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OVERTOPPING OF SMALL DAMS

AN ALTERNATIVE FOR DAM SAFETY

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ABSTRACT

When an embankment dam is expected to overtop during or near a PMF (probable maximum flood) event, the Bureau of Reclamation, as well as most of the engineering profession, assumes the dam will fail. Experience has shown that it is not always practical or cost effective to modify small existing embankment dams to accommodate the PMF by enlarging the spillway or raising the dam crest.

A research effort was initiated in 1983 by the Bureau of Reclamation to gain insight in the development of cost-effective modifications to small embankment dams which would enable them to withstand overtopping. A small flume, 3 ft wide by 4 ft high by 30 ft (914 mm by 1219 mm by 9144 mm), was constructed to contain the 2.12-ft-high (646-mm) soil test embankments and flow. The nature and modes of erosion of the small embankments were observed during overtopping flow.

Introduction

It is commonly assumed by engineers that when an embankment dam is overtopped, erosion on the downstream slope and toe of the dam will lead to dam failure. Consequently, overtopping is not permitted in the design of an embankment dam. Of the approximately 8,500 dams inspected by the Corps of Engineers under the non-Federal Dam Safety Program, over 3,000 dams were found to be potentially unsafe. Of these potentially unsafe dams, about 85 percent were due to inadequate spillway capacity or insufficient dam height which could result in dam failure due to overtopping.

The PMF has been used by the Bureau of Reclamation as the IDF (inflow design flood) for new storage dam designs and for modification of existing dams if the dam failure could cause potential loss of human life or significant property damage. Due to the larger storm data base now being considered, the PMS (probable maximum storm) and PMF magnitudes used for design of new dams and modification of existing dams have increased significantly. As a result, design of new dams and spillways or modification of existing dams for the revised and often larger PMF becomes very expensive. In some instances, it may not be physically or

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economically feasible to accommodate large floods without overtopping. In fact, some embankment dams have been removed because of excessive costs. However, some embankments have survived moderate overtopping. Therefore, it has been decided that some existing embankment dams [especially those less than 50 ft (15.2 m)] could be modified to safely permit overtopping without losing the reservior pool.

A research effort including a model study was initiated in 1983 by the Bureau of Reclamation to gain insight in the development of cost-effective modifications to small embankment dams which would enable them to withstand overtopping. In addition, the Bureau, in assisting the National Park Service, has initiated design studies to modify two dams in the Blue Ridge Parkway, North Carolina, to allow overtopping without the loss of their reservoir pools.

Using embankment dams owned by the National Park Service as a basis, it was decided that:

1. The model should represent a 32-ft-high (9.8-m) dam constructed of materials similar to those of the Blue Ridge Parkway dams.

2. The model embankment and overflow system should represent 4 ft (1.2 m) of water overtopping the crest of the dam for a period of 4 to 6 hours.

Laboratory Test Facility

Water was supplied by a portable laboratory pump. An 8-in (203-mm) orifice venturi meter was used to measure and set discharge by reading the differential pressure. One side of the flume had windows for viewing scour and flow action. A 12-in-diameter (305-mm) pipe with gate valve was used to pass flow around the test embankments. Upon closing the valve, the reservoir water level would rise and the preset discharges would flow over the test embankments.

Soil Tested

The soil used for all test runs is a local clayey sand. The soil has 49 percent fines passing the No. 200 sieve defined as less than 0.074 mm in diameter and 6 percent gravel greater than 4.76 mm or not passing the No. 4 sieve. The soil has a liquid limit (LL) of 25 percent and a plasticity index (PI) of 9 percent.

The soil compaction properties were used to obtain the desired test density of 95 percent standard Proctor for all but one test run placement. The placement for Test Run 8 was overcompacted to 102 percent. Soil compaction was controlled by determining the weight of stock pile soil required to fill volumes of 3-in-thick (76-mm) layers of model embankment and determining the amount of moisture that needed to be added to obtain optimum compaction.

The top of each finished layer was scarified before placing new test embankment and providing scour protective treatments.

A 6:1 slope was used for early tests but was later increased to 4:1 because of the high stability experienced with earlier tests. A 6:1 slope is the erosion stability breakpoint. Noncohesive material on this slope has about 75 percent of the critical tractive shear resistance of that for the same material on a flat bed. As slope increases erosion resistance decreases rapidly; as slope decreases erosion resistance increases slowly. Noncohesive material on a 4:1 slope has about 60 percent erosion resistance relative to flat bed flow resistance.

