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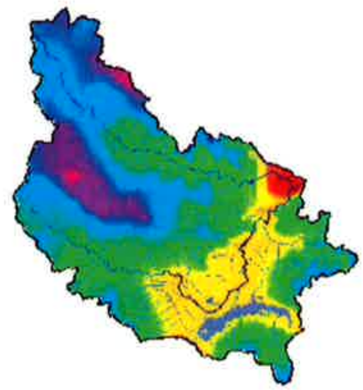

South Fork Licking River Watershed Initiative

Project # DNR 736 736-98-011

Project Memorandum

FINAL
50% PMF Analysis of the Buckeye Lake Dam

Prepared for:
Ohio Department of Natural Resources
Division of Engineering
Columbus, Ohio

July 2003

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July 1, 2003

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Mr. Steve Manilla, P.E.
Ohio Department of Natural Resources
Division of Engineering
1889 Fountain Square Court, F-3
Columbus, Ohio 43224

Re: South Fork Licking River Watershed Initiative
Project Memorandum
DNR 736 736-98-011

Dear Mr. Manilla:

We are pleased to submit five copies of the Final Project Memorandum on the 50% PMF Analysis of the Buckeye Lake Dam for the South Fork Licking River Watershed Project. This analysis was conducted in accordance with the ODNR Dam Safety Engineering Program's Critical Flood Analysis Guidelines. In addition to a discussion of the applicability of the 50% PMF as the Critical Flood for the Buckeye Lake Dam, the report also presents research into the likelihood that a dam breach would occur during the 50% PMF.

Your comments to the Draft version of this memorandum that we submitted in late 2002 have been incorporated into this final document. Please call if you have additional comments or questions.

Sincerely,
FULLER, MOSSBARGER, SCOTT AND MAY
ENGINEERS, INC.

Richard R. Noss, Ph.D., P.E.
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JUL - 7 2003

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PROJECT MEMORANDUM

50% PMF Analysis of the Buckeye Lake Dam

INTRODUCTION

Background

Buckeye Lake is a 4.3 square mile lake owned and operated as a State Park by the Ohio Department of Natural Resources (ODNR). Buckeye Lake is 25 miles east of Columbus, Ohio. The Buckeye Lake Dam is considered a Class I dam because the storage volume of Buckeye Lake exceeds 5000 acre-ft and because failure of the dam may result in loss of life. Class I dams are required to contain or safely pass the Probable Maximum Flood (PMF), unless it can be shown that there is a lesser flood (the "Critical Flood") for which there will be "no additional potential for loss of life, health or property in the critical routing reach from overtopping failure of the dam when compared to the potential for loss of life, health or property caused by the flood in the absence of a dam overtopping failure" (OAC 1501:21-13-02(B)). Critical Flood Analyses may be performed for existing dams or incorporated into the planning and design of new dams.

Following a series of studies in the 1980's, a plan was developed to bring the Buckeye Lake Dam into compliance with the requirements for Class I dams. The Sellers Point Spillway, which was constructed in the early 1990's, was designed to handle the 50% PMF, with storage to accommodate the remainder of the PMF. Some of the elements of the plan were never implemented, and, consequently, the dam does not meet current regulatory requirements. Subsequent to the construction of the Sellers Point Spillway, the Ohio Dam Safety Regulations have been revised to permit dams to be designed for the Critical Flood rather than the full flood specified for their classification, if the lesser flood is supported by a satisfactory Critical Flood Analysis.

The determination of the Critical Flood assumes that the dam fails by overtopping and is based on a comparison of two flood situations: 1) the "base-flow" flood and 2) the corresponding situation with the "most severe hypothetical dam failure [by overtopping] that is possible" (ODNR Critical Flood Guidelines). A Critical Flood Analysis does not evaluate the structural integrity of the dam. To complete the Critical Flood Analysis the two flood simulations are routed through the downstream reach that is the recipient of the potential floodwater damages. The initial test for additional potential for loss of life, health or property is based on the incremental rise in flood elevations and the increased floodplain flow velocity resulting from the failure of the dam. According to the regulations, additional potential for loss of life, health or property is expected at flood magnitudes where the "with dam failure" scenario increases flood elevations by more than 2 feet or where the product of the elevation increase and the overbank velocity is greater than 7 ft-ft/sec. If the impacts of a flood are within these criteria, then an additional analysis of specific conditions in the critical routing reach and in the vicinity of

the dam is required to assess the potential for additional loss of life, health or property before that flood can be designated as the Critical Flood for the facility.
This Project Memorandum summarizes the application of Critical Flood Analysis procedures to analyze the performance of the Buckeye Lake Dam for the 50% PMF. The standard Critical Flood Analysis has been modified to analyze only the 50% PMF because the construction of the Sellers Point Spillway is complete and the ODNR does not have any plans to raise the dam as was planned when the Sellers Point Spillway was constructed.

ODNR is currently completing a study of alternative designs for a new structural wall to be constructed on the lake side of the existing earthen dam to provide protection against failure of the existing dam (in effect, a new dam will be constructed inside the existing dam). This new wall will be freestanding (i.e., independent of the earthen dam) and will be designed to withstand overtopping (i.e., the Critical Flood Analysis assumption that the dam fails by overtopping will be moot). The new structure is expected to be constructed in stages. Construction is expected to begin in 2004 and may be phased over as many as ten years. This 50% PMF Analysis addresses the current condition of the dam.

Discussion of Buckeye Lake Conditions

Constructed in the 1820's, the Buckeye Lake Dam is an uneven 4.1 mile earthen dike that contains most of the west and northwest extents of the lake. The dam is referred to as the North Bank or the West Bank, with the Sellers Point Spillway area being the approximate dividing line. The top of the dam varies from elevation 894.5 to 897 feet[1]. Sheetpiling has been added over the years to reinforce portions of the dam. Some of the sheetpiling dates to the 1940's; most was installed in the 1960's. The West Bank from Mud Island to Lieb's Island does not have any sheetpiling.

Buckeye Lake is a shallow, off-stream reservoir with natural tributaries. Prior to damming what was known as Big Swamp to form the lake, one small stream connected the wetland to the South Fork Licking River (SFLR). Currently, the Lake Outlet, which conveys the flow from the Amil Gate Spillway (max. discharge 610 cfs), follows the alignment of that historical outlet from the wetland (known to some as the "Waste Weir"). The SFLR passes within 2,500 feet of the North Bank for a 1+ mile stretch in the vicinity of the Sellers Point Spillway. The SFLR floodplain in this vicinity is very broad and very flat, behaving almost as a reservoir during floods. Most of this floodplain is farmland. The Village of Buckeye Lake abuts the lake at the northeast end of the dam, and the dam itself is lined with houses for its entire length. Other than the Sellers Point and Amil Gate Spillway outlet channels which handle only the spillway flows, there is no downstream valley in a traditional sense that would receive flows in the event of a dam failure. Instead, any overflow or breach flow from the lake would spread out across the SFLR floodplain before heading downstream.

[1] All elevations are reported as NGVD (1929). The project GIS and the HEC-HMS and HEC-RAS models use the NAVD (1988) datum, which is 0.66 ft. lower than the NGVD (1929) datum in the vicinity of Buckeye Lake.

Overview of Approach

Critical Flood Analyses are driven by the assumption that the dam fails by overtopping, which in turn implies the development of a dam breach through erosion of the earthen dam rather than a catastrophic collapse. Computer simulation models are required for simulation of the rainfall, lake inflow and outflow, dam failure calculations and estimation of water levels in the river and floodplain in the "critical routing reach." After the calculations are completed, the results are compared with the criteria described earlier (2 ft and 7 ft-ft/sec), and the initial assessment of whether the flood under consideration may be considered as a Critical Design Flood is made. In addition to comparisons with the criteria, the sensitivity of the modeling results to the data and assumptions used in the analysis is assessed, and site-specific considerations of the potential for additional loss of life, health or property are reviewed.

This 50% PMF analysis was performed according to the following outline:

A. Dam Breach (Assumed)
   1. Identify most sensitive dam breach location.
   2. Determine breach parameters to be used in the modeling.

B. H&H Modeling (Hydrologic and Hydraulic Modeling)
   1. Calculate runoff from the 50% PMF and route runoff hydrographs through the SFLR system (watershed hydrology).
   2. Calculate dam breach, spillway and dam overtopping discharges from Buckeye Lake for the dam failure scenario (DAMBRK).
   3. Calculate water surface elevations and flood characteristics (river and floodplain hydraulics).

C. Evaluation of Results
   1. Compare to Critical Flood criteria.
   2. Assess sensitivity to data and assumptions.
   3. Review site specific loss of life, health or property potential.
   4. Summarize findings.

The results of the analyses are presented in the following sections.

ANALYSIS AND RESULTS

Dam Breach (Assumed)

Siting the Dam Breach. The intent of the Critical Flood Analysis Guidelines is to site the assumed dam breach where the impacts of the dam failure would be most severe. The severity of the impacts depends in part on the magnitude of the breach and on the sensitivity of development downstream of the breach. These factors are discussed in the following paragraphs.

The first screen for siting the assumed breach was that it be located where dam overtopping flow would first occur. There are five areas (total of 600 LF) with a dam crest of 894.5+' (the minimum height of the dam according to the ODNR Dam Safety Information Sheet). Of these, the location with the greatest dam height (13.5 feet) was selected as the site of the assumed breach. A breach at this point is expected to have the greatest rate of discharge because the height of the dam is greatest. The breach location, approximately 600 LF south of the Licking County – Fairfield County line, is indicated in Figure 1, on the following page.

Figure 1. Buckeye Lake Dam: Assumed Breach Location.

The development downstream of the assumed breach includes individual houses (on the dam and 200 feet distant), a trailer park (900+ feet downstream) and residential and commercial development of the Village of Buckeye Lake (1000+ feet laterally downstream). Flows from the breach will cross 2,700 feet of floodplain before they reach the SFLR channel. The development downstream of the assumed breach location can also be seen in Figure 1.

Dam Breach Parameters. ODNR's Critical Flood Guidelines state that the recommendations of the Federal Energy Regulatory Commission (FERC) and the U.S. Army Corps of Engineers should be followed in selecting the dam breach parameters to be used in the analysis. The parameter ranges from FERC and the Corps of Engineers[2] for earthen dams are presented in the table below.

