Edison Pond Dam may have resulted in a larger overtopping of the downstream dam and hence possible failure.

View of breach across crest with masonry wall on upstream face

View of breach immediately below masonry wall

View of breach through earthen embankment

Views of Edison Pond Dam Failure

The masonry wall along the upstream face of the Edison Pond Dam was approximately 3 feet wide. This, in combination with the narrow breach width downstream and a low hydraulic head on the wall probably prevented the total failure of the wall. But, had this dam overtopped for an extended period of time, a more significant portion of the earthen dam may have been eroded making a wider area of exposed masonry wall. This wall may not have been able to withstand the water pressures and may have experienced a total structural failure. With no earth pressures on the downstream side of the wall, a near instantaneous failure could have been expected, resulting in a large release of stored water.

Additional Issues and Research Needs

Additional issues and research needs that were identified by State engineers in the Northeast Region as part of the survey are:

- Refinement of breach parameters for dams with core walls or vertical concrete or masonry walls on the upstream face.
- Research into and refinement if necessary of breach parameters for small dams.
- NWS DAMBRK model has problems with large lateral inflows being added downstream of a dam
- State engineers unaware of the latest on the new FLDWAV model. Little or no training available.
- How do models handle debris flow in the flood wave? Currently engineers concerned with this issue are using high 'n' values in the overbank areas.

- Forensic Team. The Research Subcommittee of ICODS recommended to FEMA the development of a Forensic Team. The intent is that this team would be dispatched to the location of dam failures to gather data with respect to the breach and the impacts of the failure. State dam safety staffs are spread thin, and when failures occur, particularly in a wide spread area similar to the many failures that occurred along the east coast as a result of Hurricane Floyd, state engineers have little or no time to gather pertinent information with respect to the breach parameters and resultant damages. The data gathered by the Forensic Team would be useful for future research on dam safety analysis.

B-9

Issues, Resolutions, and Research Needs Related to Dam Failure Analysis Workshop
Oklahoma City, Oklahoma
June 26 -28, 2001

Embankment Dam Failure Analysis
by
Francis E. Fiegle II, P. E.
Georgia Safe Dams Program

The Kelly-Barnes Dam failed on November 6, 1977 near Toccoa, Georgia and killed 39 people that fateful Saturday night. That incident led to the passage of the Georgia Safe Dams Act and the formation of the Georgia Safe Dams Program. Since that date, there have been over 300 dam failures recorded in Georgia. Some of these have been catastrophic and two of these have resulted in loss of life. The Kelly Barnes Dam failed in 1977 and unnamed farm pond dam failed between Plains, Georgia and Americus which resulted in three deaths on July 5, 1994.

Kelly Barnes Dam Failure
Toccoa Georgia

A few of these dam failures have had good investigative follow-up where the size of the breach, initial conditions, and the resulting flood wave depths were measured. For instance, the Kelly Barnes Dam failure was thoroughly detailed by a Federal Investigative Board. The following breach parameters were detailed in the report for the Kelly Barnes Dam which was 38 feet tall:

- Breach side slopes - right 0.5 H to 1.0V
  - left 1.0 H to 1.0 V
- Base width of breach - 40 ft
- Sudden failure
- Estimated peak flow - 24,000 cfs

However, most of the dams that fail in Georgia have not had these detailed measurements. Every year in Georgia there are usually three or four dam failures of unregulated dams that are reported to our office. The majority of the dam failures have occurred during major rainfall events such as Tropical Storm Alberto in 1994, the 100-year flood in middle Georgia in 1998, and Tropical Storm Allison in 2001.

Lake Collins Dam
Sumter County
Tropical Storm Alberto
July 1994

Clayton County Waste Water Pond Dam
1982

Pritchard's Lake Dam
Morgan County
March 2001

Unknown Dam
Putnam County
Tropical Storm Allison
June 2001

Over the years, our office has noted that most of the dam breaches have had the following general parameters:

- Side slopes 1.0 H to 1.0 V
- Base width of breach equal to height of the dam

The side slopes are sometimes steeper in more clayey soils and flatter in sandy soils. The breach width maybe wider if there is a large impoundment (>than 25 acres).

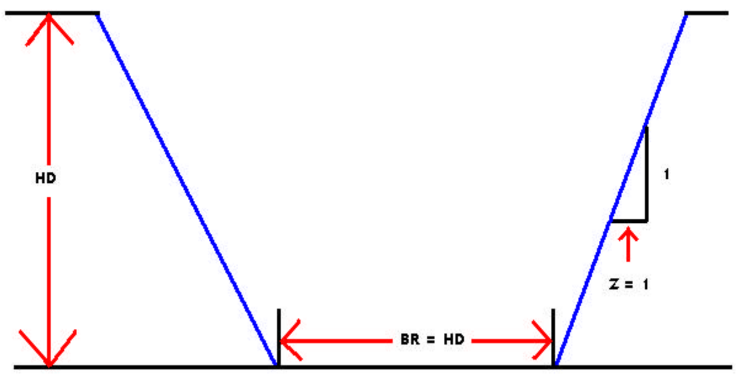
The breach parameters recommended in the Georgia Safe Dams Program's Engineering Guidelines mirror the Breach Parameters recommended by FERC and have been modified by our field observations of numerous dam failures.

Table I - Breach Parameters

Type of Dam	Breach Width BR (Feet)	Breach Side Slope Z	Time to Failure Hours
Arch	W	Vertical or Slope of Valley Walls	0.1
Masonry; Gravity	Monolith Width	Vertical	0.1 to 0.3
Rockfill	HD		
Timber Crib	HD	Vertical	0.1 to 0.3
Slag; Refuse	80% of W	1.0 – 2.0	0.1 to 0.3
Earthen – non-engineered	HD	1.0	0.1
Earthen-engineered	HD	1.0	0.5

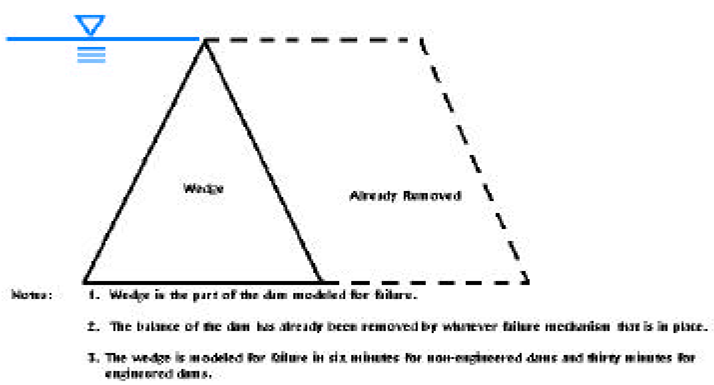
Table 2 – Breach Parameters
Definitions

- HD - Height of Dam
- Z - Horizontal Component of Side
- - Slope of Breach
- BR - Base Width of Breach
- TFH - Time to Fully Form the Breach
- W - Crest Length



Typical Sketch of Breach of Earth Embankment

Our office uses the Boss Dambreak software, which is based on the NWS Dambreak, developed by Dr. Danny Fread, P. E. Our office assumes that wedge erosion occurs (see following sketch). Furthermore, the time to failure is conservative for hazard classification of dams. We use a 6-minute time failure for earth fill dams that are not engineered fills or that we have no construction/design information for and 30 minutes for failure of engineered dams.



In Georgia, we use dambreak modeling for the following purposes:

- Hazard classification
- Flood inundation mapping
- Emergency action planning
- Incremental spillway capacity design

In closing, over the years our office has used or has seen dambreak modeling and routings use the following methods:

- NRCS TR66 (1978 to 1982)
- NWS Dambreak
- HECI Dambreak in conjunction with HECII or HECRAS stream routing
- Boss Dambreak (currently used by our office)

In preparation for this workshop, I surveyed the states east of the Mississippi River. I received responses from Virginia, North Carolina, West Virginia, South Carolina, Kentucky, Tennessee, Florida, Ohio, Georgia, and Indiana. The following questions were asked and the responses are detailed.

1. **Who does the dambreak routings?**
   - Owner of dam - SC, NC, VA, KY, WV, for new dams - FL, TN
   - State - SC, OH, GA, NC; TN for existing dams

2. **Breach Parameters:**
   - Breach width varies from height to twice the height of the dam
   - Side slopes varies from 0.5 H to 1.0 H to 1V
   - Time to failure varies from 6 minutes to 60 minutes

3. **What type of dams are routed?**
   - High hazard - SC, WV, VA, KY, TN, OH, NC, GA
   - Significant Hazard - SC, WV, VA
   - Low hazard - none
   - In Florida and Indiana - various hazards are routed

4. **Definition of High Hazard Dams:**
   - Floods a building that is occupied
   - One foot above finished floor
   - Well-traveled roadways 6 inches deep
   - Use BurRec Guidelines
   - Loss of life likely to probably
   - Any dam over 60 feet in height or stores more than 5000 acre-feet (Ohio)
   - Application of damage index

5. **Type of analyses used**
   - NWS Dambreak (variation of) - TN, SC, KY, OH, WV, NC, GA
   - HEC1 Dambreak/HECII and HECRAS - TN, SC, OH, WV, NC, VA
   - Visual Observation - NC

6. Reinventory of Dams Timeframe:

- Annually - high hazard only -SC
- Two Years - high hazard only - NC
- Three Years - significant hazard - NC, SC
low hazard - SC
- Five Years - all hazards - OH, GA
low hazard - NC
- Six Years - all hazards - VA
- Kentucky only reinventories if hazard is noticed
- West Virginia reinventories during routine inspections
- Florida is locally determined (Water Management Districts)
- Tennessee when doing a safety inspections

As a result of this survey, there were several issues identified that need attention. It is clear that states need to take the following actions:

- Regularly reinventory dams of all hazard classifications
- Have consistent hazard classification guidelines
- Adopt Quality Assurance/Quality Control Procedures
- Improve technical expertise

The states have requested the following guidance from this workshop based on the assembled expertise:

- Time to failure guidelines
- Is there time for emergency response to make a difference?
- When to use which model or sets of models (field conditions, etc)?
- What is the level of accuracy for each model?
- Advantages/disadvantages for each model(s)

As a result of the survey, the following **immediate** dam safety needs were identified by the states:

- Combine HECI and HECII or HECHMS and HECRAS into integrated model(s)
- Finish Floodwave Model
- Provide in depth, hands on training in the use of all models

Finally, the states identified the following **Research Needs:**

- Input parameters for breach development for earth and rockfill dams
- Depth of overtopping that causes failure
- How does the crest protection influence overtopping failure development?
- How does the embankment protection influence overtopping failure development?
- Forensic investigation of breach failures including the condition of the dam
- Influence of the size of the drainage basin on a "storm in progress" failure

In closing, I wonder if we in the dam safety community are meeting the public's expectations in regulating dams, or better yet, are we classifying dams for regulation to meet our perception of the public's expectation or some variation there of? If we are using our paradigms without adequate explanation to the public and feedback from the "at risk" population, then likely we are imposing additional risk to the "at risk" populations that is not justified.

B-10

ADJUSTING REALITY TO FIT THE MODEL

MATTHEW LINDON, P.E.

- DEPT OF NATURAL RESOURCES
- STATE ENGINEERS OFFICE
- DAM SAFETY HYDROLOGIST

CREDENTIALS

AMATEUR ACADEMIC MODELER

- Prep School - Math, Science, Computers, Statistics
- Engineering - Calculus, Physics, Thermo, Fluids
- Grad Courses - Modeling, Meteorology, Hydrology
- Computers - Punch Cards, Batch Files, XT, AT, PC, Math Chip, 286, 386, 486, Pentium I, II….

MODELING EXPERIENCE

DAM SAFETY HYDROLOGY
20 YEARS

- ACOE - HEC I, II, HMS, RAS
- NWS - DAMBRK, BREACH, SMPDBK, DWOPER
- DHM, FLO2D, TR20, PIPE NETWORK
- STORM, SPIPE, FLDRTE, BACKWAT
- SIDECHAN, SPILLWAY, STABLE, QUAKE….

Awakening

From the Hypothetical to the Real World

- HEC I, HEC II courses and experience
- NWS - DAMBRK/BREACH Course - Exercise
- Quail Creek dam failure - Calibration opportunity
- Necessity is the mother of invention
- Measure, survey, interview, history of event

CALIBRATION

CORRELATION TO REALITY

- NO correlation of BREACH model with reality
  - Piping channel start sensitivity
  - Can't model actual breach shape and timing
- NO correlation of DAMBRK model with reality
  - Manning Roughness Coefficients unreal - 0.1-0.25
  - sensitive to breach size and timing
  - Can't model trapezoidal migration
  - Can't converge with Manning increase with depth
- Sensitive to Black Box Variables - Mannings
  - Friction, bulking, debris, turbulence, eddys
- Limited by time steps and reach lengths - converge?
- Supercritical to subcritical hydraulic jumps

Doubt and Disillusionment

Pity the man who doubts what he's sure of

- HEC I
  - Sensitive to Time Step, Reach Length, Basins
  - Black box for infiltration, lag, runoff, melt….
  - Hydrological routing - No Attenuation
  - Designed for flat farms not wild mountains
- HEC II
  - Manning Black Box,
  - 1 dimension limits, boundary conditions
  - Designed for labs and canals - not rivers
- Old equations on new high speed computers
- Developed by mathematicians, statisticians and Computer Geeks

Basis of Uncertainty

Close counts in horseshoes, hand grenades & hydrology

- ➢ Sensitivity analysis of input variables
- ➢ Probabilistic approach
- ➢ Monte Carlo combinations of all variables
  - Most probable answer
  - Not best answer
  - Not worst case
- ➢ Fuzziness of results

Apparent Veracity

Computers lie and liars use computers.

- ➢ Easy inout, user interface, GUI, ACAD, GIS
- ➢ Garbage in garbage out
- ➢ Slick output, graphics, color
- ➢ Windows, WYSIWYG, 3D, Iso views
- ➢ Computer Credibility - must be FACT
- ➢ Models using old theories and methods
- ➢ Lagging physically based, spatial and temporal
- ➢ Computers effect modeling like writing styles

New age modelers

Post-modern hydrology - form before function.

- Ease of operation encourages the unqualified or unscrupulous to take advantage
- Not familiar with theory and methods
- Have not done calculations in head or by hand
- Don't understand complex non linear nature of these multidimensional problems.
- Know exactly what the models does or don't use it

Problem Solutions

Good math and science don't always make good models

- Better Models
  - Eliminate Balck Boxes
  - 2D, 3D - less assumptions
  - incorporate new theory and methods
  - Use spatial and temporal improvements
- Qualified modelers
  - Better modelers for better models
  - educate, train, help screens, documentation
  - Use models for intended purpose, scope and scale
- Calibrate, Correlate, Calculate
  - Sensitivity analysis on input variables
  - Interpolate rather than extrapolate
- Express degree of uncertainty of output
  - Probabilities, confidence, fuzziness, chaos

Get out of the box.

To think outside the box you must get outside.

- ➢ Natural phenomena are fantastically complex systems
  - Understand little
  - Describe less
  - Model and reproduce even less
- ➢ Math and Science just our best guess
  - They are tools like slide rules, computers, hammers.
  - "Ology" is the study of, not the perfect understanding.
  - Use a large grain of salt
- ➢ Observe present and past
  - Paleohydrology
  - Walk up and downstream
  - What does nature want to do
- ➢ Connect the model with reality

B-11

A Simple Procedure for Estimating Loss of Life from Dam Failure

FEMA/USDA Workshop
Issues, Resolutions, and Research
Needs Related to Embankment Dam Failure Analysis
26-28 June 2001

Wayne J. Graham, P.E.

INTRODUCTION

Evaluating the consequences resulting from a dam failure is an important and integral part of any dam safety study or risk analysis. The failure of some dams would cause only minimal impacts to the dam owner and others, while large dams immediately upstream from large cities are capable of causing catastrophic losses. Dam failure can cause loss of life, property damage, cultural and historic losses, environmental losses as well as social impacts. This paper focuses on the loss of life resulting from dam failure. Included is a procedure for estimating the loss of life that would result from dam failure. No currently available procedure is capable of predicting the exact number of fatalities that would result from dam failure.

PROCEDURE FOR ESTIMATING LOSS OF LIFE

The steps for estimating loss of life resulting from dam failure are as follows:

Step 1: Determine dam failure scenarios to evaluate.
Step 2: Determine time categories for which loss of life estimates are needed.
Step 3: Determine area flooded for each dam failure scenario.
Step 4: Estimate the number of people at risk for each failure scenario and time category.
Step 5: Determine when dam failure warnings would be initiated.
Step 6: Select appropriate fatality rate.
Step 7: Evaluate uncertainty.