Model Scaling

Froude scaling can be applied only to the flow near the top of the dam where friction does not dominate. Length scaling of 1:15 was selected to make the 2.12-ft-high (646-mm) model dam represent a typical National Park Service embankment dam about 32 ft high (9.8 m). Equations for frictional flow and noncohesive sediment transport do not apply for shallow flow, steep and rapidly accelerating flow, nor for chutes and pool flow. Also, no adequate governing equation for cohesive soil erosion exists. Without these equations, no reliable sediment transport time nor velocity scaling relationships can be determined. This is not only a problem of using a small scale model but it is also a hinderance in making predictions from experience with one full size dam to another full size dam. To estimate sediment transport time without equations, modelers generally compare a nonrandom prototype event of significant sediment transport quantity and time duration with model performance. The random nature of soil particles and construction techniques makes test replications desirable. Thus, these model results are considered qualitative, more likely indicating which treatment of those tested worked better rather than determining how much better.

Model Operation

A total of nine tests were conducted. For most test arrangements, the unit discharge represented was 40 $ft^3/s/ft$ (3.7 $m^3/s/m$). Test Run 7 arrangement was replicated for Test Run 8 but operated at a unit discharge representing 87 $ft^3/s/ft$ (8.1 $m^3/s/m$). Test Run 1 lasted only 17 minutes because erosion was considered to be excessive and more consideration needed to be given to the boundary effects of the model and the smoothness of the hard cap at the crest. Therefore, 17 minutes (about 1 hour prototype time) was used as a common model test run time interval for the remaining tests to compare test arrangement erosion. Test Runs 4, 6, 7, and 8 were also operated an additional hour which would be representative of 5 hours of overtopping which is expected for the National Park Service dams being considered for modification.

Results

Table 1 contains a brief summary of model embankments tested and results of each test run. Following are results and implications noted during the studies, tempered with experience from other sediment model studies and conversation with people experienced with overtopping cases.

During Test Run 1 with a 6:1 slope, the flow rapidly transformed into a chutes and pool mode. This type flow is initiated by shallow bank surface waviness or by jets and vortices caused by flow around and over isolated projecting rocks in the fill material. Brush and trees would cause local areas of increased scour. Scour holes tended to lift flow resulting in intermediate areas of less scour between areas of increased jet scour. Road pavements, curbs, parapets, and bulging or sagging of top faces of gabion compartments can initiate and affect location of scour.

The erosion data for the 5-hour test runs were used to see if the rough later flow had a greater or less transport rate than the earlier smoother flow. For these tests the sediment transport rate for the last 4 hours averaged about 1/4 of the rate for the first hour. Thus, it appears that the transition from chutes and pool mode flow reduced erosion relative to earlier smooth flow erosion. For some unexplained reason no erosion was detected for the last hour of Test Run 8 using the techniques for measuring volume.

During Test Run 1, the smooth vertical walls of the model appeared to have exaggerated scour causing deep holes in the embankment near the flume walls and the vertical curtain wall of the hard protective crest cap. Reservoir side and bottom roughness and form roughness cause eddies that are lifted by the upstream slope of the dam and stretch out parallel to the bed. These bed parallel vortices cause local areas of relatively more intense scour. Crest end treatments that contract the flow sideways also cause vortices that intensify scour.

When flow over a fixed bed makes a transition to over a soil bed or vice versa, scour occurs. This is caused by boundary layer being forced to adjust to a change in boundary rugosity. Going from the hard crest cap to soil is an example of this type of scour. For Test Run 2, pea gravel was epoxyed to the downstream sloping part of the hard crest cap simulating 3- to 6-in (76- to 152-mm) cobble roughness to increase depth and dampen vortex action. This increase of roughness resulted in a significantly less, and more uniform, scour than for Test Run 1. This effect also occurs for flow making the transition from over gabions to over soil.

Despite the previously discussed modeling limitations, velocity, depth, and discharge were scaled by Froude law for comparison. The scaled velocities of all test runs rapidly reached more than 30 ft/s (9.1 m/s). Riprap design methods do not account for the combination of high velocity, steep downslopes, and shallow flow. During Test Run 3, model riprap representing 6- to 24-in-diameter (152- to 610-mm) became fluidized and eroded out immediately. Rock contained in mesh compartments was tested next. Gabions and mattress pods modeled for Test Runs 4, 5, and 6 represented about 4-in (102-mm) mesh compartments filled with angular rock up to 12-in (305-mm) maximum diameter on a filter bed. During these tests, there was no indication of a threat of them being dislodged by the overtopping flow. However, manufacturers of gabions have no backup data for velocities greater than about 24 ft/s (7.3 m/s). On the steeper slope, 4:1, the main erosion was just near the downstream end of the protection. Whereas, for the 6:1 slope the main scour hole was 45 ft (13.7 m) downstream. Comparing Test Runs 4 and 5 indicates that the effect of increasing the down slope from 6:1 to 4:1 increased the scour volume about 5 times. The maximum depth of scour was about two times greater. A minor part of the differences can be attributed to sagging and bulging of gabion compartment tops.