Table 1
Suggested Breach Parameters for Earthen Dams

Parameter	FERC	Corps of Engineers
Breach Width (*)	1 to 5 dam heights	0.5 to 3 dam heights
Side Slope of Breach	0.25:1 to 1:1	0:1 to 1:1
Failure Time (hrs)	0.1 to 1.0	0.5 to 4
(*) *FERC specifies this as average width; the Corps of Engineers does not state whether this is the bottom or average width.*		

A breach width of 5 dam heights, a side slope of 0:1, and a failure time of 0.5 hours were selected for the initial dam breach simulations. The breach width and side slope values are the most conservative of the recommended ranges. The breach formation time is conservative, but not the extreme. The breach analysis assumed that any sheetpiling or structures present would be removed by the breach flow and would not affect the development or behavior of the breach.

H&H Modeling

The 50% PMF analysis used computer simulation models to calculate the discharges and physical characteristics of flooding and dam failure necessary to make the comparisons to the Critical Flood Analysis criteria. The functions of the models were to calculate:

1. The watershed hydrology resulting from the 50% PMF;

2. The spillway, dam overtopping and breach discharges from the lake during "with breach" and "without breach" conditions;

3. The combination of discharges from Buckeye Lake with other watershed runoff and routing through the critical routing reach; and

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[2] FERC *Engineering Guide* (obtained from the FERC website), Chapter 3, "Selecting and Accommodating Inflow Design Floods for Dams" and Chapter 16, "Dam Break Analysis" of the Corps of Engineers *Engineering and Design Manual for Hydrologic Engineering Requirements for Reservoirs* (EM 1110-2-1420) (which refers to the parameter ranges in Corps of Engineers HEC Research Document 13, Table 1).

4. The water surface elevations and physical characteristics of the resulting flooding in the critical routing reach, including the floodplain adjacent to the dam.

The models, and the data and parameters used to develop the models, comply with numerous criteria stated in the ODNR Guidelines and the underlying OAC Regulation (OAC 1501:21-13-02).

Watershed Hydrology to Calculate 50% PMF Runoff. The HEC-HMS model (U.S. Army Corps of Engineers, Hydrologic Engineering Center, "Hydrologic Modeling System") developed and calibrated by FMSM for the ODNR's SFLR Watershed Initiative was used to calculate the runoff from the watershed for the 50% PMF. The PMP (Probable Maximum Precipitation) rainfall was developed in accordance with National Weather Service (NWS) procedures as described in NWS publications HR-51 and HR-52.[3] The PMP modeling was reported to ODNR in an earlier technical memorandum dated November 28, 2000. HEC-HMS processed the PMP and simulated the watershed behavior for the 50% PMF.

For the 50% PMF (no breach), the maximum water level in Buckeye Lake was 895.13' or approximately 7 inches overtopping the lowest points on the dam. The total duration of overtopping (i.e., lake levels higher than 894.5') was 8.7 hours.

For the "base flood" (i.e., no breach), HEC-HMS calculated the discharges through the Amil Gate Spillway and the Sellers Point Spillway and the flow overtopping the dam. These hydrographs were routed through the SFLR along with the runoff and routed flows from the rest of the watershed. For the "with breach" case, the inflow to Buckeye Lake, as calculated by HEC-HMS, was input to the DAMBRK model and DAMBRK was used to simulate the outflows from the lake and the development and discharge from the breach.

Calculation of Dam Breach, Spillway and Dam Overtopping Discharges. DAMBRK (originally developed by the National Weather Service; obtained from Boss International, Inc.) was used to model the breach development using the parameters discussed above. The breach was assumed to begin when the water level in the lake reached 895.0' (i.e., six inches above the crest of the dam) and to be fully developed one-half hour later. At its full size, the breach was 67.5 feet wide and extended to the toe of the dam at elevation 881.0'. The maximum discharge through the breach for the 50% PMF was 11,000 cfs.

DAMBRK also simulated the release of water through the Amil Gate Spillway (610 cfs, maximum), the Sellers Point Spillway (16,000 cfs, maximum), and over the top of the dam (other than through the breach, 600 cfs, maximum). The maximum water level in Buckeye Lake during the 50% PMF with the breach was 895.03'. When DAMBRK was run for the 50% PMF assuming no breach to check its validity, the calculated maximum lake level was within 0.01 ft. of that calculated by HEC-HMS; the maximum Sellers Point

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[3] U.S. Department of the Commerce, National Weather Service, "Probable Maximum Precipitation Estimates, United States East of the 105th Meridian," Hydrometerorological Report No. 51 (HR-51), June 1978, and U.S. Department of the Commerce, National Weather Service, "Application of Probable Maximum Precipitation Estimates – United States East of the 105th Meridian" Hydrometerorological Report No. 52 (HR-52), August 1982.

Spillway discharge was within 1 percent; and the peak dam overtopping flow was within 10 percent.

For the "with breach" scenario, the breach, spillway and dam overtopping discharges calculated by DAMBRK were substituted into the HEC-HMS model for the spillway and overtopping discharges calculated for the "no breach" scenario and the model was rerun to route the "with breach" discharges through the SFLR system. For modeling purposes, the breach discharge was assumed to enter the SFLR at the point closest to the breach location.

River and Floodplain Modeling to Calculate Flood Elevations. The peak discharges from the HEC-HMS modeling were input to the HEC-RAS (U.S. Army Corps of Engineers, Hydrologic Engineering Center, "River Analysis System") model that was developed and calibrated for the ODNR SFLR Watershed Initiative. The model incorporated ODNR's plan to increase the SFLR capacity between the Sellers Point Spillway confluence and the Amil Gate Spillway outlet by increasing the channel bottom to 50 feet wide.

The results of modeling the with- and without-breach scenarios for the 50% PMF are presented in Table 2 and discussed in a subsequent section.

Table 2
Buckeye Lake Dam
Summary of Results – 50% PMF with- and without-Breach

Milepoint	50% PMF - No Breach				50% PMF - with Breach (Breach bottom 881.0'; width 67.5')			
	Discharge (cfs)	Water Surface Elevation (ft)	Overbank Velocity (ft/sec) Left	Right	Discharge (cfs)	Water Surface Elevation (ft)	Overbank Velocity (ft/sec) Left	Right
14.842	16,644	888.0	0.78	0.16	23,547	889.9	0.42	0.11
13.938	14,549	887.8	0.15	0.22	19,282	889.7	0.16	0.23
13.755	14,549	887.7	0.24	0.20	19,282	889.7	0.25	0.20
13.113	14,467	886.0	0.93	0.61	18,124	888.4	0.74	0.60
11.207	14,435	878.4	1.46	1.45	18,000	879.3	1.32	1.61
9.340	14,345	873.7	0.69	0.31	17,719	875.1	0.72	0.31
6.214	14,345	858.5	1.85	1.30	17,719	859.7	2.07	1.58
	Water surface elevations and velocities were obtained from the HEC-RAS model assuming widening of the SFLR to an 80-foot bottom. Elevations are NGVD (1927)							

In addition to the 50% PMF, other scenarios were also modeled in HEC-HMS and HEC-RAS. The results of these runs were used to develop a functional relationship between the discharge in the river and the corresponding water surface elevation. This relationship was used to estimate flood levels for the SFLR in the vicinity of the hypothetical breach location from the HEC-HMS hydrographs at different times during the 50% PMF. Buckeye Lake water levels and the SFLR flood levels during the 50% PMF are presented in Figure 2, on the following page.

A comparison of the water levels in Figure 2 indicates that the river begins to flood about the same time that the Sellers Point Spillway begins to discharge (1.5 hours after the start of the storm). Buckeye Lake reaches its peak elevation much more quickly than the river. When the lake level exceeds the top of the dam, 5.7 hours after the start of the storm, the floodwater against the outside of the dam is about 4.5 feet deep (river elevation 885.6'). When the lake level peaks 3 hours later, the downstream flooding is about 5.5 feet deep. The river continues to rise as the lake level recedes. The SFLR flooding crest is 10 hours after the lake level crest, at which time downstream flooding is approximately 7 feet deep.

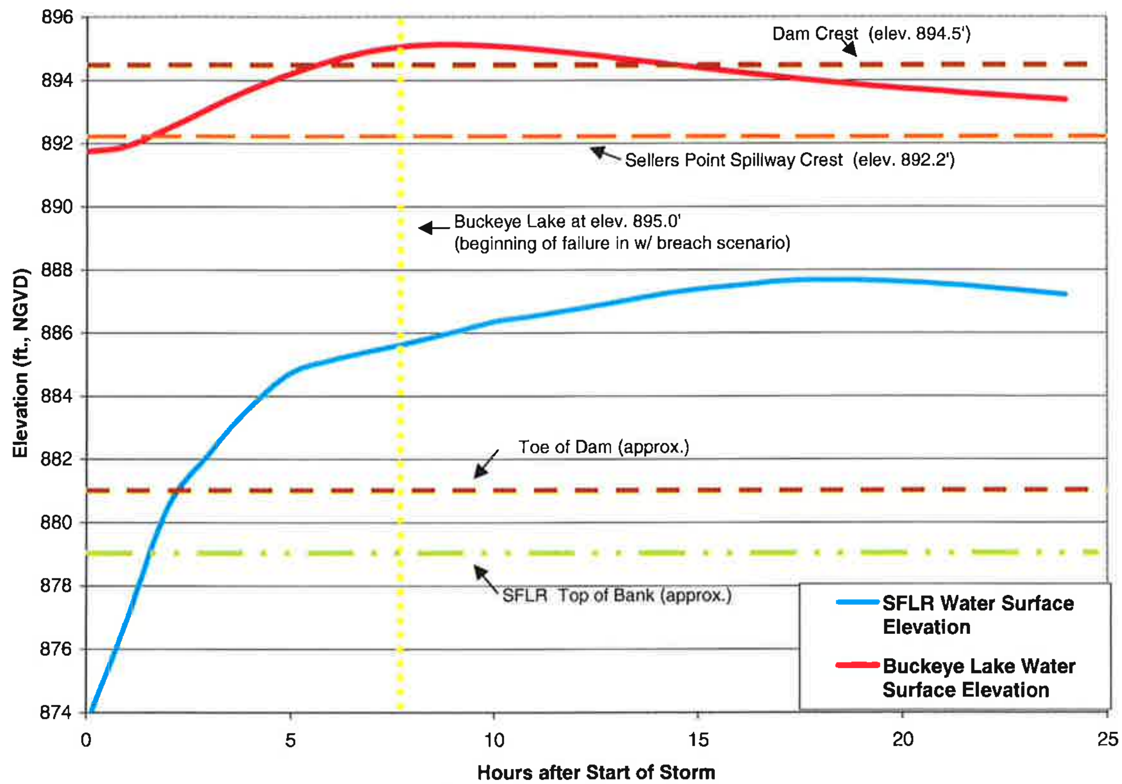

Figure 2. SFLR and Buckeye Lake Levels during the 50% PMF (no Breach.)

