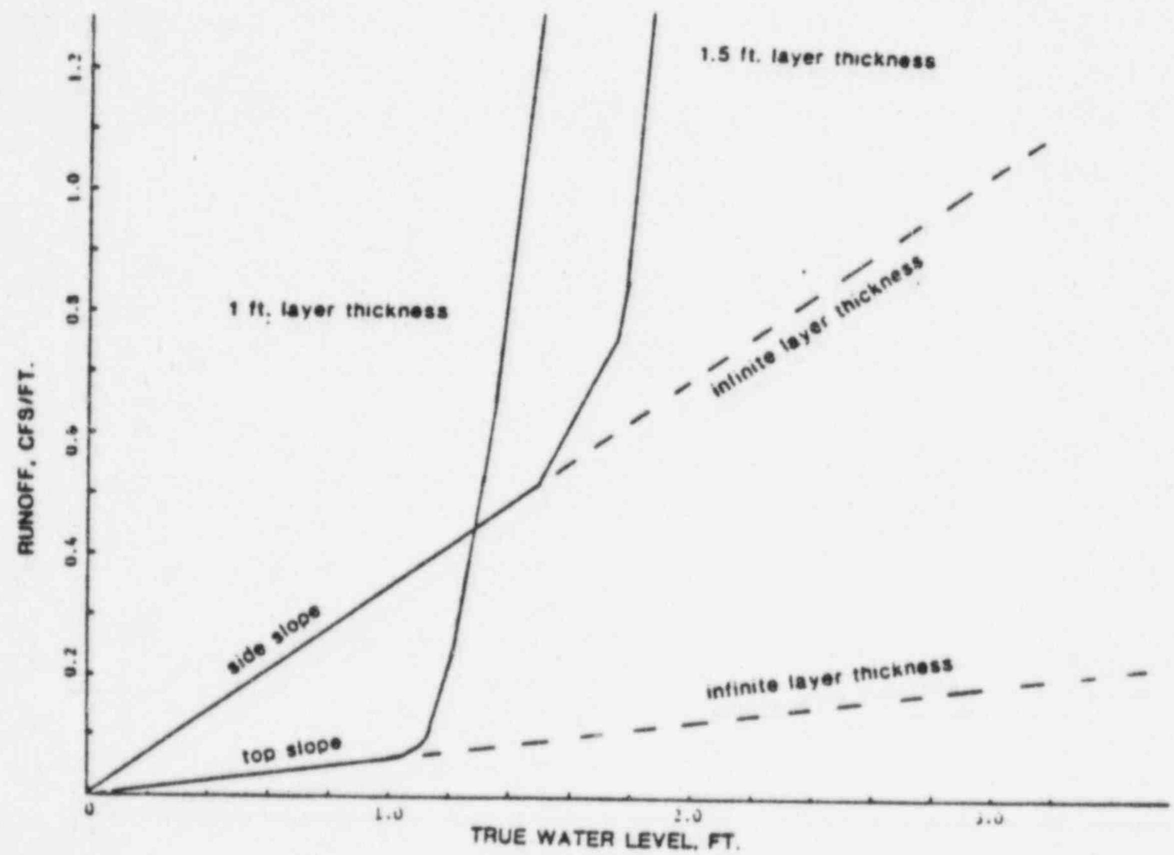


Figure A5 - Rating Curves for Shiprock Example

DRAFT

Rating curves for flowrate vs. water depth for the Shiprock example are shown in Fig. A5. The much higher carrying ability of the over-top layer is evident from this figure.

Boundary Conditions

The runoff problem is set up in terms of straight upper and lower slopes. The upper slope is usually gentle (2 to 5%), while the lower slope is usually steep (about 20%). The upper end of the top slope is assummed to be a no-flow boundary:

$$v=0, \qquad \partial v/\partial y = 0 \qquad \text{(A14)}$$

The break point between the upper and lower slopes does not require a formal boundary condition but in order to avoid sharp discontinuities between the intrinsic properties of the two slopes in the numerical solution, the values of slope, rock diameter and rock layer thickness are averaged at that point.