The details of each step are as follows:

Step 1: Determine Dam Failure Scenarios to Evaluate

A determination needs to be made regarding the failure scenarios to evaluate. For example, loss of life estimates may be needed for two scenarios - failure of the dam with a full reservoir during normal weather conditions and failure of the dam during a large flood that overtops the dam.

Step 2: Determine Time Categories For Which Loss of Life Estimates Are Needed

The number of people at risk downstream from some dams is influenced by seasonality or day of week factors. For instance, some tourist areas may be unused for much of the year. The number of time categories (season, day of week, etc.) selected for evaluation should accommodate the varying usage and occupancy of the floodplain. Since time of day can influence both when a warning is initiated as well as the number of people at risk, each study should include a day category and a night category for each dam failure scenario evaluated.

Step 3: Determine Area Flooded for Each Dam Failure Scenario

In order to estimate the number of people at risk, a map or some other description of the flooded area must be available for each dam failure scenario. In some cases, existing dam-break studies and maps may provide information for the scenarios being evaluated. Sometimes new studies and maps will need to be developed.

Step 4: Estimate the Number of People at Risk for Each Failure Scenario and Time Category

For each failure scenario and time category, determine the number of people at risk. Population at risk (PAR) is defined as the number of people occupying the dam failure floodplain prior to the issuance of any warning. A general guideline is to: "Take a snapshot and count the people." The number of people at risk varies during a 24-hour period.

The number of people at risk will likely vary depending upon the time of year, day of week and time of day during which the failure occurs. Utilize census data, field trips, aerial photographs, telephone interviews, topographic maps and any other sources that would provide a realistic estimate of floodplain occupancy and usage.

Step 5: Determine When Dam Failure Warnings Would be Initiated

Determining when dam failure warnings would be initiated is probably **the most important** part of estimating the loss of life that would result from dam failure. Table 1, "Guidance for Estimating When Dam Failure Warnings Would be Initiated," was prepared using data from U.S. dam failures occurring since 1960 as well as other events such as Vajont Dam in Italy, Malpasset Dam in France and Saint Francis Dam in California. An evaluation of these dam failure data indicated that timely dam failure warnings were more likely when the dam failure occurred during daylight, in the presence of a dam tender or others and where the drainage area above the dam was large or the reservoir flood storage space. Timely dam failure warnings were less likely when failure occurred at night or outside the presence of a dam tender or casual observers. Dam failure warnings were also less likely where the drainage area was small or the reservoir had little or no flood storage space, i.e, when the reservoir was able to quickly fill and overtop the dam. Although empirical data are limited, it appears that timely warning is less likely for the failure of a concrete dam. Although dam failure warnings are frequently initiated before dam failure for earthfill dams, this is not the case for the failure of concrete dams.

Table 1 provides a means for deriving an initial **estimate** of when a dam failure warning would be initiated for the failure of an earthfill dam. The availability of emergency action plans, upstream or dam-site instrumentation, or the requirement for on-site monitoring during threatening events influences when a dam failure warning would be initiated. Assumptions regarding when a warning is initiated should take these factors into account.

Table 1
Guidance for Estimating When Dam Failure Warnings Would be Initiated (Earthfill Dam)

Dam Type	Cause of Failure	Special Considerations	Time of Failure	When Would Dam Failure Warning be Initiated?	
				Many Observers at Dam	No Observers at Dam
Earthfill	Overtopping	Drainage area at dam less than 100 mi^2 (260 km^2)	Day	0.25 hrs. before dam failure	0.25 hrs. after fw reaches populated area
		Drainage area at dam less than 100 mi^2 (260 km^2)	Night	0.25 hrs. after dam failure	1.0 hrs. after fw reaches populated area
		Drainage area at dam more than 100 mi^2 (260 km^2)	Day	2 hrs. before dam failure	1 hr. before dam failure
		Drainage area at dam more than 100 mi^2 (260 km^2)	Night	1 to 2 hr. before dam failure	0 to 1 hr. before dam failure
	Piping (full reservoir, normal weather)		Day	1 hr. before dam failure	0.25 hrs. after fw reaches populated area
			Night	0.5 hr. after dam failure	1.0 hr. after fw reaches populated area
	Seismic	Immediate Failure	Day	0.25 hr. after dam failure	0.25 hr. after fw reaches populated area
			Night	0.50 hr. after dam failure	1.0 hrs. after fw reaches populated area
		Delayed Failure	Day	2 hrs. before dam failure	0.5 hrs. before fw reaches populated area
			Night	2 hrs. before dam failure	0.5 hrs. before fw reaches populated area

Notes: "Many Observers at Dam" means that a dam tender lives on high ground and within site of the dam **or** the dam is visible from the homes of many people or the dam crest serves as a heavily used roadway. These dams are typically in urban areas. "No Observers at Dam" means that there is no dam tender at the dam, the dam is out of site of nearly all homes and there is no roadway on the dam crest. These dams are usually in remote areas. The abbreviation "fw" stands for floodwater.

Step 6: Select Appropriate Fatality Rate

Fatality rates used for estimating life loss should be obtained from Table 2. The table was developed using data obtained from approximately 40 floods, many of which were caused by dam failure. The 40 floods include nearly all U.S. dam failures causing 50 or more fatalities as well as other flood events that were selected in an attempt to cover a full range of flood severity and warning combinations. Events occurring outside of the U.S. were included in the data set. The following paragraphs describe the terms and categories that form the basis for this methodology.

Flood Severity along with warning time determines, to a large extent, the fatality rate that would likely occur. The flood severity categories are as follows:

1) Low severity occurs when **no** buildings are washed off their foundations. Use the low severity category if most structures would be exposed to depths of less than 10 ft (3.3 m) or if DV, defined below, is less than 50 ft^2/s (4.6 m^2/s).

2) Medium severity occurs when homes are destroyed but trees or mangled homes remain for people to seek refuge in or on. Use medium flood severity if most structures would be exposed to depths of more than 10 ft (3.3 m) or if DV is more than 50 ft^2/s (4.6 m^2/s).

3) High severity occurs when the flood sweeps the area clean and nothing remains. High flood severity should be used only for locations flooded by the near instantaneous failure of a **concrete** dam, or an earthfill dam that turns into "jello" and washes out in seconds rather than minutes or hours. In addition, the flooding caused by the dam failure should sweep the area clean and little or no evidence of the prior human habitation remains after the floodwater recedes. Although rare, this type of flooding occurred below St. Francis Dam in California and Vajont Dam in Italy. The flood severity will usually change to medium and then low as the floodwater travels farther downstream.

The parameter **DV** may be used to separate areas anticipated to receive low severity flooding from areas anticipated to receive medium severity flooding. DV is computed as follows:

$$DV = \frac{Q_{df} - Q_{2.33}}{W_{df}}$$

where:

Q_{df} is the peak discharge at a particular site caused by dam failure.

$Q_{2.33}$ is the mean annual discharge at the same site. This discharge can be easily estimated and it is an indicator of the safe channel capacity.

W_{df} is the maximum width of flooding caused by dam failure at the same site.

Warning Time influences the fatality rate. The warning time categories are as follows:

1) No warning means that no warning is issued by the media or official sources in the particular area prior to the flood water arrival; only the possible sight or sound of the approaching flooding serves as a warning.

2) Some warning means officials or the media begin warning in the particular area 15 to 60 minutes before flood water arrival. Some people will learn of the flooding indirectly when contacted by friends, neighbors or relatives.

3) Adequate warning means officials or the media begin warning in the particular area more than 60 minutes before the flood water arrives. Some people will learn of the flooding indirectly when contacted by friends, neighbors or relatives.

The warning time for a particular area downstream from a dam should be based on when a dam failure warning is initiated and the flood travel time. For instance, assume a dam with a campground immediately downstream and a town where flooding begins 4 hours after the initiation of dam failure. If a dam failure warning is initiated 1 hour after dam failure, the warning time at the campground is zero and the warning time at the town is 3 hours.

The fatality rate in areas with medium severity flooding should drop below that recommended in Table 2 as the warning time increases well beyond one hour. Repeated dam failure warnings, confirmed by visual images on television showing massive destruction in upstream areas, should provide convincing evidence to people that a truly dangerous situation exists and of their need to evacuate. This should result in higher evacuation rates in downstream areas and in a lowering of the fatality rate.

Flood Severity Understanding also has an impact on the fatality rate. A warning is comprised of two elements: 1) alerting people to danger and 2) requesting that people at risk take some action. Sometimes those issuing a flood warning or dam failure warning may not issue a clear and forceful message because either 1) they do not understand the severity of the impending flooding or 2) they do not believe that dam failure is really going to occur and hence do not want to unnecessarily inconvenience people. People exposed to dam failure flooding are less likely to take protective action if they receive a poorly worded or timidly issued warning. Warnings are likely to become more accurate after a dam has failed as those issuing a warning learn of the actual failure and the magnitude of the resultant flooding. Precise warnings are therefore more probable in downstream areas. This factor will be used only when there is some or adequate warning time.

The flood severity understanding categories are as follows:

1) Vague Understanding of Flood Severity means that the warning issuers have not yet seen an actual dam failure or do not comprehend the true magnitude of the flooding.

2) Precise Understanding of Flood Severity means that the warning issuers have an excellent understanding of the flooding due to observations of the flooding made by themselves or others.

Table 2
Recommended Fatality Rates for Estimating Loss of Life Resulting from Dam Failure

Flood Severity	Warning Time (minutes)	Flood Severity Understanding	Fatality Rate (Fraction of people at risk expected to die)	
			Suggested	Suggested Range
HIGH	no warning	not applicable	0.75	0.30 to 1.00
	15 to 60	vague	Use the values shown above and apply to the number of people who remain in the dam failure floodplain after warnings are issued. No guidance is provided on how many people will remain in the floodplain.	
		precise		
	more than 60	vague		
		precise		
MEDIUM	no warning	not applicable	0.15	0.03 to 0.35
	15 to 60	vague	0.04	0.01 to 0.08
		precise	0.02	0.005 to 0.04
	more than 60	vague	0.03	0.005 to 0.06
		precise	0.01	0.002 to 0.02
LOW	no warning	not applicable	0.01	0.0 to 0.02
	15 to 60	vague	0.007	0.0 to 0.015
		precise	0.002	0.0 to 0.004
	more than 60	vague	0.0003	0.0 to 0.0006
		precise	0.0002	0.0 to 0.0004

Step 7: Evaluate Uncertainty

Various types of uncertainty can influence loss of life estimates. Quantifying uncertainty is difficult and may require significant time to achieve.

Step 1 of this procedure suggests that separate loss of life estimates be developed for each dam failure scenario. Various causes of dam failure will result in differences in downstream flooding and therefore result in differences in the number of people at risk as well as flood severity.

Step 2 suggests that the dam failure be assumed to occur at various times of the day or week. It is recognized that the time of failure impacts both when a dam failure warning would be initiated as well as the number of people who would be at risk.

Dam failure modeling serves as the basis for step 3. Dam failure modeling requires the estimation of: 1) the time for the breach to form, 2) breach shape and width and 3) downstream hydraulic parameters. Variations in these parameters will result in changes in the flood depth, flood width and flood wave travel time. This will lead to uncertainty in the: 1) population at risk, 2) warning time and 3) flood severity.

Estimating the number of people at risk, step 4, may be difficult, especially for areas that receive temporary usage. A range of reasonable estimates could be used.

Step 5 focuses on when a dam failure warning would be initiated. This warning initiation time could be varied to determine sensitivity to this assumption.

The last type of uncertainty is associated with the inability to precisely determine the fatality rate, step 6. There was uncertainty associated with categorizing some of the flood events that were used in developing Table 2. Similarly, some of the factors that contribute to life loss are not captured in the categories shown in Table 2. This type of uncertainty can introduce significant, but unknown, errors into the loss of life estimates. Some possible ways of handling this uncertainty would be to 1) use the range of fatality rates shown in Table 2, 2) when the flooding at a particular area falls between two categories (it is unclear if the flood severity would be medium or low, for example) the loss of life estimates can be developed using the fatality rate and range of rates from all categories touched by the event and 3) historical events can be evaluated to see if there are any that closely match the situation at the site under study.

B-12

Workshop on Issues, Resolutions, and Research Needs Related to Dam Failure Analysis

Current Practice
Natural Resources Conservation Service

by Bill Irwin [1]

Introduction

The Natural Resources Conservation Service (NRCS) formerly Soil Conservation Service (SCS) is the engineer-of-record on over 26,000 of the roughly 77,000 dams currently identified in the National Inventory of Dams (NID). NRCS has also engineered over 3,000,000 dams and ponds that are smaller than the minimum size dam included in the National Inventory. Typical dams in the NRCS portfolio are relatively small embankment dams built over 30 years ago. Data on NRCS dams in the NID is shown in Figure 1.

NID size dams	26000		26000		26000
25ft+ high	15000	50AF+ storage	23000	30yrs+ old	15000
45ft+ high	2000	500AF+ storage	7000	40yrs+ old	5000
65ft+ high	400	5000AF+ storage	600	50yrs+ old	1000
100ft+ high	40	15000AF+ storage	100	60yrs+ old	400

Figure 1 – NRCS Dam Portfolio

Current Criteria

The NRCS has developed a significant set of design criteria over the years to accomplish this work. The SCS established three levels of hazard classification over as far back as anyone can remember and defined the high hazard classification almost fifty years ago as structures "…where failure may result in loss of life, damage to homes, industrial and commercial buildings, important public facilities, railroads and highways." [2] This classification and subsequent design criteria approach inherently requires evaluation of dam failure parameters. The NRCS has provided increasing degrees of criteria and guidance on selection of such parameters as techniques for analyzing the consequences of dam failures have advanced.

Current NRCS failure analysis guidance was initially published the late 1970's as Technical Release Number 66 (TR-66), "Simplified Dam Breach Routing Procedure". This procedure is a combined hydrologic-hydraulic method. The hydraulic portion is a simplified version of a simultaneous storage and kinematic routing method which accepts a breach hydrograph at the upstream end of the reach and routes the flood wave downstream, continuously in time and space. The hydrologic portion develops the breach hydrograph based on estimated downstream flow characteristics, total volume of flow from dam pool, and expected maximum breach discharge (Qmax). The Qmax parameter was estimated from a curve fit of the peak discharges from historic dam failures available at the time.

[1]National Design Engineer, USDA/NRCS, Washington, DC
email: bill.irwin@usda.gov phone: (202)720-5858

[2]SCS Engineering Memo No. 3, July 16, 1956

The procedure was intended to provide a practical "hand-worked" method appropriate for typical NRCS dam work. One published report [3] compared the TR-66 procedure with three other methods available at the time including the National Weather Service (NWS) and Hydraulic Engineering Center (HEC) models. For a 36ft high embankment dam subjected to a PMP event, the four methods produced comparable breach profile depths, while the TR-66 method computed the lowest peak flow at the dam. Computed peak discharges were 71,355cfs by TR-66, 76,000cfs by Keulegan, 85,950cfs by NWS, and 87,000cfs by HEC-1.

Current NRCS breach peak discharge criteria was initially published in the late 1980"s in Technical Release Number 60 (TR-60), "Earth Dams and Reservoirs". The criteria specifies the peak breach discharge (Qmax) to be used to delineate the potential dam failure inundation area below the dam and subsequently to determine the dam hazard classification. The criteria does not specify downstream breach routing or other hydraulic methodologies to be used. Regardless of the stream routing techniques to be used, the minimum peak discharge is as follows:

1. For depth of water at dam (H_w) at time of failure   103 feet,

$$Qmax = 65\ H_w^{\ 1.85}$$

2. For depth of water at dam (H_w) at time of failure < 103 feet,

$$Qmax = 1000\ B_r^{\ 1.35} \quad \text{but not less than } 3.2\ H_w^{\ 2.5} \text{ nor more than } 65\ H_w^{\ 1.85}\text{, where,}$$

$$B_r = V_s\ H_w / A \quad \text{and,}$$

B_r = breach factor, acres
V_s = reservoir storage at failure, acre-feet
A = cross-sectional area of the embankment, square feet

3. When actual dam crest length(L) is less than theoretical breach width (T) such that,

$$L < T = (65\ H^{\ 0.35}) / 0.416 \quad \text{use,}$$

$$Qmax = 0.416\ L\ H^{\ 1.5} \quad \text{in lieu of } 65\ H_w^{\ 1.85} \quad \text{in category 1 or 2 above, where,}$$

H = height of dam at centerline, from bottom of breach to top of dam, feet

This suite of expressions for Qmax was derived from a data set of 39 dam failures available in the profession or collected from NRCS sources at the time. Figure 2 taken from the original work shows the relationship between the peak breach discharge from the 39 sites and the peak break discharge predicted by the Qmax criteria.

[3] Safety of Existing Dams, National Research Council, National Academy Press, 1983.