Evaluation of Results

Comparison with Critical Flood Criteria. The projected impacts of the with- and without-breach discharges are to be compared for the "critical routing reach." The regulations define the critical routing reach as beginning at the dam and extending downstream where life, health or property is potentially affected by failure of the dam. For the SFLR, the critical routing reach extends to the confluence of the SFLR and Ramp Creek. Previous modeling work has shown that the peak discharges in the SFLR downstream of Ramp Creek are controlled by local inflows that originate downstream of the eastern I-70 bridge over the SFLR. Thus, there is a well-defined downstream limit of the critical routing reach. The data on water surface elevations and overbank velocities presented in Table 2 have been used to calculate the values needed for comparison with the Critical Flood Analysis numerical criteria.

As can be seen by comparison with the numerical criteria from the regulations presented in Table 3, there are two locations where the water surface increase exceeds the 2.0 foot criterion. All locations meet the 7.0 ft-ft/sec criterion for the (flow velocity) * (increased water surface elevation). The incremental depth of flooding resulting from the dam breach occurs in addition to the significant flood damage caused by the 50% PMF. In such cases, the additional flooding may not significantly increase the actual damages.

Table 3
Buckeye Lake Dam
Comparison of with- and without-Failure Scenarios
for the Critical Routing Reach
50% PMF and Breach to Elevation 881.0' (NGVD)
Breach Width 67.5 ft.

SFLR Location	Miles downstream of Breach	Water Surface Elevation Increase over 50% PMF (ft)	Maximum Overbank Velocity (ft/sec)	Product of Overbank Velocity and Water Surface Elevation Difference (ft-ft/sec)
SFLR closest to Breach	0.56	1.8	0.42	0.8
SFLR at I-70 (east crossing)	1.47	1.9	0.23	0.4
SFLR between I-70 and SR 79	1.65	2.0	0.25	0.5
SFLR at Confluence with Lake Outlet	2.29	2.5	0.74	1.8
SFLR downstream of US 40 (incl. Hebron Run)	4.20	0.9	1.61	1.5
SFLR at Ridgely Tract Rd. (USGS gage site)	6.06	1.4	0.72	1.0
SFLR upstream of Ramp Creek Confluence	9.19	1.0	2.1	3.1
Critical Flood Analysis Guidance Criteria	n/a	< 2.0	n/a	< 7.0

Sensitivity to Data and Assumptions. The "with breach" scenario for the above analysis used conservative values from the ranges of parameters recommended by the Corps of Engineers and FERC. The sensitivity of the simulation results, especially the resulting water surface elevations, was tested with other parameter values and assumptions. The sensitivity analyses indicate that:

1. Modeling results are insensitive to the time to develop the breach, even if an almost instantaneous collapse (0.1 hr –the shortest permitted by DAMBRK) is used.

2. Modeling results are insensitive to the lake level at which the breach is assumed to occur (the most conservative case evaluated was for the breach to occur when the lake level was only 0.1 ft above the dam crest).

3. Based on a simplified analysis, the water surface increase would be less than the 2.0 ft criterion if the bottom of the breach was at 882 to 883 ft (1 to 2 feet higher than was used in the simulation) (keeping the width at 67.5').

4. Based on a simplified analysis, the water surface increase would be less than the 2.0 ft criterion if the width of the breach was less than 50 +/- ft (a height to width ratio of 3.5:1 instead of the 5:1 ratio that was used in the simulation; keeping the bottom at 881.01').

Virtually all past dam failures, from which the breach parameter ranges were developed, were for taller dams (many were much taller) with larger impoundments than the Buckeye Lake Dam. Neither the Corps of Engineers nor the FERC provide guidance regarding the selection of breach parameters from the ranges provided. Thus, there is no guidance whether the values used are realistic or appropriate for this application.

It is possible that the presence of the floodwaters on the outside of the dam, siltation on the inside of the dam, or the presence of the sheetpiling would prevent the breach from forming all the way to the outside toe of the slope. As indicated by the sensitivity analyses, reductions in the depth or width of the breach may reduce the breach outflow sufficiently that the Critical Flood criteria for the 50% PMF would be met.

Review of Site Specific Loss of Life, Health or Property Potential. Meeting the numerical criteria does not necessarily indicate that the flood being analyzed is the Critical Flood per the regulations. A review must also be made of site specific conditions that may pose special potential for additional loss of life, health or property. Examination of the Buckeye Lake setting identified two situations to be addressed qualitatively: the effect of breach location; and potentially affected structures.

A qualitative screening of all potential dam overtopping locations was performed to assess the effect of the location of the assumed breach on the Buckeye Lake Dam failure scenario and the flood impact comparisons. In addition to the general area in which the assumed breach is located, six other sections of the dam are below 895.0' and will experience overtopping flow during the 50% PMF. These six areas and the area adjacent to the assumed breach location were reviewed for the presence of features that could be especially susceptible to breach-related flood impacts.

After reviewing existing development downstream from the lake, it was concluded that several other locations had houses built into the dam and one or two houses approximately 200 feet downstream, similar to the development at the site of the assumed breach. The likely flow path of breach waters from these other locations to the SFLR was primarily over vacant or agricultural land. None of the other potential breach locations faced development as sensitive as the trailer park 900+ feet downstream of the assumed breach location. Thus, it is believed that none of the other breach locations would embody a greater potential for loss of life, health or property than the assumed breach location.

Structures close to a breach face an additional potential peril as the water cascading from the breach spreads out and transitions from the lake water surface elevation to the flood elevation in the floodplain. For the assumed breach conditions, the water surface elevation difference between the lake elevation and the floodplain flooding is 9.6 feet at the initiation of the breach. As the water pours from the breach, it falls and spreads out, such that at some (undetermined) distance from the breach, the breach discharge flow is indistinguishable from the general flood flow from the SFLR.

The behavior of flow along the principal flow path of the plume from the breach is of concern if there are structures downstream that would be exposed to higher velocities or transient water depths from the plume that are not reflected in the riverine flood modeling with HEC-RAS. A quantitative analysis of the breach flow downstream from the breach would require modeling that is beyond the capability of this analysis. Instead, similar phenomena have been researched and are used to provide guidance.

Common experience indicates that the height of the plume from the breach will fall rapidly. Similar examples of unconfined water flows include the nappe in weir overflow, the behavior of storm sewer outfall flow as it cascades from a full outfall pipe and water coming out of a hose lying on pavement. In all of these examples, the free water surface falls rapidly to become a shallow, almost level spreading flow away from the source. This fall-off will be more marked in the case of a dam breach because, unlike the examples, the water cascading from the breach has negligible velocity prior to emerging from the breach.

The second aspect of concern is the velocity of the flow from the breach. As the water moves away from the breach, some will remain in an effective flow regime, while other water will pass off to the side and be removed from the flow (i.e., entering storage areas off the flow path). Because of the large volume of storage available for the plume to spread into on either side of the plume from the breach, it is unlikely that the velocity of the plume from the breach will pose a threat to downstream development.

A third concern is unique to Buckeye Lake – the fate of houses built into the dam at the site of the assumed breach. The Critical Flood Regulations require that dam failure by overtopping be assumed. Based on aerial photography of the houses built into the dam in the vicinity of the assumed breach, it is unlikely that a breach greater than 10 feet could form unless an adjacent structure fails. To assume a breach width greater than 10 feet (as was the case in this analysis) is to assume that a structure fails. Failure of a structure would indicate a potential for loss of life. Without information on the structures and foundations, and assumptions about the nature of the erosion adjacent to the houses, specific statements about their fate are not possible. Nonetheless, the additional potential for loss of one or more houses relative to the base case i.e., without the breach) exists.

Summary of the Evaluation of Results. Using the conservative breach parameters from the Corps of Engineers and FERC parameter ranges, the with- vs. without-breach analysis does not pass the numerical criterion for maximum allowable water surface elevation increase so the 50% PMF cannot be considered the Critical Design Flood for Buckeye Lake. However, using values 20 to 25 percent less than the most conservative parameters for breach height and/or breach width (still within the acceptable ranges), it appears that all numerical criteria would be met. An evaluation of the additional potential for loss of life, health or property must address the fate of the houses built into the dam at the site of the assumed dam breach. If the assumed failure scenario includes the collapse of a structure or entails conditions that pose a substantial risk of structure failure, then the failure of the dam would result in additional potential of loss of life, health or property.

During the review of the modeling results and again in the consideration of the fate of buildings built into the dam, the question arose whether the dam really would fail during the 50% PMF. Whether the dam fails or not makes a big difference to the structures lining the dam. As described above, during the 50% PMF, Buckeye Lake levels exceed the lowest points along the crest of the dam for less than nine hours, and the maximum depth of overflow, 7 inches, occurs for less than two hours of that time.

Given the limited duration and magnitude of the dam overtopping flows, the analysis of the 50% PMF turned to the Critical Flood Analysis assumption that the dam would fail by overtopping. The following section describes the mechanisms and processes involved in dam overtopping failures. This knowledge base is then applied to assess the likelihood of an overtopping failure at the hypothetical Buckeye Lake Dam breach location.

CRITICAL REVIEW OF THE ASSUMPTION THAT THE BUCKEYE LAKE DAM FAILS DURING THE 50% PMF

Mechanics of Dam Breach by Overtopping

An embankment behaves as both a weir and channel when overtopping flow occurs. The discharge over the top of the dam can be quantified by the broad crested weir equation and is a function of the depth of the overflow and the width of the overflow zone. After passing through critical depth at the outer edge of the crest, the overtopping flow becomes supercritical as it flows down the outer slope of the embankment. The velocities increase as flow passes through critical depth, so that there is some erosion that begins on the downstream shoulder of the dam crest, but the greatest energy is at the base of the slope, and it is here that the more significant erosion process typically occurs. The discharge across the dam is not expected to increase appreciably until the downstream shoulder erosion works back to the upstream edge of the crest, or until the headcut erosion progresses from the downstream toe to the upstream edge of the crest (assuming a constant lake elevation).

Figures 3 and 4 are pictures of a field embankment breaching experiment at the U.S. Department of Agriculture Agricultural Research Service (ARS) facility in Stillwater, Oklahoma. The pictures present two of the three major processes in the failure of an embankment by overtopping:

1. The development of the initial cut at the base of the embankment (where the energy of the overtopping water is dissipated) (Figure 3); and

2. The progression of the headcut towards the upstream edge of the embankment crest (Figure 4).

Figure 3. Initial Overflow and Energy Dissipation at Toe of Slope.