The downstream boundary of the lower slope is the only remaining boundary condition. Two boundary conditions were tried; (a) a critical flow boundary and (b) a normal flow boundary. The critical flow boundary condition assumed

that the flow on the slope was subcritical(generally the case) and approached critical flow (i.e., Froude number = 1) at the downstream terminus:

$$\xi_c = (q^2/g\, n^2)^{1/3} \tag{A15}$$

This boundary condition would apply for the condition of water emerging from the rock face into air, as in the case of a dry channel at the downstream end of the slope.

The normal flow boundary condition considers that the depth of the water layer is determined only by the balance between friction and gravity.

$$\xi_{\cancel{c}\,n} = q\,(K/g\, d^{*} n^2\, S_y)^{\frac{1}{2}} \tag{A16}$$

The normal flow boundary condition was eventually chosen for all runs. It had the distinct advantage of causing much smaller numerical perturbations to the flow equations than was the case for the critical flow boundary condition. Calculated values of runoff from the modeled slope were nearly indistinguishable between the normal flow and critical flow boundary conditions.

DRAFT

Kinematic Approximation

Many open channel and overland flow problems have been solved using the "kinematic flow" approximation of the Saint Venant equations (Ref. ~~A5~~ ster). Kinematic flow assumes that frictional and gravitational forces exactly balance each other, and that the inertial terms are negligible. Under this assumption, Eq. A2 becomes: steady uniform flow approximation.

$$n\frac{\partial \xi}{\partial y} + \frac{K v|v|}{g n^2 d^*} - S_y = 0 \qquad \text{(A17)}$$

Equation A17 can be solved directly for v if the gradient is positive:

$$v = \sqrt{(S_y - n\,\partial\xi/\partial y)\,(n^2 g\, d^*/K)} \qquad \text{(A18)}$$

The kinematic approximation greatly simplifies the numerical solution and increases its numerical stability. The inertial terms from Eq. A2 have been found to be negligable for the Shiprock example.

DRAFT

Numerical Solution

Equations ~~A2~~ A1 and A18 are solved by means of a staggered grid, time centered finite difference method. The grid for this method is illustrated in Fig A6. The variables v, S_y, d and K are represented between nodes, while the water depth ξ is represented on the nodes. This arrangement minimizes the distance over which the finite difference operator must be taken for the gradient $\partial\xi/\partial y$, and also allows a compact, straightforward matrix inversion in the numerical solution.

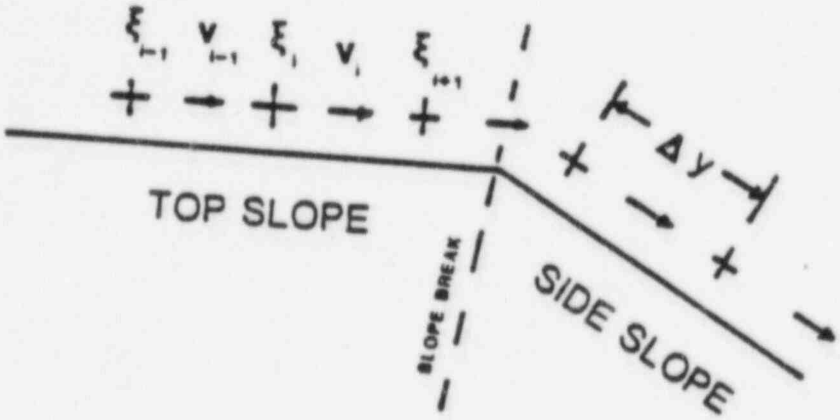


Figure A6 - One-Dimensional Finite Difference Grid

DRAFT

A two-step, implicit-explicit algorithm is used for the numerical solution. The first step is implicit, and takes the water level and velocity from time t to time t+ Δt. Equations ~~A2~~ A1 and A18 are written:

$$\xi_i^{n+1} = \xi_i^n + \frac{R\Delta t}{n} - \frac{\Delta t}{n\Delta y}\left[\frac{(\xi_i^{n+1}+\xi_{i+1}^{n+1})v_i^n}{2} - \frac{(\xi_i^{n+1}+\xi_{i-1}^{n+1})v_{i-1}^n}{2}\right] \quad \text{(A19)}$$

$$v_i^n = \sqrt{\frac{g d_i n^2}{K_i}\left(S_{yi} - n\frac{\xi_{i+1}^n - \xi_i^n}{\Delta x}\right)} \quad \text{(A20)}$$

what are supscript n and n+1 for? time steps?