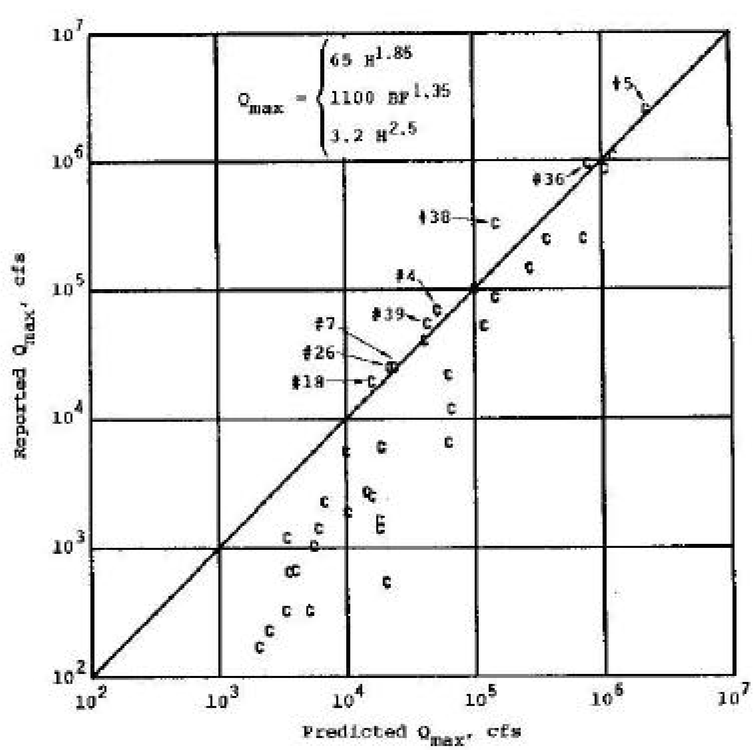


Figure 2 – Comparison of Predicted vs. Reported Qmax for 39 site data set

Failure Experiences

NRCS has experience a relatively small number of dam failures considering the magnitude of its portfolio. However, information from some dramatic NRCS dam failures provides insight into NRCS experienced failure modes.

Figure 3 – Obion Creek #36 – looking upstream into reservoir

Obion Creek #36 is a typical NRCS flood control dam from the 1960's. It was built in 1963 and failed a year later during the first reservoir filling storm. An Engineering Investigation concluded that dispersive soils were a major factor in the failure. Note that the dam was constructed with anti-seep collars along the principal spillway pipe as was typical at the time.

Although this failure occurred several years ago, it is still representative of similar dams that were built around the same or earlier time periods before needed treatments of dispersive soils or needed filter diaphragms around pipe penetrations were recognized. NRCS has had several similar piping type failures and does have many similarly designed flood control dams that have not yet experienced a significant first filling.

Figure 4 – Coon Creek #41 – note remaining embankment in upper right

Coon Creek #41 is also a typical NRCS flood control dam from the 1960's. It was built in 1962 and failed in 1978 during the first significant reservoir filling. An Engineering Investigation concluded that stress relief fractured rock in the steep abutment was the major factor in the failure. This site was constructed with minimal foundation investigation and foundation treatments as was typical at the time. Although this site failure occurred several years ago, it is still representative of similar dams that were built in similar geologic settings around the same or earlier time periods before such foundation hazards were widely recognized or routinely investigated. NRCS has similar flood control dams which have not yet experienced a significant first filling. Most recently, Bad Axe #24, a similar site in a similar setting built in 1963, failed in a similar fashion last year.

Figure 4 – Ascalmore #11 – looking upstream, note pipe outlet on left


Figure 5 – Ascalmore #11 – looking upstream

Ascalmore #11 was built in 1959 and failed last year after trash blocked the pipe spillway and a storm quickly filled the flood reservoir. An Engineering Investigation concluded that dispersive soils and animal burrow damage in the upper portions of the embankment were major factors in the failure. The rough appearance of the embankment surface is due to removal of extensive woody vegetation after the breach occurred and before the picture was taken. It is interesting to note that the embankment breached in two separate locations and the energy of the stored water was not sufficient to erode either breach to the base of the dam.

Practical Criteria Needs
The principal NRCS need related to embankment failure analysis continues to be determination of the breach inundation area below the dam for purposes of determining population at risk, hazard classification, and emergency action planning.

The “hand-worked” hydraulic portion of the old TR-66 method is adequate for only very basic hazard class screening on typical rural NRCS dams and ponds. Current software for breach flow profile analysis supported with modern computer capabilities is the professional norm for developing breach inundation maps and eventually emergency action plans. New hydraulic routing software currently being developed in the profession will further advance this aspect of dam failure consequence analysis.

The hydrograph portion of the old TR-66 method and subsequently the Qmax equations approach of the current TR-60 criteria are still important. This approach can still provide adequate dam failure analysis criteria for typical NRCS dams since the embankments are small, the area at risk is close to the dam, and agency experience has been a wide variety of failure modes. The NRCS workload involving dams requires that a large number of existing and potential dams be evaluated without significant topographic or soil site data. Such workload without much physical data does not justify a complex analysis. The current

Qmax equation needs to be updated considering newer dam failure data available in the profession.

Another NRCS need related more to embankment non-failure analysis is allowable overtopping. Agency experience has repeatedly shown that well vegetated dams built with well compacted cohesive materials can sustain substantial overtopping flow with minimal damage. As NRCS begins a rehabilitation program to rehabilitate aging watershed dams, a major issue is increasing the height or spillway capacity of the existing dams to accommodate larger required design storms. Research that can increase the confidence level of the dam safety profession to accept limited overtopping flow in upgrading these dams could eliminate the need for expensive structural upgrades on many existing agency dams.

A last NRCS need related to dam failure analysis is a better tool for risk assessment. Recent new authority for NRCS to provide rehabilitation assistance came with the requirement to give priority consideration to those existing dams that are the greatest threat to public health and safety. NRCS has adopted a risk index system based on the common approach that total risk is the product of the probability of loading, the probability of adverse response to that loading, and the probability of consequence due to adverse response. Dam failure research could provide better tools to define the probability of adverse response.

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Dam Failure Analyses Workshop

Oklahoma City, OK

June 26-28, 2001

James Evans
FERC
Division of Dam Safety and Inspections

Important Areas to Consider in the Investigation and Evaluation of Proposed and Existing Dams

- **The Embankment Must be Safe Against Excessive Overtopping by Wave Action Especially During Pre-Inflow Design Flood**
- **The Slopes Must be Stable During all Conditions of Reservoir Operations, Including Rapid Drawdown, if Applicable**
- **Seepage Flow Through the Embankment, Foundation, and Abutments Must be Controlled so That no Internal Erosion (Piping) Takes Place and There is no Sloughing in Areas Where Seepage Emerges**

Important Areas to Consider in the Investigation and Evaluation of Proposed and Existing Dams (Continued)

- **The Embankment Must Not Overstress the Foundation**
- **Embankment Slopes Must be Acceptably Protected Against Erosion by Wave Action from Gullying and Scour From Surface Runoff**
- **The Embankment, Foundation, Abutments and Reservoir Rim Must be Stable and Must Not Develop Unacceptable Deformations Under Earthquake Conditions**

Design Factors of Safety for Embankment Dams

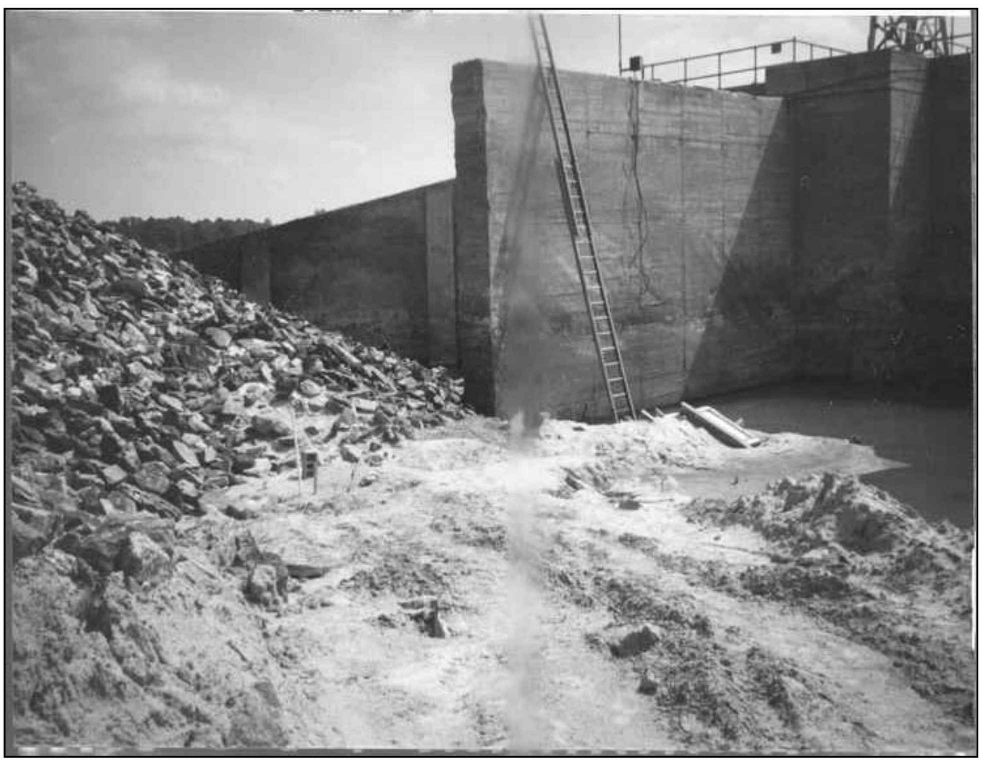
- **End of Construction--FS > 1.3**
- **Sudden Draw Down From Maximum Pool------------------------------FS > 1.1**
- **Sudden Draw Down From Spillway Crest or Top of Gates----------FS > 1.2**
- **Steady Seepage with Maximum Storage Pool---------------------------FS > 1.5**
- **Steady Seepage With Surcharge Pool-------------------------------------FS > 1.4**
- **Seismic Loading Condition Factor of Safety----------------------------FS > 1.0**
  - **For Zones with Seismic Coefficients of 0.1 or Less – Pseudostatic Analysis is Acceptable if Liquefaction Does not Trigger.**
  - **Deformation Analysis are Required if pga $\geq$ 0.15g For Newmark Procedures, Deformation Should be $\leq$2.0 feet.**

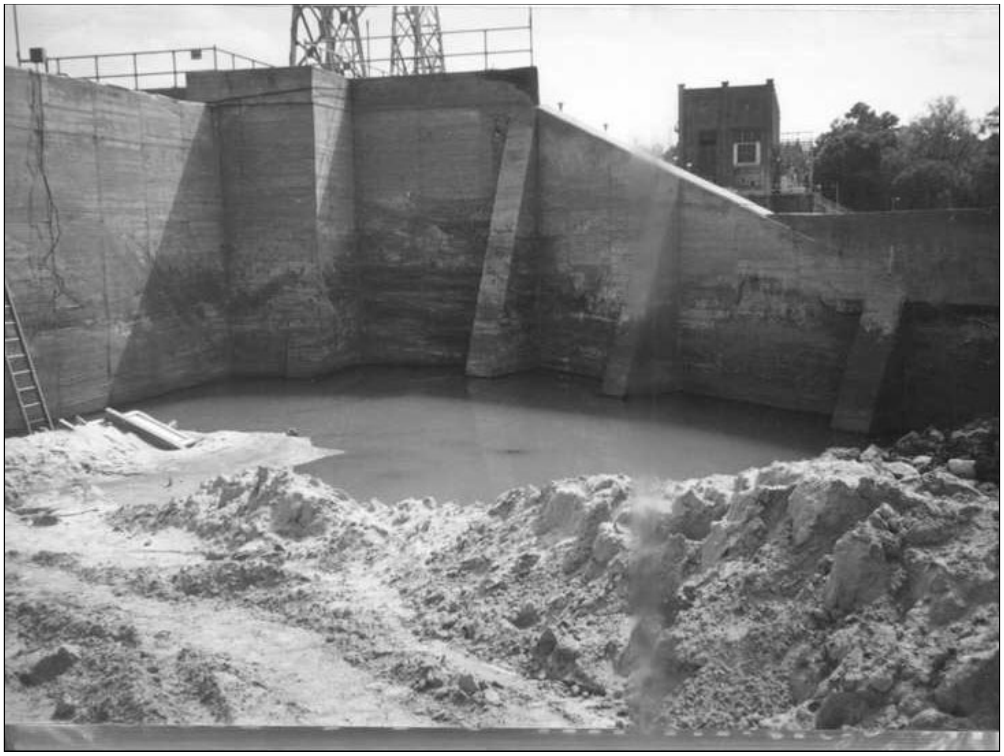
Stability Analyses Programs to Determine the Factors of Safety for the Various Loading Conditions

- **Computer Programs Such as UTEXAS3 are Used to Determine the Factors of Safety for the Various Loading Conditions Previously Discussed**
- **COE Hand Calculation Method From EM 110-2-1902 are Used to Confirm a Computer Program Critical Failure Surface for Important Projects**

Lake Blackshear Dam

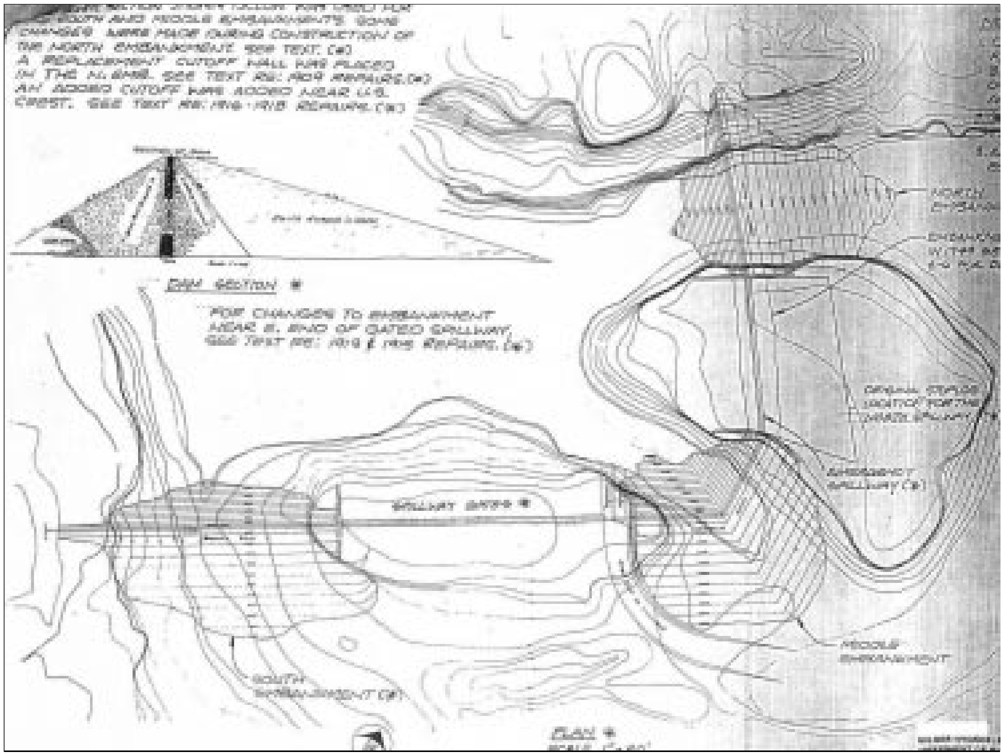
- **Reasons to Prevent Overtopping of an Embankment Even When Covered With a Good Growth of Grass**

