WM s/f (426.1) DGillen, WMLU
WMGT r/f TJohnson, WMGT
NMSS r/f
RBrowning
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JBunting
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MEMORANDUM FOR: LEO B. HIGGINBOTHAM, CHIEF
WMLU

FROM: MALCOLM R. KNAPP, CHIEF
WMGT

SUBJECT: RESPONSE TO TAR OF JANUARY 2, 1986

In response to your request, we have reviewed DOE's letter on the rainfall-duration curve for UMTRAP embankments transmitted to DOE on October 18, 1985 (Ref.1). We acknowledge a typographical error in the equation presented. A term, F_i was inadvertently omitted from the equation and table. The revised table, attached, includes the corrected equation and a column of F_i values. The curve fit originally presented was correct.

We have compared the NRC spline fit to the regression equation suggested by DOE. We note, however, that there is an apparent error in the DOE equation. The exponents of both coefficients should be preceded by minus signs. The comparison is presented below for 2.5 to 15 minute times of concentration:

Time of concentration minutes	Fraction of 1 hour PMP		
	NRC	DOE	NRC/DOE
2.5	0.3145	0.2843	1.11
5	0.45	0.4528	0.99
10	0.6206	0.6437	0.96
15	0.74	0.7489	0.99

The comparison shows a discrepancy of up to 11% for the 2.5 minute time of concentration, but much closer agreement at the 5 minute time of concentration.

The DOE letter urged that the analyses not require a time of concentration for the rainfall-duration curve of less than 2.5 minutes. We have tentatively agreed that 2.5 minutes was probably short enough for typical embankments. As backup to this assertion, we performed a simulation using the finite difference runoff model described in Reference 2 for the baseline (unslumped) embankment case. A draft of this paper is attached. We simulated runoff for the cases of a local PMP with minimum times of concentration of 2.5 and 5 minutes. The peak runoff rate for the 2.5 minute case was only about 1% greater than the 5 minute case. This indicates that the peak runoff would not be sensitve to the 11% discrepency between the DOE and NRC curves, and that shorter times of concentration should not be necessary.

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We therefore conclude that DOE's regression equation for rainfall-duration is acceptable. Furthermore, we conclude that a time of concentration of less than 2.5 minutes is not necessary for calculation of design basis runoff at tailings embankments.

/S/

Malcolm R. Knapp, Chief
WMGT

Enclosures:
1. Revised table
2 Reference 2 (draft)

References

1. Letter from Leo Higginbotham to John G. Themelis, October 18, 1985

2. R.B. Codell, "Runoff from Armored Slopes", Paper to be presented at Geotechnical and Geohydrological Aspects of Waste Management, Colorado State University, Fort Collins CO, Feb. 1986

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Table 1 - Cubic Spline Curve for Rainfall Intensity vs. Duration

i	t_i min.	Range_i min.	$C_{i,1}$	$C_{i,2}$	$C_{i,3}$	F_i
1	0.0	0.0 - 1.0	.1616205E+00	0.	-.7820493E-02	0.0
2	1.0	1.0 - 5.0	.1381590E+00	-.2346148E-01	-.7820493E-02	0.1538
3	5.0	5.0 -15.0	.3967789E-01	-.1158800E-02	-.7820493E-02	0.45
4	15.0	15.0 -30.0	.1923221E-01	-.8857681E-03	-.7820493E-02	0.74
5	30.0	30.0 -45.0	.4822097E-02	-.7490628E-04	-.7820493E-02	0.89
6	45.0	45.0 -60.0	.3479401E-02	-.1460676E-04	-.7820493E-02	0.95

EQUATION:

$$R = ((\,C_{i,3} \times D + C_{i,2}) \times D + C_{i,1}\,) \times D + F_i$$

where R = fraction of one hour PMP accumulation,
D = Duration - t_i, min.

Runoff from armored slopes

Richard B. Codell
U.S. Nuclear Regulatory Commission, Washington D.C. 20555

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1 INTRODUCTION

Uranium mill tailings embankments in the United States are required to be stabilized and protected from natural phenomena for a period of 200 to 1000 years (EPA 1985). Embankments and diversion channels are often protected from erosion by rock armor, which must withstand runoff from intense precipitation. Protection from the effects of the Probable Maximum Flood (PMF) adjacent to the site and the runoff from the local Probable Maximum Precipitation (PMP) would satisfy this requirement. Lesser events would also be acceptable if they could be adequately justified.