where t is the time step

Δy is the grid spacing

ξ_i^n is the water level at point i

v_i^n is the velocity at point i+½

S_{yi} is the slope at point i+½

K_i is the friction factor at point i+½

d_i is the effective rock diameter at point i+½

g is the acceleration of gravity

Not reviewed beyond this page.

[illegible signature]

9/5/85.

DRAFT

Equation A19 can be rewritten in matrix form

$$\begin{bmatrix} \ddots & \ddots & \ddots & & \\ & a_{i,i-1} & a_{i,i} & a_{i,i+1} & \\ & & \ddots & \ddots & \ddots \end{bmatrix} \begin{bmatrix} -- \\ \xi_i^{n+1} \\ -- \end{bmatrix} = \begin{bmatrix} -- \\ b_i \\ -- \end{bmatrix} \qquad \text{(A21)}$$

or

$$\bar{\bar{A}}\,\bar{Y} = \bar{B} \qquad \text{(A22)}$$

The superscripts n and n+1 are the time levels, corresponding to times t and $t + \Delta t$ respectively.

where

$$a_{i,i-1} = -v_{i-1}^{n}\,\Delta t/2n\Delta y$$

$$a_{i,i} = 1 + (\Delta t/2n\ \Delta t)\ (v_i^{n} - v_{i-1}^{n})$$

$$a_{i,i+1} = \Delta t\ v_i^{n}/2\Delta x$$

DRAFT

$$b_i = \xi_i^{\,n} + \Delta t\ R/n$$

The boundary conditions on Eqs. A19 and A20 are represented as elements of the $\overline{\overline{A}}$ matrix and $\overline{B}$ vector:

$$a_{1,1} = 1$$

$$a_{1,2} = 1$$

$$b_1 = 0$$

$$a_{N,N} = 1 + \Delta t\ (g\ d_N\ S_{yN}/\Delta y\ K_N)$$

$$a_{N,N-1} = -v_{i-1}\ \Delta t/2n\Delta y$$

$$b_N = \xi_N^{\,n} + R\Delta t/n$$

Matrix $\overline{\overline{A}}$ is tridiagonal. Equation A22 is solved for $\overline{Y}$ by Gaussian elimination using the Thomas algorithm (Ref.A6)

DRAFT

The second part of the numerical solution is used to bring the water level and velocity from timestep t+Δt to t+2Δt:

$$\xi_i^{n+2} = \xi_i^{n+1} + \frac{R\Delta t}{n} - \frac{\Delta t}{n\Delta x}\left[\frac{(\xi_i^{n+1} + \xi_{i+1}^{n+1})}{2} v_i^{n+1} - \frac{(\xi_i^{n+1} + \xi_{i-1}^{n+1})}{2} v_{i-1}\right] \qquad \text{(A24)}$$

$$v_i^{n+1} = \sqrt{\frac{g d_i n^2}{K_i}\left(S_{yi} - \frac{(\xi_{n+1}^{n+1} - \xi_n^{n+1})}{\Delta y}\right)} \qquad \text{(A25)}$$

Equations A23 and A24 are solved explicitly in terms of water levels and velocities from the previous timestep. Boundary conditions are represented by the following explicit equations:

$$\xi_1^{n+2} = \xi_2^{n+2} \qquad \text{(A25)}$$

$$v_i^{n+1} = 0 \qquad \text{(A26)}$$

$$\xi_N^{n+2} = \xi_N^{n+1} + \frac{\Delta t R}{n} + \frac{\Delta t}{2n\Delta y}\left(\xi_i^{n+1} + \xi_{i-1}^{n+1}\right) v_{i-1}^{n+1} - \frac{\Delta t}{n\Delta y}\xi_N^{n+1}\sqrt{g d_N n^2 S_{yN} / K} \qquad \text{(A27)}$$