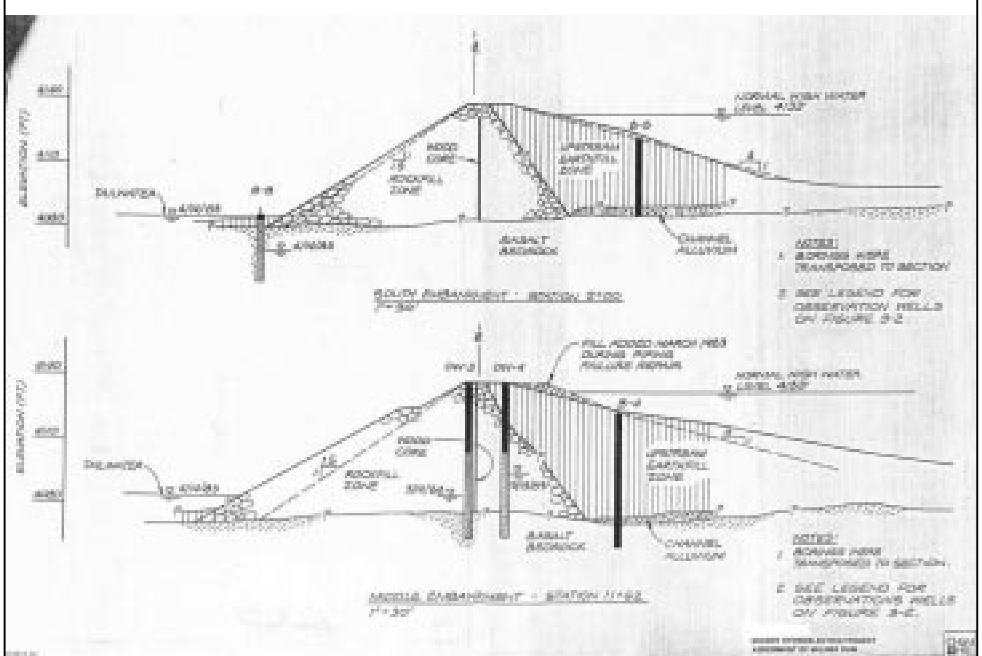


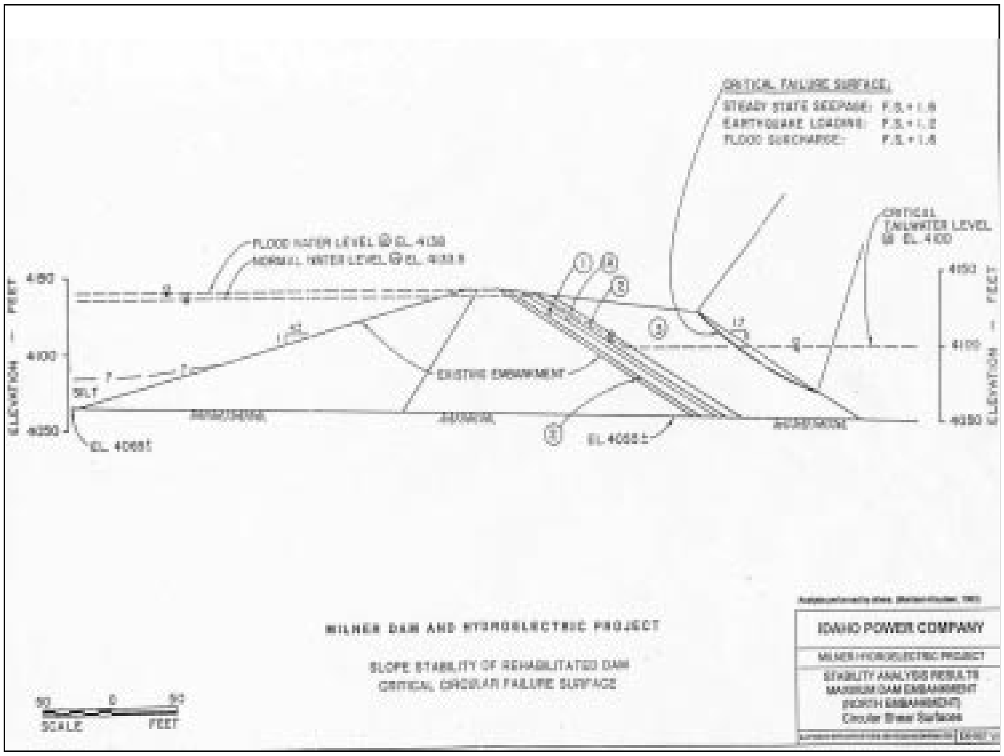


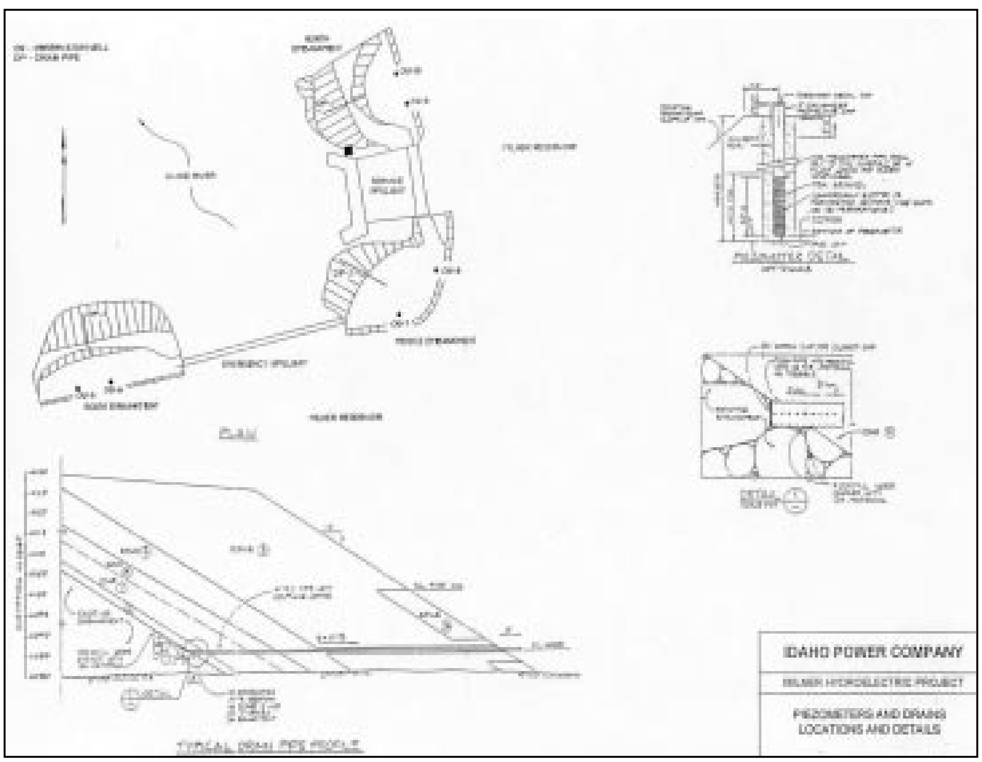
Milner Dam

- Control of Seepage Flow Through the Embankment, Foundation, And Abutments to Prevent Internal Erosion (Piping)
- The Slopes Must be Stable During all Conditions of Reservoir Operations
- Do Not Permit Unacceptable Deformations Under Earthquake Conditions


DAM SECTION *
FOR CHANGES TO EMBANKMENT NEAR S. END OF GATED SPILLWAY, SEE TEXT RE: 1916 & 1918 REPAIRS. (*)
SPILLWAY GATES *
NORTH EMBANKMENT
EMERGENCY SPILLWAY (*)
MIDDLE EMBANKMENT
SOUTH EMBANKMENT (*)
PLAN *

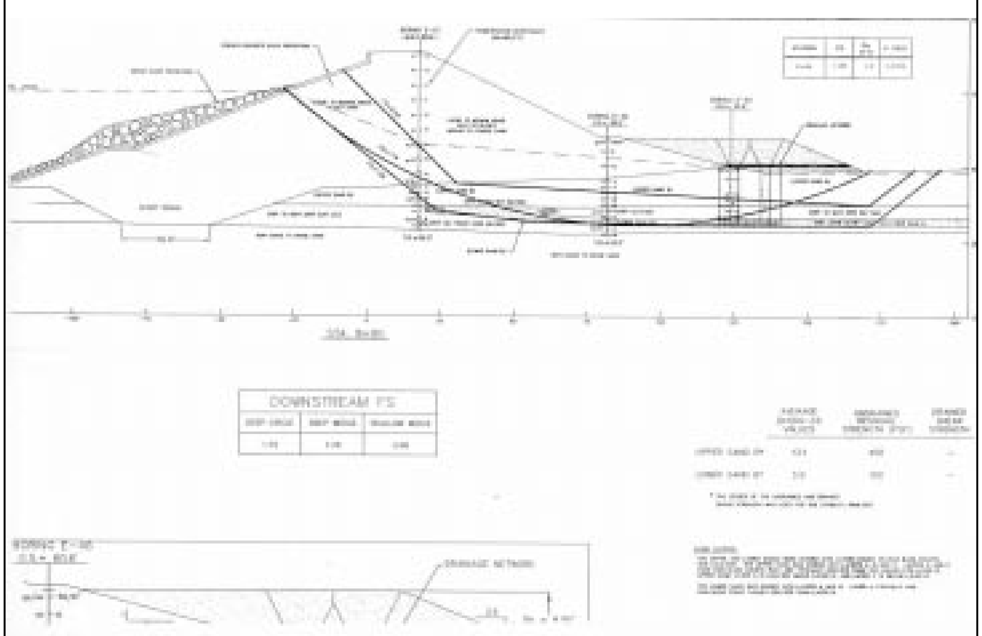

NORMAL HIGH WATER LEVEL 4132
ROCKFILL ZONE
UPSTREAM EARTHFILL ZONE
BASALT BEDROCK
CHANNEL ALLUVIUM
TAILWATER
SOUTH EMBANKMENT - SECTION 3+00
MIDDLE EMBANKMENT - STATION 11+65
NOTES:
SEE LEGEND FOR OBSERVATION WELLS ON FIGURE 3-2.

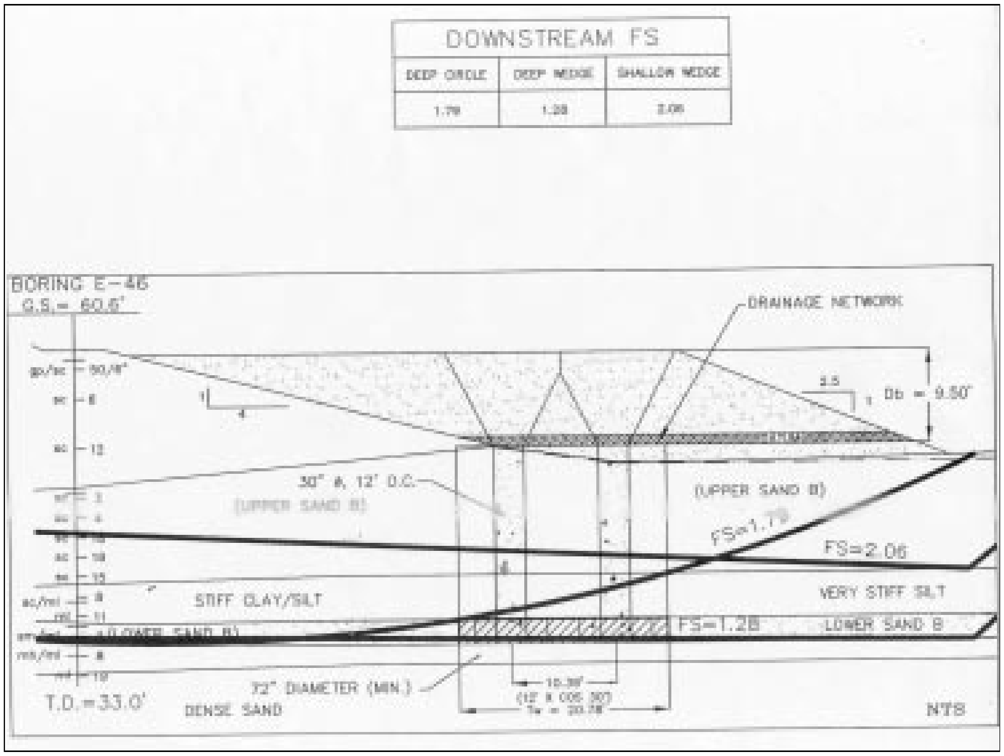

CRITICAL FAILURE SURFACE:
STEADY STATE SEEPAGE: F.S.=1.8
EARTHQUAKE LOADING: F.S.=1.2
FLOOD SURCHARGE: F.S.=1.8
CRITICAL TAILWATER LEVEL @ EL. 4100
FLOOD WATER LEVEL @ EL. 4139
NORMAL WATER LEVEL @ EL. 4138.5
EXISTING EMBANKMENT
SILT
EL. 4068±
ELEVATION — FEET
MILNER DAM AND HYDROELECTRIC PROJECT
SLOPE STABILITY OF REHABILITATED DAM
CRITICAL CIRCULAR FAILURE SURFACE
SCALE FEET
IDAHO POWER COMPANY
MILNER HYDROELECTRIC PROJECT
STABILITY ANALYSIS RESULTS
MAXIMUM DAM EMBANKMENT
(NORTH EMBANKMENT)
Circular Shear Surfaces


PLAN
TYPICAL DRAIN PIPE PROFILE
IDAHO POWER COMPANY
MILNER HYDROELECTRIC PROJECT
PIEZOMETERS AND DRAINS
LOCATIONS AND DETAILS

Santee Cooper East Dam

- The Embankment, Foundation, Abutments, and Reservoir Rim Must be Stable and Must Not Develop Unacceptable Deformations Due to Earthquake Loadings
- Use of Hand Calculations to Confirm Computer Calculations




DOWNSTREAM FS
DEEP CIRCLE
DEEP WEDGE
SHALLOW WEDGE
1.79
1.28
2.06
BORING E-46
G.S.= 60.6'
DRAINAGE NETWORK
2.5
Db = 9.50'
30" Ø, 12' O.C.
(UPPER SAND B)
(UPPER SAND B)
FS=1.79
FS=2.06
STIFF CLAY/SILT
VERY STIFF SILT
FS=1.28
LOWER SAND B
(LOWER SAND B)
72" DIAMETER (MIN.)
T.D.=33.0'
DENSE SAND
NTS

Dayton Dam Canal Embankment Failure & Repair

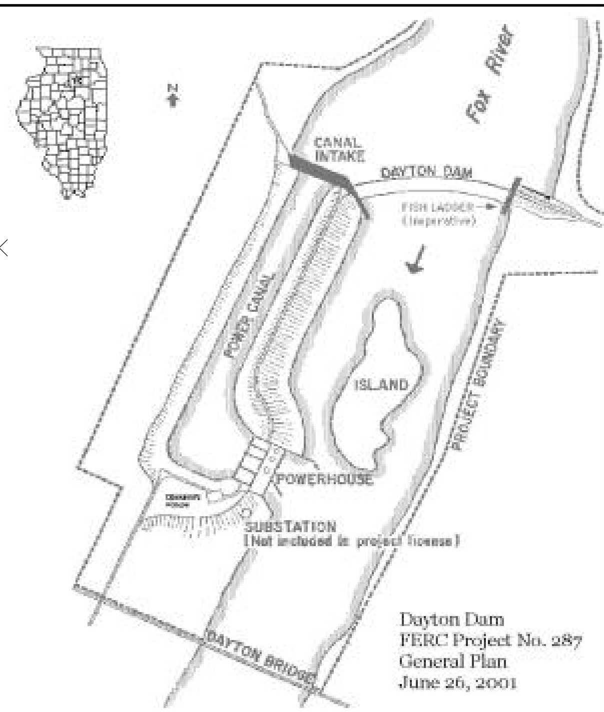
Michael S. Davis - Lead Engineer

Chicago Regional Office

Federal Energy Regulatory Commission

FERC-CRO June 2001

2/12/02
1



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Pertinent Data

- Licensed: 1923
- Built: 1925
- Hazard Potential: Low
- Flood of Record: 47,100 cfs (11/10/55)
- Canal Dike Height: 28 feet
- Canal Dike Length: 725 feet
- A substation, owned by a separate entity, is located adjacent to the powerhouse.

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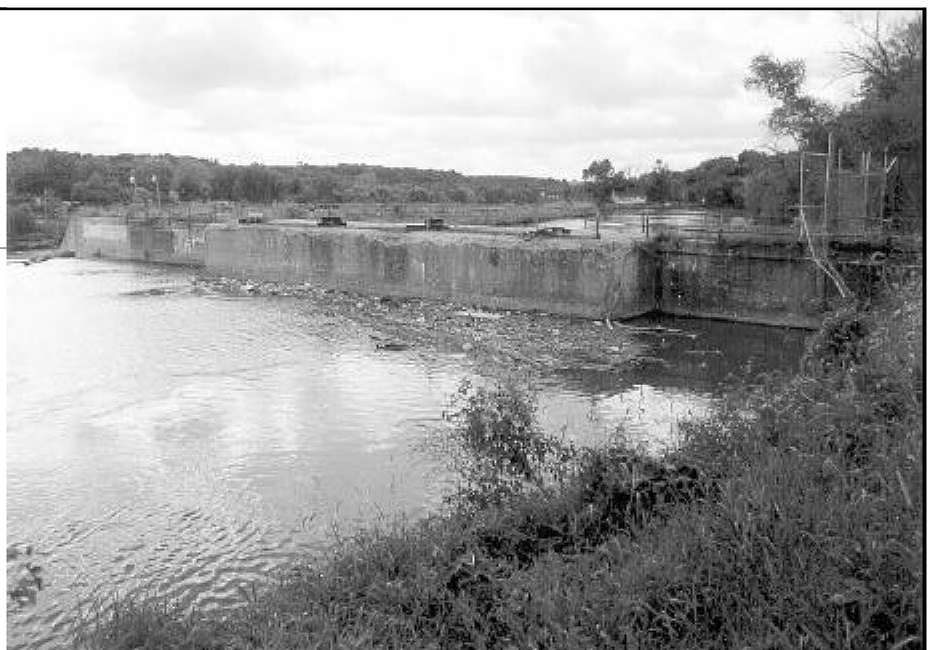

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Events Leading Up to the Breach

- On July 17, 1996, 16.91 inches of rain over an 18-hour period was recorded at the Aurora precipitation gage, which is located about 40 miles upstream of the dam. As a result, the reservoir rose throughout the following day.
- At 4 p.m., on July 18th, the water level was at about 2 feet below the crest of the headgate structure, and rising about 1 foot per hour. The tailwater was also still rising and was beginning to encroach on the substation.
- At that time, the substation owner ordered the plant be taken off line so that the power could be cut to the substation.
- As a result, the level in the canal rose about 3 feet at the powerhouse, and began to overtop the canal embankment around both sides of the powerhouse.