Typical embankments at these sites have surface areas of only a few acres, with gentle top slopes (0-2%) and a steep side slope (10-20%). The tailings are generally covered with a 6-to-8 ft layer of compacted silt and clay to reduce the diffusion of radon into the atmosphere. Rock armor is usually placed on top of the radon barrier in thicknesses of 1-2 ft, with a mean diameter d_{50} of 1-2 in on the gentle slopes, and 3-12 in on the steeper slopes.

Models exist for calculating overland flow on hillsides (Morris 1980) but no models have been found which explicitly deal with runoff from armored slopes. Flow on armored slopes differs from overland flow, because substantial flow occurs beneath the surface of the rock layer at low runoff, and both above and below the surface for high runoff. In addition to the lack of a suitable model, no estimates of the PMP exist for such small areas and for very short durations.

This paper develops a model for calculating runoff from armored embankments. The model considers the effect of slope, drainage area and "flow concentration" caused by irregular grading or slumping. A rainfall-duration curve based on the PMP is presented which is suitable for very small drainage areas. The development of the runoff model and rainfall-duration curve is presented below, along with a demonstration of the model on the design of a hypothetical tailings embankment.

2 RUNOFF MODEL

Rain falling on an armored slope will flow downhill except for the fraction infiltrating the ground which for the present case can be neglected. Referring to Fig.1, the flow of water on the slope may be described for a two dimensional case by a macroscopic mass balance and the kinematic approximation of the energy balance (Overton, 1976). The kinematic

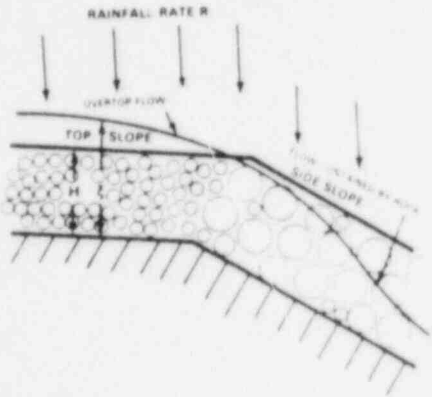

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Figure 1. Embankment in profile

approximation neglects acceleration, which can be shown to be small, and balances friction versus hydraulic gradient only. The kinematic equations for runoff are stated:

$$\frac{\partial(\xi u)}{\partial x} + \frac{\partial(\xi v)}{\partial y} + \frac{\partial \xi}{\partial t} = R \qquad (1)$$

$$\theta \frac{\partial \xi}{\partial x} + \frac{Ku\sqrt{u^2 + v^2}}{gn^2 d^*} - S_x = 0 \qquad (2)$$

$$\theta \frac{\partial \xi}{\partial y} + \frac{Kv\sqrt{u^2 + v^2}}{gn^2 d^*} - S_y = 0 \qquad (3)$$

where ξ = water depth above an impermeable layer, u = flux of water across the slope, v = flux of water down the slope, n = rock void porosity, t = time, R = rainfall rate, g = acceleration of gravity, K = friction factor, and d^* = representative rock diameter.

For flow over the top of the rock layer, the depth ξ becomes a virtual depth; that is, the depth which the water would have to assume if the rock layer were infinitely thick. The factor θ is used to adjust the gradient of the virtual water surface for flows which overtop the rock layer. It is equal to unity if flow is below the level of the rock layer and equal to the porosity n if flow is above the rock layer surface. The coefficient K for flows confined below the surface of the rock is a function of rock shape, roughness and Reynolds number $Re = Ud^*/n\nu$, where ν is the kinematic viscosity, and U is the water flux in the direction of flow. For the large Reynolds numbers (i.e., greater than 1000) expected through the rock layers, Stephenson (1979) observed that K is a function only of roughness and shape.

$$K = \frac{800}{Re} + K_t \qquad (4)$$

where K_t = 1 for smooth polished spheres, 2 for semi-rounded rocks, and 4 for angular rocks.

2.1 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 effective resistance will decrease once the water depth exceeds the thickness of the rock layer.