Model test results of Test Runs 8 and 9 indicated that increasing standard Proctor compaction from 95 to 102 percent resulted in 1/2 the erosion with similar protective treatments tested.

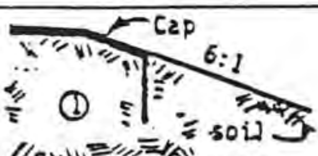
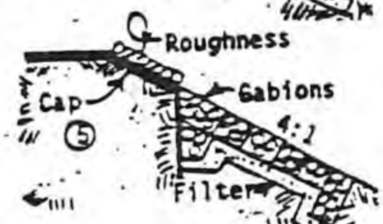
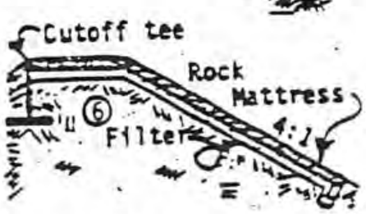
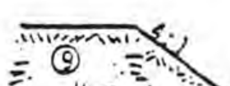
Comparing Test Runs 7 and 9 test results showed that increasing unit discharge from 40 to 87 $ft^3/s/ft$ (3.7 to 8.1 $m^3/s/m$) resulted in about 40 percent increase in erosion.

Despite exercising care during construction and inspection, there always are low areas along the crest. Low areas can occur after construction because of crest traffic, nonuniform settling, and lack of maintenance. Erosion mode could depend on the shape of hydrograph during overtopping. For example, with slow rising or low constant flow the erosion may start and remain at one low point causing gulley type erosion. If flow rose fast enough, the erosion could act more like sheet flow erosion despite a low area.

A well coordinated interagency team approach is necessary to fund and make positive progress in solving a problem of this magnitude and expense. The results of these studies clearly illustrate that erosion during overtopping flow is a multivariable and multidisciplinary problem. Random aspects such as the many variables, lack of true soil homogeneity, different soil classifications, and hydrograph variations present a strong case for more repetition of model tests and uniform documentation of failures in the field.

Much long term effort is needed to develop adequate governing equations for rate of sediment transport on steep slopes. In reality these equations are needed not only for scaling sediment time and velocity and mathematical modeling, but for making more rational inferences from one prototype experience to another and to new design cases.

Table 1. - Summary of Results of Overtopping Flow Model

Schematic sketch and run numbers in circles	Unit discharge ft^3/s/ft (m^3/s/m)	Time (hrs)	Erosion of available volume of material (%)	Least depth of soil on + 2.5:1 slope ft (m) Below − Location from crest ft(m)	Deepest scour hole ft(m) Location from crest ft(m)
	40 (3.7)	1	16	-8(-2.4) 15(4.6)	10(3.0) 15(4.6)
	40 (3.7)	1	7	0(0.0) 10(3.0)	9(2.7) 106(32.3)
	40 (3.7)	1	13	-6(-1.8) 10(3.0)	8(2.4) 10(3.0)
	40 (3.7)	1	2	9(2.7) 41(12.5)	2(0.6) 84(25.6)
		5	5	8(2.4) 41(12.5)	4(1.2) 95(29.0)
	40 (3.7)	1	12	-1(-0.3) 53(16.2)	3(0.9) 59(18.0)
	40 (3.7)	1	4	4(1.2) 59(18.0)	6(1.8) 104(31.7)
		5	8	2(0.6) 84(25.6)	10(3.0) 84(25.6)
	40 (3.7)	1	9	-0(-0.0) 0(0.0)	5(1.5) 77(23.5)
		5	14	-2(-0.6) 0(0.0)	9(2.7) 59(18.0)
	87 (8.1)	1	6	-0(-0.0) 0(0.0)	4(1.2) toe
		5	6	-1(-0.3) 0(0.0)	6(1.8) toe
	87 (8.1)	1	13	-1(-0.3) 0(0.0)	3(0.9) toe

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KEY WORDS

Embankment dams, spillway capacity, safety, overtopping, floods, erosion protection, scour rate, design, modification, model tests, riprap, gabions.