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The Breach

- With the reservoir still rising and the flow now overtopping the canal embankment, the canal embankment breached at about 7 p.m. The breach was initially measured to be about 50 feet wide when it reached the foundation. Several secondary breaches formed on the embankment as well.
- The resulting breach lowered the canal level and caused a differential pressure on the raised headgates. As a result, the chains holding the gates failed and all four gates slammed into the closed position and were severely damaged.
- The flood peaked at about 55,000 cfs later that night (setting a new flood of record) with the reservoir at about elevation 507.8 feet, which is 8.9 feet over the crest of the spillway, and about 3 inches over the crest of the headgate structure. The tailrace reached a peak elevation of 488.9 feet at the powerhouse.

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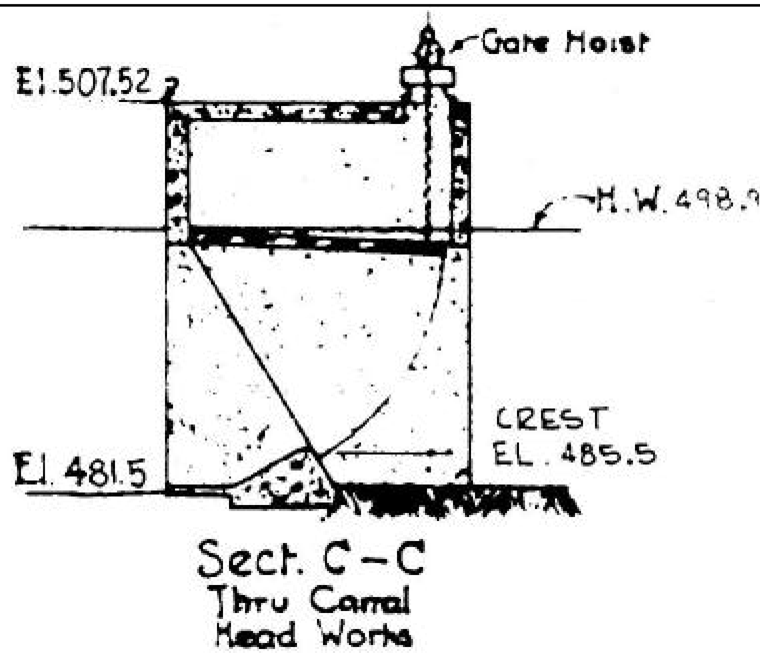

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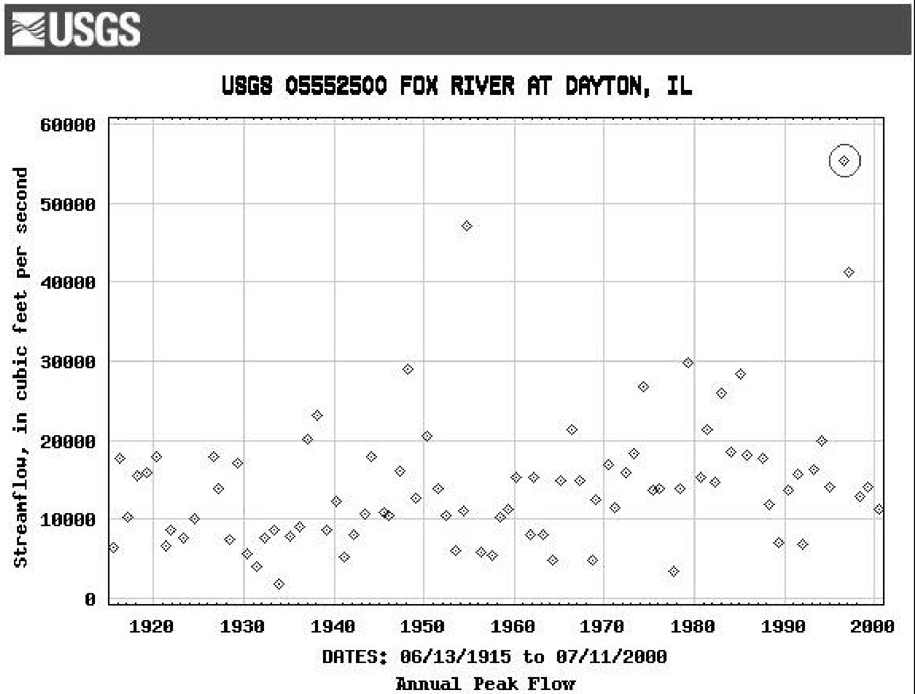

July 20, 1996 Inspection

- The next four photographs show the condition of the project structures two days after the breach.
- The inspection was unannounced and was done on a Saturday. Access to the site was limited to the left bank opposite the side of the canal.
- The tailrace had receded about 15 feet since the breach.

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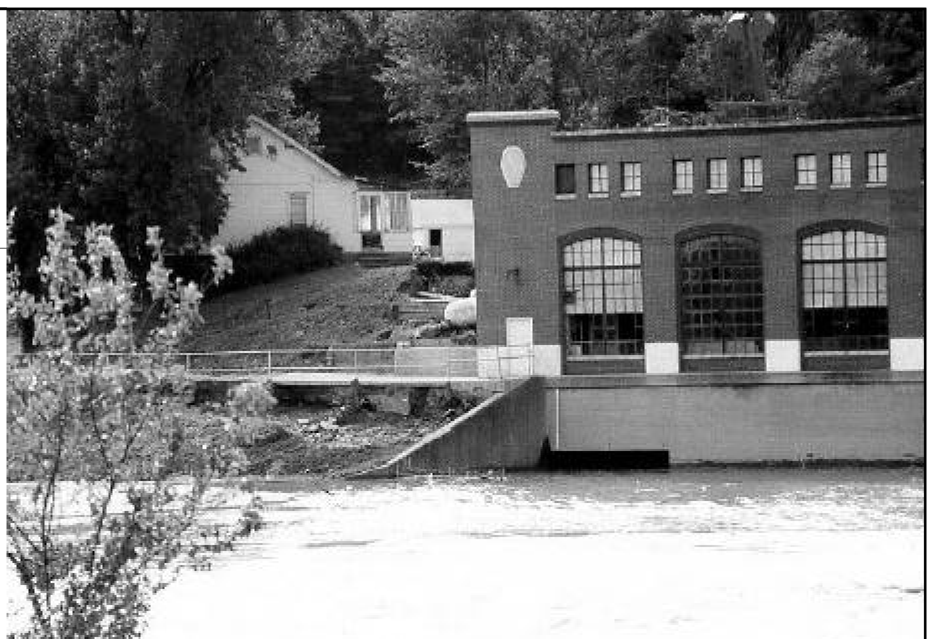

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July 22, 1996 Inspection

- The next nine photographs show the condition of the project structures four days after the breach.
- The tailrace had receded about another 6 to 8 feet since the July 20 inspection.

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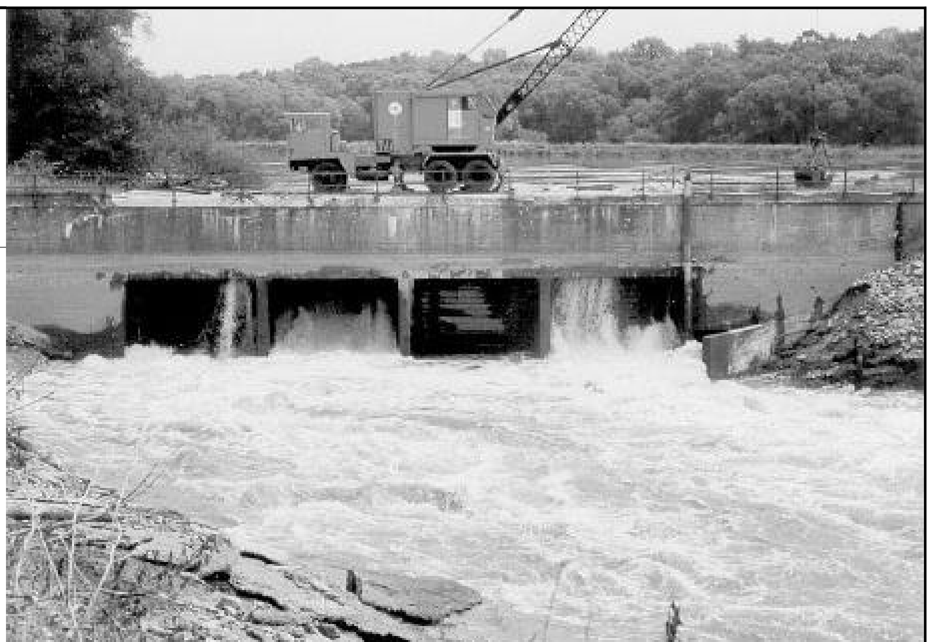

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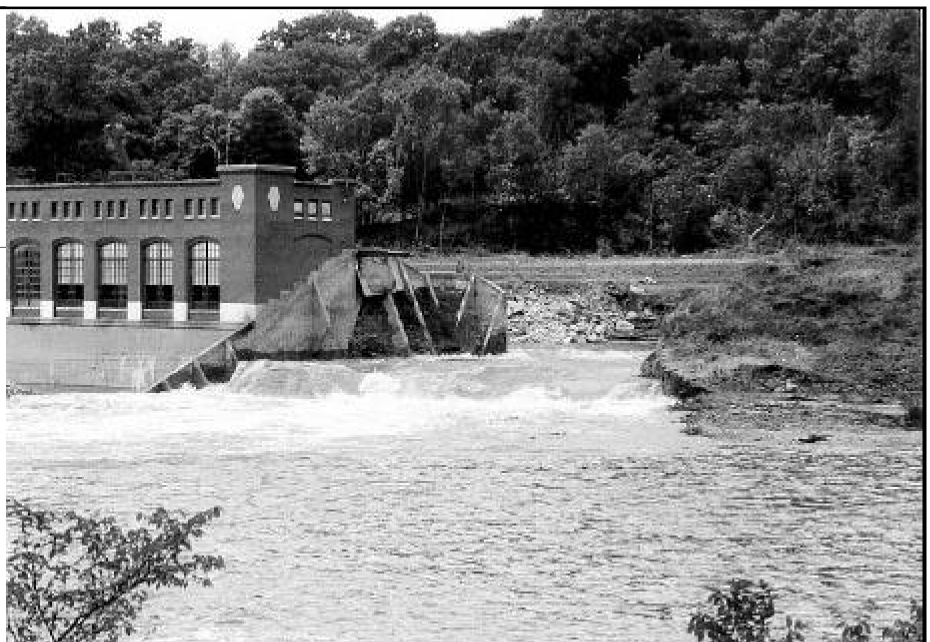

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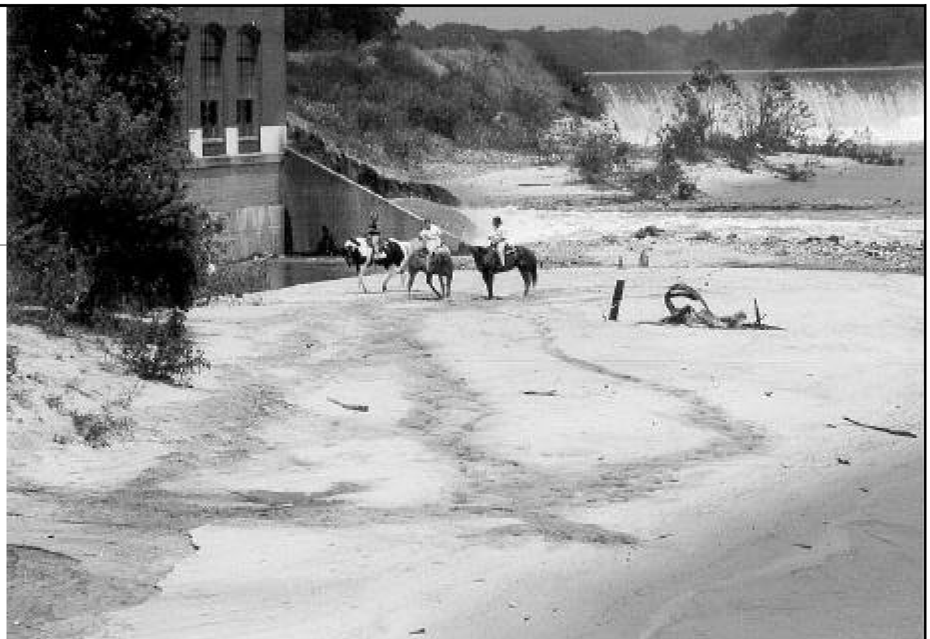

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September 30, 1996 Inspection

- On July 25, 1996, the licensee began to place large trap rock upstream of the headgate structure to cut off the flow.
- The cofferdam was completed on July 30, 1996.
- The next four photographs show the condition of the substantially dewatered canal.

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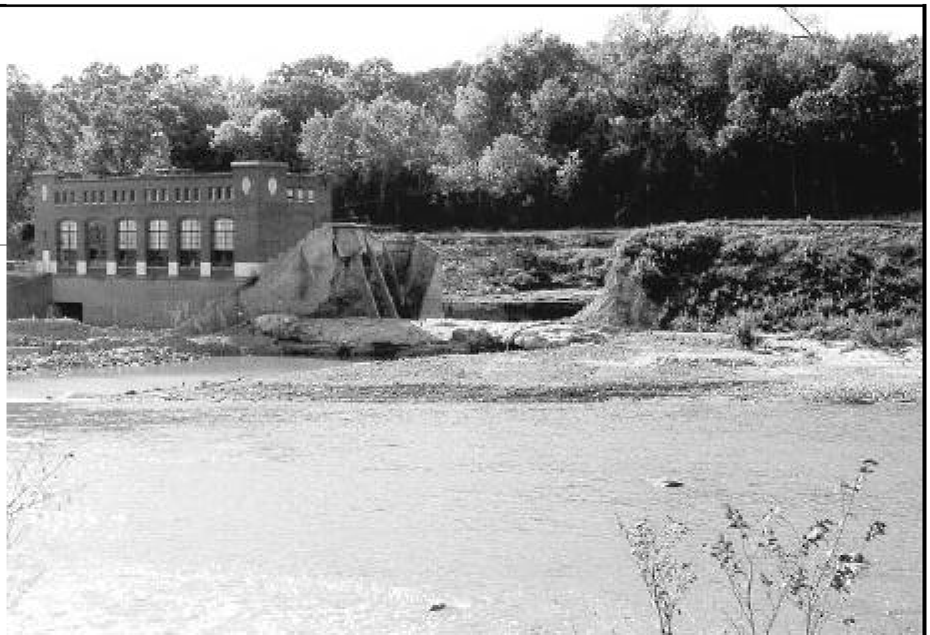

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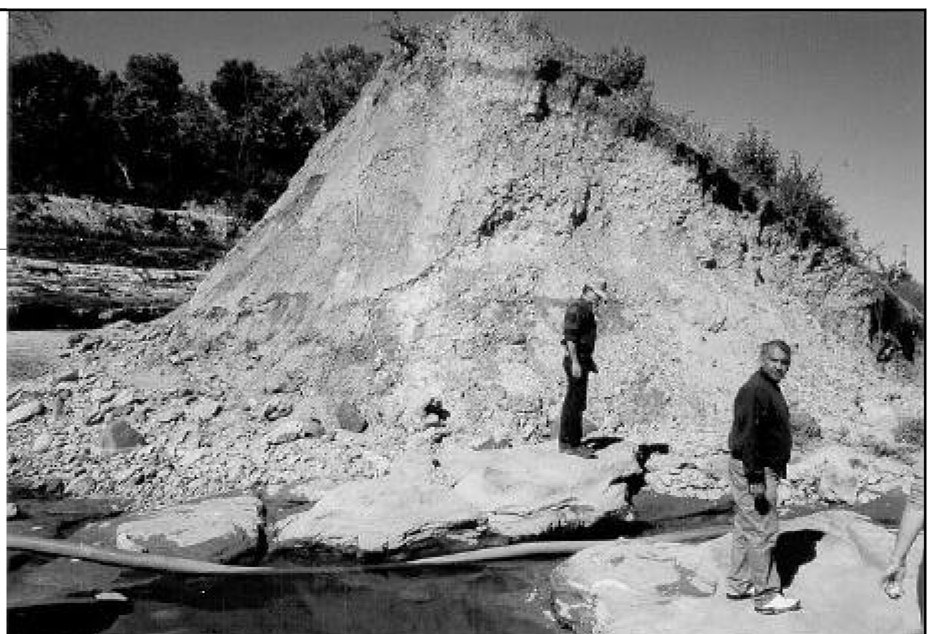

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Investigation and Evaluation

- Primary breach was located from Station 6+40 to Station 7+25, which is 85 feet at the crest.
- Secondary breaches were located at Stations 1+50, 3+00, and 4+00.
- Nine borings were taken of the existing embankment and foundation. The material was found to be heterogeneous, varying from clays, to silt and silty sand, to poorly graded sand. Blow counts ranged from 6 to 18.
- A loose-to-medium-dense silty sand layer about 2 feet thick was encountered on the bedrock from the centerline of the embankment to its toe.
- The foundation rock is a fine-grained, hard-jointed sandstone with numerous horizontal joints and fractures.