DRAFT

The friction factor K is adjusted according to Eq. A11 if the water level averaged over the middle of the node exceeds the rock layer thickness. In addition, if the water layer thickness at any node exceeds the rock layer thickness H, the effective depth ξ and the surface water gradient term $\partial\xi/\partial y$ must be reduced by the rock porosity:

$$\xi' = H + (\xi - H)\, n \tag{A28}$$

$$\partial\xi'/\partial y = n\, \partial\xi/\partial y \tag{A29}$$

Two Dimensional Model

A two dimensional, areal x,y model was developed for cases which cannot be represented as simple, infinitely wide tilted planes. The two dimensional model can be used to study the phenomenon of flow concentration caused by irregular basin shapes and slumping of the slopes. It is presently limited to slopes which can be represented by up to 4 tilted subslopes, which are symmetrical around the vertical axis, as illustrated in Fig. A7.

The two dimensional kinematic equations are given below:

DRAFT

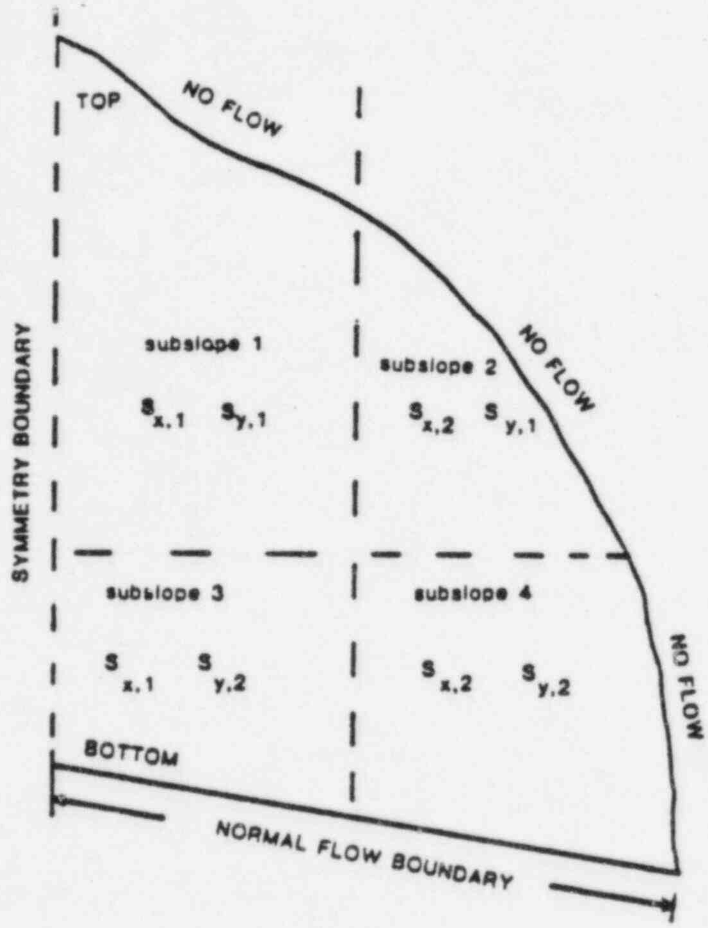


Figure A7 - Two-Dimensional Model of Slopes

DRAFT

$$\frac{\partial}{\partial x}(\xi u) + \frac{\partial}{\partial y}(\xi v) + n\frac{\partial \xi}{\partial t} = R \tag{A30}$$

$$\frac{K u\sqrt{u^2+v^2}}{g d n^2} = S_x - \frac{\partial \xi}{\partial x} \tag{A31}$$

$$\frac{K v\sqrt{u^2+v^2}}{g d n^2} = S_y + \frac{\partial \xi}{\partial y} \tag{A32}$$

For the present problem, the boundary conditions are represented as free slip conditions on the borders, no flow past the solid borders, and normal flow on the downstream end of the lower slope, as shown in Fig. A7.