Consider for the time being only the flow down a slope, which is covered by a uniform layer of rock. The total flow Q past a point on the slope is the sum of the flows through the rock layer, Q_1 and over the rock layer, Q_2:

$$Q = Q_1 + Q_2 = v_1 H + v_2 (\xi - H) n \tag{5}$$

where v_1 is the flux in the rock layer, v_2 is the flux 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:

$$v_2 = \left[\frac{8\ g\ R_h (S_y - n \frac{\partial \xi}{\partial y})}{f}\right]^{\frac{1}{2}} \tag{6}$$

where R_h is the hydraulic radius, and f is the Darcy-Weisbach friction factor. The hydraulic radius is approximated as the water depth over the top of rock:

$$R_h = n\ (\xi - H) \tag{7}$$

Hey (1979) presents a correlation of f for flow in gravel river beds, in terms of R_h and the d_{84} rock diameter:

$$1/\sqrt{f} = 2.03\ \log_{10}\ 13.46\ R_h\ /3.5\ d_{84} \tag{8}$$

An effective resistance factor K' for the total flow in and over the rock layer can be derived by combining Eqs. 4 through 7:

$$K' = \frac{d^*\ \xi^2}{H(d^*/K)^{\frac{1}{2}} + (\xi - H)\ (8\ \frac{(\xi - H)n}{f})^{\frac{1}{2}}} \tag{9}$$

Equations 1,2, and 3 are solved with the effective value, K' substituted for K when ξ is greater than the rock layer thickness H. Since friction is likely to only decrease as the flow overtops the rock layer, the value of K' is limited to K:

$$K' \leq K \tag{10}$$

Rating curves for flowrate vs. water depth at steady-state for the example are shown in Fig. 2. The much higher carrying ability of the overtop layer is evident from this figure.

The upper end of the top slope is assummed to be a no-flow boundary:

$$v = 0, \qquad \frac{\partial v}{\partial y} = 0 \tag{11}$$

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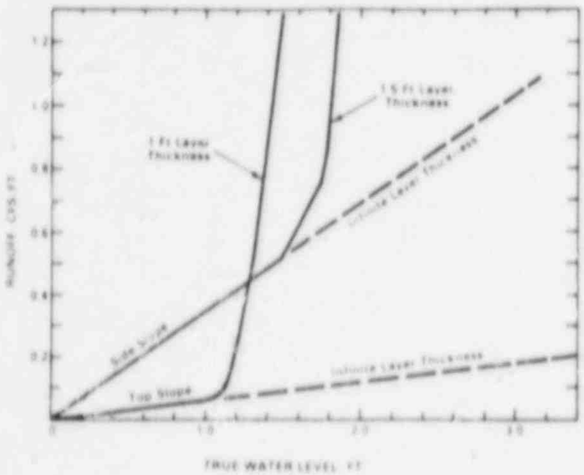

Figure 2. Rating curves for example - 2% top and 20% side slopes

The water level is continuous across the slope break. Free slip and no flow are assummed at the lateral boundaries. The flow boundary condition at the base of the lower slope considers that the depth of the water layer is determined only by the balance between friction and gravity:

$$\xi = \frac{g}{n} \left(\frac{K'}{gd^*} S_y \right)^{\frac{1}{2}} \tag{12}$$

The partial differential equations 1, 2, and 3 are reduced to their finite difference form in a staggered grid, and are solved by the "Leapfrog" explicit algorithm (Roache 1972). The model is presently limited to slopes which can be represented by up to 4 tilted subslopes.

3 PRECIPITATION MODEL

The rainfall-duration curve is developed from Hydrometerological Report 49 (NOAA 1977) for durations of 15 minutes to 2 hours, and from estimates made by the staff of the U.S. National Weather Sevice (NWS) for durations shorter than 15 minutes (Hansen 1985). It is most suited to the Colorado and Great Basin drainages of the Western United States, but rainfall-duration curves for other regions ould be developed along similar lines.

The NWS estimated that the 5 minute duration PMP for the area covered by HMR-49 was 45 ±5% of the 1 hour PMP. For durations shorter than 5 minutes, NWS advised that maximum rainfall rates could be estimated from record rainfall amounts measured at mid-latitudes on the globe. NRC, therefore, used the U.S. record for 1 minute of 1.23 in, measured at Unionville, MD, July 4, 1956.

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 would be an increasing intensity of precipitation, with the last 2.5 minutes of the first hour and the first 2.5 minutes of the second hour being the most intense. Total precipitation for the first hour was 8 inches. Precipitation for the second hour was 14% of that for the first hour. The hyetograph is presented in Fig. 3.