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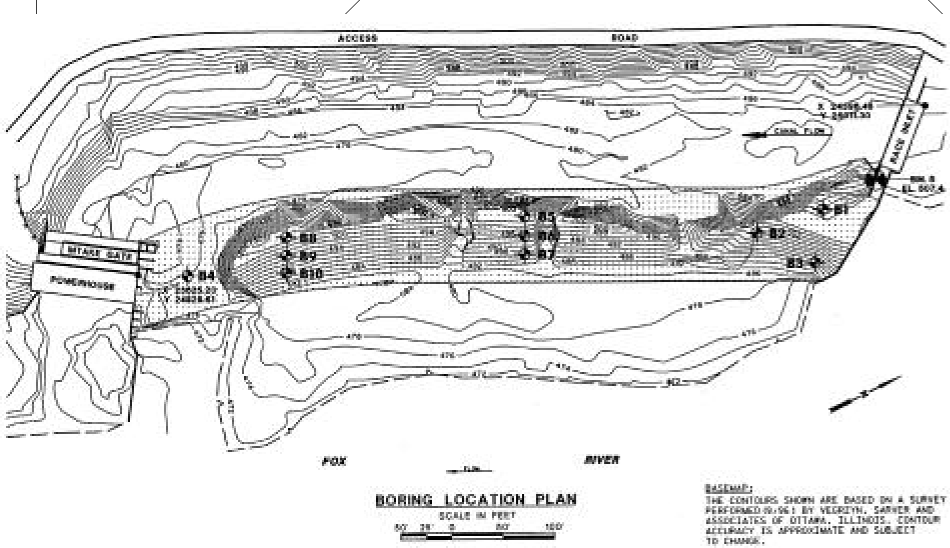

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Reconstruction of the Canal Embankment

- Construction began on July 31, 1997.
- The canal embankment was completed in late November 1997.
- Repairs to the headgate structure were completed in April 1998.
- Generation resumed on May 11, 1998.
- Cost of repairs was about $1,600,000.

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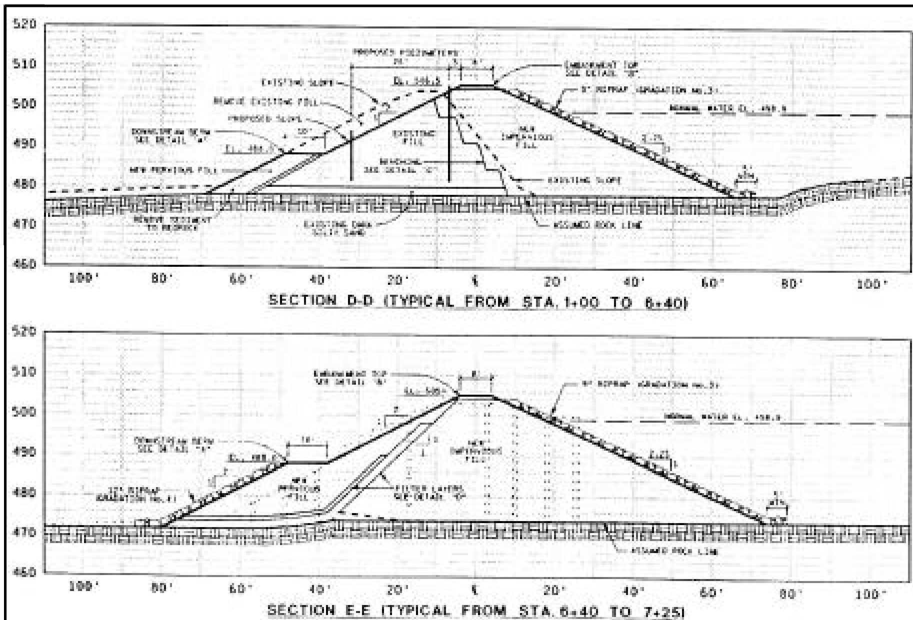

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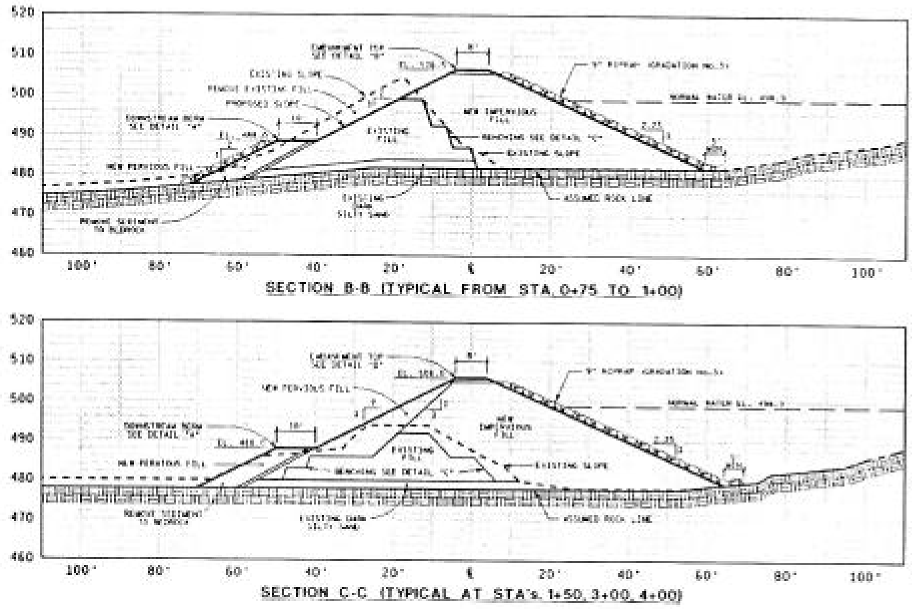

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August 27, 1997 Inspection

- The following two photographs show the progress of the reconstruction work.
- At the time of this inspection, the contractor was reconstructing the canal invert.

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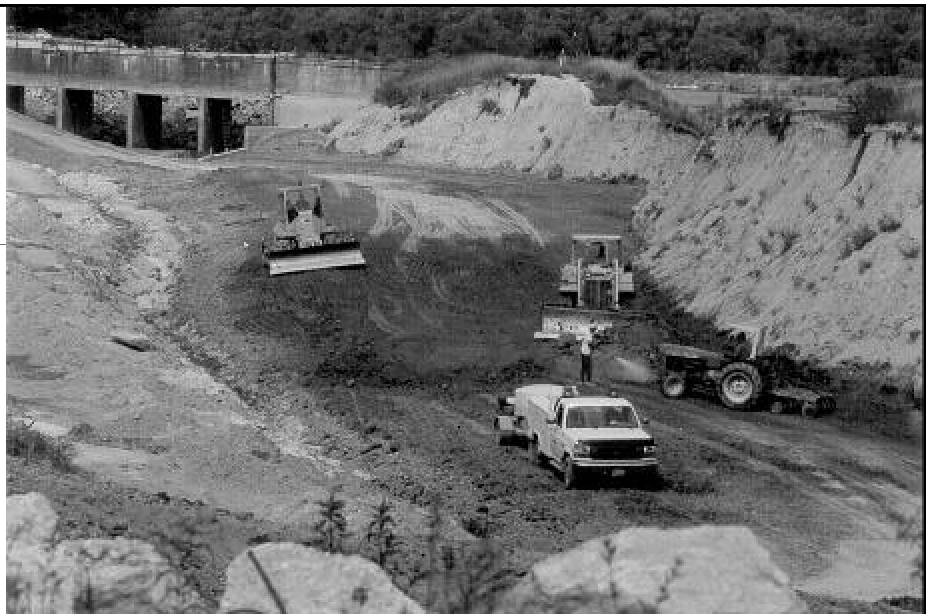

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September 30, 1997 Inspection

- The next four photographs show the canal embankment substantially completed.

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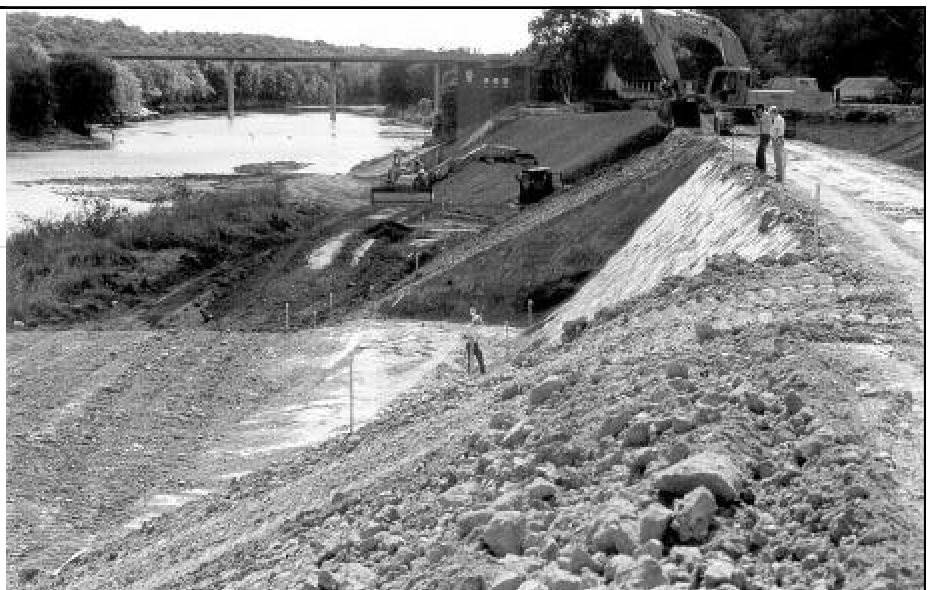

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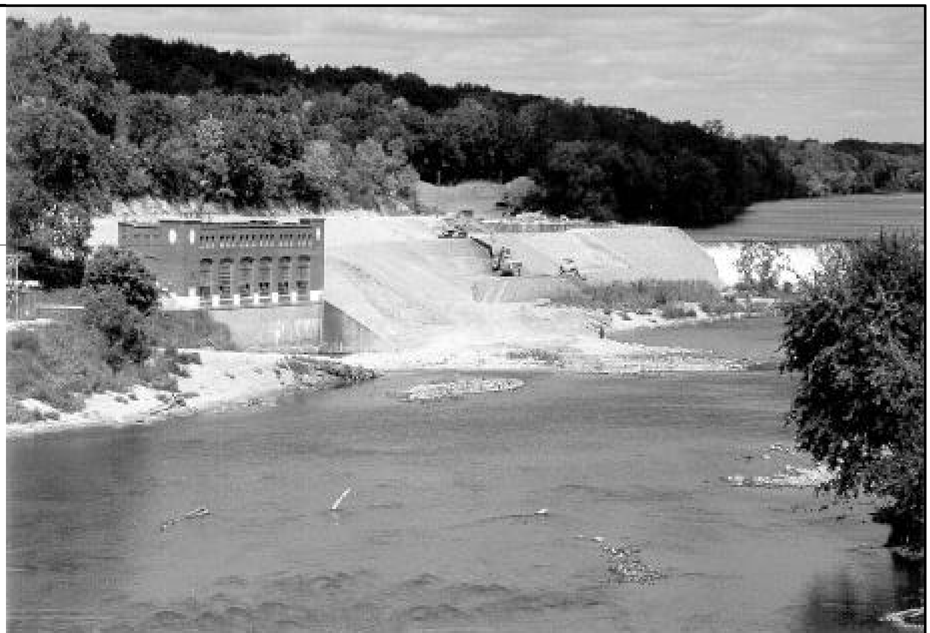

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Final Inspection – January 15, 1998.

- Embankment work was completed.
- New gates were being installed in the headgate structure.

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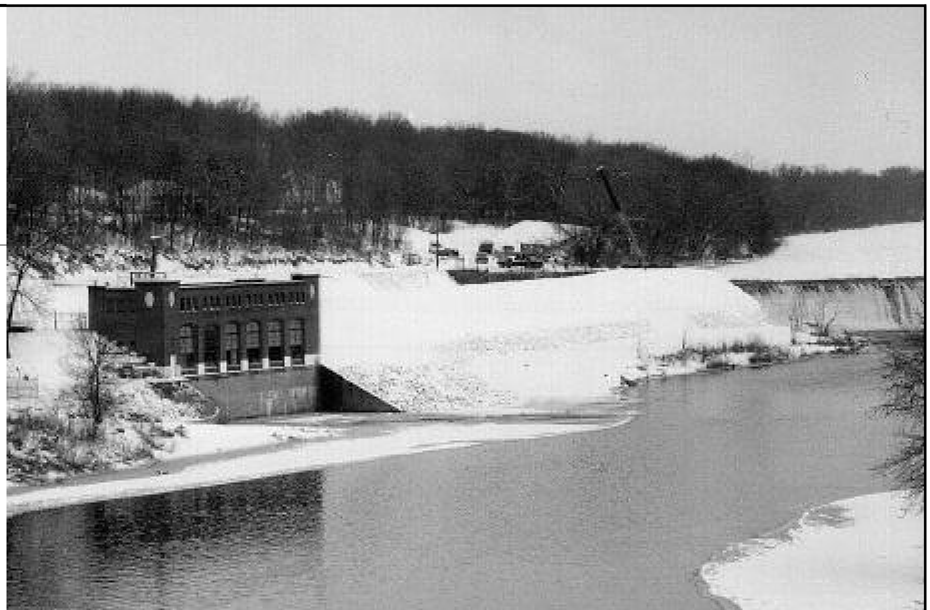

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B-14

Presentation for FEMA / USDA Workshop – 27 June 2001:
Workshop on Issues, Resolutions, and Research Needs Related to Dam Failure Analysis

BC HYDRO INUNDATION CONSEQUENCES PROGRAM

Derek Sakamoto, P.Eng.
BC Hydro

Introduction
BC Hydro is currently working on a program to define a methodology for assessing the consequences resulting from a potential dam breach. This Inundation Consequence (IC) Program was initiated in February 2000, with the goal of defining guidelines for performing the consequence investigation, which is to be followed by the completion of consequences investigations on all BC Hydro's dam facilities.

Included in this overview of the IC Program is a brief discussion of BC Hydro and its assets, a summary of the legislation and guidelines defining the requirements of the program, followed by a discussion highlighting the key components of the IC Program.

Primary Rationale & Objectives
The key focus of this program is to provide an improved investigative tool for safety management planning. In the case of dam-breach emergency planning, this program will provide decision-makers with realistic characterizations of the various situations to which they may have to respond. Investigation into the effect of parameters such as dam breach scenarios and temporal variation related to the flood wave propagation can be performed. Additionally, severe "non-dam-breach" scenarios, such as the passing of extreme floods, can be investigated.

A valuable product from these investigations will also be in providing powerful communication tools. This will benefit decision-makers by ensuring they are well informed of the magnitude of potential impacts related to dam breaches, thus enabling emergency precautions that are proportionate to the consequences and uncertainties. Additionally, meeting regulatory approvals and due diligence are key factors.

BC Hydro
British Columbia (BC) is the western most of Canada's provinces, located on the Pacific Ocean along the West Coast. BC Hydro itself is a crown corporation, meaning it is a corporation that is owned by the province. The corporation, however, is run like a business without subsidies from the government; and like other commercial businesses its dividends are provided to its owner which, in this case, is the Province of BC.