Figure 4. Headcut Progression to almost Full Breach.

The third process (but first process in time) is the failure of the vegetative cover, exposing the underlying soil to erosion.
Although there is some, downcutting erosion by the water passing over the top of the embankment, the primary soil wasting process is the development and progression of headcuts into the slope and at the toe of the embankment. Once the headcut reaches the upstream edge of the embankment crest, then additional downcutting significantly increases the discharge over the embankment and the development of the full breach

(depth and width) is relatively rapid. The development of the initial cut is delayed by the presence of a vegetative cover. The condition of the vegetative cover, especially its coverage and uniformity, determines how well it can resist failure. Any discontinuities present will concentrate erosive forces, to the detriment of the entire cover. Vegetation offers little protection once the headcutting begins.

In general, the higher the dam, the greater the energy of the overtopping flow as it hits the toe of the slope. The height of the Buckeye Lake Dam at the hypothetical breach location is 13.5 ft., but at the time that overtopping flow begins it is only 9 feet to the downstream floodwater. The floodwater becomes the "toe" of the slope. This has two consequences; it:

1. Reduces the length of fall of the overtopping flow (9 feet, and decreasing as the downstream floodwaters continue rising); and

2. Dissipates some of the energy of the overtopping flow so that less energy is available to erode the embankment.

Assessment of Buckeye Lake Dam Breach Development

Assessing dam breach development is very similar to assessing the integrity of an earthen spillway during an overflow event. For the Buckeye Lake situation the primary difference is the steeper exterior slope and the fact that less information is available concerning the embankment materials. Three methodologies for assessing the development of a breach were identified in the professional literature. The first, and oldest, compares expected overtopping conditions with the Maximum Permissible Velocity. The second approach uses the qualitative and quantitative results of on-going research conducted by the Agricultural Research Service at their Stillwater, Oklahoma field research station. The third analysis is to model the Buckeye Lake conditions with the SITES reservoir design and evaluation model. The following sections discuss the application of these methods to the Buckeye Lake Dam.

Maximum Permissible Velocity. A Maximum Permissible Velocity[4], for a stated vegetation and soil condition, is the velocity below which erosion of the channel would not be expected to occur. It follows, that if velocities exceed the Maximum Permissible Velocity then erosion is expected. Generally, a velocity of around 5 ft/sec is considered the maximum permissible for a clay soil with grass cover. Permissible Velocities were developed for vegetated channels, so their application to steeper situations such as a spillway or dam embankment may not be entirely applicable.

The velocities of the dam overtopping flows are presented on the following page in Table 4, which presents discharge, velocities and flow depths for an assumed 10-foot wide strip of embankment between adjacent houses on the dam for the 9-hour period that flow overtops the dam. In the vicinity of the hypothetical breach location, the distance between houses ranges from approximately 3 to 21 feet, with a mean width of 10 feet between the 6 houses on either side of the breach location. The assumed width of the exposed dam affects the discharge, but not the velocities. As the table entries indicate,

[4] U.S. Department of Agriculture, Soil Conservation Service, "Handbook of Channel Design for Soil and Water Conservation," TP-61, 1954.

the velocities of flow down the embankment become high enough to erode through the vegetation and scour exposed soil for almost all levels of overtopping.

Table 4
Buckeye Lake Dam
Characteristics of Flow Overtopping the Dam
50% PMF, no Breach

Depth of Flow over Dam Crest (ft)	Total Duration (hr:min)	Discharge (cfs)	Flow on Downstream Slope of Dam (supercritical)	
			Maximum Velocity (ft/sec)	Flow Depth (ft)
0.6 - 0.613	1:50	12.8	8.90	0.1
0.5 - 0.6	1:50	11	8.40	0.1
0.4 - 0.5	1:10	8.2	7.40	0.1
0.3 - 0.4	1:00	5.6	6.40	0.1
0.2 - 0.3	1:00	3.4	5.20	0.1
0.1 - 0.2	1:00	1.6	3.9	0.0
0.0 - 0.1	1:00	0.3	2	0.02

8:50 Total duration of overtopping flow

Table Notes:
1. Discharge, Velocity and Depth values are for the midpoint of the depth range.
2. Discharge is for a 10-foot wide section of the dam.

The Permissible Velocity method does not provide any information about the magnitude or rate of erosion. To assess the significance of the erosion during periods when the velocities exceed the Maximum Permissible Velocities, other characteristics of the overtopping flows must be considered. Of particular note in Table 4 are the very shallow depths of flow and the limited times over which each velocity occurs. In addition, the presence of floodwaters from the SFLR will limit the ability of the overtopping flows to transport the eroded soil away from the dam face. It is unlikely that the very shallow depths of flow and the short exposure to the highest velocities would be sufficient to breach the 20-foot wide crest of the dam.

Application of ARS Field Research Results. ARS staff have published numerous papers on their field-scale experiments on overtopping failure of earthen embankments. Figures 3 and 4 were pictures from an overtopping experiment with 6-foot high embankments.[5] Figure 5, from the ARS field research shown in Figures 3 and 4, presents the rate of headcut advance through a 6 foot high embankment composed of 63% sand, 31% silt and 6% clay with a 3:1 side slope. The vertical profiles drawn on the embankment profile are the location of the headcut as it advances upstream. For these experiments, a head of 1.5 feet was maintained on the crest of the embankment. Figure 5 (on the following page) illustrates that the headcut reached the upstream edge of the crest after about 5 hours for this experiment. A similar test embankment composed of 25% sand, 49% silt and 26% clay did not breach after 20 hours of testing.

[5] Hanson, G.J., Cook, K.R., and Hahn, W., "Evaluating Headcut Migration Rates of Earthen Embankment Breach Tests," ASAE Meeting Paper 01-012080, presented at the 2001 ASAE International Meeting, Sacramento, CA, St. Joseph, MI: American Society of Agricultural Engineers, 2001. This paper is reproduced in Appendix A.

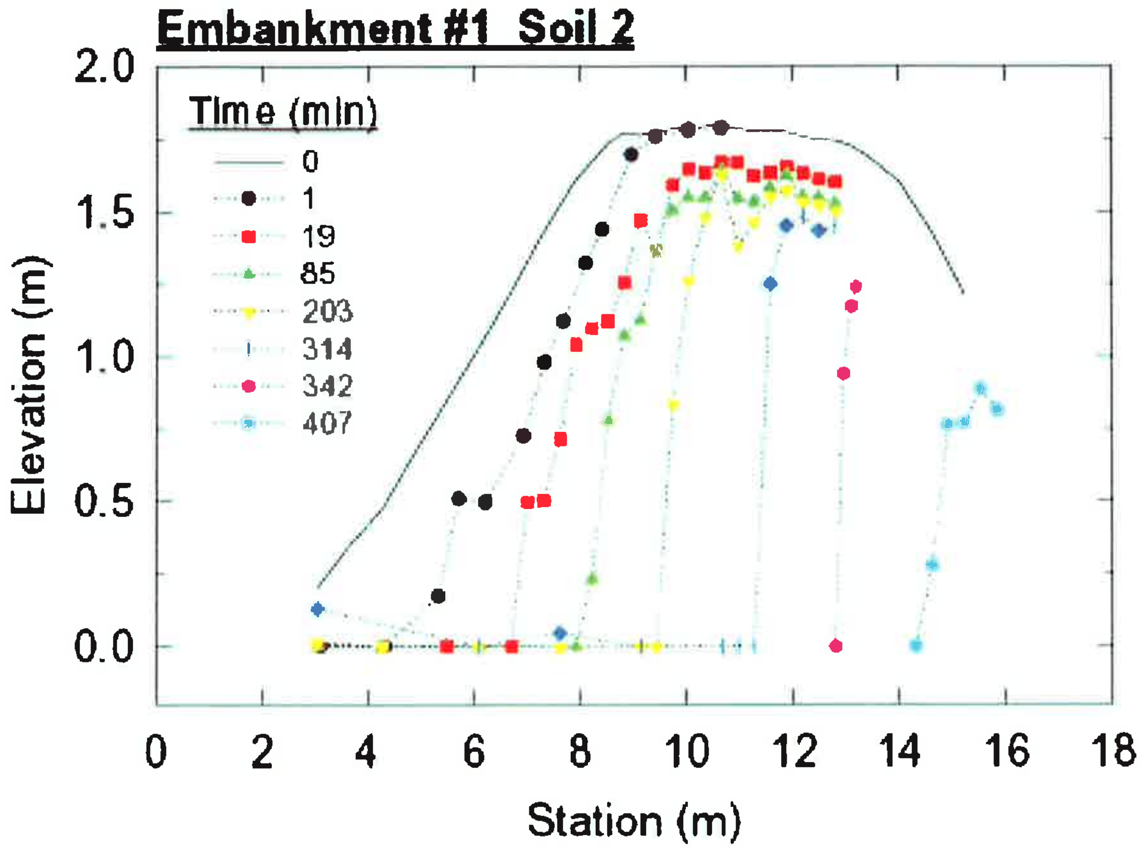

Figure 5. Headcut Progression with Time.

Geotechnical results from a soil boring[6] near the hypothetical breach location were reviewed for comparison with the ARS field research results. This area of the dam is composed of 25% sand (and larger), 42% silt and 33% clay. In contrast to the ARS experimental conditions, the modeled maximum overtopping head for the 50% PMF is 0.6 feet and it occurs for only 2 hours. The higher clay content and lesser overtopping depth and duration would make the Buckeye Lake Dam less likely to breach than the experimental embankment, though the greater height (9+ feet versus 6 feet) and steeper side slope (2.3:1 versus 3:1) would tend to make the Buckeye Lake Dam less resistant to breaching than the experimental embankment. In balance, the comparison to the 26% clay test embankment indicates that the Buckeye Lake Dam would not breach.

The ARS prepared two predictive relationships for the advance of the headcut. The relationships take into account the energy of the overtopping water, quantified as the product of the unit discharge and the height of the headcut. The material of the embankment is characterized by either the unconfined compressive strength or the soil erodibility coefficient. The soil erodibility coefficient is difficult to estimate from other

[6] The soil borings were performed by Dodson-Lindblom Associates and reported in the "Buckeye Lake Dam Report" (ODNR, 1987). Boring 5 of cross section 4 was located approximately 600 feet southwest of the hypothetical breach location. The soil sample was collected from the 4.0 to 4.5-foot interval.

parameters. The standard penetration tests from three soil borings[7] near the hypothetical breach location were used to estimate an unconfined compressive strength of 2 tons/SF. Using this value and the relationship from the ARS, the rate of headcut advance (assuming the depth of headcut = 9 feet) is 0.016 ft/min. At this rate, the 20-foot wide crest section of the dam would withstand overtopping at the 0.6 foot depth for 20 hours.