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4 MODEL RESULTS

The modeled embankment is typical of those found at uranium mill tailings sites. The embankment is assumed to be triangular and symmetrical around the vertical centerline, similar to that shown in Fig.4a. It is 700 feet long from top to bottom, and 1200 feet wide at the base. The top portion of the embankment is 440 ft long, with a slope of 2% and a rock-layer thickness of 1 ft. The lower portion of the embankment is 260 ft long with a slope of 20% and a rock layer thickness of 1.5 ft. The harmonic mean diameters of the rock are 0.1 ft and 0.3 ft for the top and side slopes respectively. The d_{84} diameters are 0.32 and 0.75 ft respectively. Friction factor for flow through the rock is assumed as K = 2.0.

Runoff per unit width from the toes of the top and side slopes of the sample embankment are shown in Fig. 3. These and subsequent results are also summarized in Table 1. In the present case, the top and side slopes are assumed to be unfailed. Peak flow is nearly coincident with the peak precipitation rate. Runoff from the side slope shows a small disturbance after its peak, which is caused by the routing of the peak flow from the top slope.

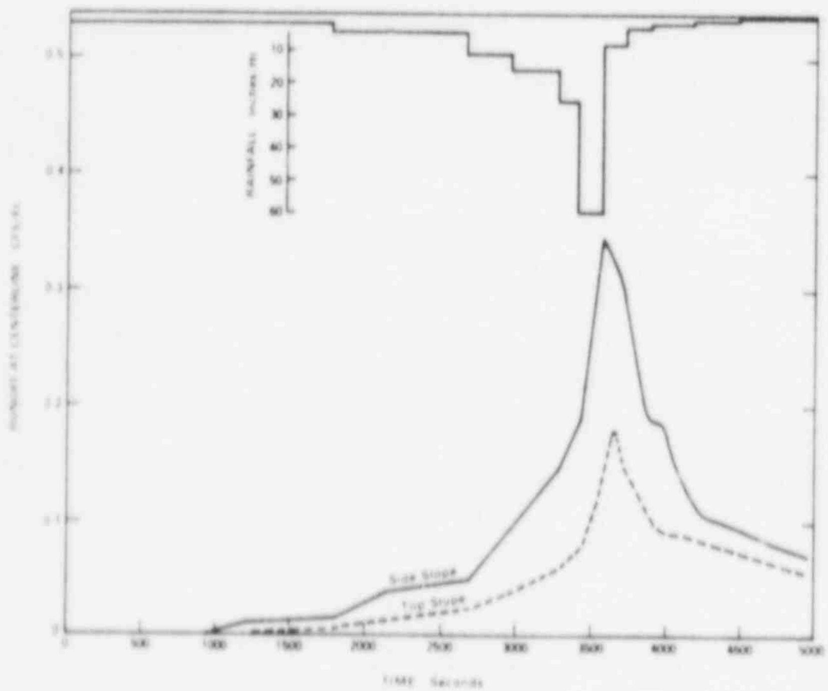

Figure 3. Hyetograph and runoff from benchmark embankment

4.1 Flow concentration

Flow concentration is a term to describe the preferential flow paths on the embankments caused by nonuniformities of the embankment profile. The most likely cause of flow concentration, given that good grading and rock placement practices are followed, is a failure or differential settlement of the earthwork with subsequent subsidence or 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.

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The smaller grade and lower water carrying ability on the top slope would accentuate the effects of settlement on flow concentration, Settlement of from one to several feet might be possible (Wardwell 1984). The effect of settlement can probably be overcome by good engineering

Table 1 - Summary of model experiments

Experiment	Peak runoff, cfs/ft Top Slope	Side slope
Benchmark, 0% slump	0.15	0.35
Halve d*	0.29	0.28
Double d*	0.16	0.42
½ layer thickness	0.45	0.40
½% slump	0.55	0.90
1% slump	1.03	0.55
infinite layer, 1% slump	0.20	0.46