BC Hydro owns 61 dams located within 6 operating areas:

- Columbia River Basin – encompassing the upper region of the Columbia River and draining into Washington State, this area produces approximately 50% of BCH power.
- Peace River Basin – the second largest power generating area, the Peace River joins with the Athabasca River in Alberta.
- Coastal Region – Several smaller dam facilities located along the BC coast.
- Lower Mainland (Vancouver Region) – housing facilities located near the City of Vancouver.
- Fraser River Basin – 4 dam facilities draining into the Fraser River.
- Vancouver Island – a number of dam facilities located on Vancouver Island (southwestern corner of BC), taking advantage of the high precipitation of the Pacific West Coast.

BC Hydro's assets range from the extremely large Mica Dam to the smaller Salmon River Diversion Dam. Mica Dam, a 243 metre high earth-fill dam, is located at the headwaters of the Columbia River; the Salmon River Dam is a 5.5 metre earth-fill diversion dam located on the Campbell River system on Vancouver Island.

BC Dam Regulations and Guidelines
Legislation for dam safety has been recently updated. Managed on a provincial level, the Province of BC passed its Dam Safety Regulations in late 1999. The need for defined safety regulations arose from such recent incidents as the failure of a private dam in May 1995. Although the breach of this small (6 metre high) dam did not result in any loss of life, over $500,000 in damage to property and infrastructure along with massive sediment loading into a local river resulted. The Province of BC has established regulations which define hazard classifications (Very High, High, Low & Very Low) for dams based on their consequence. Based on these hazard classifications, frequency of inspection to ensure the safe operations of the dam facilities are outlined as follows:

Item	**Very High Consequence**	**High Consequence**	**Low Consequence**	**Very Low Consequence**
Site Surveillance [a]	WEEKLY	WEEKLY	MONTHLY	QUARTERLY
Formal Inspection [b]	SEMI-ANNUALLY	SEMI-ANNUALLY or ANNUALLY	ANNUALLY	ANNUALLY
Instrumentation	AS PER OMS * MANUAL	AS PER OMS * MANUAL	AS PER OMS * MANUAL	N/A
Test Operation of Outlet Facilities, Spillway gates and other Mechanical Components	ANNUALLY	ANNUALLY	ANNUALLY	ANNUALLY
Emergency Preparedness Plan	UPDATE COMMUNICATIONS DIRECTORY SEMI-ANNUALLY	UPDATE COMMUNICATIONS DIRECTORY SEMI-ANNUALLY	UPDATE COMMUNICATIONS DIRECTORY ANNUALLY	N/A
Operation, Maintenance & Surveillance Plan	REVIEW EVERY 7 - 10 YEARS	REVIEW EVERY 10 YEARS	REVIEW EVERY 10 YEARS	REVIEW EVERY 10 YEARS
Dam Safety Review [c]	EVERY 7-10 YEARS [d]	EVERY 10 YEARS [d]	[d]	[d]

Further information regarding the BC regulations can be found at the web site:
http://www.elp.gov.bc.ca/wat/dams/reg_final.html

In addition to the BC regulations a consortium of dam owners in Canada called the Canadian Dam Association (CDA) has established guidelines defining key design parameters for dam construction. As with the inspection requirements of the provincial regulations, the level of the design requirements are based on the consequence classification:

Consequence Category	**Earthquake Criteria Maximum Design Earthquake (MDE)**		**Inflow Design Flood (IDF) Criteria**
Very High	Maximum Credible Earthquake (MCE)	1/10,000	Probable Maximum Flood (PMF)
High	50% to 100% MCE	1/1000 to 1/10,000	1/1000 to PMF
Low	-	1/100 to 1/1000	1/100 to 1/1000

Further information regarding the Canadian Dam Association can be found at web site:
http://www.cda.ca

Inundation Consequence Program
Building upon the legislative, design and safety requirements, the IC program is focused on defining the consequences associated with potential dam breaches. In doing this assessment, four key tasks have been defined:

- Hydraulic modeling

- Life Safety Model
- Environmental / Cultural Impact Assessment
- Economic / Social Impact Assessment

Hydraulic Modeling

Previous breach assessment work done by BC Hydro was done during the 1980's. This analysis provided inundation mapping for assumed dam breach scenarios, which were completed using the NWS DAMBRK model. In looking to update what was "state of the art" of its time, BC Hydro has opted to update these breach studies using the 2-dimensional hydraulic model TELEMAC-2D. The decision to use the 2-dimensional model is driven by two aspects. The 2-D model offers the ability to simulate complex flow patterns, which will be valuable tool in simulating the spread of flood waves over wide areas, or circulation and backwater of flows. Additionally, the 2-D output is an integral part of the Life Safety Model discussed later.

There is, however, a need to identify the data requirements in selecting the correct computer model. In cases of "low consequence" dams, it may not be necessary to go to the expense and level of effort required of the 2-D model when a 1-D model can provide the same, or sufficient results. Two levels of assessment in the IC Program may be performed, with 1-D or coarse 2-D models being used on "low consequence" dams, and the more detailed 2-D models being used on the "high consequence" facilities.

Life Safety Model

BC Hydro is developing the Life Safety Model (LSM), a 2-D computer model which will be used to estimate Population at Risk (PAR) and Loss of Life (LOL) in the event of a dam breach. The power of the LSM is in the ability to simulate the movement of people over real space as they becoming aware of the dam breach, and models how people will escape from a flood. Using national census data, the PAR can be distributed over areas being assessed. Various scenarios are prepared distributing the PAR based on time of day, day of week, or season.

This dynamic model will use the flood wave hydrograph produced by TELEMAC-2D as input, simulating the movement of the PAR in real time as the flood-wave propagates. The LSM model then simulates how the people react to the flood-wave, and their means of escape. A key aspect of this modeling is in providing a valuable tool in defining evacuation routes, potential "bottle-necks" in the evacuation plans, and highlighting problems associated with high risk areas such as hospitals or schools.

Environmental / Cultural Impact Assessment

The dam breach could result in a number of environmental and cultural impacts in areas both upstream and downstream of the dam. Three *Consequence Types* were identified (Physical, Biological, and Human Interaction) with 18 resulting *Consequence Categories*:

<u>Physical</u>
terrain stability, river channel changes, soil loss / deposition, mobilization of debris, & water quality
<u>Biological</u>
vegetation, fish, fish incubating, wildlife, productivity of reservoir, & productivity of receiving systems
<u>Human Interaction</u>
forest, agricultural resources, mineral resources, biological resources, settlement, recreation, & heritage

Evaluation of these individual consequence categories is based on the net impact the potential dam breach could have on them. For each category, a series of "linkage diagrams" have been established. Each link defines the resulting effect that the breach can have on the specific category in varying degrees of severity. The more severe the impact, the higher up the linkage diagram.

Economic / Social Impact Assessment

The initial and key challenge in the economic assessment model was in the identification of all structures (residential, institutional, businesses, industries etc.) at risk. Geographic Information Systems (GIS) is being utilized to link the various available databases, the location of areas at risk, and the magnitude of the impending hazard. Databases that were used to identify areas at risk include:

- hydraulic model inundation polygon provided in UTM coordinates (TELEMAC-2D output);
- BC Hydro customer database (providing building location with UTM coordinates & address);
- BC Assessment Authority database (providing property values, property improvement value, construction material, structure use, age, number of floors, etc.)

GIS also provides a valuable assessment tool in yielding a powerful graphical representation of properties at risk. It also yields an easily queried database to assess economic impact based on various scenarios. Future work will entail linking the economic losses with respect to social impact on communities in the inundation zone.

IC Program Future
A pilot program is currently under way to establish guidelines for completing the Inundation Consequence assessments. A draft of these guidelines is planned for completion during the summer of 2001 and finalized in 2002. Ultimately, inundation consequence assessments will be completed for all the BC Hydro sites.

Presenter: Derek Sakamoto is an engineer with BC Hydro's Power Supply Engineering (PSE) group, and works in the Civil Engineering / Water Resources team. Having been with PSE for just over one year, Derek brings to his team over five years in consulting with a focus in design, construction and assessment work in hydraulic/hydrologic related projects.

Contact at: (604) 528-7812 (phone); (604) 528-1946 (fax); derek.sakamoto@bchydro.com (email)
BC Hydro - 6911 Southpoint Drive (E13), Burnaby, BC, CANADA, V3N 4X8 (mail)

B-15

Analyzing Flooding Caused by Embankment Dam Breaches: A Consultant's Perspective

By Ellen B. Faulkner, P.E.
Mead & Hunt, Inc.
Madison/Eau Claire, Wisconsin

Introduction

As engineering consultant to owners of dams throughout the United States, Mead & Hunt performs dam safety assessments which must be responsive both to the needs of the dam owner and to the requirements of state and federal regulatory agencies. Frequently, these dam safety studies include the simulation of a hypothetical dam failure for the purpose of hazard classification, emergency action planning, or design flood assessment. Each dam failure study begins with the identification of a critical, but plausible, mode of failure and the selection of specific parameters which define the severity of the failure. These parameters include the ultimate dimensions of the breach, the time required to attain these dimensions, and (in the case of overtopping failures of embankment dams) the depth of overtopping required to initiate a failure.

None of these quantities is easily identified. In many cases, the obvious solution is to choose the "path of least resistance" - that is, the parameters which will most easily meet with regulatory acceptance. However, choosing excessively conservative breach parameters may impose significant costs on the dam owner in the form of new design work and remedial actions, additional safety studies, or unnecessarily complex or inefficient emergency action plans.

Clearly, the design, construction, and material composition of an earthen embankment significantly affect how a breach will form. As consultants we are aware that analytical approaches exist, based on theory, experiment, and experience with real dam failures, for relating breach size and speed of formation to the characteristics of the embankment. However, these approaches are not yet well-established enough to use in the regulatory settings in which we work. One Mead & Hunt study from northern Wisconsin, now almost ten years old but still fairly representative of the difficulties that may be encountered in this type of study, illustrates how different approaches to simulating an embankment breach can lead to substantially different conclusions with respect to design and safety requirements.

Case Study Setting

The Chalk Hill hydroelectric project is located on the Menominee River on the border between northeast Wisconsin and Michigan's Upper Peninsula. Three miles downstream is the White Rapids project, owned by the same utility. Both dams contain concrete spillway sections and long earth embankments, but Chalk Hill's embankment is significantly higher (37 feet) than that at White Rapids (25 feet). The river valley below both dams is lightly developed, with a mix of year-round and seasonal residences located near the river and potentially in the dam failure inundation area.

The studies described below were performed in 1992. They were the most recent of a series of dam break studies for the dams, which began in 1983 with a HEC-1 storage routing model which indicated that the hazard related to overtopping embankment failure of either dam was minimal. In 1987, failures of the embankments were re-analyzed using the NWS-DAMBRK dynamic routing model. In both the 1983 and 1987 studies, the assumed breach dimensions were consistent with then-current guidelines, which called for a breach bottom width equal to the height of the dam.

In 1988, the Federal Energy Regulatory Commission (FERC) issued new guidelines regarding breach assumptions used for emergency action plans, inflow design flood studies, and hazard classifications. In the case of breaches in earth embankments, the assumed average breach width was to be as much as five times the dam height. Reviewing the 1987 reports under these guidelines, FERC requested a re-analysis for Chalk Hill and White Rapids using a wider breach. These analyses were conducted in early 1992. For White Rapids, where the height of the embankment was just 25 feet and downstream development relatively high on the valley walls, the re-analysis still indicated no incremental hazard due to overtopping flows; that is, the existing spillway capacity was adequate. For Chalk Hill, however, the use of a breach width in the high end of the stipulated range led to an IDF determination about twice the existing spillway capacity.

Part of the problem at Chalk Hill was an assumed domino failure of White Rapids Dam. Although White Rapids did not pose a downstream hazard by itself, an overtopping failure of White Rapids in conjunction with the peak of the dam failure wave from Chalk Hill would affect residences which were not in the inundation area of White Rapids alone. However, the assumed failure of White Rapids, consistent with FERC's approach, occurred at the peak overtopping stage after the Chalk Hill failure. This scenario would require that the White Rapids embankment survive about three feet of overtopping before finally failing. If White Rapids could be assumed to fail at some lesser depth of overtopping, the downstream consequences would be less.

In most inflow design flood studies, the addition or removal of a few structures to the dam break inundation zone is of little consequence, as long as at least one inhabited structure is affected by the flood. In this case, however, each structure determined to be affected was important because one of the owner's alternatives was to purchase affected properties.

Physical Model Analysis (NWS-BREACH)

The inflow design flood determination for Chalk Hill Dam involved a number of separate breach assumptions: the breach formation time and dimensions at Chalk Hill; the formation time and dimensions at White Rapids; and the depth of overtopping which would certainly cause failure at White Rapids. (Another issue, related to the evaluation of alternatives for upgrading the spillway capacity, was whether initially confining Chalk Hill overtopping flows to a low section of the embankment would successfully promote a non-critical failure in that section.)

We questioned whether the extreme breach parameters used in the first 1992 study were appropriate, considering two characteristics of the dam. First, the embankments were engineered and well-constructed -- unlike many dams whose actual historical failures formed the database that was apparently the foundation for the guidelines. Second, both reservoirs were small and drawdown would happen quickly. In an attempt to determine whether breach dimensions in the middle or lower end of the FERC's suggested range would be consistent with the site-specific characteristics of the dams, we used the NWS-BREACH program to assess the breach formation characteristics at both Chalk Hill and White Rapids dams.

BREACH is a physically based erosion model for embankment dams, and generates a time sequence of breach dimensions and an outflow hydrograph given an inflow hydrograph, the dam and reservoir capacity, and geometry and material properties for the embankment. The data requirements for BREACH include dike material data such as cohesion, angle of internal friction, void ratio, unit weight, plasticity, and gradation. The availability of almost all of these data through recent boring studies was another factor which made the physical model approach practicable for the Chalk Hill study.

Using the NWS-BREACH model was a new approach in our experience, and one not approved by the FERC. To anticipate reviewers' concerns about the accuracy and conservativeness of the model, we adopted an approach in which we simulated the breach using the most critical lab test values from both borings at each site. We tested one input variable at a time, choosing the single test value from the two borings which gave the most severe breach. When two tests values were not available, we chose the worst-case value of the input variable, based on ranges given in the BRAECH program documentation.

The resulting breach description was significantly different from any of those postulated, based on written guidelines, in previous studies. Table 1, below, summarizes the differences between the analyses.

Table 1
Comparison of Chalk Hill Embankment Breach Characteristics Using FERC Guidelines and NWS-BREACH Program

Breach Assumptions	Depth of Overtopping Required to Initiate Breach (ft)	Breach Formation Time	Time for Breach to Erode to Bottom of Dam	Ultimate Breach Bottom Width	Breach Side Slope (H:V)
Pre-1988 Studies	0.5	1 hour	1 hour	37 feet (1 x dam height)	1:1
Studies Based on 1988 Guidelines	0.5	0.3 hour	0.3 hour	111 feet (3 x dam height)	1:1
NWS-BREACH	0.6	> 2 hours	0.1 hour	25 + feet	1:1

There were two major differences between the BREACH results and earlier assumptions. First, the BREACH program gave a much smaller breach after 2 hours (the time step limit in the program) than we had previously assumed for a one-half-hour formation time. Second, the simulated breach eroded very quickly to the bottom of the dam, at which time the peak reservoir outflow occurred, then continued to slowly widen as the reservoir level dropped. At two hours, the average breach width was about 1.7 times the height of the dam -- within the range given in the FERC Guidelines, but near the lower end. The breach was still widening when the program halted due to time step limits, but the peak outflow had long since passed.

We also performed a sensitivity analysis to the individual material properties input to the program. Varying each one by plus or minus 25 percent, we found that the maximum changes in peak breach outflow were + 10 percent and -28 percent. The most sensitive parameters were cohesive strength and friction angle. We did not vary any of the parameters in combination with others, so did not determine how the various properties interacted in affecting the breach.

A separate NWS-BREACH analysis was conducted for the White Rapids project. There, the simulated breach was larger relative to the dam height. The average breach width at White Rapids was more than twice the dam height. The White Rapids embankment materials had a lower unit weight than those at Chalk Hill, were more poorly graded, and were assumed to have zero cohesion (due to a lack of test data).

Inflow Design Flood Determination

NWS-BREACH develops an outflow hydrograph but does not route it downstream. Therefore, we used the breach parameters predicted by the NWS-BREACH model as input to the NWS-DAMBRK model. The resulting Inflow Design Flood was 85,000 cfs -- still more than the calculated spillway capacity, but much less than the IDF indicated by the previous study.