Numerical Solution

The numerical solution of the two dimensional model as presently implemented employs the "Leapfrog" explicit algorithm (Ref.A7). This method was chosen over the implicit-explicit algorithm employed for the one dimensional model because it was much easier to program, and appeared to give acceptable results

DRAFT

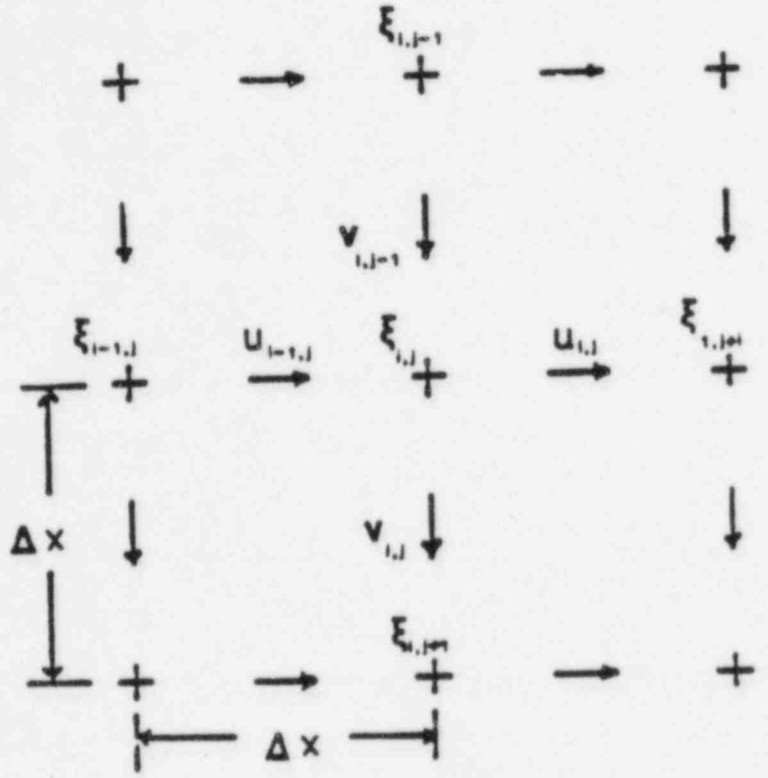


FIGURE A8 - TWO DIMENSIONAL FINITE DIFFERENCE GRID

DRAFT

when compared to problems solvable by the one dimensional model. The time centered, implicit-explicit solution for the two dimensional case may be programmed at a later time, however.

The staggered finite difference grid employed for the two dimensional model is illustrated in Fig. A8. The finite difference grid blocks are square, and of equal size throughout. The variables in the finite difference equations are defined on the corners of the grid blocks as shown in this figure.

The continuity equation, A29 is represented in finite difference form:

$$\begin{aligned}\xi^{n+1}_{i,j} = \xi^{n}_{i,j} + \frac{R\Delta t}{n} + (\xi^{n}_{i-1,j} + \xi^{n}_{i,j})\, u^{n}_{i,j} \frac{\Delta t}{2n\Delta x} \\ - (\xi^{n}_{i+1,j} + \xi^{n}_{i,j})\, u^{n}_{i,j} \frac{\Delta t}{2n\Delta x} + (\xi^{n}_{i,j-1} + \xi^{n}_{i,j})\, v^{n}_{i,j-1} \frac{\Delta t}{2n\Delta x} \\ - (\xi^{n}_{i,j+1} + \xi^{n}_{i,j})\, v^{n}_{i,j} \frac{\Delta t}{2n\Delta x}\end{aligned} \quad \text{(A33)}$$

The solution of the velocity equations A31 and A32 is more difficult than for the case of the one dimensional model, since u and v are coupled through the absolute velocity term $(u^2 + v^2)^{\frac{1}{2}}$. Since the v velocity will almost certainly be much larger than the u velocity in these runoff calculations,