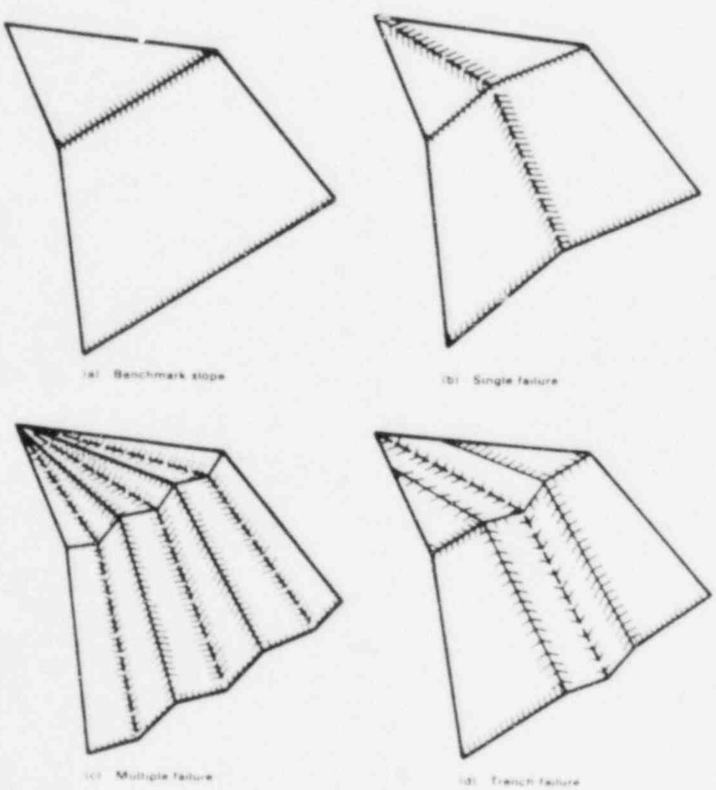

Figure 4. Slope failure scenarios for flow concentration

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practice. Nevertheless, several scenarios of embankment failure have been postulated and studied with the numerical runoff model. Figure 4b shows a uniform inward slumping of the embankment toward the centerline. Multiple failures as illustrated in Fig. 4c would probably cause less severe flow concentration, because the drainage area for each sub-basin is smaller than the single failure case. Other failures are possible, such as the opening of a trough by slumping and erosion of an otherwise-unfailed slope (Fig.4d).

Two cases of embankment slumping of the type illustrated in Fig. 4b are presented in order to demonstrate flow concentration: (1) Uniform inward slope of ½% toward centerline and (2) Uniform inward slope of 1%.

Flow concentrations resulting from steady rainfall are presented in Fig. 5 as the ratio of runoff per unit width at the embankment centerline to runoff per unit width from an infinitely-wide slope with no slumping. Flow concentrations for the unfailed (Benchmark) case are less than unity, especially on the top slope, and are relatively insensitive to the rate of precipitation.

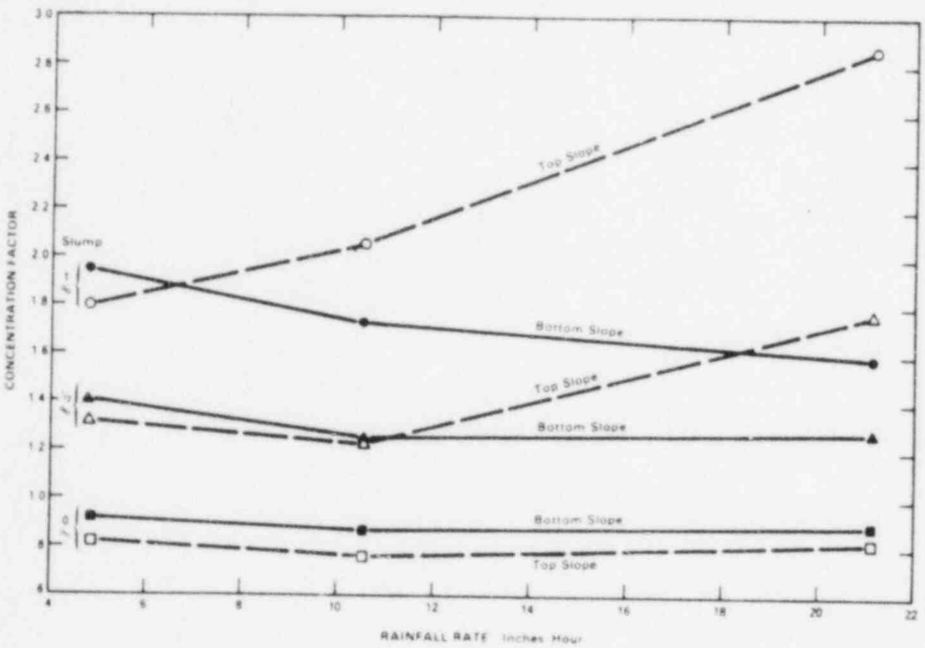

Figure 5. Flow concentrations at steady state

Flow concentration for the ½% and 1% slump 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 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% downward gradient of the original slope. There is significantly less flow concentration on the steep side slopes. Overtopping would occur only at points above the slope break. Peak flow rates are attenuated within the rock layer of the side slope.