The NWS-BREACH study never met with regulatory approval, apparently due to the very limited track record of the BREACH model and the startlingly less severe breach it predicted than had been assumed in previous studies. Returning to the more severe guidelines-based breach used in the previous study, however, we still had an unanswered question. This was the determination of risk below White Rapids Dam, which was presumed to fail as a result of the Chalk Hill breach wave. However, for some of the IDF cases considered, White Rapids would be overtopped by several feet before the Chalk Hill failure occurred. The BREACH model -- even if it had been accepted -- was of little help in this question, because it predicted a rapid failure at just 0.5 foot of overtopping. Although that may have been the most likely event, none of the parties involved were comfortable with it as a “worst-case” scenario. Finally, it was agreed, on the basis of professional judgement alone, that the White Rapids embankment need not be assumed to withstand more than two feet of overtopping.

Eventually, the owner of the projects addressed the IDF in two ways. One -- demonstrating that the best ideas are the simplest -- was to retest the radial opening of the spillway gates. The gates proved to open considerably farther than shown in the design drawings, resulting in a spillway capacity about 20 percent higher than had previously been computed. Secondly, the owner purchased outright or in easements the remaining affected properties, which were relatively few in number.

B-16

EMBANKMENT DAM FAILURE ANALYSIS

Private Consultant Experience*

by

Catalino B. Cecilio, P.E., P.H.
Consulting Engineer

1. Introduction

Following the near failure of Lower San Fernando dam in the San Fernando Valley earthquake of February 9, 1971, the California State Legislature enacted Senate Bill 896 relating to dam safety. This became effective March 7, 1973. It was subsequently amended by Senate Bill 1632 on May 31, 1974. Under the provisions of the new section of the code, the State Office of Emergency Services (OES), after consultation with the State Department of Water Resources (DWR), is required to identify those dams, the partial or total failure of which could cause death or personal injury due to flooding of the area below the dam. The owner of each dam so identified must then prepare and file inundation maps which show the areas of potential flooding in the event of sudden failure of the dam.

In 1980, the Federal Energy Regulatory Commission (FERC) included in their FERC Order 122, a requirement to prepare inundation maps for hypothetical failure of all dams under their jurisdiction and to prepare emergency action plans related to such hypothetical failures.

This paper will not provide any original contributions to the existing or past knowledge in dam failure analysis of an embankment dam. It will only describe the methods of approach used by the author in the implementation of the two government requirements. Only the assumptions on breach parameters are addressed in this paper. The flood routing is not included in this paper since the workshop is limited to breaching characteristics.

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* Presentation at the USDA/FEMA Workshop, "Issues, Resolutions, and Research Needs Related to Embankment Dam Failure Analysis," June 26-28, 2001, Oklahoma City, Oklahoma.

2. General

As mentioned in several public documents and other technical papers in the past, the analyses and effects of dam failures are complex and most failures are not well understood. The greatest uncertainty lies in the likely cause, mode and degree, and duration of failure.

In the early days, all dam-break solutions are based on theoretical equations, empirical equations or model studies. In 1978 Dr. Danny L. Fread developed a workable dam-break computer model, he called the NWS DAMBRK. This facilitated the work in dam-break analysis. Nevertheless, even with the presence of the NWS DAMBRK model, the application of all the mentioned solutions requires several assumptions based on engineering judgments for reasonable prediction of the flood hydrograph. The shape of the failure hydrograph depends on many variables such as the reservoir level, size, shape and position of the breach, and inflow into the reservoir.

3. Embankment Erosion

3.1 Cristofano's Equation

Embankment dams were considered to fail by erosion. At the beginning of the project, the rate of erosion used the relationship developed by E. A. Cristofano (3) for the Bureau of Reclamation. Although the original equation was intended for earthfill dams, rockfill embankments in our applications were treated with the same equation. The equation relates the volume of fill material eroded (or removed) to the water flowing through the breach for a unit area in the overflow channel. The assumed opening remains a trapezoid with a constant base and erosion along the sides is not considered. The equation also assumes that the slope of the breach in the direction flow remains constant and is equal to the developed angle of friction. Cristofano's formula is:

$$\frac{\text{Qsoil}}{\text{Qwater}} = e^{-x}$$

$$\text{where: } X = \frac{b \tan \Phi_d}{H}$$

The application of the above equation requires a trial and error solution with a time increment of a few seconds. A computer program developed by the Tennessee Valley Authority (4) facilitated solution of the equation. An initial breach length has to be assumed for the analysis.

There are times when the erosion process is so slow that the dam never fails at all. However, because the intent of the analysis is to fail the dam, a minimum value of erosion ratio had to be assumed to insure failure. It has also been demonstrated by the equation that during a few minutes after the breach has been initiated, the peak outflow and failure are reached in less than one hour. This pattern almost simulates an instantaneous failure. Also, the author recommended that the developed angle of friction will vary between the range of 11° to about 15°.

Even with the aid of TVA's computer program, the analyses for embankment dam failure were getting to be complicated and expensive. It was finally decided that the simpler approach would be applied.

3.2 Simplified Triangular Method

With no better terminology to call it, this method approaches the erosion failure in a very simplistic way. This method was used for the dambreak studies prepared for the California Office of Emergency Services (OES) in 1973. The steps and procedures are described as follows:

1. Assume that the failure of the dam is by erosion so that one-half of the reservoir volume or capacity is required to erode the initial breach to natural ground level.
2. Assume the final breach shape to be trapezoidal or parabolic in shape with the size related to the dam size, construction material and reservoir size. For trapezoidal shape of breach, the average width ranges from ½ to 3 times of dam height, side slope ranges from ¼ to 2 with setting vertical as a unit.
3. Assume that the maximum outflow would occur when the reservoir is one-half emptied.
4. The maximum outflow from the breach is estimated by:

$$Q_{max} = CH^{2.5}$$

where: C = coefficient that varies with breach shape. Examples are: C = 1.2 for triangular breach where slope (horizontal : vertical) is ½:1; C = 5.0 for parabolic breach with top width about three times the depth.

H = depth of water in feet at one-half reservoir capacity.

Sometimes when a trapezoidal breach is assumed, the most efficient hydraulic trapezoidal cross section is used to estimate the Q_{max}. The dimension of this efficient hydraulic cross section can be found in any hydraulics textbook. The basic weir formula is used to calculate the Q_{max} through the cross section.

5. Check the reasonableness of the estimated Q_{max} obtained from the previous step with the historical plots of dam failures prepared by Kirkpatrick (5) of the U. S. Bureau of Reclamation and shown as Figure 1 in this paper. If there is a significant difference in Q_{max}, then an adjustment of the time of failure, or the width and height of the breach is made.

6. The maximum discharge from the failure is assumed to occur at the midpoint of the outflow hydrograph. Using an isosceles triangle with Q_{max} at the apex, the rising and falling limbs of the hydrograph are adjusted so that the area under the hydrograph represents the storage volume in the reservoir.

3.3 NWS DAMBRK Method

In 1978, Dr. Danny Fread of the National Weather Service, released his first public version of the model NWS DAMBRK. PG&E obtained a working copy of the model and applied it to develop dam failure analyses to 178 dams to comply with the requirement of the Federal Energy Regulatory Commission (FERC) to prepare Emergency Action Plans. Of the 178 dams, 27 are earthfill, 24 are a combination of earth and rockfill or rock wall and 18 are rockfill. A 1980 version of the NWS DAMBRK model was later released. All dam-break analyses performed for PG&E dams were done with the 1980 version.

Breach shapes using the NWS DAMBRK were assumed to be trapezoidal for all embankment dams. The time of failure, which is identified in the program as TFH, was assumed depending upon the size of the reservoir. It was found from experience that an ideal failure hydrograph could be produced when the time of failure is between 0.1 hr to 1.0 hr.

Much of the guidance of breach shapes and time of failure were based on subsequent papers prepared by Dr. Danny Fread regarding the breach parameters.

All my dambreak analyses, regardless of type of dam, are done with the aid of the NWS DAMBRK model. I have probably performed some 250 dam break analyses in my career, which includes Aswan High Dam.

4. Case Studies

All dambreak analyses performed nowadays utilize assumed breach parameters provided by regulatory guidelines such as those issued by the Federal Energy Regulatory Commission. (6) Table 1 attached is taken from such reference showing the suggested breach parameters to be used in dam failure analyses for various types of dams. However, such guidelines should always be used with the thought that results from the assumed breach need to be compared with historical estimates such as shown in Figure 1.

For example, in estimating the peak failure flow of Aswan High Dam, it was found out that a time of failure of one hour for the embankment dam would not work with the NWS DAMBRK program which is based on the Saint Venant equation. The volume of the reservoir was so huge that it was found that the most reasonable time of failure would be between 12 and 24 hours. It can be seen that the 12 and 24 hours are outside the suggested values in Table 1.

In another case, a letter to one of my clients suggested using conservative values of breach parameters and compared the values obtained using the NWS DAMBRK. The result from the NWS DAMBRK run produced a peak failure flow of 46,500 cfs. Meanwhile, the staff using conservative values of breach parameters and the empirical equation for estimating peak flows taken from reference 6 obtained a peak failure flow of 151,100 cfs. However, in studying the results of the empirical equation, it was found that the peak flow of 151,100 cfs would release more than twice the volume of water that was available in the reservoir. This was pointed out to the staff member and the issue was resolved.

5. Conclusion

Use of the NWS DAMBRK has improved the modeling of dam failure even though a considerable amount of assumptions were employed. Dr. Fread developed another model called BREACH in 1988, but we never applied it because of the difficulty of using it and the amount of assumptions needed to apply it. We found that DAMBRK was more than adequate to satisfy our needs for developing dam-break analyses for Emergency Action Plans.

We found that the inappropriate use of empirical equations will produce unreasonable estimates of the peak failure flow. Use of conservative values is not the proper way to apply empirical equations.

However, we believe that an embankment breach model would be useful to the profession if it can be developed such that minimal amount of assumptions are applied.

6. References

1. State of California, Senate Bill No. 896, Chapter 780, "An Act to add Section 8589.5 to the Government Code, relating to dam safety, Approved by Governor August 11, 1972, filed with Secretary of State August 11, 1972.

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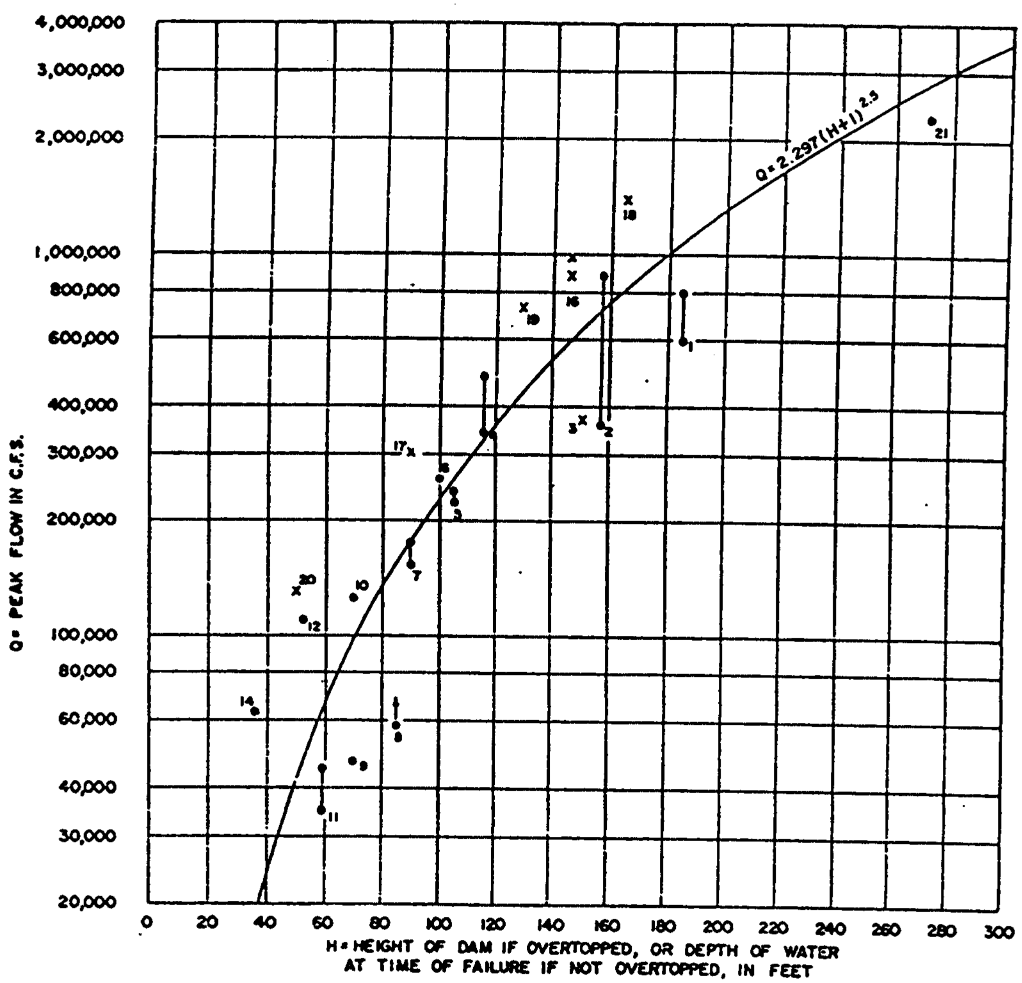
4. Tennessee Valley Authority, "Computer Program for Dam Breaching," Knoxville, Tennessee, 1973.

5. Kirkpatrick, Gerald W., "Evaluation Guidelines for Spillway Adequacy (Bureau of Reclamation)," Engineering Foundation Conference Proceedings, Asilomar Conference Grounds, Pacific Grove, California, November 28-December 3, 1976, Published by ASCE, NY, 1977.

6. Federal Energy Regulatory Commission, "Engineering Guidelines for the Evaluation of Hydropower Projects," FERC 0119-2, Office of Hydropower Licensing, Washington, DC, April 1991 with updates up to December 1994.



NAME OF DAM, LOCATION, YEAR OF FAILURE

1. St. Francis, California 1928
2. Swift, Montana 1964
3. Hypothetical Computation (Existing Dam)
4. Oros, Brazil 1960
5. Apishapa, Colorado 1923
6. Hell Hole, California 1964
7. Schaeffer, Colorado 1921
8. Granite Creek, Alaska 1971, discharge at 5 miles downstream
9. Little Deer Creek, Utah 1963
10. Castlewood, Colorado 1933
11. Baldwin Hills, California 1963
12. Hatchtown, Utah 1914
14. Lower Two Medicine, Montana 1964

16-20. Hypothetical Computations (Existing Dams)

21. Teton Dam, Idaho 1976

LEGEND

- • Actual Failure
- x Hypothetical Computation

BUREAU OF RECLAMATION

ESTIMATED FLOOD PEAKS FROM DAM FAILURES

REVISED F. A. B.–G. W. K. 1976

Source: Kirkpatrick, Gerald W., "Evaluation Guidelines for Spillway Adequacy (Bureau of Reclamation)," Engineering Foundation Conference Proceedings, Asilomar Conference Grounds, Pacific Grove, California, November 28 - December 3, 1976, Published by ASCE, NY, 1977

Figure 1

TABLE 1
SUGGESTED BREACH PARAMETERS
(Definition Sketch Shown in Figure 1)

Parameter	Value	Type of Dam
Average width of Breach ($\overline{BR}$) (See Comment No. 1)*	$\overline{BR}$ = Crest Length	Arch
	$\overline{BR}$ = Multiple Slabs	Buttress
	$\overline{BR}$ = Width of 1 or more Usually $BR \leq 0.5\ W$	Masonry, Gravity Monoliths,
	$HD \leq \overline{BR} \leq 5HD$ (usually between 2HD & 4HD)	Earthen, Rockfill, Timber Crib
	$\overline{BR} \geq 0.8 \times$ Crest Length	Slag, Refuse
Horizontal Component of Side Slope of Breach (Z) (See Comment No. 2)*	$0 \leq Z \leq$ slope of valley walls	Arch
	$Z = 0$	Masonry, Gravity Timber Crib, Buttress
	$\frac{1}{4} \leq Z \leq 1$	Earthen (Engineered, Compacted)
	$1 \leq Z \leq 2$	Slag, Refuse (Non-Engineered)
Time to Failure (TFH) (in hours) (See Comment No. 3)*	$TFH \leq 0.1$	Arch
	$0.1 \leq TFH \leq 0.3$	Masonry, Gravity, Buttress
	$0.1 \leq TFH \leq 1.0$	Earthen (Engineered, Compacted) Timber Crib
	$0.1 \leq TFH \leq 0.5$	Earthen (Non Engineered Poor Construction)
	$0.1 \leq TFH \leq 0.3$	Slag, Refuse

Definition:
- HD - Height of Dam
- Z - Horizontal Component of Side Slope of Breach
- BR - Average Width of Breach
- TFH - Time to Fully Form the Breach
- W - Crest Length

Note: See Page 2-A-11 for definition Sketch

**Comments: See Page 2-A-9 - 2-A-10*

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Colorado State University

Current Dam Safety Research Efforts

Presented By:
Steven Abt