Application of SITES Model. The SITES model[8] was developed to assist in the design of earthen dams and spillways. The most recent version (2000-1) includes algorithms to evaluate the ability of earthen spillways to withstand erosion during overtopping events. The SITES model was used to simulate the Buckeye Lake Dam overtopping by entering the characteristics of the dam profile as the auxiliary spillway and the overtopping flow hydrograph from HEC-HMS as the inflow hydrograph. The SITES model has not been tested for applications with slopes as steep as 2.3:1, therefore the SITES model results are only indications of the significance of erosion during overtopping. The model also provided an opportunity to observe the sensitivity to model parameters.

For the 50% PMF at the hypothetical Buckeye Lake Dam breach location, the SITES model predicted that the headcut would not progress beyond the downstream shoulder of the crest. The vegetative cover failed and concentrated flow developed 5 hours into the overtopping event. When the assumed quality of the vegetative cover was changed from very poor to fair, the vegetative cover did not fail at all and there was no headcutting. Variations of +/-50% in the Headcut Erodibility Index[9], the parameter that describes the resistance of the embankment soil to erosion, did not change the model results, indicating that the controlling factor limiting erosion is the limited energy and short duration of the overtopping flows. SITES does not model the effects of downstream inundation of the toe of the embankment.

Discussion of Dam Failure Critical Review

The above evaluations of the potential for dam failure have used the 50% PMF overtopping hydrograph from HEC-HMS and geotechnical information from the 1987 Buckeye Lake Dam report because they are the best estimates of the PMF event and dam characteristics available. This analysis has focused on the hypothetical breach location described on page 3 because the areas of lowest crest elevation will be subjected to the greatest overtopping flow. The West Bank is reportedly in poorer condition than the North Bank. This analysis did not investigate the West Bank because the dam crest is higher there. Also, the failure mechanism under consideration is erodibility. This investigation has not addressed the structural integrity of the dam.

Other factors may increase or decrease the magnitude or duration of the overtopping flows or modify the characteristics of the dam. The effect of such changes could be

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7 Borings 4, 4A and 5, all at cross section 4. The blow counts (blows/ft) for tests in material characterized as "Fill" were averaged.

8 U.S. Department of Agriculture, Natural Resources Conservation Service, "Water Resources Sites Analysis Program," 2001. Computer program and users manual obtained from the NRCS web site. http://www.wcc.nrcs.usda.gov/water/quality/sites/sites/html

9 U.S. Department of Agriculture, Natural Resources Conservation Service, "Headcut Erodibility Index," Photo Reference DMT-9-'93, 1993. Obtained from the NRCS web site. http://www.wcc.nrcs.usda.gov/water/quality/sites/sites/html

evaluated by re-running the calculations or comparisons with the new data. An extensive evaluation of the sensitivity of the results to a range of alternative data is beyond the scope of the current analysis. Instead, the effects of variations in the assumed 50% PMF conditions are discussed in general terms.

Factors that could increase the lake level and therefore the duration and magnitude of overtopping flows include wave action, wind set-up, changed PMP hydrology or location of the PMP. Clearly, as the head on the dam and the duration of overflows increase, more erosion of the downstream embankment occurs. The comparison with the ARS high-clay embankment, the calculation of the headcut advance rate, and the SITES modeling all indicated that the 50% PMF as evaluated would cause only minor damage to the embankment. Therefore, the dam can likely withstand additional overtopping flow beyond that modeled, though the cutoff point beyond which a breach is inevitable cannot be calculated or predicted except in very broad terms given the current state of the art. The assessment that only minor damage would occur from the 50% PMP as modeled contributes a factor of safety to the conclusion that the dam would not breach.

The Critical Flood Analysis Guidelines, upon which the analyses in this report were based, do not consider actions that may be taken to reduce the likelihood of a dam failure or to reduce the impact of a dam breach if one does occur. Buckeye Lake is an actively monitored and managed ODNR facility. Although ODNR does not usually draw the lake down to provide flood control capacity, the means to do so are available and the lake has been drawn down on several occasions in the past. The lake is regularly drawn down three feet during non-boating months (i.e., late fall to early spring), which provides additional storage for winter and spring storms. ODNR maintains full time staff at the Buckeye Lake State Park office so that changed conditions can be recognized and addressed. The ODNR has mobilized to place sandbags at weak spots and along low areas of the dam crest during past storms. If, despite ODNR's response activities, a dam breach by overtopping appears imminent, ODNR staff will be on hand to initiate emergency evacuation actions in conjunction with local and state authorities.

CONCLUSION

This Project Memorandum has summarized an assessment of the performance of the Buckeye Lake Dam relative to the 50% PMF. Based on a review of research on erosion of earthen embankments, applications of the SITES model to configurations similar to the Buckeye Lake Dam, calculations of headcut advancement, and a consideration of the characteristics of the Buckeye Lake Dam and modeled watershed runoff conditions, the dam is not likely to fail from an overtopping breach during the 50% PMF. This is a specific conclusion drawn from a specific set of circumstances and assumptions as described in this Project Memorandum. The analyses and evaluations would change, and at some point the conclusion of whether the dam is likely to fail would change also, if storms greater than the 50% PMF or if other assumptions were considered. In addition, since this analysis does not address the structural integrity of the dam, no conclusions about the structural integrity can be drawn from this effort.

The current combination of spillway capacity and lake storage (dam height) does not unambiguously satisfy the conditions necessary for the 50% PMF to be considered the Critical Flood for regulatory and design purposes because the incremental water surface elevation exceeded the 2.0 ft criterion for the most conservative breach parameters analyzed. Also, the available information does not resolve questions concerning the potential for additional loss of life, health or property with regard to the fate of the structures built into the dam in the event of overtopping flow, erosion and dam breaching. Despite these uncertainties, we do not believe additional analysis to determine the Critical Flood in accordance with the regulations is warranted, considering that a new, stand-alone structural dam with a crest elevation of 895.0 is being designed.

APPENDIX A

Evaluating Headcut Migration Rates of Earthen Embankment Breach Tests

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EVALUATING HEADCUT MIGRATION RATES OF EARTHEN EMBANKMENT BREACH TESTS

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Abstract. *There are a number of recognized technology advancement needs associated with watershed structure re-evaluation and rehabilitation. One of those needs, addressed by embankment breach research, is technology for predicting performance during extreme events. Preliminary evaluation of the headcut migration rates during overtopping and breaching tests on large-scale models are described in this paper. Two large-scale embankments, 2.3 m and 1.5 m high, have been tested. Each embankment had three homogeneous test sections with 3H:1V slopes on both the upstream and downstream sides, consisting of a different soil material in each section, two non-plastic SM silty sand materials and a CL lean clay. The test sections for the 2.3 m embankment are 7.3 m wide and the test sections for the 1.5 m embankment are 4.9 m wide. Headcut migration rates differed from 0.15 to 0.003 m/min for the 2.3-m high embankment and 0.15 to 0.001 m/min for the 1.5-m high embankment depending on material type. Measured soil properties help account for the difference in migration rates of the materials.*

Keywords. Embankment, breach, headcuts, energy dissipation, erodibility, soil parameters.

EVALUATING HEADCUT MIGRATION RATES OF EARTHEN EMBANKMENT BREACH TESTS

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ABSTRACT

There are a number of recognized technology advancement needs associated with watershed structure re-evaluation and rehabilitation. One of those needs, addressed by embankment breach research, is technology for predicting performance during extreme events. Preliminary evaluation of the headcut migration rates during overtopping and breaching tests on large-scale models are described in this paper. Two large-scale embankments, 2.3 m and 1.5 m high, have been tested. Each embankment had three homogeneous test sections with 3H:1V slopes on both the upstream and downstream sides, consisting of a different soil material in each section, two non-plastic SM silty sand materials and a CL lean clay. The test sections for the 2.3 m embankment are 7.3 m wide and the test sections for the 1.5 m embankment are 4.9 m wide. Headcut migration rates ranged from 0.15 to 0.003 m/min for the 2.3-m high embankment and from 0.15 to 0.001 m/min for the 1.5-m high embankment depending on material type. Measured soil properties help account for the difference in migration rates of the materials.

INTRODUCTION

Interest in the occurrence and effects of overtopping of earth embankments by storm runoff has existed for years. Based on conclusions made by Ralston (1987) there are about 57,000 dams on the national dam inventory that have the potential for overtopping. Reservoirs overtop as a result of inflow exceeding the capacity of the reservoir and spillway system, since this risk can never be completely eliminated, the challenge is determining how these embankments will perform in advance of overtopping.

Ralston (1987), in his discussions of dam overtopping, distinguishes between cohesive and non-cohesive soils and their erosion characteristics. For cohesive soils in embankments with backslopes in the range of 2.0 or 3.0 horizontal to 1.0 vertical, overtopping results in eventual degradation of the surface, formation of a discontinuity, and development of an overfall or headcut. The headcut advances progressively headward as the base of the headcut deepens and widens. Failure and breach occur when the headcut migrates through the upstream crest of the dam. The headward advance was attributed to a combination of a) insufficient soil strength to stand vertically due to the height of the face, stress relief cracking and induced hydrostatic pressure in the stress cracks and, b) loss of foundation support for the vertical face due to the waterfall flow plunging effect and its associated lateral and vertical scour. This process of combined slope mass wasting and hydrodynamic action has been described briefly for naturally occurring gullies (Piest et al., 1975). Al-Qaser (1991) also observed similar

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processes in his flume studies on embankment overtopping. The physical processes involved in the erosion of an embankment are of importance in determining rate and amount of damage to the embankment, as well as the amount and timing of discharge from the reservoir if it does breach.

Stein and Julien (1993) describe two modes of headcut migration: 1) Rotating headcuts that tend to flatten as they migrate; and 2) stepped headcuts that tend to retain nearly vertical faces. Rotating headcuts and stepped headcuts depend on the relative erosion rates upstream and downstream of the headcut. The morphology of rotating and stepped headcuts depends on the time scales of the upstream and downstream scour. The upstream time scale denotes the time for upstream vertical scour to reach the toe of the vertical face. The downstream time scale denotes the time required for the impingement scour to develop horizontally and reach the toe of the vertical face. Stein and Julien (1993) hypothesized that the ratio of these two time scales could be used to delineate the mode of headcut migration. Conceptually rotating headcuts would be obtained if upstream erosion was dominant and the ratio of time scales was less than 1. Stepped headcuts would be maintained if downstream erosion was dominant and the time scale ratio was greater than 1. For the limiting case of homogeneous cohesive soils, the time-scale ratio was determined to be a simple function of the Froude number, F and the aspect ratio or headcut height to normal flow depth.