Transient runoffs from the top and side slopes resulting from the local PMP are presented in Fig. 6 for the 1% slump scenario. There is a considerable degree of flow concentration in this case, particularly on the top slope. An interesting observation is that peak runoff may occur at the toe 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.

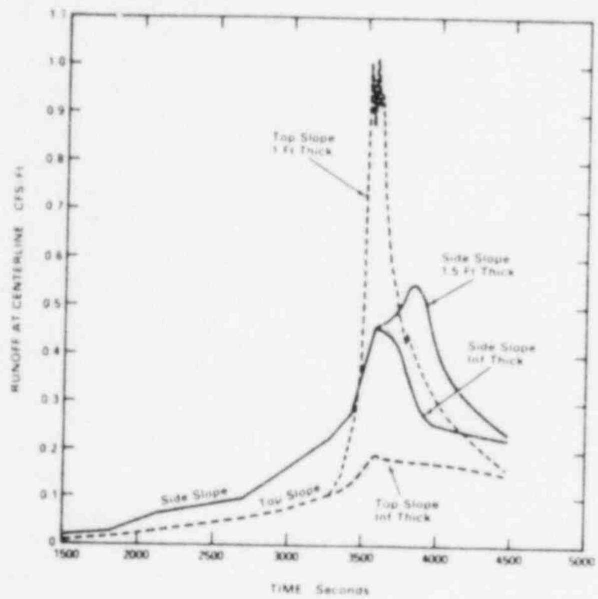

Figure 6. Flow concentration for transient case - 1% slump case

4.2 Sensitivity of runoff to design parameters and uncertainties

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 attenuation of peak flow from the slope. This effect will be diminished however if the rock layer is too thin, its friction too great or its porosity too small.

The capacity of the rock layer to conduct flow is related to its thickness and on the grouping of terms KU^2/gd^*n^2. Reducing porosity n would both increase friction and reduce the water-carrying ability of the rock layer, forcing more flow to the surface, all other factors being equal.

The effect of doubling and halving the estimate of d^* is given in Table 1. Doubling d^* lowers the internal friction, increasing the peak runoff. Interestingly, halving d^* increases friction, but causes the flow on the top slope to exceed the carrying capacity of the rock layer, resulting in an increase in the peak runoff at the toe of the top slope.

The grouping is directly proportional to the friction factor K. 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.

The transient case was rerun for the 1% slump scenario, but with an essentially infinite layer thickness which eliminates the possibility of overtopping. The results of this run are shown in Fig. 6 along with the runoff for the normal rock layer thickness. Peak runoffs for this case are dramatically lowered. 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 example embankment to completely contain the peak flows are about 3 ft. on the top slope and 1.3 ft. on the side slope.

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One area of uncertainty is the effective diameter d^*, which is necessary to calculate resistance for flow through the rock layer. Leps (1973) suggests the d_{50} rock diameter. Stephenson recommends the harmonic mean (i.e., the mean weighted by the inverse) diameter d^r. 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 example is about 1.3. Ratios calculated on typical grades of crushed rock are in the range $d_{50}/d^r = 2$ to 3. The harmonic mean diameter places heavier emphasis on the smaller rock sizes.

5 CONCLUSIONS

Runoff from armored compound slopes on tailings embankments resulting from intense precipitation has been studied by means of a mathematical model for kinematic flow. Several interesting conclusions can be drawn from the mathematical experiments with the model:

1. The calculation of runoff must consider flow both through and over the top of the armor layer.
2. Irregularities in the surface of the slopes may lead to large concentrations of flow along preferential paths.
3. The peak runoff from the gentler top slope can frequently be greater 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. This observation indicates that the larger armor used on the side slope should extend a distance above the break in the slope, onto the less-steep slope.
4. The use of larger-diameter rock and thicker rock layers tends to diminish peak runoff from the top slope.
5. The effects of flow concentration caused by geotechnical failure or slumping can be greatly diminished by having an adequate rock layer thickness.

Several uncertainties were identified during the development of an analytical method 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 methods. To evaluate the significance of these uncertainties, NRC is sponsoring technical assistance at Colorado State University to collect experimental data on flow resistance of typical rockfill layers, and the ability of the rock to withstand the erosive effects of the runoff.

6 DISCLAIMER

The opinions expressed in this paper are those solely of the author, and do not necessarily represent the official policy of the Nuclear Regulatory Commission.

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