DRAFT

Eq.A31 is solved for v, using the u and v velocities from the previous timestep in a correction factor:

$$v_i^{n+1} \left(\frac{g n^2}{k C'} |S| \right)^{1/2} \operatorname{Sign}(S) \tag{A34}$$

$$\text{where } C' = \left[1 + \left(\frac{\overline{\overline{U}}^{x\,\kappa\,y}}{V} \right)^2 \right]^{1/2} \tag{A35}$$

$$\overline{\overline{U}}^{x\,\kappa\,y} = \frac{u_{i,j}^n + u_{i-1,j}^n + u_{i,j-1}^n + u_{i-1,j-1}^n}{4} \tag{A36}$$

$$V = \sqrt{\left(\overline{\overline{U}}^{x\,\kappa\,y}\right)^2 + \left(v_{i,j}^n\right)^2} \tag{A37}$$

$$S = S_{y,j} - \frac{\left(\zeta_{i,j+1}^{n+1} - \zeta_{i,j}^{n+1} \right)}{\Delta x} \tag{A38}$$

The u velocities are then solved once all of the v values have been generated:

$$u_{i,j}^{n+1} = \left[S_x - \frac{\zeta_{i+1,j}^{n+1} - \zeta_{i,j}^{n+1}}{\Delta x} \right] \frac{g n^2 V}{k d_j} \tag{A39}$$

$$\text{where } V = \sqrt{\left(u_{i,j}^n\right)^2 + \left(\overline{V}^{x\,\kappa\,y}\right)^2}$$

$$\overline{V}^{x\,\kappa\,y} = \frac{v_{i,j}^{n+1} + v_{i+1,j}^{n+1} + v_{i,j-1}^{n+1} + v_{i-1,j-1}}{4}$$

DRAFT

Velocities normal to all borders except the downstream boundary are defined as zero. The gradient $\partial\xi/\partial x$ is zero at the ends of each row vector. The gradient $\partial\xi/\partial y$ is zero at the end of each column vector. Normal flow in the +y di-ection is assumed at the downstream end of each column vector, and is implemented in the finite difference solution as:

$$\xi^{n+1}_{i,N} = \xi^{n}_{i,N} + \frac{R\Delta t}{n} + \frac{\Delta t}{2n\Delta x}\Big[\left(\xi^{n}_{i-1,N} + \xi^{n}_{i,N}\right)u^{n}_{i-1,N} - \left(\xi^{n}_{i+1,N} + \xi^{n}_{i,N}\right)u^{n}_{i,N} + \left(\xi^{n}_{i,N-1} + \xi^{n}_{i,N}\right)v^{n}_{i,N-1}\Big] - \frac{\Delta t}{n\Delta x}\left(\xi^{n}_{i,N}\sqrt{\frac{g d_N n^2 S_{yN}}{k}}\right) \quad \text{(A40)}$$

References, Appendix A

1. D. Stephenson, Rockfill Hydraulic Engineering, Elsevier Scientific Publishing Company, Amsterdam, 1979

DRAFT

2. T.M. Leps, "Flow Through Rockfill" in Embankment Dam Engineering, Casagrande Volume, by R.C. Hirshfeld and S.J. Poulos (Editors), John Wiley and Sons, New York, pp 87-108, 1973

3. D.E. Overton, M.E. Meadows, Stormwater Modeling, Academic Press, New York, 1976

4. R.D. Hey, "Flow Resistance in Gravel Bed Rivers", Journal of the Hydraulics Division, ASCE, HY4, pp 365-379, April 1979

5. E.M. Morris and D.A. Woolhiser, "Unsteady One-Dimensional Flow over a Plane: Partial Equilibrium and Recession Hydrographs", Water Resources Research, Vol.16, no.2, pp 355-360, April 1980

6. L. Lapidus, Digital Computers for Chemical Engineers, McGraw-Hill, 1962

7. A. Grammeltved, "A Survey of Finite Difference Schemes for the Primitive Equations for Barotropic Fluids", Monthly Weather Review, vol.97,no.5