Predicting the rate of headcut migration is a complex problem. Simple relationships for headcut migration have almost universally focused on energy at the overfall as the driving mechanism (De Ploey, 1989, Temple 1992, Moore et al. 1994, Temple and Moore, 1997). These formulations typically use some form of the unit discharge q, unit weight of water γ_w, and the headcut height H to describe the hydraulic attack in terms of energy dissipation at the headcut. Temple (1992) proposed a simple model describing headcut migration dX/dt based on a material dependent coefficient C and a hydraulic attack parameter A such that:

$$dX/dt = C(A) \qquad (1)$$

$$A = q^a H^b \qquad (2)$$

where:

a and b = exponents.

Moore et al. (1994) proposed that for earthen spillway applications headcuts would not advance until a hydraulic attack threshold was exceeded for a given headcut erodibility index. The headcut advance threshold was evaluated based on an energy dissipation term:

$$E = q\gamma_w H \qquad (3)$$

The headcut erodibility index parameter is a strength and structure based parameter that enables the user to characterize earth materials on a continuum from soil to rock (Moore, 1997) for resistance to headcut migration in earthen spillways.

Temple and Moore (1997) proposed an equation for headcut migration based on the premise of a material dependent rate coefficient and an excess hydraulic attack relation:

$$\frac{dX}{dt} = C(A - A_o) \qquad (3)$$

where

C = a material-dependent advance rate coefficient,

A = the hydraulic attack, and

A_o = a material dependent threshold level of attack below which advance does not occur.

The hydraulic attack A, is expressed in the form of energy dissipation raised to a power.

$$A = (q\gamma_w H)^{1/3} \qquad (4)$$

The material-dependent threshold A_o and the material-dependent advance rate coefficient C are a function of the headcut erodibility index.

An exception to the energy-based models is the simplistic headcut migration model based on soil strength, erodibility, and hydraulic stresses (Hanson et al. 2001).

$$\frac{dX}{dt} = \frac{Tk_d(\tau_e - \tau_c)}{E_v} \qquad (5)$$

where

T = distance of movement at each mass failure.

E_v= required erosion on vertical face required to cause the headcut to become unstable.

k_d = erodibility coefficient,

τ_e = effective stress, and

τ_c = critical shear stress.

This equation is based on the idea that headcut movement occurs in a cyclic manner, in which a mass-wasting event takes place. Even though movement occurs in a cyclic manner, the rate of migration may be described as the distance of movement T at each mass failure. Hanson et al. (2001) assumed the distance T of each movement was equivalent to H/2. The amount of erosion on the overfall face E_v, required to cause mass failure is a function of the soil strength and forces on the soil matrix.

Soil Erodibility and headcut migration are influenced by compaction water content, dry unit weight and soil type (Hanson et al. 1998, 1999). There are also other recognized factors that influence erodibility such as water temperature, water and soil chemistry, weathering, etc. Embankments are constructed of soil material and therefore are affected by these factors in an overtopping event. Therefore it is important to measure embankment soil properties to determine anticipated performance.

Experiments have been and are currently being conducted on the erosion processes associated with embankment overtopping of soil materials. The purpose of this paper is to describe 1) the experimental set-up, 2) observations of headcut migration during embankment overtopping tests, 3) compare observed headcut migration rates to measured soil parameters such as soil strength and soil erodibility, and 4) include a simple parameter of hydraulic attack qH for comparison of results.

EXPERIMENTAL SETUP

Embankment Experiments

Construction of embankment overtopping test facilities (Figure 1) started in the spring of 1998, which included 2.3-m and 1.5-m high embankments with 3:1 slopes on both the upstream and downstream sides. Overtopping tests for the 2.3-m high and 1.5-m high embankment were conducted in 1999 and 2000, respectively. Each embankment had three test sections. Each test section for the 2.3-m high embankment was 7.3 m wide, while the test sections for the 1.5-m high embankment were each 4.9 m wide. The homogenous test sections were constructed from three different materials, two non-plastic SM silty sand materials and a CL lean clay. Trapezoidal flow conveyance notches were cut into each test section to contain flow for the overtopping event (Figure 2). The dimensions of the notch for the 2.3 m high embankment were 0.46-m deep by 1.83-m wide at the base with 3H:1V side slopes, and for the 1.5 m high embankment the notch was 0.30-m deep by 1.22-m wide at the base with 3H:1V side slopes. In order to test each soil material individually, the notches in the other two test sections were filled on the upstream end with a soil plug. The height of the embankment at the notch crests was 1.83 m and 1.22 m for the 2.3 m and 1.5 m high embankments, respectively. The 2.3 m high embankment was seeded with Fescue to protect against rainfall. The stand was young and in generally poor condition at the time of testing. The vegetal condition at the time of testing soils 1 and 2 was poor to non-existent. The vegetal condition at the time of testing soil 3 was fair but had only been in place three months. The vegetal condition of the 1.5-m high embankment was poor to non-existent for all test sections.

Chart recorders were utilized to record inflow and outflow hydrographs. A 10-m wide overhead rolling carriage with attached point gage was utilized to obtain bed profiles, cross sections, and water surface elevations during testing (Figure 2). Inflow to the reservoir during testing was supplied by a canal and measured at the test site with a modified Parshall flume. The reservoir upstream of the test embankments was filled in advance of testing to a depth of 1.2 m and 1.0 m for the 2.3 m and 1.5 m high embankments respectively. On the day of testing, the reservoir was completely filled and the embankment test section overtopped. Maximum overtopping head attained prior to breach was 0.46 m and 0.30 m for the 2.3 and 1.5-m high embankments respectively. The inflow discharge stabilized quickly during each test, and was then maintained at a relatively constant flow of about 1 m^3/s and 0.3 m^3/s for the 2.3 m and 1.5 m high embankment respectively. This relates to a unit discharge of approximately 0.1 and 0.06 $m^3/s/m$ for the 2.3 and 1.5 m high embankment respectively. Flow was measured both as it entered the reservoir and at a point downstream. The downstream measuring device had a maximum measuring capacity of 1.7 m^3/s. This was not adequate capacity for all flows during the breaching process. Therefore, reservoir elevation and storage records were an essential part of evaluating the breach outflow. The storage outflow was determined by three means: (1) downstream V-notch weir; (2) a downstream H-flume; and (3) the change in storage measurements.

Reservoir stage-storage curves were determined by two methods: (1) The reservoir was surveyed and the volume determined by area enclosed at given contour elevations; and (2) The reservoir was filled with water and the volume of water required to fill to given elevations was measured. A best-fit stage-storage rating curve was developed from these two methods.

Pre and Post Test Soil Parameter Determinations

Pre-test soil material parameters were measured on the soil materials in each of the test sections of the embankments (Table 1). Dry unit weight, γ_d, and moisture content, ω, were determined for each compacted layer. Compacted lift thickness averaged 0.13 m. Particle grain size distribution and Atterberg Limits were determined from disturbed samples taken from each lift and tested by the Natural Resources

Conservation Service (NRCS) National Soil Testing Lab (NSTL) in Fort Worth, Texas. Unconfined compressive strength q_u tests were run on samples from each test section. Immediately following completion of an overtopping test, an erodibility test was performed utilizing a jet test device (Hanson and Cook, 1997) in accordance with a revised test procedure of ASTM D5852-95.

RESULTS AND DISCUSSION

Observations and data recorded during testing of the embankment sections indicated that overtopping of the crest initiated rill and microrill erosion (at different times depending on embankment material) on the downstream slopes of the embankments. The erosion eventually developed a network of rills that gradually developed into a master rill (i.e., gully). This dominant channel initially consisted of multiple cascading overfalls that in time migrated upstream through the downstream slope while simultaneously widening.

The 2.3-m embankment tests eventually formed a single large headcut that migrated upstream through the embankment (Figure 4). The headcut in soils 1 and 2 of embankment 1 migrated through the entire embankment leading to a breach and emptying of the reservoir. The headcut in soil 3 migrated approximately half way through the embankment prior to terminating the test after 20 hours of testing.

The 1.5-m embankment results were mixed. Soil 3 formed a single headcut that migrated upstream but never breached the structure even after 72 hours of testing. For soils 1 and 2, a larger headcut formed on the downstream slope but following this a combination rotating and stepped headcut were observed migrating through the embankment (Figure 5). During the early stages of headcut migration, downward erosion was observed in the crest that resulted in increased discharge due to releases of stored water from the reservoir below the original crest elevation. This may explain the tendency towards a rotating headcut as observed by Stein and Julien (1993) due to the increased upstream normal depth to the overfall height. The headcut in soils 1 and 2 of embankment 2 migrated through the entire embankment leading to breaches that emptied the reservoir.

The centerline point gage profile measurements were used to determine headcut location versus time for each test (Figure 6a-f). Headcut location for a vertical headcut was determined based on the location of the upstream face over the duration of the test. Headcut migration rates for the tests in which a combination of rotating and vertical headcuts existed were determined based on the bed location horizontally at a fixed elevation of typically 0.5-m over the duration of the test. A linear fit through the origin of the headcut location versus time was used to determine the rate of headcut migration. The headcut location versus time plots indicate that: 1) Headcut migration does not occur in a smooth continuous process but is rather cyclic in nature; 2) Headcut migration varies significantly with soil type; and 3) Headcut migration varies with scale. Even though the variation with scale is not as clear as the variation with soil type, the ratio of headcut migration rate of embankment 1 to embankment 2; for soil 1 was 1, for soil 2 was 2.7, and for soil 3 was 3.2. This indicates that the rate of headcut migration tended to increase with increase in embankment size.

In review of the models described in equations 1-5, the material dependent advance rate parameters were defined primarily by soil erodibility and soil strength. The erodibility coefficient k_d determined from jet tests on the soils following testing varied 400 fold from soil 1 to soil 3 (Table 1). The migration rate varied approximately 200 fold (Table 2). The erodibility coefficient increased as the headcut migration rate increased for each soil type within each embankment (Figure 7). The data for embankment 1 tended to plot above the data for embankment 2 but there is a definite relationship between headcut migration and soil erodibility as would be expected in the model described by equation 5.

The unconfined compressive strength q_u was observed to increase as the headcut migration rate decreased (Figure 8). The ratio of strength of soils 3 to soil 1 was approximately 4. It should be noted that the curves for the smaller scale embankment, embankment 2, in figures 7 and 8 tended to plot below those of embankment 1. Based on equations 1-4 this could possibly be explained by the difference in available energy dissipation for each of the two tests. The inflow discharge for the tests conducted on embankment 1 was 1 m^3/s or a unit discharge of 0.1 $m^3/s/m$ and the height at the notch was 1.8 m. Therefore the available energy dissipation could be estimated as qH, which is 0.18 m^3/s. The available energy dissipation for embankment 2 is 0.06 m^3/s. The ratio of the maximum available energy of embankment 1 to embankment 2 is 3.0. A comparison of the migration rate divided by the available energy dissipation (dX/dt/[60qH]) to the erodibility coefficient k_d and the unconfined compressive strength q_u is shown in Figures 9 and 10. These simple relations, shown in Figures 9 and 10, indicate that soil strength, erodibility, and energy dissipation, as described in equations 1-5 are pertinent in modeling the headcut migration erosion process during embankment overtopping.

SUMMARY AND CONCLUSIONS

Headcut erosion of embankment overtopping and breaching tests conducted on large-scale models was evaluated and described in this paper. The test set-up included a reservoir with two large-scale embankments, 2.3 m and 1.5 m high, respectively, on one end of the reservoir. Each embankment contained three homogeneous test sections, 7.3 m in width for the 2.3-m high embankment and 4.9 m wide for the 1.5-m high embankment. The embankments were constructed with 3H:1V slopes on both the upstream and downstream sides. The three test sections consisted of two non-plastic SM silty sand materials and a CL lean clay. Part of the erosion process during overtopping consisted of a headcut migrating either as a rotating headcut or a stepped headcut. The headcut migration rates for each test section were evaluated and compared to measured soil properties (i.e., erodibility and soil strength), and an available energy dissipation term qH. The rate of headcut migration varied as much as 200 fold between soil types and by as much as 3 times between the two embankment heights. The measured erodibility varied as much as 400 fold between soil types and headcut migration rate increased as erodibility increased. The measured soil strength varied as much as 4 fold between soil types and headcut migration rate increased as soil strength decreased. These results were in agreement with the simple equations describing headcut migration. A simple qH term taken from energy dissipation equations helps explain the differences in headcut migration rates between the two embankment scales tested.

The erodibility coefficient and unconfined compressive strength showed the anticipated relative relationship to rates of headcut migration. The next step in the analysis process will be evaluation of the erosion rates of the embankment during overtopping and determining the relationships between soil parameters, hydraulic parameters, and the discharge hydrograph. This will include: 1) a detailed analysis of the outflow hydrograph, energy dissipation, and hydraulic stresses over the duration of the overtopping test; and 2) a detailed analysis of the rate erosion width to headcut migration. This work should have an impact on future flood breach models.

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Table 1. Soil properties.

Emb	Soil	Sand[1] %	Silt[1] %	Clay[1] %	Disp[2] %	PI[3]	WC[4] %	γ_d[5] Mg/m^3	q_u[6] kPa	k_d cm^3/N-s	τ_c Pa
1	1	70	25	5	69	NP	8.7	1.72	20	10.3	0.14
1	2	63	31	6	0	NP	12.1	1.73	32	2.0	0.14
1	3	25	49	26	18	17	16.4	1.65	63	0.039	15
2	1	67	30	3	0	NP	11.5	1.73	22	14.2	0.14
2	2	65	29	6	2	NP	14.5	1.74	31	8.0	0.14
2	3	27	48	25	22	16	17.8	1.67	82	0.038	10

Note: The following ASTM Standards were used to measure soil properties: [1]D 422, [2]D 4221, [3]D 4318, [4]D 4959, [5]D 2937, and [6]D 2166

Table 2. Embankment height, testing discharge, and observed migration rates

Embankment	Soil	Height @ Notch m	Rate Of Inflow m^3/s	dX/dt m/min
1	1	1.8	0.98	0.147
1	2	1.8	0.98	0.010
1	3	1.8	0.98	0.0026
2	1	1.2	0.33	0.146
2	2	1.2	0.33	0. 0039
2	3	1.2	0.33	0.0008

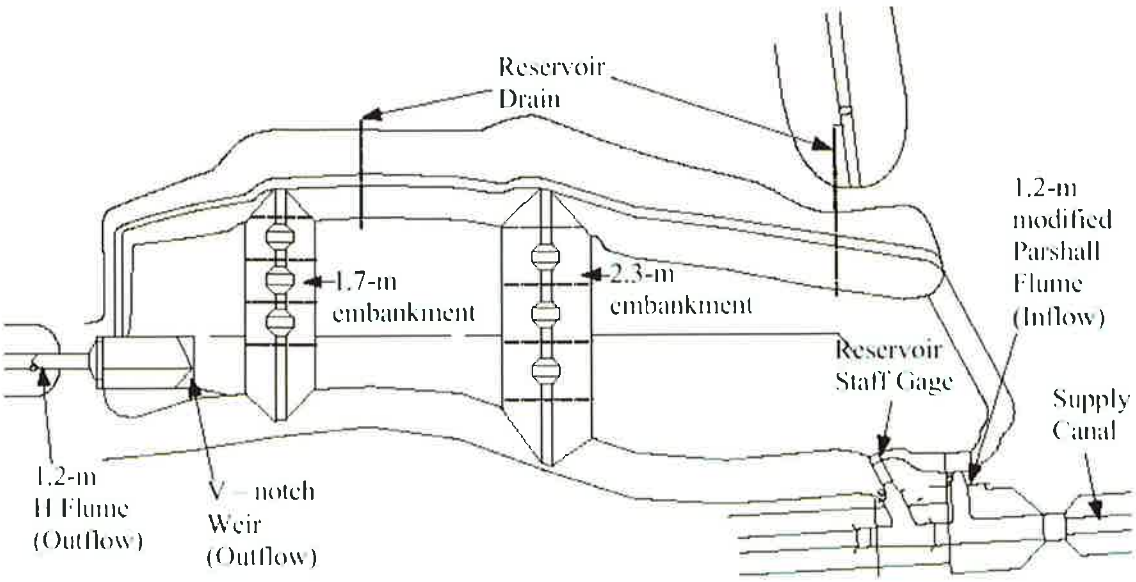

Figure 1. Schematic of embankment overtopping facilities.

Figure 2. Embankment overtopping test with trapezoidal notch to conduct flow.

Figure 3. Overhead carriage used to take point gage readings of erosion profiles.

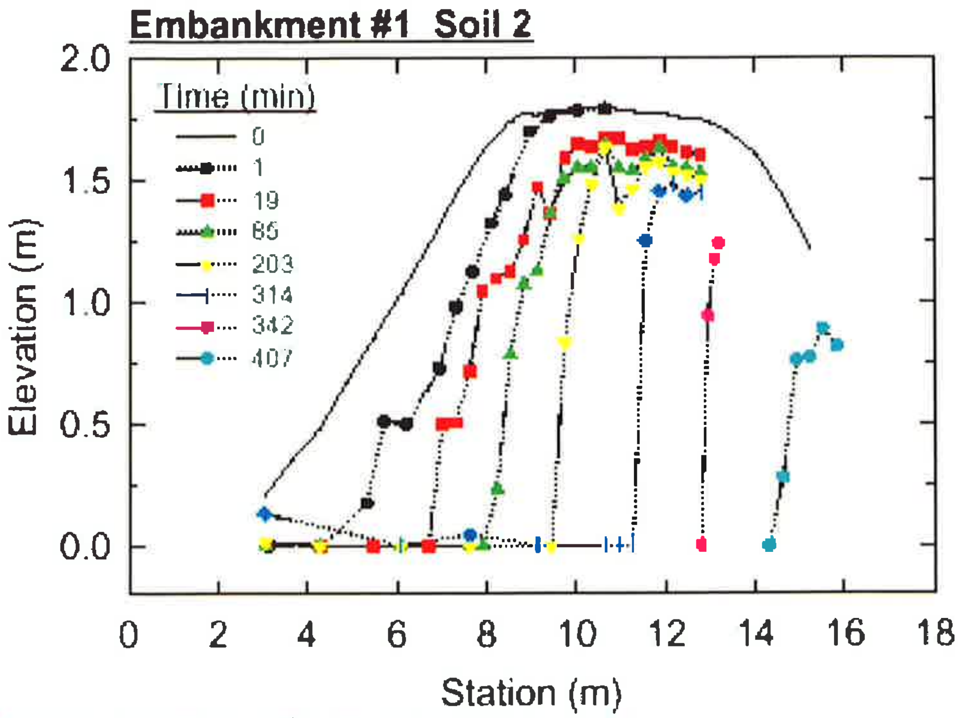

Figure 4. Centerline profile for embankment 1, soil 2.

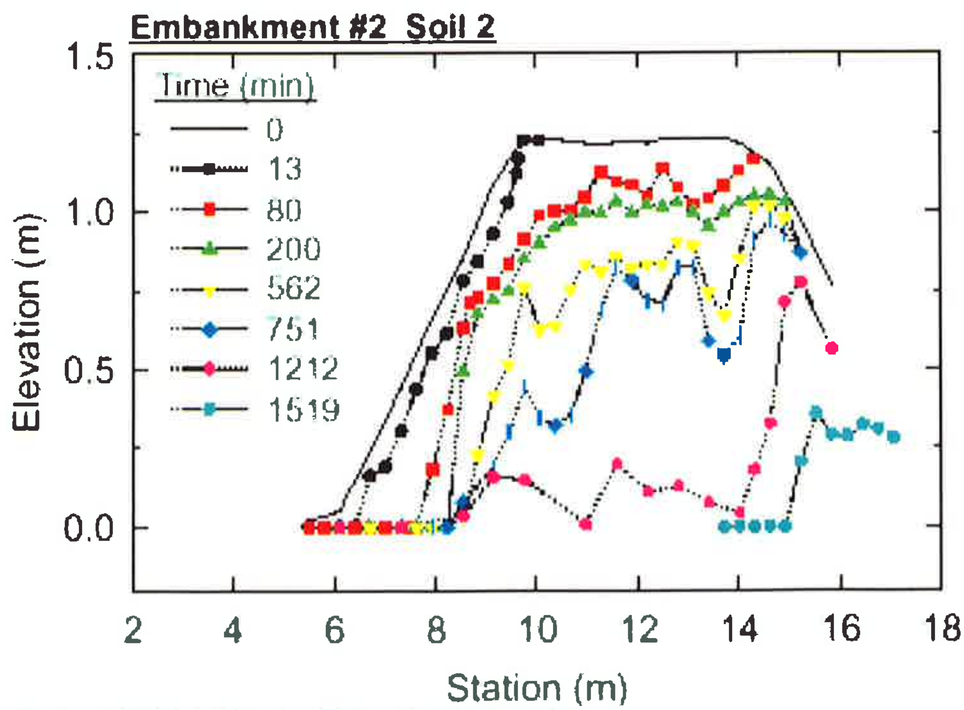

Figure 5. Centerline profile for embankment 2, soil 2.

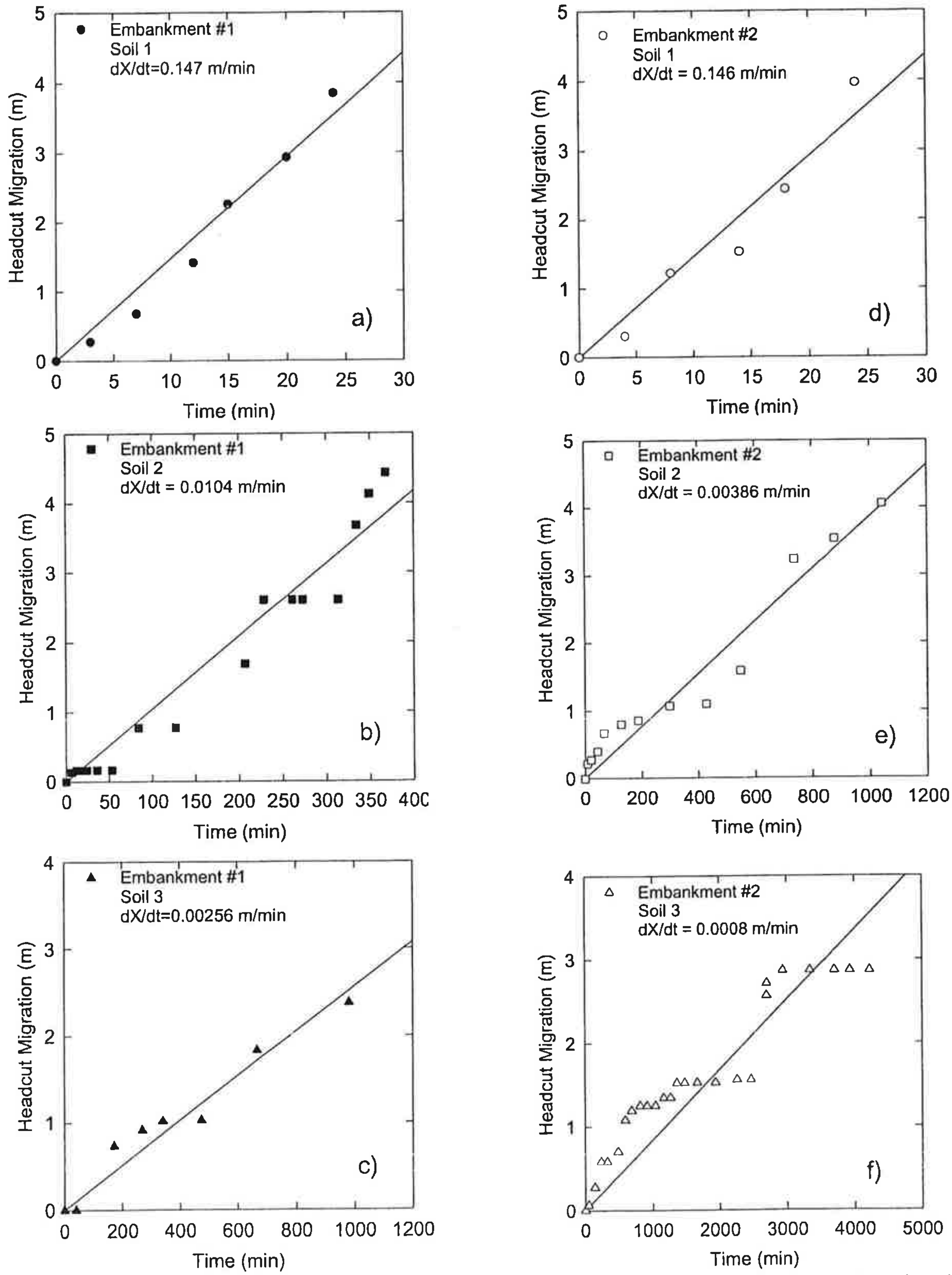

Figure 6. Headcut migration versus time for embankment tests; Embankment 1 a) soil 1, b) soil 2, c) soil 3; and Embankment 2: d) soil 1, e) soil 2, f) soil 3.

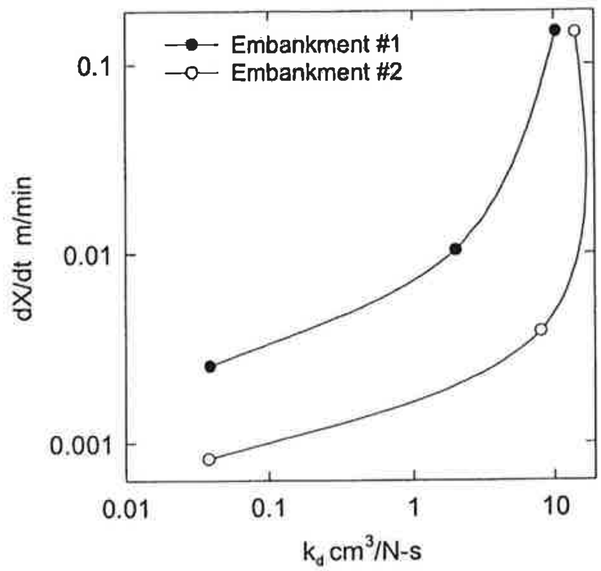

Figure 7. Migration rate dx/dt versus the erodibility coefficient k_d.

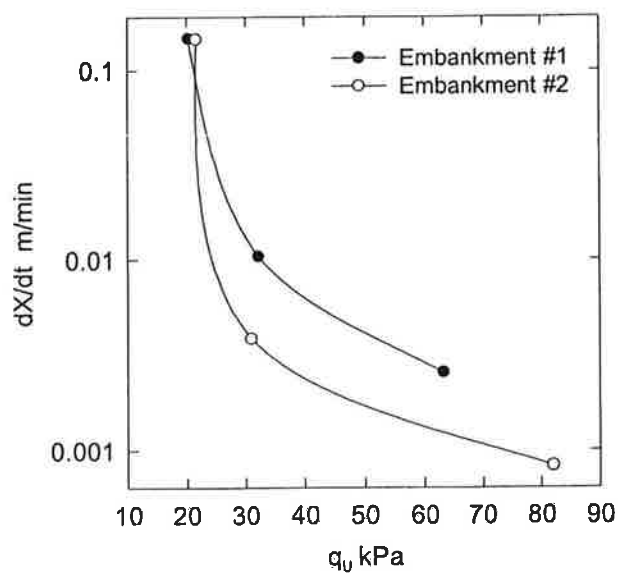

Figure 8. Migration rate dx/dt versus the unconfined compressive strength q_u.

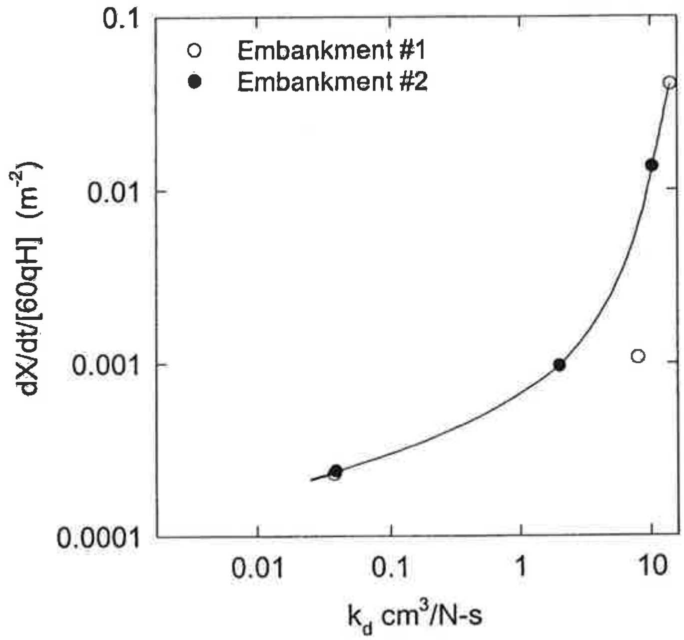

Figure 9. The migration rate dx/dt divided by 60qH versus the erodibility coefficient k_d.

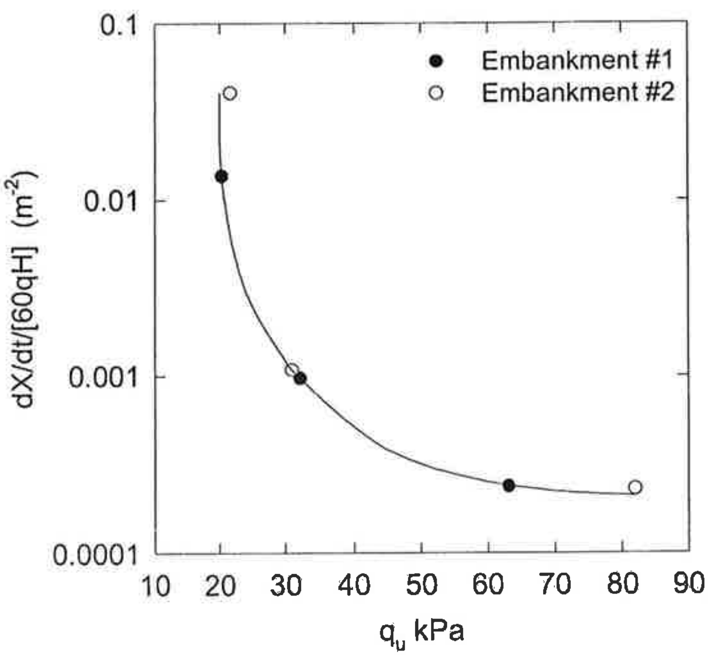

Figure 10. The migration rate dx/dt divided by 60qH versus the unconfined compressive strength q_u.